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The **International Ferocement Information Center (IFIC)** was founded in October 1976 at the **Asian Institute of Technology** under the joint sponshirship of Institute's Division of Structural Engineering and Construction and the Library and 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over the world, **IFIC** attempts to collect information on all forms of ferrocement applications either published or unpublished.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 publication, reference and reprographic services and technology transfer activities. All information collected by **IFIC** are entered into a computerized data base using ISIS system. These informations 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, bibliographics 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 compling 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, conference 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 training programs for Indonesia and Malaysia; a regional symposium and training course in India; a seminar to introduce ferrocement in Malaysia; another seminar to introduce ferrocement to Africans; 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

The International Ferrocement Information Center (IFIC) has always been concerned with people, putting them in touch with each other, encouraging them to exchange resources and experiences, providing forum for the exchange and upgrading their knowledge through training courses, seminars, workshops and symposia. The International Symposium on Ferrocement, held every three years, particularly achieves these objectives. During the Fourth International Symposium on Ferrocement in Cuba last October many new developments in material properties, applications and techniques of construction were presented. To instill confidence to many users, we reprint selected papers in this issue and in subsequent issues of the *Journal of Ferrocement.*

The birth of the International Ferrocement Society (IFS) in Havana, Cuba last 25 October 1991, was specially rewarding for the staff of IFIC and the members of the Steering Committee under the leadership of Dr.Ricardo P. Pama. They worked for the formation of the IFS to give the ferrocement industry an international identity. The IFS will unify all users of ferrocement and provide opportunities for collaboration among members. IFS will provide leadership, direction and organization for research and development on ferrocement.

Users of ferrocement worldwide, you are invited to be a member of IFS. Join IFS, influence the direction of the society, play a vital role and make a difference.

Congratulations to the awardees during the Fourth International Symposium on Ferrocement. These pioneering individuals were recognized for their contribution to research, development and transfer of ferrocement technology. All IFIC staff are proud of the award for IFIC, this is a recognition that IFIC have attained its objectives effectively.

The Editor

Aspects Concerning the Behavior of Ferrocement in Flexure+

Train Onet* , C. Magureanu and V. Vescan****

The ferrocement elements of plate and beam have a good behavior under working load due to the fact that the width of cracks appears to be very small than in the reinforced concrete. The good behavior at failure regarding the aspect of ductility and ultimate moment of the elements shows the capability of using ferrocement efficiently. The present paper presents some specific aspects concerning the behavior of ferrocement in a short time bending.

INTRODUCTION

The uniform distribution of the reinforcement of a relatively small diameter in the composite material, which is ferrocement, offers special performances, a mechanical behavior and distinct potential applications as compared with the conventional reinforced concrete. Consequently it has been classified and researched as a special material, for which there have been elaborated a code or design guide [1].

Such concerns began in Romania too, in 1989, within a program for promoting the use of this material in constructions [2].

The results obtained in this program concerning the physical and mechanical properties of the ferrocement constitute the subject of an independent paper.

TESTS ON FERROCEMENT PLATES

The experimental members with the dimensions from Fig. 1 have been reinforced with 2,4 and 6 layers of woven hexagonal meshes, uniformly distributed across the thickness, respectively with 3, 4 and 5 layers of welded square meshes. On both types of meshes, in each layer, two superimposed meshes were placed. The achieved volumetric percentages (p_v) are indicated in Table 1. The microconcrete grade was Bc 25.

The plates have been equiped for measuring the concrete strains, the deflections and the opening of the cracks. The plates were tested according to the scheme from Fig. 1 by a step loading of 1/10 from calculated ultimate load.

The Cracking of the Plates

Table 1 presents the cracking state of the plates reinforced with hexagonal meshes, defined by: the loading step P/P_r (P_r - representing the ultimate load) at the appearance of the cracks (α_f 0.05) respectively, at their medium opening of 0.1 mm (α_f0.1), the loading step corresponding to the working

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load (P_e / P_r = 1/1.3 = 0.769), the maximum measured crack width (α_f^e max) and calculated one according to [3] (α_f^e max), corresponding to the working load and the maximum number of cracks.

The following were noticed:

- at reduced reinforcing percentages (p_v = 0.212%) the appearance of the cracks coincides with the failure of the elements while, at high reinforcing percentages (p_v = 0.636%) the appearance of the cracks took place at a loading step, of about 0.525;

-the number of the cracks increases with the volumetric reinforcing percentage;

- the maximum crack width, experimentally measured, increases with the volumetric reinforcing percentage while, the calculated one decreases;

- the value of the crack width calculated according to [3] is conservative only at reduced percentages of reinforcements. But it is true that the calculation relation from the mentioned guide has been concluded for the square meshes.

Concerning the reinforced plates with square meshes, the appearance of the cracks in the experiments has coincided, in all cases, with the failure of the units. Therefore, such elements were considered as having unsatisfactory behavior.

Deformation of the Plates

The maximum values of deflections (f) measured at the appearance of cracks ($\sigma_f^{0.05}$) and, at an opening of 0.1 mm ($\alpha_f^{0.01}$) are presented in Table 1. The evolution of the measured deflections, at the plate reinforced with hexagonal meshes, is presented in Fig. 2. There may be seen that, although the flexibility of the elements is obvious, for a loading step corresponding to a α_f = 0.1 mm crack opening the measured deflection does not exceed the 1/200th of the span.

With the help of the deflection values, experimentally measured (f^e) there have been calculated the values of the modulus of rigidity at short term bending using the relation:

$$K_S = \frac{23}{216} \frac{Ml^2}{f^e} \qquad \text{........ (1)}$$

and these values are presented in Fig. 3, in accordance with the loading step (M/M_r^e) as well as the volumetric reinforcing percentage, (p_v).

It was noticed that the gradual decrease of the stiffness is simultaneous with the increase of loading, especially after the cracks appeared.

The tests led to the conclusion that at loading steps below 60% from the failure load the modulus of rigidites may be evaluated by the relation:

$$K_S = E_b \left(1 - \lambda \right) I_b \qquad \text{........ (2)}$$

without considering the reinforcement.

In the above, relation, E and I , are the modulus of elasticity and the modulus of inertia of the concrete section respectively and is the plasticity factor, whose value is determined experimentally.

Failure of Plates

The values of the (M_r^e) ultimate bending moments experimentally measured as well as the (M_r^c) calculated according to the American guide [3], are presented in Table 2. Table 2 also presents the ultimate strain in the compressed concrete (ε_{bu}) and in the extreme extended reinforcement (ε_a).

Table 1

Type of mesh	p_V	$\alpha_f^{0.05}$ P/P_r	$\alpha_f^{0.05}$ f	$\alpha_f^{0.1}$ P/P_r	$\alpha_f^{0.1}$ ε_b^f	$\alpha_f^{0.1}$ ε_a^f	$\alpha_f^{0.1}$ f	$\alpha^{\varepsilon}_{fmax}$	α^{c}_{fmax}	$\frac{P_e}{P_r}=\frac{1}{3}$	Total number of cracks
	%	--	mm		%	%	mm	mm	mm		
1	2	3	4	5	6	7	8	9	10	11	12
HEXA				1.00	0.210	0.251	1/738				1
GONAL				1.00	0.200	0.249	1/685				1
	0.212			1.00	0.205	0.263	1/685	0.100	0.127	0.768	1
				1.00	0.203	0.275	1/640				1
		*		1.00	0.204	0.260	1/685	0.100			
				1.00	0.975	1.330	1/369	0.180			2
				0.95	0.895	1.317	1/320	0.180	0.080	0.768	2
	0.424			0.681	0.950	1.285	1/480	0.150			3
				0.762	0.960	1.270	1/480	0.260			3
		*		0.848	0.945	1.300	1/400	0.193			
		0.513	1/564	0.640	2.590	5.770	1/400	0.150			3
		0.486	1/505	0.794	2.660	5.975	1/240	0.200	0.067	0.768	5
	0.634	0.545	1/331	0.727	2.600	5.900	1/140	0.200			4
		0.555	1/615	0.611	2.615	6.111	1/369	0.350			4
		* 0.525	1/486	0.693	2.616	5.939	1/287	0.225			

* Mean Values

Table 2

No.	p_V	M_r^e	M_r^c	$\frac{M_r^e}{M_r^c}$	$\varepsilon_{b,u}$	ε_a
	%	N . mm	N . mm		%	%
1	0.212	194.960	107944	1.806	0.200	0.240
2	0.424	250.400	238066.5	1.052	2.700	6.600
3	0.636	380.800	337205.8	1.129	3.650	15.00

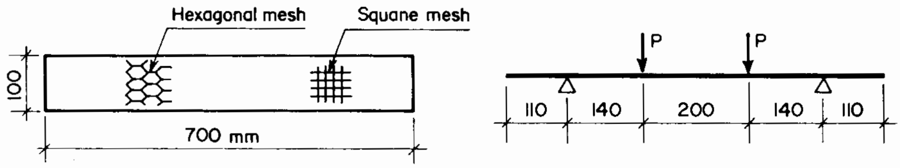


Fig. 1. Dimension and loading of experimental members.

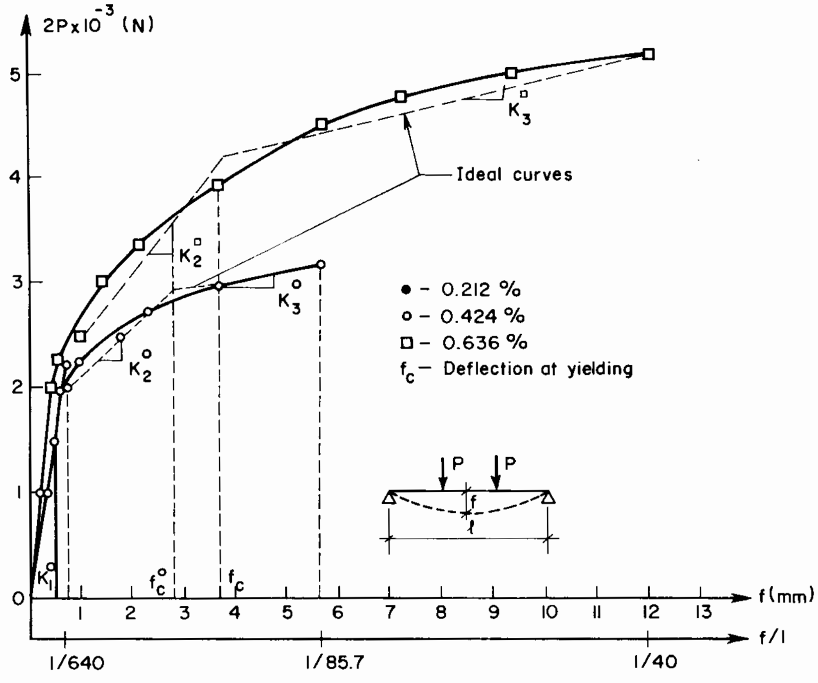


Fig. 2. Deflection of the plate reinforced with hexagonal meshes.

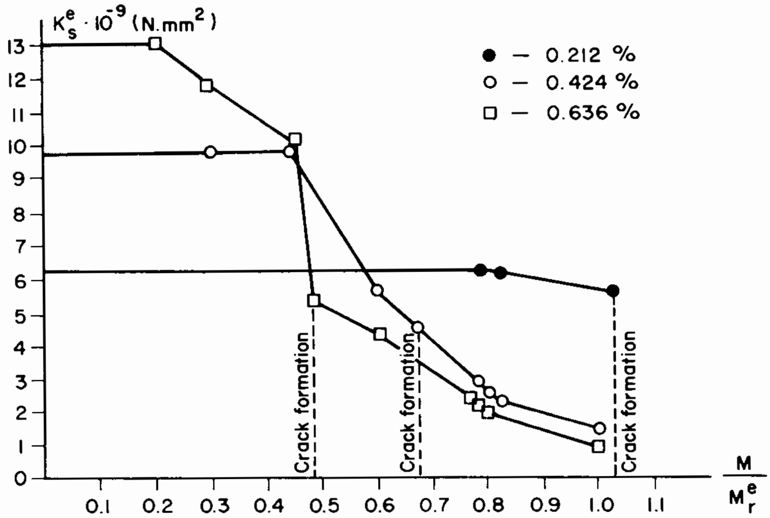


Fig. 3. Modulus of rigidity at short term bending.

There is a good correspondence between the calculated and measured values of the bending moments. Exceptions are the elements with low reinforcing percentage (p_v =0.212%), which, in fact, have factured without preventing, at the same time with the appearence of the first cracks.

TEST ON FERROCEMENT BEAMS

The tested beams, six in number, had the dimensions of 100 mm x 250 mm x 3200 mm and the reinforcement of hexagonal meshes, as it is seen in Fig. 4.

The letter *F* from the symbol of the beam indicates that, in preparing the microconcrete the FLUBET admixture has been used. The presumed and performed microconcrete grade was Bc 25.

The beams were simply supported and were tested under two concentrated loads applied on the third of the span. The measuring device used was identical with that of the ferrocement plates.

The stages of behavior noticed while testing the beams coincide with those described by Paul B.K. and Pama R.P.[4].

Cracking of the Beams

The values of the cracking moments, experimentally established (M_f^e) are presented in Table 3 comparatively with the calculated values (M_f^c) admitting the calculation diagrams in Fig. 5. As an experimental value for the cracking moment, the value corresponding to an α_f=0.05 mm crack width has been admitted.

From the data presented in the table it results that at reinforcing percentages, P_v < 0.746%, the best correspondence is obtained admitting a trapezoidal diagram of strains in the tensile concrete and a factor of plasticity λ =0.7. At higher values of the reinforcing percentage, the triangular diagram leads to the most satisfactory results.The cracking state of the beams, after testing, is presented in Fig.6.

Table 4 gives the maximum and medium values of the crack width (α_f^e max, α_f^e med) measured experimentally at different load steps, the maximum number of cracks, the medium crack spacing and the maximum values (α_f^c max) calculated according to the American guide [3], It should be mentioned that although this relation of calculation was deduced for elements reinforced with square meshes, while the experimental elements are reinforced with hexagonal meshes, yet it leads to satisfactory results.

It may be noticed that the cracking took place at reduced load steps (0.34 ... 0.46), that the number of crackings has rapidly increased with the load step but their width grew slowly as reported from the researches performed by Jiang E.E. [5].

The cracking process is established around a load step of M/M_f= 0.6 further increasing the crack width.

Deformation of Beams

The evolution of the deflections according to the load step is similar with that from Fig. 2. The flexibility of the elements is very high, reason for which the American guide [3], recomends to take into account in design, other criteria than the deformations limitaiton. However, at load steps below M/M_f=0.6 the maximum values of the deflections represent 1/366 - 1/255 of the span of the beams.

The modulus of rigidity at short time bending, calculated with the Eq. (1) increases with the volumetric reinforcing percentage (p_v) and decreases with the load step (M/M_r). The drastic decrease of the stiffness after the cracking of the elements, is obvious as shown in Fig. 3.

The numerical tests led to the conclusion that, in the uncracked stage the modulus of rigidity in

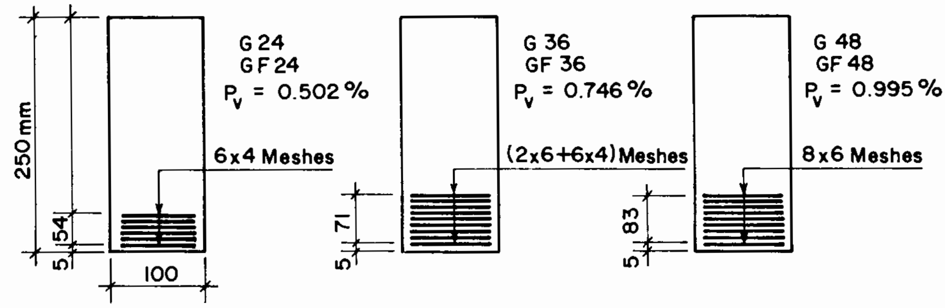


Fig. 4. Cross section of beams.

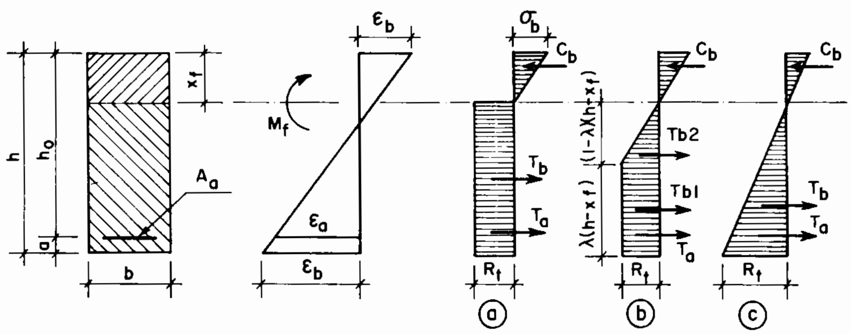


Fig. 5. Stress diagram.

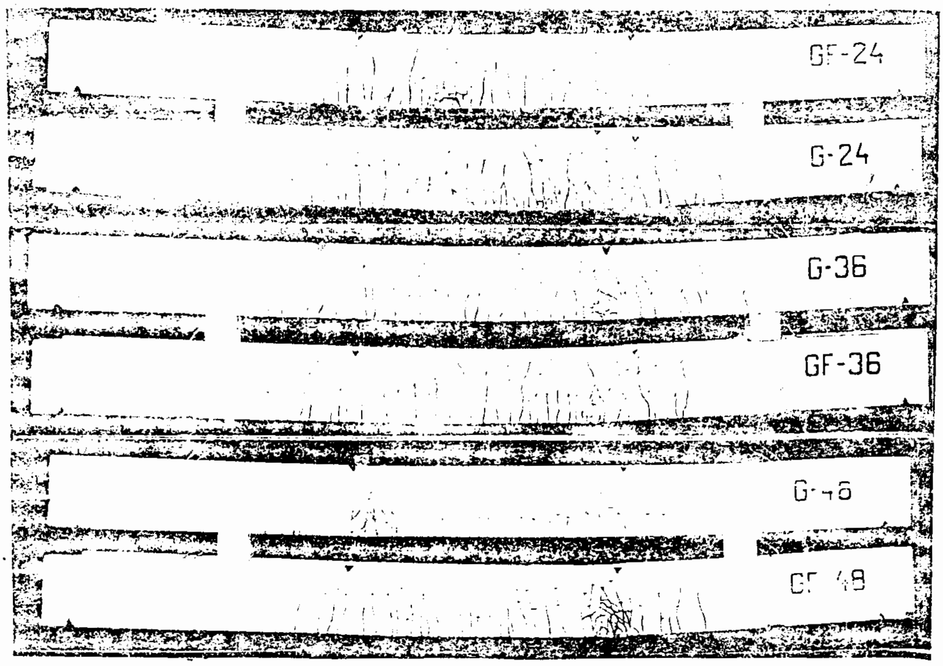


Fig. 6. Cracking state of the beam.

Table 3

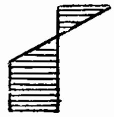
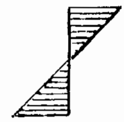
p_v	$M_f^c \times 10^{-6}$						
		$M_f^c \times 10^{-6}$	$\frac{M_f^e}{M_f^c}$	$M_f^c \times 10^{-6}$	$\frac{M_f^e}{M_f^c}$	$M_f^c \times 10^{-6}$	$\frac{M_f^e}{M_f^c}$
%	N mm	N mm		N mm		N mm	
0.502	4.100	0.740	5.540	4.015	1.021	6.148	0.667
0.746	5. 000	0.820	6.097	3.903	1.281	4.174	1.198
0.995	6.000	0.782	7.672	3.808	1.575	4.795	1.251

Table 4

p_v	Load Step	$\frac{M}{M_r}$	$\alpha^{\varepsilon}_{f\,max}$	$\alpha^{e}_{f\,med}$	Number of Cracks	λ_f^{med}	$\alpha^{c}_{f\,max}$
%			mm	mm		mm	mm
0.502	0.340	$\frac{M_{0.05}}{M_r}$	0.05	0.030	5	283.3	0.250
	0.476	$\frac{M_{0.10}}{M_r}$	0.10	0.061	14	95.4	
	0.714	$\frac{M_{0.20}}{M_r}$	0.20	0.137	27	57.8	
0.746	0.452	$\frac{M_{0.05}}{M_r}$	0.05	0.030	5	223.3	0.249
	0.638	$\frac{M_{0.12}}{M_r}$	0.12	0.053	30	468.0	
	0.851	$\frac{M_{0.20}}{M_r}$	0.20	0.105	39	465.0	
0.995	0.460	$\frac{M_{0.05}}{M_r}$	0.05	0.020	10	356.0	0.248
	0.520	$\frac{M_{0.10}}{M_r}$	0.10	0.300	20	83.1	
	0.689	$\frac{M_{0.17}}{M_r}$	0.17	0.065	30	51.7	
	0.996	$\frac{M_{0.20}}{M_r}$	0.20	0.088	32	50.9	

short time bending may be estimated with the Eq. (2). After the cracks appeared the modulus of rigidity at bending may be using the calculus relation estimated from the Romanian code [6] for the reinforced concrete elements.

Failure of the Beams

The values of the bending moments at failure measured experimentally (M_r^e) are given in Table 5 in accordance with the volumetric reinforcing percentage (p_v). In the same table, for comparison are presented values of bending moments at failure, calculated (M_r^c):

- after the procedure proposed by Mansur and Paramasivan [7] for elements of ferrocement;
- according to the procedure in the Romanian code of design [6] for reinforced concrete elements, that supposes a rectangular stress block in compression concrete and a stress in the reinforcements distributed on the tension zone proportional with the distance to the neutral axis.

It is obvious that the two calculation procedures lead to close results in between and in a satisfactory concordance with the experimental data.

Table 5

p_v	$M_r^e \times 10^{-6}$	$M_r^c \times 10^{-6}$ [7]	$M_r^c \times 10^{-6}$ [6]	Col 2 / Col 3	Col 2 / Col 4	$\varepsilon_{b,u}$
%	N mm	N mm	N mm			%
1	2	3	4	5	6	7
0.502	10.50	7.583	7.510	1.384	1.398	4.05
0.746	11.75	10.614	10.55	1.107	1.113	3.95
0.995	14.25	13.437	13.41	1.060	1.063	3.05

CONCLUSIONS

The achieved tests have led to the following conclusions:

a. The ferrocement elements of plate and beam types tested have a good behavior under working load due to the fact that the width of cracks appears to be very small than in the reinforced concrete. Consequently, the impermeability, stiffness and durability of the ferrocement elements is much improved.
b. The good behavior at failure regarding the aspect of ductility and ultimate moment of the elements shows the capability of using efficiently the ferrocement in the country.

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Deflection Evaluation of Ferrocement Plates in Bending+

A.W. Ballarin* and J.B. de Hanai**

This paper presents some considerations about deflection evaluation of ferrocement plates in bending, when using large opening welded wire meshes. Flexion tests were made on forty specimens, with thickness of 15 mm to 35 mm and steel ratio of 100 kg/m³ to 250 kg/m³ of mortar. Square meshes (50 mm x 50 mm) and rectangular meshes (25 mm x 50 mm) with wire diameter of 2.5 m (f_{yk} = 600 MPa) were used.

Experimental plate deflections were compared with several theoretical formulations, mainly related to conventional reinforced concrete and ferrocement standards.

INTRODUCTION

Industrialized ferrocement applications in Brazil have been accomplished with an extensive use of large opening welded wire meshes [1,2,3]. Due to this, ferrocement general formulations have not adequately responded to all of the mechanical property prediction needs [3]. Therefore, to provide a more rational basis for design, an extensive series of tests has been made to evaluate ferrocement mechanical properties when using several mesh types with larger openings than the usual ones.

This paper presents a part of this work, mainly related to large opening welded wire meshes, discussing the deflection evaluation of ferrocement plates in bending, by means of theoretical and experimental result comparisons.

EXPERIMENTAL PROGRAM

Ferrocement flexural specimens - here called plates - were 1000 mm long and 210 mm wide. The plates had a variable thickness, according to the reinforcement arrangement for each series.

Square openings (50 mm x 50 mm) and rectangular openings (25 mm x 50 mm) welded wire mesh reinforcement were used. The wires were 2.5 mm diameter and the nominal yielding tensile strength was 600 MPa.

The mortar consisted of ordinary portland cement, conforming to NBR 5732 [4], river sand and water, with a 0.40 water-cement ratio and a 2.0 sand-cement ratio, by weight.

Forty specimens gathered in five groups with distinct reinforcement arrangement were tested (Fig. 1). In each group two series - square and rectangular openings reinforcement - were performed with four specimens in each series (Table 1).

Flexion tests were made in a specially designed testing frame. A two point loading system with 900 mm span was used, and so a 300 mm constant moment zone was obtained (Fig. 2). Plates were loaded until failure.

Deflection measurement was made with Mitutoyo mechanical gauges with 0.01 mm sensibility and 50 mm range. Deflection higher than 50 mm was evaluated with standard scale.

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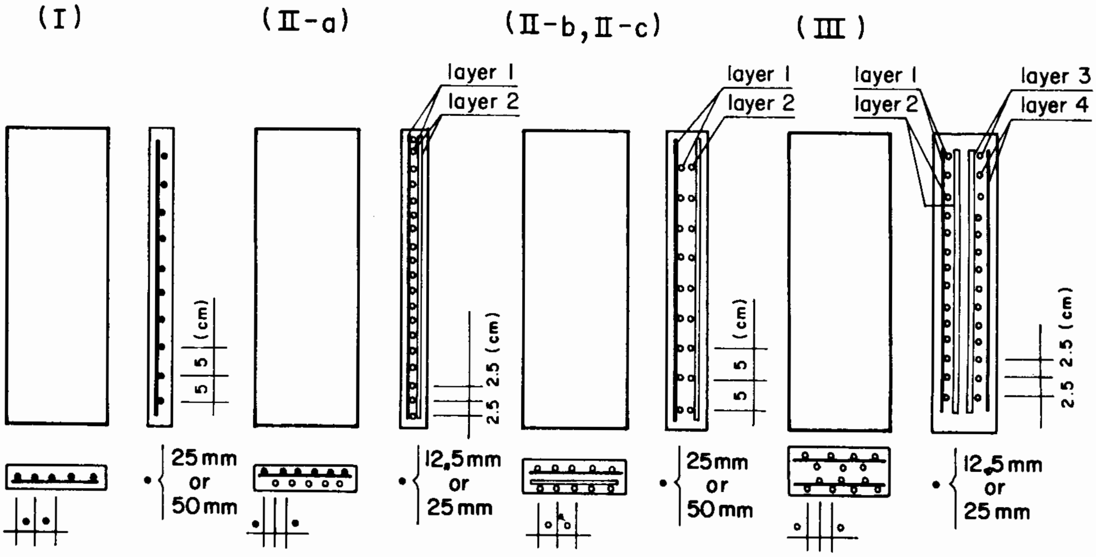

Fig. 1. Reinforcement arrangements

Experimental mortar modulus of elasticity was evaluated in cylindrical samples of 50 mm mm length using electric strain gages.

Experimental plate deflections were compared with theoretical formulation proposed by strength of materials theory. In this considerations, several theoretical values of I (moment of inertia) were used according to the referred test stage ($P < P_r$ or $P > P_r$, where P_r is the cracking load).

Likewise, several theoretical and experimental values of E (modulus of elasticity) were adopted.

TEST RESULTS AND ANALYSIS

Load-deflection experimental diagrams obtained from tests are showed in Fig. 3. Each diagram results from four samples on average.

Table 1 Characteristics of the Specimens

Group	Mesh opening	Thickness (mm)	Series	Characteristics			Steel ratio
				(%)	A_{sl} (mm^{-1})	A_{sl1} (mm^{-1})	
I	Square	15	FIQ15	0.78	0.0249	0.0124	109
	Rectangular		FIR15	1.40	0.0336	0.0224	158
II-a	Square	17.5	FIIQ17.5	1.20	0.0334	0.0192	177
	Rectangular		FIIR17.5	2.27	0.0545	0.0363	260
II-b	Square	25	FIIQ25	0.93	0.0299	0.0149	131
	Rectangular		FIIR25	1.68	0.0404	0.0269	190
II-c	Square	30	FIIQ30	0.78	0.0249	0.0124	109
	Rectangular		FIIR30	1.40	0.0336	0.0224	158
III	Square	35	FIIIQ35	1.20	0.0384	0.0192	177
	Rectangular		FIIIR35	2.27	0.0545	0.0363	260

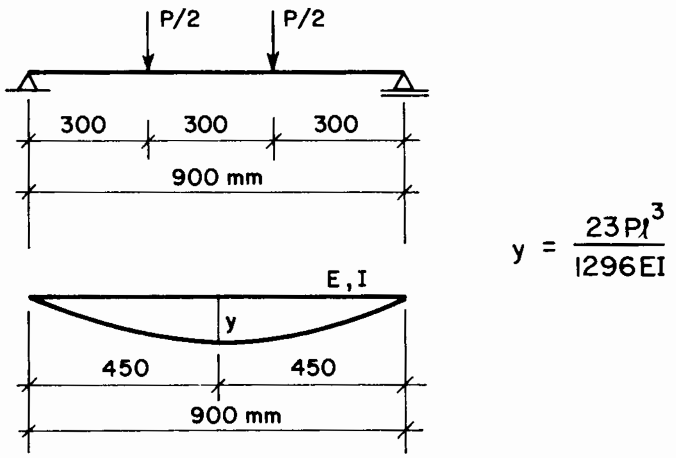

Fig. 2. Two point loading flexion test.

Theoretical central deflection for two point loading flexion test (Fig. 2) can be calculated by:

$$y = \frac{23pl^3}{1296EI} \qquad \text{........(1)}$$

where,

P = acting load;
l = plate span;
E = modulus of elasticity;
I = moment of inertia.

According to the test stage, theoretical deflections may be calculated with:

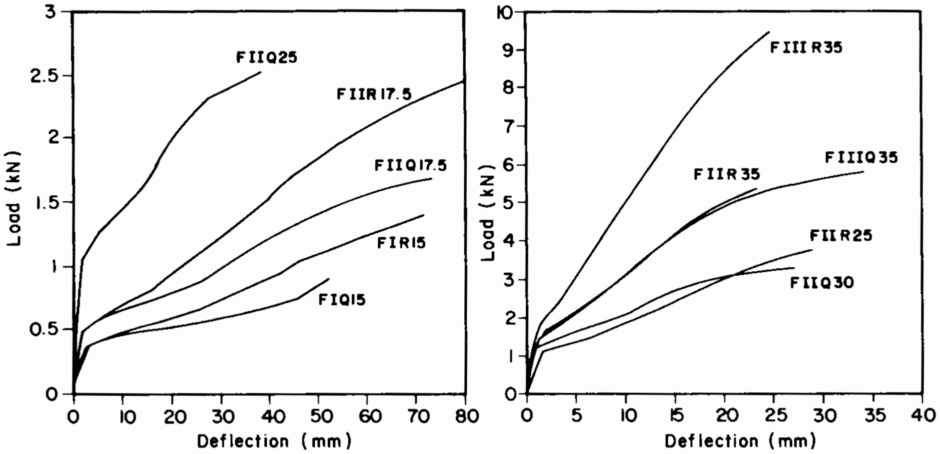

Fig. 3. Load-deflection experimental diagrams.

$I = I_1$ = moment of inertia referred to non-cracked section, when $P < P_r$

$I = I_2$ = moment of inertia of cracked section, when $P > P_r$

$I = I_2$ = moment of inertia of cracked section, when $P \geqslant P_r$

Furthermore, when $P \geqslant P_r$, Branson's [5,6] equation can be utilized in order to take into account the variable rigidity of the plates :

$$I = I_3 = \left(\frac{M_r}{M}\right)^3 . I_1 + \left[1 - \left(\frac{M_r}{M}\right)^3\right] . I_2 \qquad \text{........ (1)}$$

where,

I_1 = moment of inertia, non-cracked section;

I_2 = moment of inertia, cracked section;

M = acting moment;

M_r = cracking moment.

Brazilian Ferrocement Codes [7] have suggested the theoretical formulation of equation (2) for deflection evaluation.

The modulus of elasticity to be adopted in Eq.1 should be selected from five distinct values, as showed:

$E_1 = E$ = tangent experimental modulus of elasticity (obtained from 50 mm x 100 mm cylindrical samples) ;

$E_2 = 0.85\ E$ = secant experimental modulus of elasticity (+) ;

$E_3 = 6600 . \sqrt{f_{cd}}$, conforming to *Brazilian Concrete Codes* [8], where f_{cd} is the design strength of concrete;

$E_4 = 0.80\ (6600 . \sqrt{f_{cd}})$ (++) ;

$E_5 = 0.85 \ . 0.80\ (6600\ . \sqrt{f_{cd}})$.

Table No. 2 shows experimental/theoretical deflection relationships. Experimental deflection values were obtained from Fig. 3. Theoretical deflection values were obtained from Eq. 1 with defined values of moment of inertia (according to test stage) and modulus of elasticity.

From Table 2 one can say that experimental / theoretical relationships with $I = I_3$ (Eq. 2) did not show good results. Studying these data and others not exposed here, it was noted that Eq. 2 gives good values of moment of inertia only to test stages surrounding the cracking load ($P_r \leqslant P < 1.8\ P_r$). In the majority of these tests $P_u = 4.0\ P_r$.

Table 2 shows that E_1 and E_4 (tangent modulus of elasticity) when combined to I_1 in Eq. 1 give a good deflection prevision in non-cracked test stages (experimental/theoretical relationships of 1.05).

Likewise, it can be seen that good results are achieved when using E_2 and E_5 combined to I_2, in cracked test stages (experimental/theoretical relationships of 1.00 and 1.01).

Modulus of elasticity E_3 - obtained from *Brazilian Concrete Codes* [8] - in Eq. 1 suggested lower theoretical deflection values.

(+) The factor 0.85 utilized to obtain secant modulus of elasticity, is an assumption of the *Brazilian Ferrocement Codes* [7].
(++) The factor 0.80 is a suggestion of *Brazilian Ferrocement Codes* [7] to obtain ferrocement modulus of elasticity from concrete modulus of elasticity.

Table 2 Experimental / Theoretical Deflection Relationships

Series	E_1			E_2			E_3			E_4			E_5		
	I_1	I_2	I_3	I_1	I_2	I_3	I_1	I_2	I_3	I_1	I_2	I_3	I_1	I_2	I_3
FIQ15	1.03	1.05	2.70	0.87	0.89	2.30	1.30	1.33	3.42	1.04	1.06	2.73	0.88	0.90	2.32
FIR15	0.87	1.24	1.59	0.74	1.05	1.35	1.07	1.53	1.96	0.86	1.22	1.57	0.73	1.04	1.33
FIIQ17.5	0.36	1.17	1.79	0.73	0.99	1.52	1.25	1.69	2.60	1.00	1.35	2.08	0.85	1.15	1.76
FIIR17.5	0.91	1.57	1.76	0.77	1.33	1.50	1.17	2.02	2.27	0.94	1.61	1.82	0.80	1.37	1.54
FIIQ25	1.07	1.10	2.15	0.91	0.94	1.03	1.31	1.36	2.64	1.05	1.00	2.11	0.89	0.92	1.80
FIIR25	1.12	1.04	1.31	0.95	0.88	1.11	1.38	1.28	1.61	1.11	1.03	1.29	0.94	0.87	1.10
FIIQ30	1.11	1.01	1.79	0.94	0.86	1.52	1.30	1.10	2.10	1.04	0.95	1.68	0.88	0.80	1.43
FIIR30	0.96	0.95	1.12	0.81	0.81	0.96	1.12	1.19	1.41	0.96	0.95	1.12	0.81	0.81	0.95
FIIIQ35	1.59	1.43	1.80	1.35	1.22	1.53	1.87	1.69	2.12	1.49	1.35	1.69	1.27	1.14	1.44
FIIIR35	0.95	1.22	1.30	0.81	1.04	1.11	1.28	1.64	1.75	1.02	1.31	1.40	0.87	1.12	1.99
$\bar{x}$	1.05	1.18	1.73	0.89	1.00	1.47	1.31	1.49	2.19	1.05	1.19	1.75	0.89	1.01	1.49
s	0.21	0.20	0.46	0.18	0.17	0.39	0.22	0.40	0.59	0.17	0.21	0.47	0.15	0.18	0.40

CONCLUSIONS

From this test program, for ferrocement plates with large opening welded wire meshes, the results are:

a. Theoretical formulation proposed by strength of materials theory can be applied on deflection evaluation of ferrocement plates in bending;
b. In deflection theoretical formulation, plate rigidity can be utilized as the product of:
 - tangent modulus of elasticity (either experimental or theoretical value) and non-cracked moment of inertia, for non- cracked behavior stage analysis;
 - secant modulus of elasticity (either experimental or theoretical value) and cracked moment of inertia, for cracked behavior stage analysis;
c. Ferrocement theoretical modulus of elasticity can be obtained from concrete's theory using a reduction factor - in this work, the adopted factor was 0.80;
d. Branson's moment of inertia give good results in plate defection analysis only at the stages surrounding the cracking load.

ACKNOWLEDGEMENT

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Mathematical Mode of Determination of Critical Cracking Force at Tension Zone of Ferrocement

R. Walkus* and R. Gackowski*

Generally, it has been established that during setting and hardening time of concrete the microcracks in the reinforced concrete-matrix composites have been formed, as a result of contact stresses on the contact surfaces "binder-aggregate" and as shrinkage effect and technological treatment. Cracks of this type are known in the literature as "structural microcracks". Exposing specimen of ferrocement to the action of axial tension it has been established that the structural microcracks have their direction being perpendicular to the tensile force and broadening themselves with the increase of the applied load.

EFFECT OF AXIAL TENSION ON FERROCEMENT BEHAVIOR

In the sixties of the20th century the problem of deformability of reinforced concrete has emerged as a result of many experimental tests. Some of the researchers that have been dealing with this problem have different opinions in the matter of ferrocement extension at the moment of cracking. Ciakreli G.D. [2] maintained that at the moment of cracking the cracks of the order $\varepsilon_t = 0.7 + 1.2\%$ exist; Lysenko E.F. [3] assumed that the first crack appears with the longitudinal strain of the order $\varepsilon = 0.1 + 0.2\ \%$ saying that the size of the cracks does not differ from the cracks in concrete.

At a certain stage of ferrocement specimen operation the cracks called "operational microcracks", [5] are developed.

In Fig. 1 the ideographic process of ferrocement tension, reinforced concrete and concrete [4] are shown, and also shown is the successive growing and development of the formed "operational microcracks". One may observe that the prolonged phase of the microcracks in which the cooperation of microreinforcement with the matrix is noted, constitutes the specific feature of the ferrocement. Stresses and deformation of material in this phase are dependent upon microcracks development.

Behavior of ferrocement being axially stretched has been shown in Fig. 2[4].

ANALYSIS OF CRACKING AS A FUNCTION OF LOAD

Ferrocement specimens of the strip type have been analysed with axial tension. Distribution of the internal forces and stresses existing in every node of the net have been shown in Figs. 3 and 4[1]. The aim of this work is to determine the critical tensile force which is causing the first microcrack. The authors have assumed that at the point of contact "matrix microreinforcement" the adherence stresses, τ_p have been formed as a result of axial tensile of the ferrocement specimen. This results in a kind of cooperation between microreinforcement and the matrix.

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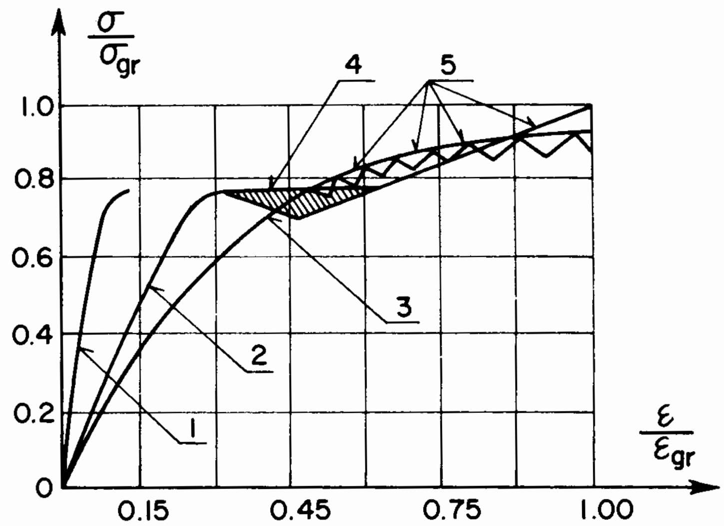

Fig.1. Ideographic process of tension.

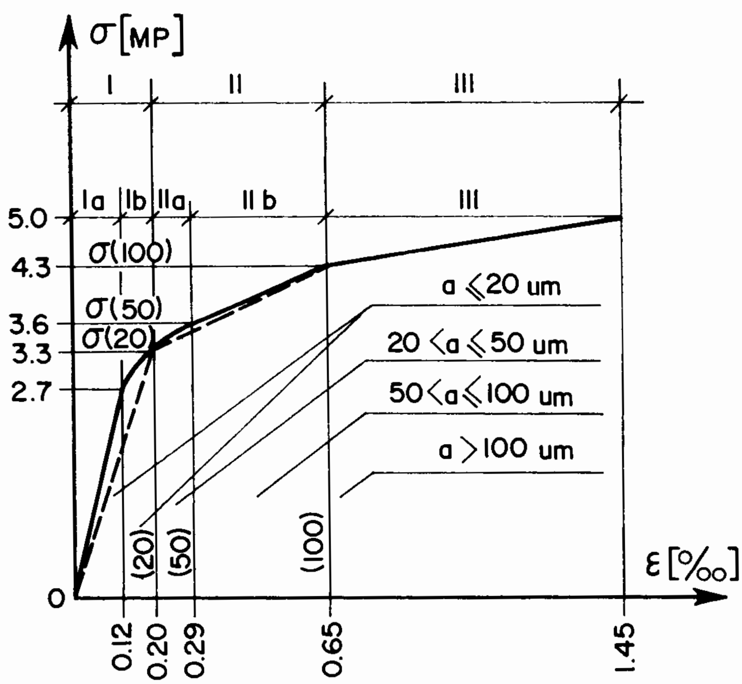

Fig.2. Behavior of ferrocement in axial tension.

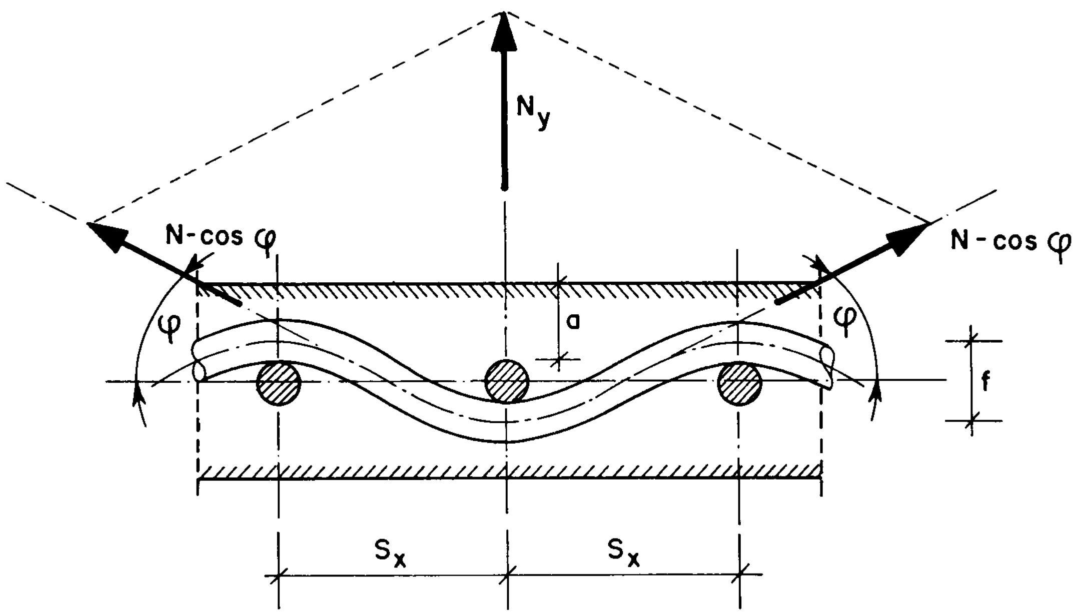

Fig.3. Distribution of internal forces in axial tension.

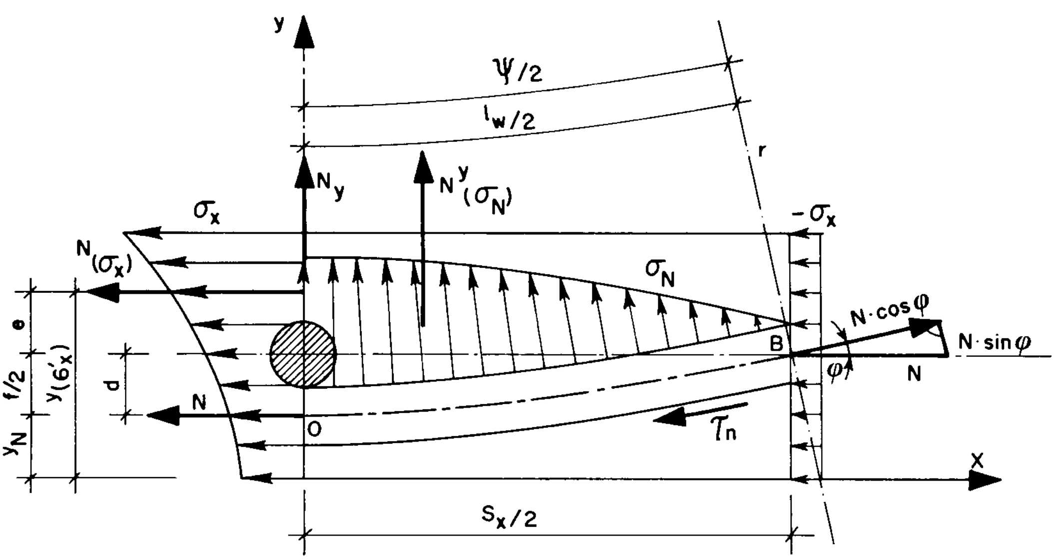

Fig.4. Stresses existing in every node of the net.

As the tensile force increases, the stresses τ_p approaches the critical value. At this value, loss of bond between the microreinforcement and matrix occurs. The longitudinal wires of the net are then stretched, resulting in their gradual straightening (Fig.3) in the direction of the tensile force. Straightening effects the cross-wire of the net on the casing, generating an increase of the compressive stresses on the contact surface between the cross-reinforcement and the casing. It should be noted that in the lateral direction, the net wires have been waved more and more and thus the self-tensionning phenomenon in this direction is taking place. After exceeding the concrete strength on the local compression, cracking of the specimen occurred. Value of the applied external load, at the moment of crack, may be treated as the critical force, N_{kr}. The magnitude of N_{kr} depends on the thickness of casing and concrete strength on compression at the contact points.

The unit elongation of the wire resulting from its complete straightening may be determined approximately from the geometrical relationship (Fig. 3):

$$\varepsilon=\sqrt{1+\left(\frac{f_w}{S_x}\right)^2}-1 \qquad \text{........ (1)}$$

where:

f_w = is deflection of the axis of longitudinal net wire,
S_x = is the size of the net mesh (assuming the square dimension of net ($S_x = S_y$)

To ascertain what is the effect of the angle φ, contained between longitudinal wire of the net and direction of the force operation, on the value of N_{kr} - the adherence analysis of the fiber group to the matrix has been carried out.

Assuming that P_n is the force, which is causing the separation of the fibers from the matrix (loss of bond), the nominal bond should be calculated as the quotient between component of the pulling force P_n on the fiber axis, and contact surface of the fiber with the matrix:

$$\tau_n=\frac{P_n \cos\phi}{n_w \pi d\, l_w} \qquad \text{........ (2)}$$

under assumption of the uniform force distribution on all fibers (longitudinal wires of the net). where

d = diameter of the longitudinal wire of net,
l_w = the length of the longitudinal wire of net,
n_w = number of wires in the cross-section of the specimen.

Length of the longitudinal wire can be calculated according to the formula:

$$l_w = \int_{x_B}^{x_A} \sqrt{1+\left(\frac{dy}{dx}\right)^2}\, dx \qquad \text{........ (3)}$$

As the longitudinal wires of the net have the parabolic shape (Fig.5) of the equation:

$$y = \frac{4d}{S_x^2} x^2 \qquad \text{........(4)}$$

thus

$$l_w = \int_{-\frac{S_x}{2}}^{\frac{S_x}{2}} \sqrt{1 + \left[\frac{d}{dx}\left(\frac{4d}{S_x} x^2\right)\right]^2}\, dx = \int_{-\frac{S_x}{2}}^{\frac{S_x}{2}} \sqrt{1 + \left(\frac{8d}{S_x}\right)^2 x^2}\, dx$$

Substituting

$$\delta^2 = \frac{64 \cdot d^2}{S_x^2} \qquad \text{........(5)}$$

we can obtain

$$l_w = \int_{-\frac{S_x}{2}}^{\frac{S_x}{2}} \sqrt{1 + \varepsilon^2 x^2}\, dx = \delta \int_{-\frac{S_x}{2}}^{\frac{S_x}{2}} \sqrt{\frac{1}{\delta^2} + x^2}\, dx$$

$$l_w = \delta \left[\frac{1}{2} \cdot x \cdot \sqrt{x^2 + \frac{1}{\delta^2}} + \frac{1}{2\delta^2} \cdot \ln\left| x + \sqrt{x^2 + \frac{1}{\delta^2}} \right| \right]_{-\frac{S_x}{2}}^{\frac{S_x}{2}}$$

$$l_w = \delta \frac{1}{2\delta^2} \cdot \ln \left| \frac{\frac{S_x}{2} + \sqrt{\frac{S_x^2}{4} + \frac{1}{\delta^2}}}{\frac{S_x}{2} + \sqrt{\frac{S_x^2}{4} + \frac{1}{\delta^2}}} \right|$$

After some transformations

$$l_w = \frac{S_x^2}{64\,d} \cdot \ln \frac{\sqrt{16\,d + S_x} + 2d}{\sqrt{16\,d + S_x} - 2d} \qquad \text{........(6)}$$

Taking into account the length of the net wire l_w, on which the force $N.\ N.\cos\phi$ is acting in the wire axis, all the variable have been made dependent upon coordinates along the axis of the wire.

It was assumed that skeleton of the net is undeformable in the direction being perpendicular to its axis. Poisson's ratio has been assumed equal to zero ($\nu = 0$). Between the matrix and the wire exist only the action of the tangential forces but after loosing the contact, there is no interaction. Also, linear relation between τ_d on the wire surface and displacement $u\ (x)$ of the wire in x-axis direction has been assumed:

$$\tau_d\ (x) = -k.\ u(x) \qquad \ldots\ldots\ldots (7)$$

where k = adherence (bond) parameter.

Loss of the adherence on the contact: wire - matrix is taking place after reaching the critical value of the wire displacement $u\ (x) = u_{gr}$.

Taking into account the equilibrium state of the wire section of the length dx (Fig. 6) it has been obtained

$$\sigma_n = \frac{N}{F} \qquad \ldots\ldots\ldots (8)$$

where F = cross-sectional area of the single wire of skeleton
The equation of equilibrium is as follows:

$$F\ .\ \sigma_n\ .\ \cos\phi\ - F\ .\ \sigma_n\ .\ \cos\phi\ - F\ \ (\partial\sigma_n/\partial x)\ dx\ -\ \tau_n . S\ .\ dx\ = 0 \qquad \ldots\ldots\ldots (9)$$

where S = circumference of the wire equal to πd,
d = diameter of the wire
Axial deformation $\frac{du}{dx}$ of the wire is equal to:

$$\frac{du(x)}{dx} = \frac{\sigma}{E_a} \qquad \ldots\ldots\ldots (10)$$

Making use of the Eqs. (7) and (9) one can obtain:

$$F.E_a.\frac{\partial^2 u\ (x)}{\partial x^2} dx + k\ u\ (x) S\ dx\ = 0 \qquad \ldots\ldots\ldots (11)$$

Introducing the parameter $\omega_0 = \sqrt{\frac{E_a F}{kS}}$

differential equation has been obtained:

$$\frac{\partial^2 U\ (x)}{\partial x^2} - \frac{U\ (x)}{\omega_0^2} = 0 \qquad \ldots\ldots\ldots (12)$$

Solution of the Eq. (12) is in the form:

$$u\ (x) = C_1 \cdot e^{\left(-\frac{x}{\omega_0}\right)} + C_2 \cdot e^{\left(-\frac{x}{\omega_0}\right)} \qquad \ldots\ldots\ldots (13)$$

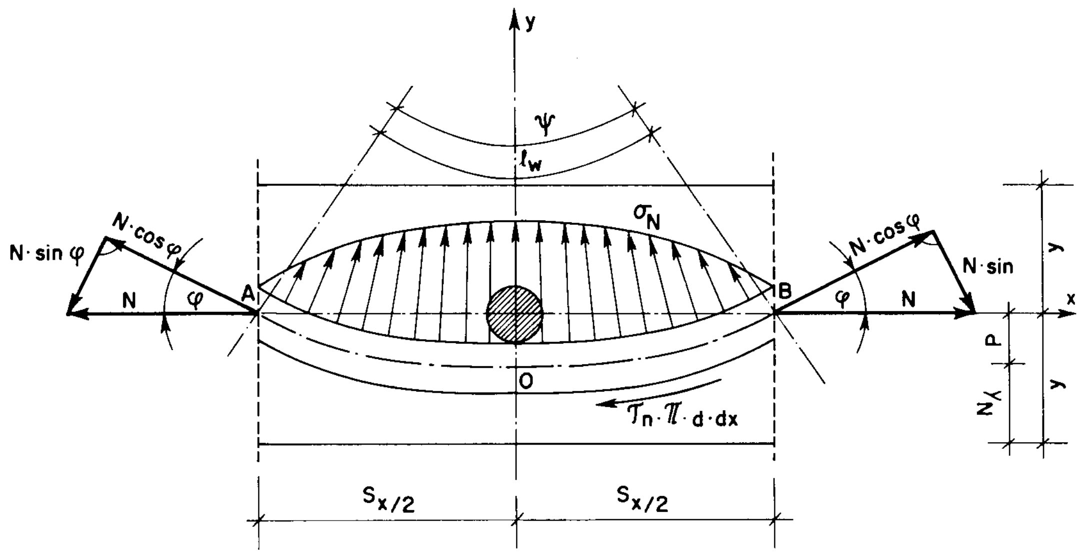

Fig.5. Stresses in the longitudinal wires of the net.

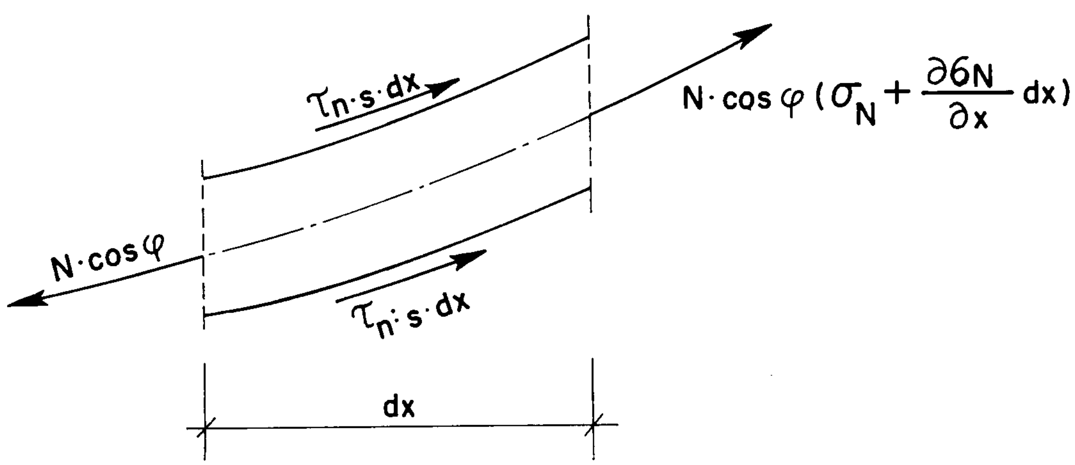

Fig.6. A wire section of length *dx* in a state of equilibrium.

Intergration constants have been derived after using the boundary conditions:

For $x = 0$ $\sigma(x) = \sigma_n$

For $x = l_w/2$ $\sigma(x) = 0$

After differentiation of the Eq. (13) one may obtain:

$$\frac{du(x)}{dx} = -\frac{1}{\omega_0} C_1 \cdot e^{\left(-\frac{x}{\omega_0}\right)} + \frac{1}{\omega_0} C_2 \cdot e^{\left(\frac{1}{\omega_0}\right)}$$

$$\sigma(x)\Big|_{x=0} = \sigma_n \longrightarrow E_a \cdot \frac{du(x)}{dx} = \sigma_n$$

$$E_a\left[-\frac{1}{\omega_0} C_1 \cdot e^{\left(-\frac{x}{\omega_0}\right)} + \frac{1}{\omega_0} C_2 \cdot e^{\left(\frac{x}{\omega_0}\right)}\right] = \sigma_n \text{ ;for } x = 0$$

$$-\frac{1}{\omega_0} C_1 + \frac{1}{\omega_0} C_2 = \frac{\sigma_n}{E_a} \longrightarrow -C_1 + C_2 = \frac{\sigma_n \cdot \omega_0}{E_a}$$

$$\sigma(x)\Big|_{x=\frac{l_w}{2}} = 0 \longrightarrow E_a\left[-\frac{1}{\omega_0} C_1 \cdot e^{\left(\frac{-l_w}{2\omega_0}\right)} + \frac{1}{\omega_0} C_2 \cdot e^{\left(\frac{l_w}{2\omega_0}\right)}\right] = 0$$

$$C_1 \cdot \frac{1}{e^{\left(\frac{l_w}{2\omega_0}\right)}} = C_2 \cdot e^{\left(\frac{l_w}{2\omega_0}\right)} \longrightarrow C_1 = C_2 \cdot e^{\left(\frac{l_w}{\omega_0}\right)}$$

$$-C_2 \cdot e^{\left(\frac{l_w}{\omega_0}\right)} + C_2 = \frac{\sigma_n \cdot \omega_0}{E_a}\left(\frac{l_w}{\omega_0}\right)$$

Finally,

$$C_1 = -\frac{\sigma_n \cdot \omega_0 - \sigma_n \cdot \omega_0\left[1 - e^{\left(\frac{l_w}{\omega_0}\right)}\right]}{E_a\left[1 - e^{\left(\frac{l_w}{\omega_0}\right)}\right]} = \frac{\sigma_n \cdot \omega_0\, e^{\left(\frac{l_w}{\omega_0}\right)}}{E_a\left[1 - e^{\left(\frac{l_w}{\omega_0}\right)}\right]}$$

$$C_2 = \frac{\sigma_n \cdot \omega_0}{E_a\left[1 - e^{\left(\frac{l_w}{\omega_0}\right)}\right]} \qquad \ldots\ldots\ldots(14)$$

The equation of displacement is in the form:

$$u(x) = \frac{\sigma_n \cdot \omega_0 e^{\left(\frac{l_w}{\omega_0}\right)} e^{\left(-\frac{x}{\omega_0}\right)}}{E_a\left[1 - e^{\left(\frac{l_w}{\omega_0}\right)}\right]} + \frac{\sigma_n \cdot \omega_0 e^{\left(\frac{x}{\omega_0}\right)}}{E_a\left[1 - e^{\left(\frac{l_w}{\omega_0}\right)}\right]} \qquad \ldots\ldots\ldots(15)$$

Stress distribution in the skeleton wires, with consideration of the Eq. (10), is the following

$$\sigma(x) = \sigma_n \cdot \omega_0 \left[\frac{e^{\left(\frac{l_w}{\omega_0} - \frac{x}{\omega_0}\right)}}{1 - e^{\left(\frac{l_w}{\omega_0}\right)}} + \frac{e^{\left(\frac{x}{\omega_0}\right)}}{1 - e^{\left(\frac{l_w}{\omega_0}\right)}}\right] \qquad \ldots\ldots\ldots(16)$$

Finally,

$$\tau_d(x) = \frac{\sigma_n k \cdot \omega_0}{E_a}\left[\frac{e^{\left(\frac{l_w}{\omega_0} - \frac{x}{\omega_0}\right)}}{1 - e^{\left(\frac{l_w}{\omega_0}\right)}} + \frac{e^{\left(\frac{x}{\omega_0}\right)}}{1 - e^{\left(\frac{l_w}{\omega_0}\right)}}\right] \qquad \ldots\ldots\ldots(17)$$

Eqs. (16) and (17) allow one to determine the stress distribution in the skeleton and the adherence stresses. Value of the coefficient k has been determined on the grounds of the experimental tests. The critical force, which is causing separation of skeleton and matrix, has been calculated accoridng to the equation:

$$N_{kv} = u_0 \pi \cdot \sqrt{\frac{\tau_d \cdot E_a \cdot d^3}{4 \cdot (-u_0)}} \left[\frac{1 + \exp\left(2 l_w \sqrt{\frac{4\,\tau_d(x)}{E_a \cdot d\,(-u_0)}}\right)}{1 - \exp\left(2 l_w \frac{4\,\tau_d(x)}{E_a \cdot d\,(-u_0)}\right)}\right]$$

where,

$$U_0 = U_x \Big|_{x=0} \longrightarrow U_0 = \frac{\sigma_n \cdot \omega_0}{E_a} \cdot \frac{\left[1 + e^{\left(\frac{l_w}{\omega_0}\right)}\right]}{1 - e^{\left(\frac{l_w}{\omega_0}\right)}}$$

$$\tau_d = \tau_d(x)$$

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Durability of Polymer-Ferrocement +

Y. Ohama* and A. Shirai**

This paper discusses the durability of polymer-ferrocement in comparison with conventional ferrocement. The polymer-ferrocement, using a styrene-butadiene rubber latex, are prepared with various polymer-cement-ratios, and tested for accelerated carbonation, chloride ion penetration and accelerated corrosion. It is concluded from the test results that the carbonation resistance, chloride ion penetration resistance and corrosion-inhibiting property of the polymer-ferrocement are remarkably improved with increase in polymer-cement ratio.

INTRODUCTION

Ferrocement is popularly used in many applications such as boats, marine structures, roofings and water tanks. For some special applications requiring superior properties, it is not advisable to use the ferrocement because of its limited durability.

In this paper, polymer-ferrocement using a styrene-butadiene rubber latex were prepared with various polymer-cement ratios, and tested for accelerated carbonation, chloride ion penetration and accelerated corrosion. From the test results, the carbonation resistance, chloride ion penetration resistance and corrosion-inhibiting property of the polymer-ferrocements which are important requirements for their practical applications are discussed.

MATERIALS

Cement and Fine Aggregate

Ordinary portland cement and river sand with a size of 1.2 mm or finer were used for all the mixes.

Polymer Dispersion

A styrene-butadiene rubber (SBR) latex was used as a polymer dispersion. Its basic properties are shown in Table 1.

Table 1 Properties of Polymer Dispersion.

Type of polymer dispersion	Specific gravity (20° C)	pH (20° C)	Viscosity (20° C, cP)	Toatal solids (%)
SBR	1.020	9.7	64	45.0

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Reinforcements

The reinforcements used were the combinations of the welded wire fabrics formed from wires of 2.6 mm in diameter and the crimped wire cloths formed from wires of 1.5 mm in diameter, specified in JIS A 3551 (*Welded Steel Wire Fabric*) and JIS A 3553 (*Crimped Wire Cloth*), respectively. The reinforcements consist of one layer of the welded wire fabrics and two layers of the crimped wire cloths, and the details of the reinforcements are illustrated in Fig. 1.

TESTING PROCEDURES

Preparation of Specimens

Polymer-modified mortars were prepared with the mix proportions given in Table 2 according to JIS A 1171 (*Method of Making Test Sample of Polymer-Modified Mortar in the Laboratory*). Ferrocement specimens 20 mm x 250 mm x 250 mm were molded, and then subjected to a 2-day-20° C-80% R.H.-moist, 5-day-20° C-water plus 21-day-20° C -50% R.H.-dry cure.

Acceleratecd Carbonation Test

Specimens were place in a non-pressurizing carbonation test chamber for 7, 14, 21, 28, 56, 91, 182 and 365 days, in which temperature, humidity and CO_2 gas concentration were controlled to be 30° C, 60% R.H. and 5.0% respectively. After accelerated carbonation, the specimens were split at the desired immersion periods, and the split cross-sections were sprayed with 1 % phenolphthalein alcoholic solution. The depth of the rim of each cross- section without color change was measured by using slide calipers as a carbonation depth as shown in Fig. 2.

Chloride Ion Penetration Test

Specimens were immersed in 2.5 % NaCl Solution at 20° C for 7, 14, 21, 28, 56, 91, 182 and 365 days for chloride ion penetration. After immersion, the specimens were split at the desired immersion periods, and the split cross-sections were sprayed with 0.1 % sodium fluorescein and 0.1 N silver

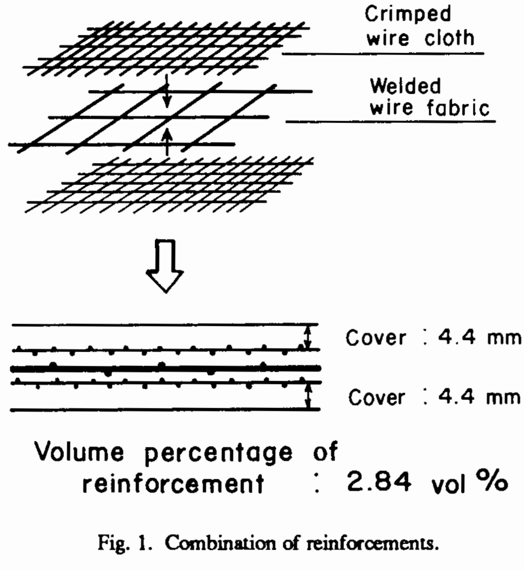

Fig. 1. Combination of reinforcements.

nitrate solutions as prescribed UNI 7928 (*Concrete-Determination of the Ion Chloride Penetration*). The depth of the rim of each cross-section changed to white color was measured by using slide calipers as chloride ion penetration depth as shown in Fig. 2.

Accelerated Corrosion Test

Specimens were tested for accelerated corrosion in two steps including pretreatment and corrosion test.

Table 2. Mix Proportions of Polymer Modified Mortars

Cement : sand (By weight)	Polymer-cement ratio (%)	Water-cement ratio (%)	Flow
1 : 3	0	59.5	166
	10	38.5	170
	20	32.0	167

Pretreatment

For carbonation, the specimens were placed in a sealed vessel, evacuated to 1 mm Hg or less at ambient temperature for 1 hour, and then exposed to pressurized CO_2 gas under a pressure of 10kgf/cm^2 (0.981 MPa) for 72 hours. The carbonated specimens were dried in an oven at 30^0 C for 168 hours. After drying, the specimens were evacuated to 1 mmHg or less for 1 hour, and then impregnated with 2.5 % NaCl solution under a pressure of 10 kgf/cm^2 (0.981 MPa) for 3 hours in the vessel for chloride ion penetration.

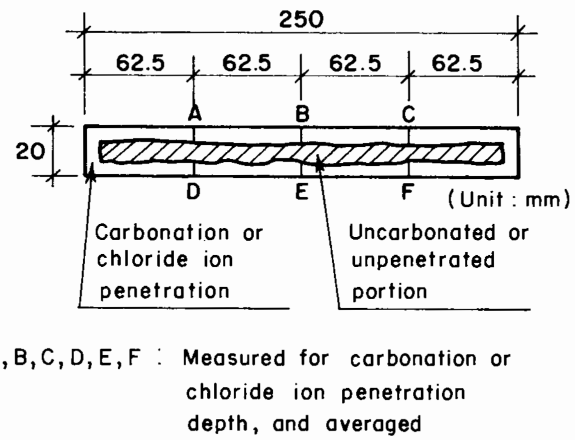

Fig. 2. Cross-section of specimen after accelerated carbonation or chloride ion penetration test.

Corrosion Test

After pretreatment, the specimens were subjected to accelerated corrosion of 1,3 and 5 cycles by the following method: Heating of the specimens wrapped in polyethylene sheets at 80 °C for 24 hours; Heating of the unwrapped specimens at 30° C for 24 hours Immersion of the unwrapped specimens in 2.5 % NaCl solution at 20° C for 24 hours. After accelerated corrosion test, the specimens were split longitudinally, and the embedded reinforcements were removed. The corroded area of the surfaces of the reinforcements was measured as a percentage of the number of corroded intersection points in the reinforcements to the number of all intersection points in the reinforcements.

TEST RESULTS AND DISCUSSION

Fig. 3 shows the relation between the exposed period and carbonation depth of the polymer-ferrocements exposed to air with a CO_2 gas concentratioin of 5.0% at 30° C and 60 % R.H. for 1 year. The carbonation depth of the polymer-ferrocement increases with additional exposure period. Particularly, the carbonation depth of the ferrocement with a polymer-cement ratio of 0 % increases sharply, and attains to 10 mm at an exposure period of 63 days, but that of the polymer-ferrocement with a polymer-cement ratio of 10% attains to 10 mm at an exposure period of 270 days. In addition, the carbonation depth of the polymer-ferrocement with a poly-cement ratio of 20 % is about 2.6 mm at an exposure period of 1 year. This can be explained by an excellent effect of inhibiting CO_2 gas diffusion in the polymer-ferrocement because of the filling of pores with the polymer. The carbonation depth of the polymer-ferrocement is affected to a great extent by factors such as exposure period,

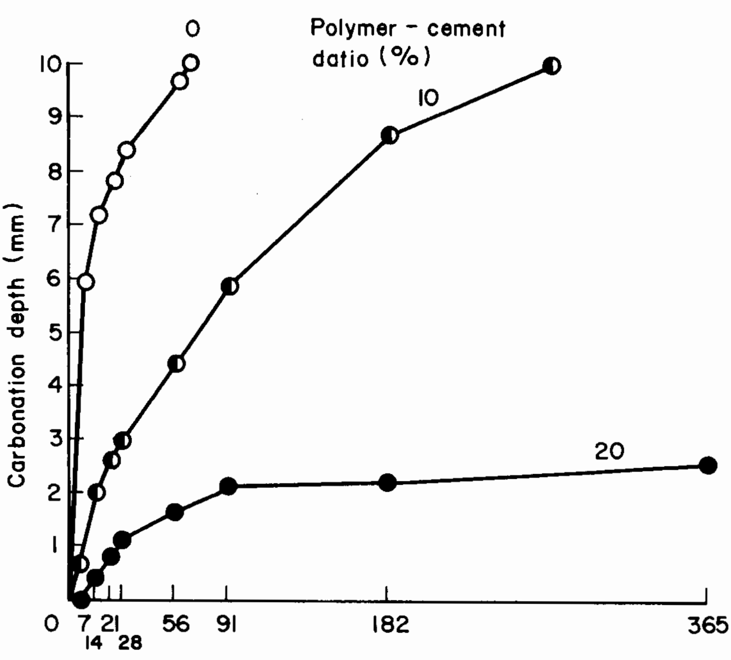

Fig. 3. Exposure period vs. carbonation depth of polymer-ferrocement exposed to air with CO_2 gas concentration of 5.0% at 30° C and 60 % R. H.

polymer-cement ratio and water-cement ratio. The carbonation depth of the polymer-ferrocement can generally be expressed as a function of these factors by the following equation:

$$DCO_2 = 15.1(1-P/C)(W/C)T^{1/4} - 4.38$$

where DCO_2 is the carbonation depth of the polymer-ferrocements, T is the exposure period, P/C is the polymer-cement ratio, and W/C is the water-cement ratio. This relationship empirically obtained is shown in Fig.4. Consequently, the prediction of the carbonation depth polymer- ferrocements is found to be possible by applying the above empirical equation.

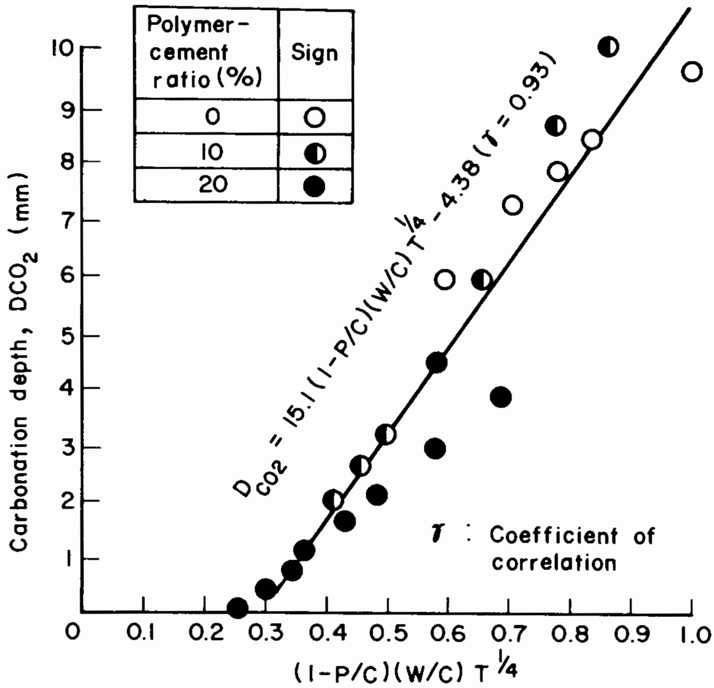

Fig. 4. Prediction of carbonation depth of polymer-ferrocement.

Fig. 5 illustrates the relation between the immersion period and chloride ion penetration depth of the polymer-ferrocement immersed in 2.5 % NaCl solution for one year. The chloride ion penetration depth of the polymer-ferrocement increases with an increase in the immersion period. In particular, the chloride ion penetration depth of the ferrocement with a polymer-cement ratio of 0 % increases markedly, and attains to 10 mm at an immersion period of 14 days. On the other hand, the chloride ion penetration depth of the polymer-ferrocement with polymer-cement ratios of 10% and 20% attains to 10 mm at immersion periods of 36 and 70 days, respectively. However, the corrosion of reinforcements in the polymer-ferrocement is not recognized at an immersion period of one year. It is considered to intercept the chloride ion penetration into the surfaces of the reinforcements because the surfaces of the reinforcements are covered with polymer films. The chloride ion penetration depth of the polymer-ferrocement is affected to a great extent by factors such as the immersion period,

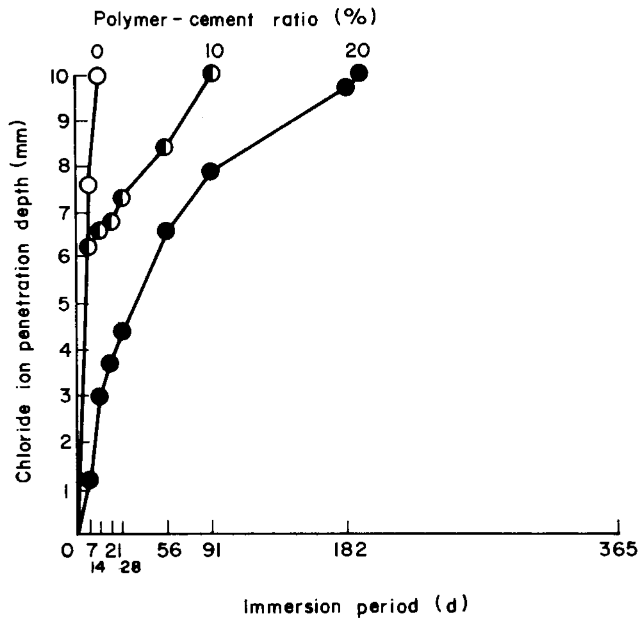

Fig. 5. Immersion period vs. chloride ion penetration, depth of polymer-ferrocement immersed in 2.5% NaCl solution.

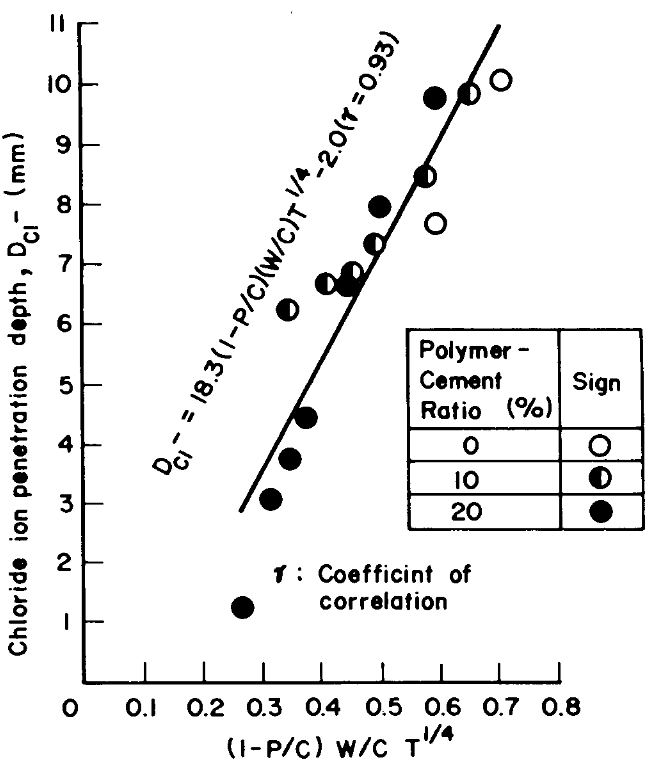

Fig. 6. Prediction of chloride ion penetration depth of polymer-ferrocement.

polymer-cement ratio and water-cement ratio. The chloride ion penetration depth of the polymer-ferrocement can generally be expressed as a function of these factors by the following equation :

$$DCI = 18.3\,(1 - P/C)\,(W/C)\,T^{1/4} - 2.0$$

Where DCI is the chloride ion penetration depth of the polymer-ferrocement and T is the immersion period. This relationship emperically obtained is shown in Fig. 6. Accordingly, the prediction of the carbonation depth of the polymer-ferrocement is found to be possible by applying the above emperical equation.

Fig. 7 indicates the relation between the corrosion rate of reinforcements and the number of wetting and drying cycles of the polymer-ferrocements subjected to accelerated corrosion. The surfaces of reinforcement in the ferrocement with a polymer-cement ratio of 0% were already corroded after pretreatment, and the corrosion rate of the ferrocement increases wih increasing number of wetting and drying cycles. The surfaces of the reinforcements in the polymer-ferrocement with a polymer-cement ratio of 10% are first corroded at a number of wetting and drying cycles of 3. The surfaces of the reinforcements in the polymer-ferrocement with a polymer-cement ratio of 20% are not corroded at a number of wetting and drying cycles of 5, and polymer-ferrocement has an excellent corrosion-inhibiting property.

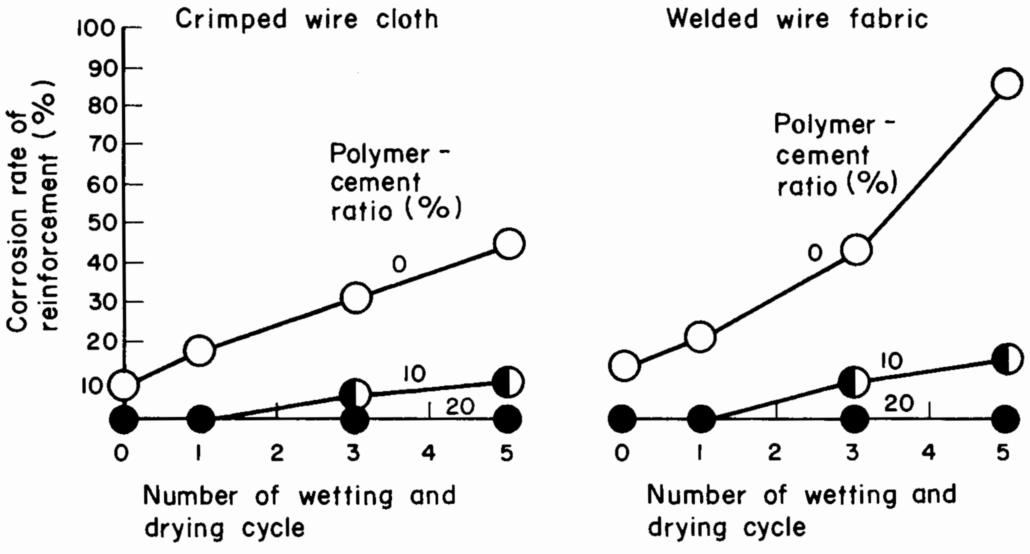

Fig. 7. Corrosion rate of reinforcement vs. number of wetting and drying cycles of polymer-ferrocement subjected to acclerated corrosion.

CONCLUSIONS

The conclusions obtained from the test results can be summarized as follows:

1. The carbonation and chloride ion penetration depths of polymer ferrocement decrease markedly with an increase in polymer- cement ratio regardless of exposure and immersion periods.
2. The carbonation and chloride ion penetration depths of polymer- ferrocements are strongly affected

by polymer-cement ratio and water-cement ratio. The carbonation and chloride io penetrtion depths of the polymer-ferrocements can generally be expressed as a function of these factors by the following equation:

$$D = a\ (1\text{-}P/C)\,(W/C)\,T^{1/4}\ - b$$

where D (mm) is the carbonation or chloride ion penetration depth of the polymer-ferrocements, and T (weeks) is exposure or immersion period, and a and b are empirical constants.

3. The corrosion-inhibiting property of polymer-ferrocement is remarkably improved with an increase in polymer-cement ratio.

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Ferrocement Prefabricated Housing: The Next Generation+

A.E. Naaman* and H. Hammoud**

Ferrocement is a construction material ideally suitable for high levels of prefabrication. However, in order for ferrocement prefabricated products to successfully penetrate the housing sector, it should be demonstrated that high quality housing can be produced with ferrocement, in as an effective way as for low cost housing. This paper describes part of an ongoing investigation which attempts to address this concern. In a first study, published earlier, advanced manufacturing techniques were considered for the production of ferrocement housing units; this led to the development of a ferrocement housing system using standardized prefabricated U and box-shaped panels for the walls, floors, and roof of a typical house. It was assumed, in all cases, that joining of the panels can be properly achieved with bolted type connections. After a brief review of existing ferrocement housing systems (which address primarily the low-cost housing sector), this paper provides a brief summary of the progress achieved so far on the study of bolted connections. Two types of connection are identified, a shear-type and a moment-type connection. Test arrangement and test set-up designed to identify various failure modes are described. Typical load-deformation response curves are presented. It is hoped that experimental results will provide the basis for calibrating analytical models of the connections. Such models can be implemented in a computer program in order to investigate a large number of parameters, and eventually develop optimum connection configurations.

INTRODUCTION

Ferrocement has been successfully used in marine applications, with a wide range of material characteristics, qualities, and finishes. However, there is a common perception that terrestrial applications such as in housing, are generally low cost, thus of a lower quality. One of the main advantages of ferrocement is that it can be constructed with a very wide range of qualities, properties, and cost, according to customers demand and budget. While most ferrocement housing applications have been so far directed toward low cost housing solutions, this does not imply that good quality housing products cannot be achieved with ferrocement. In fact some of the first applications of ferrocement used by Nervi were to replace intricate ceiling decorations usually made with gypsum lath. Indeed ferrocement can, should, and eventually will address the high quality housing sector. It is a construction material that lends itself to easy manufacturing and transportation. What is needed above all is: 1) to change the perception of architects, engineers, building authorities, and users about ferrocement, and 2) to bring the level of technology in ferrocement construction to the level of progress achieved in other industries such as the manufacturing, automobile and aerospace industries.

Today, an extraordinary confluence of new technologies and a large market for housing products worldwide can bring a revolution in the way ferrocement is used. Advanced technologies can help

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expand the applications of ferrocement and greatly improve its subjective acceptance as a high quality, high technology, luxurious, durable and cost competitive construction material.

Ferrocement is ready for new technologies. Most of its properties have been documented, and guidelines for its analysis and design have been developed [1,10,13]. While ferrocement housing components can be built using advanced manufacturing techniques, there is need to develop entire housing packages where the ferrocement structural sub-system is integrated harmoniously as part of the whole housing system and occupies a well balanced portion of it. Current advances in robotics, computerized manufacturing, machine vision, expert systems, and the like allow us to project that such advanced technologies which are already in use in the auto industry can be successfully utilized in the production of manufactured housing systems where ferrocement is the primary structural material.

OBJECTIVES AND SCOPE

It has been one of the objective of the research on ferrocement at the University of Michigan to address advanced production technologies for ferrocement, while taking advantage of the characteristics offered by ferrocement such as strength, lightweight, fire resistance, durability, ease of transportation and erection, ductility for seismic zones, and adaptability to high levels prefabrication.

In two previous studies by Naaman [9,11], ferrocement was considered and evaluated in prefabricated housing systems. In Ref. (9), ferrocement panels were considered the primary material for flooring and roofing, to reduce the total weight of the prefabricated modular box systems investigated. Ref. [11] describes a feasibility study where advanced manufacturing techniques were considered for the production of single family housing units using prefabricated ferrocement panels. One of the constraints considered was, that the housing system so produced, should be of equal if not better quality than standard single family housing units currently found on the U.S. market. The study suggested that most common housing requirements could be satisfied from a pool of about fifteen standard panel configurations. Although the ferrocement sub-system can occupy a wide range of structural and protective functions within the housing unit, it was shown that the same group of panels could be used for the skin (outside bearing wall), the floors, and the roof of the house. In all cases the connections between the various elements were assumed satisfied by bolting. The main features of the system are schematically described in Figs. 1 to 3.

The above study [11] showed that the ferrocement system developed is a technically feasible system suitable for a highly industrialized production facility at a competitive cost. It also indicated that the most pressing problem is that of the connections between the various ferrocement elements. The properties of the connections between ferrocement elements is the least documented in the technical literature and should receive high priority in future research. Connections, allowing the use of bolts to join ferrocement elements produced with high precision surfaces, can save substantial time and money, if proven structurally acceptable.

After a brief review of various ferrocement housing systems of interest as a background to the current investigation, a progress report on the status of the study of connections is given.

REVIEW - BACKGROUND ON FERROCEMENT HOUSING SYSTEMS

Numerous conceptual and developmental studies have been undertaken on prefabricated housing systems made from ferrocement [3-7,9,11,15-24]. Such systems can be as simple as a wall panel proposed for use in many non-specialized situations [7,22,23], or, as complete as the entire structural system of a house (walls, roof, floors, and foundation) [5,15,16,19,23]. However, to the best of the author's knowledge, some systems were only conceptual and never used in practice, while others led to one or a small number of prototype units. In some systems, the term ferrocement is used generically and may imply thin reinforced concrete members. In other words, the reinforcing parameters do not satisfy the minimum recommendations of the *ACI Guide on Ferrocement* [1] and other guidelines [10,13], such as particularly for the minimum amount of reinforcement and its specific surface.

Following is a brief description of five ferrocement housing systems which are relevant to the present study. They are reviewed by chronological order.

In 1979, Castro [3] reported on the use of prefabricated ferrocement panels to build as many as 350 low cost housing units in Mexico. The panels were reinforced with two to three layers of chicken wire mesh sandwiching a grid of 6 mm reinforcing bars, placed 250 mm center to center. The panels were joined using nuts and bolts. The joints were observed to behave properly even in the seismic areas of Mexico. No further report on the long term behavior of these units is available.

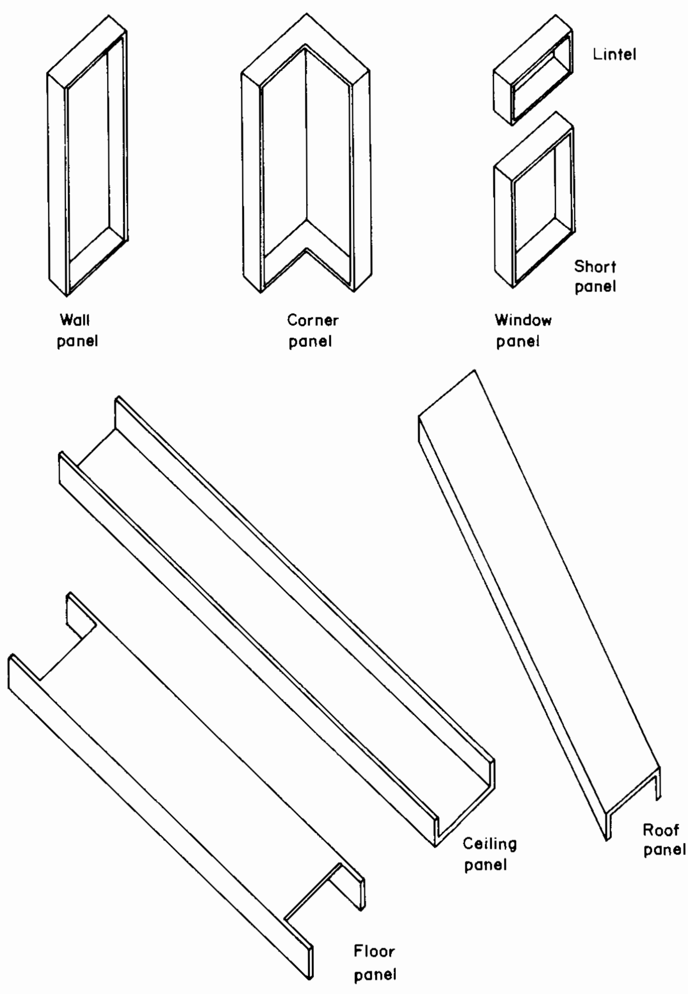


Fig. 1. Typical panels for prefabricated ferrocement housing units.

In 1981, Tatsa et al. [20] described a composite building system where ferrocement panels were joined together with cast-in-place concrete. The system consists of three basic precast ferrocement elements: 1) a horizontally spanning unit for one-way bending, 2) a two-way bending unit, and 3) a vertical shear panel unit. The beams and columns are cast in place. Continuity is assured by reinforced concrete poured in place between the ferrocement elements. The primary feature of the panels is that they incorporate a styrofoam insulation which is used as a permanent form. A system based on bolted connections instead of poured ones, was also developed later using the same concept. However, little can be found to date about the application of the system to real housing units.

Gokhale (1983) described the Castone housing system [5] developed in India. The system consists of wall panels attached to a floor slab. The panels are 3 ft x 9 ft (0.91m x 2.73 m) with a thickness of 1.25 in. (32 mm). The first floor slab (or ceiling of the ground floor) is a lattice girder hollow block

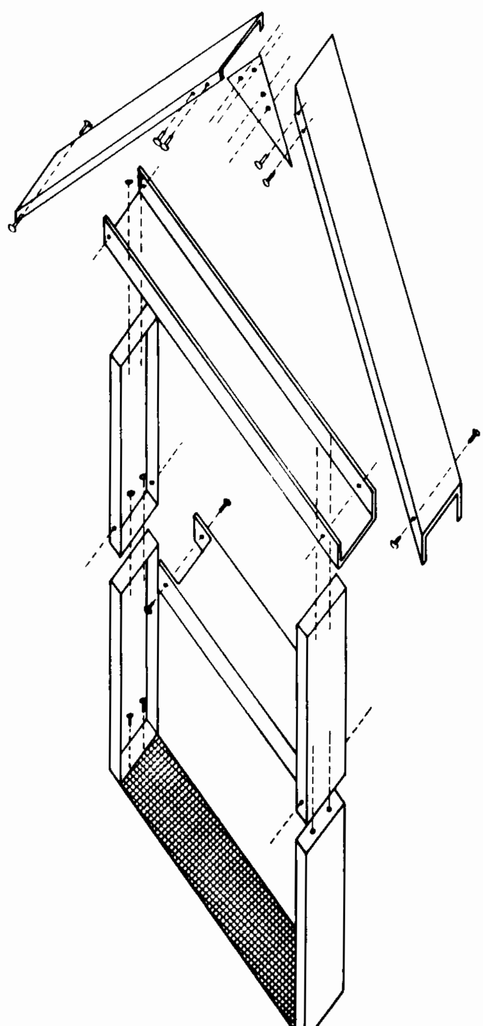


Fig. 2. Panel assembly in a typical house cross-section.

type, capable of anchoring to the top of the wall panels. The panels have ribs around their periphery with preformed holes to accept bolts. According to Gokhale, the system was used in several one or two-story houses in the Bombay area, however, we do not have any further information about the behavior of these ferrocement units during service and in the long term.

In 1985, at the Second International Symposium on Ferrocement, the F. Davis system was presented [21]. The system consists of prefabricated ferrocement U and box-shaped panels and number of other shapes that could be used for windows, water channels, and the like. For the outside walls, the ribs of the panels were positioned toward the outside. A couple of rooms (demonstration units) were built for a housing fair in France to illustrate the strength, appearance, and feasibility of the system. Very high quality panels were used. It is not known if the system was ever used in any small or large scale project.

Sandowicz (1985) described four housing systems utilizing three basic types of ferrocement channel elements; he named them the ELSA, CEE, ELWO, and Mixed Systems [19]. The ELSA panel system is used either for partitions or as a permanent formwork for cast in place concrete such as spandrel beams, colums, and floor bearing elements. The main intent of the ELSA system is to combine ferrocement with reinforced concrete to result in a monolithic structure. In the CEE system the whole house is made out of panels connected with screws and especially designed steel corner elements. There is no cast-in-place concrete and no need for foundations since the wall panels are dug directly into the ground. The system is recommended for one story houses. In the ELWO system, ELWO type ferrocement channels are joined with screws and separated by lath. The roof is covered with roof paper to allow construction of summer houses and bungalows. In the mixed system, the three previously described systems are used, namely: the framework is made of ELSA channels filled with concrete, the floor and roof are made of CEE channels, and the external walls are made of ELWO elements. It is not clear at the time of this writing, if the above systems were ever used in real applications.

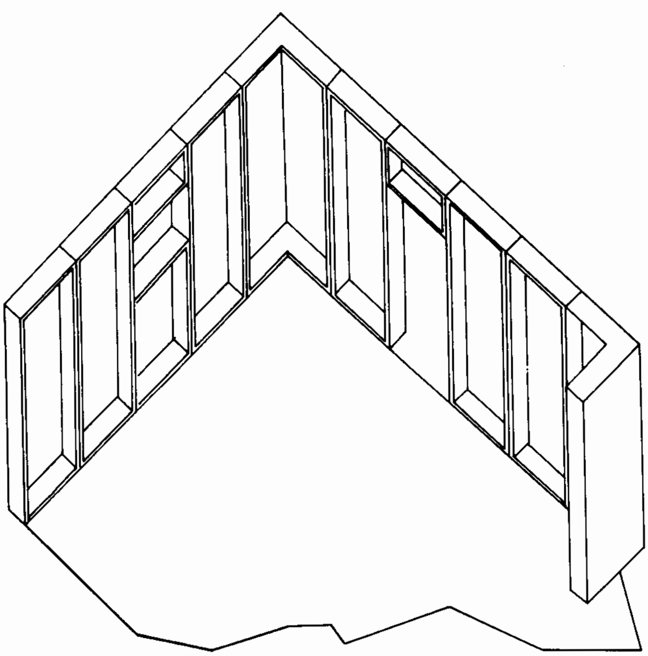


Fig. 3. Panel assembly for a typical wall and corner.

In 1988 [16] Rivas described the first ferrocement house built in Cuba in 1986. It was constructed of prefabricated ferrocement panels using hand woven wire meshes. Following the success of this first effort, Rivas reports that three factories producing ferrocement elements for housing have been set up in Cuba. Since then, numerous high quality one and two-story houses have been built using ferrocement panels, roofs, and mezzanines.

No ferrocement prefabricated system has ever been used extensively in applications other than very low cost housing, such as described by Castro in Mexico [3]. This may explain the reasons why the users and building authorities are reluctant to allow such ferrocement systems in the production of higher quality housing units. One of the main goals of the current investigation is to show that high quality, high levels of prefabrication, and competitive cost can be indeed achieved for ferrocement housing. The most pressing technical problem is that of the connections and it is being addressed.

CURRENT PROGRESS ON CONNECTIONS AT THE UNIVERISTY OF MICHIGAN

As mentioned earlier, the research at the Univeristy of Michigan has focused on developing a highly industrialized ferrocement panel system for housing, where connections are achieved by bolting. The system is described in Ref.[11] and typical details are shown in Figs. 1 to 3. While the analysis and design of the panels for bending, axial, and shear loadings has been already investigated, the primary focus, at the time of this writing, is on the connections between the prefabricated panels. Very little information can be found at present on ferrocement connections [14,25]; however, useful background on the design of joints and connections can be obtained from studies of steel joints [2,8].

In this study, ferrocement connections are classified into two main types, namely, a primarily "shear-type" connection, and a primarily "moment-type" connection. Of course a connection is generallly subjected to combined loads, but the above definition attempts to identify the load that is most critical to the connection. Typical loads transmitted by bolted connections are illustrated in Fig. 4.

A shear-type connection between two plates joined by a bolt is one where the applied external loads are parallel to the plane of the plates, and perpendicular to the axis of the connecting bolt. A typical shear-type connection is shown in Fig. 5. Note that the load is transferred by two mechanisms, friction and bearing. Understanding and quantifying the contribution of each mechanism is one of the essential goals of the current experimental investigation. Once friction is overcome under load, the bearing resistance is engaged. Five different failure modes (Fig.5) can then be observed 1) tensile failure of the critical net section of the plate, 2) shear failure or tear-out of the plate portion close to the edge of the plate, 3) crushing of the plate ahead of the hole, 4) cleavage or fracture of the plate between the hole and the plate edge, and 5) shear fracture of the bolt. This last mode of failure can always be avoided by using larger diameter and/or stronger bolts.

In the moment-type connection, the connection is subjected to a bending moment (with vector normal to the axis of the joining bolts) that induces axial forces in the bolts. Such loading occurs frequently in L-shaped joints where the bolted portion is called joined flange, while the free portion is called web. Most common failure modes of moment-type connections are: failure of the bolts due to axial loads, failure of the section of the joined material located in the corner between the flange and the web, and excessive surface separation between the two connected plates. Here also, the bolt size and pre-load can be designed in such a way that one of the failure modes can occur first.

An extensive exprimental program is being carried out on the above types of connections. Parameters include the number of mesh layers in the ferrocement plates, the distance of the bolt to the edge of the plates, and the type of mesh. Details of the experimental program and results will be given in a future publication. Here only typical preliminary results are provided.

The testing arrangement and set-up for a shear-type connection are shown in Fig.6. The corresponding load deformation curves are shown in Fig.7. It should be noted that the specimens tested in Fig.7 had the following characteristics: standard 1/2 in. (12.5 mm) diameter bolt, 1/2 in. (12.5 mm) thick ferrocement plates, with a distance of 2 in. (50 mm) from the center of the bolt to the edge of the plate. The reinforcement for Fig.7 top and Fig.7 bottom consisted of eight layers and four layers of square welded mesh respectively, with a wire spacing of 0.25 in. (6 mm) and a wire diameter of 0.025 in. (0.62 mm). A cleavage type failure (Fig.5) was observed at a maximum load of 2693 lbs (1220 kg or 11806 N) and 1318 lbs (599 kg or 5796 N) for the 8 layer and 4 layer specimens respectively. Other types of failure have been observed by varying the reinforcement and the end

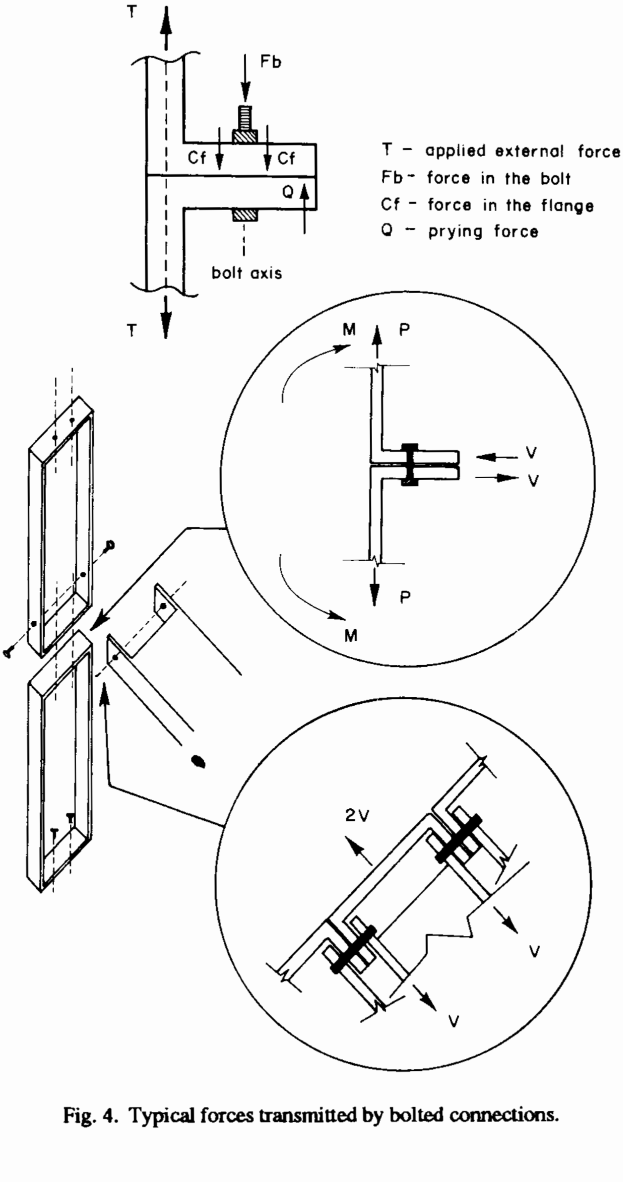


Fig. 4. Typical forces transmitted by bolted connections.

distance. The performance of the specimens have been satisfactory and further tests are being carried.

The test arrangement and set-up for the moment-type connection are shown in Fig.8. Typical load-deflection curves are shown in Fig.9. Here, the ferrocement plates had the same reinforcement as described above for the shear-type connection. Failure occurred by failure of the corner between the plates and was preceded by a small separation between the plates. The corresponding moment was of the order of 0.449 k-in (5.07×10^4 N-mm) and 0.306 k-in (3.46×10^4 N-mm) for the 8 layer and 4 layer specimens respectively. It seems that in this type of connection, the strength and spacing of the bolts can always be adjusted so that failure occurs in the plates, i.e. at the corner edge. The test results indicate that the moment resistance of the edge is smaller than in the ferrocement plate. Further tests are being carried to quantify this difference and to try strengthening procedures.

The testing program is ongoing, while simultaneously a finite element model is being developed for the connections. Once the model is calibrated, it may become possible to simulate analytically the

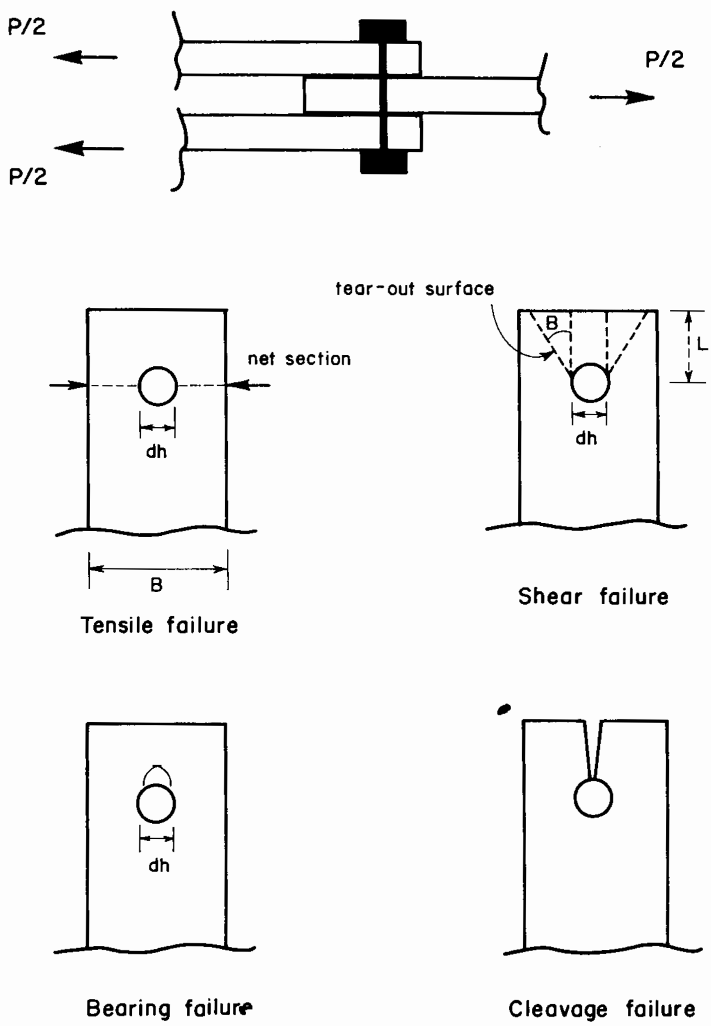


Fig. 5. Typical shear-type connections and corresponding plate failure modes.

response of any connection configuration, and thus study the effects of a large number of paramerters. This will open the way to analyzing the response of an entire house to various externally applied loads. A computerized evaluation will then allow the identification of optimum housing solutions and configurations.

CONCLUDING REMARKS

While sufficient information can be found in the technical literature on the properties, analysis, design, construction, and maintenance of ferrocement structural elements, no building code provisions or guidelines are available to address the question of ferrocement joints and connections. The connection problem is paramount to the technical feasibility and eventual success of prefabricated ferrocement housing. Thus, there is, first, a genuine need to solve the technical aspects related to the connection problem in ferrocement, and then there is need for developing related design and code recommendations. This is the main and ultimate goal of the above described investigation.

Architects, engineers, users, and building authorities are reluctant to consider ferrocement

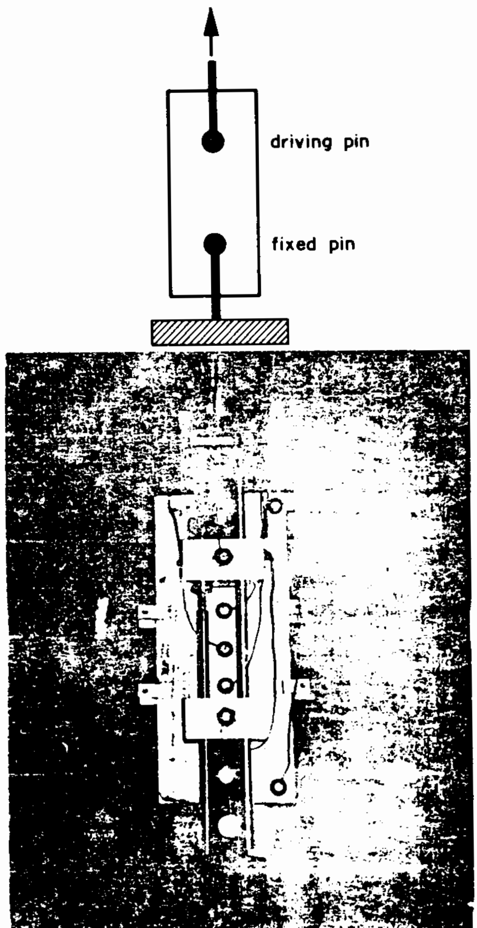
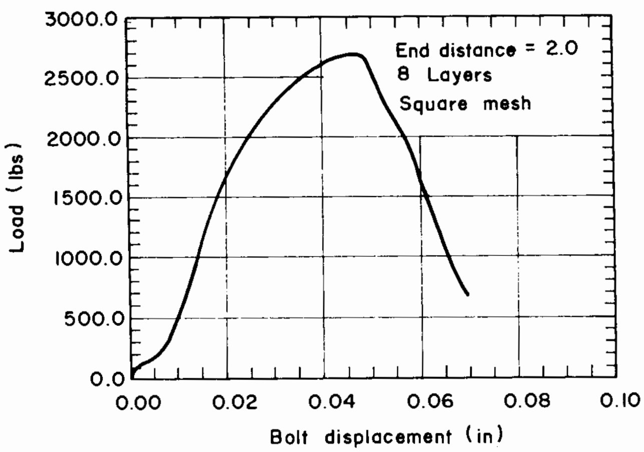


Fig. 6. Test arrangement and test set-up for shear-type connection.

systems for the large scale production of high quality prefabricated housing units, primarily because of the lack of prior experience in that sector. On the other hand, unless a large number of housing units are built with one system or another, no prior experience can be developed. This is a vicious circle which must be overcome. What may be needed is a large developer willing to take the responsibility for the planning, design, building, and maintenance of a large housing project, until satisfactory performance is proven without any doubt.

Finally, there is the question of prefabrication. How much prefabrication is optimal or necessary depends on a large number of regional and geographic factors. Available technology and manpower, site access, the means sought for transportation and erection are all issues that must be weighted in considering what level of prefabrication is needed for a given project. However, there is real ground to believe that a high level of prefabricaion can also guarantee a high quality, low cost product that can



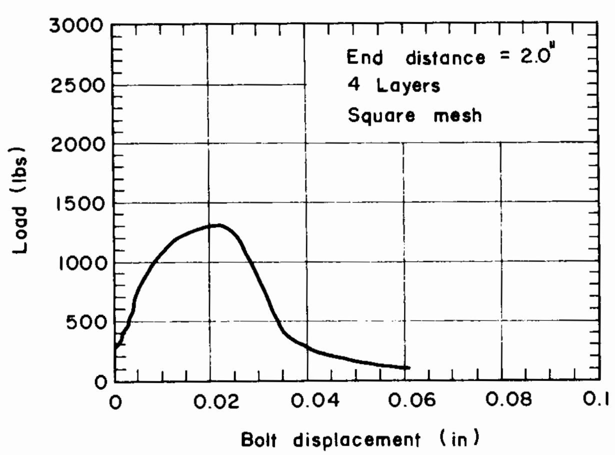


Fig. 7. Typical load-displacement response of pin-loaded plate with shear-type connection.

be competitive not only in developing countries, but also in highly industrialized countries as well.

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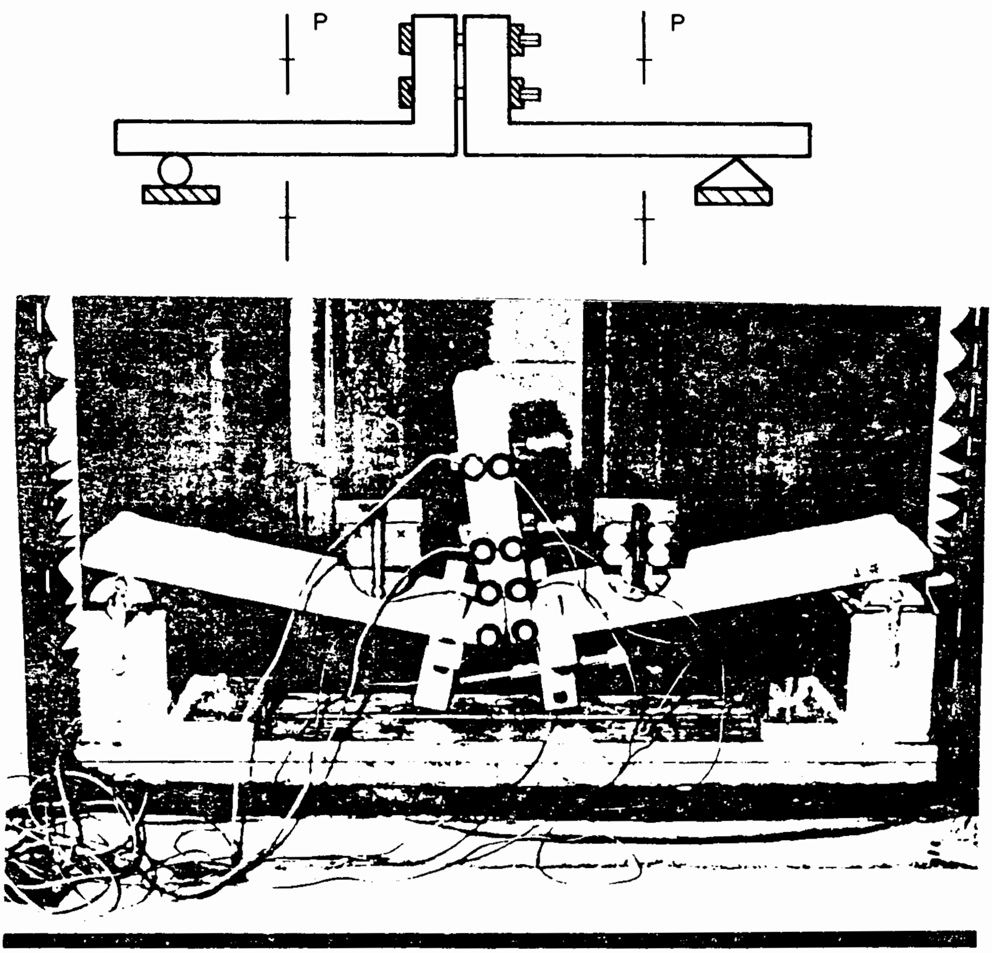
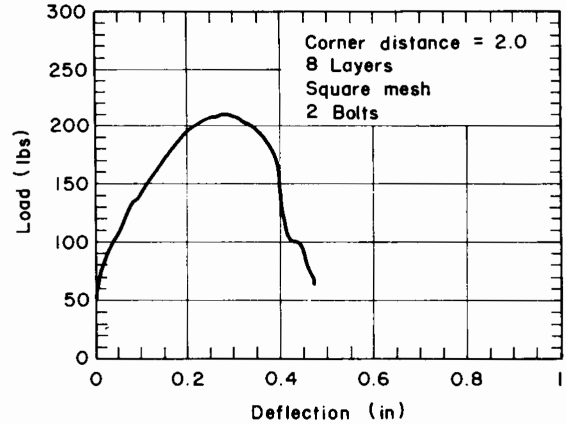


Fig. 8. Test arrangement and set-up for moment-type connection.



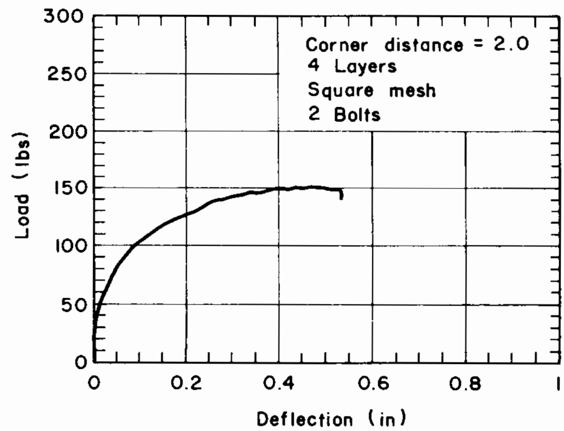


Fig. 9. Typical load-deflection response of L-shaped joined plates simulating moment type connections.

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Study of a Two Story Precast Ferrocement Model Building+

N.M. Bhandari*, D.N. Trikha and V. Syam Prakash*****

Ferrocement is an ideal material for precast industrialized construction of low cost houses due to its high strength per unit weight. In some of the countries like Cuba [1], multi-story buildings using precast ferrocement elements have already been built successfully. However, ferrocement element being comparatively very thin, jointing needs special attention. Further, there is no reported scientific study on the behavior of ferrocement buildings as assembled from precast elements to ascertain their load carrying capacity particularly with respect to serviceability requirements.

This paper presents the results of an experimental study [2] of a two story precast ferrocement building under simulated vertical and lateral loads. The model building of size 1.5 m x 1.5 m in plan and 3.0 m high was assembled from precast elements in a two story construction. Similar ribbed elements have been used both as walling and floor/roofing elements. The vertical load was applied using concrete blocks on the first floor and through hydraulic jacks, keeping the ratio between the two loads constant through each load increment. The paper presents the deformations and the behavior of the building and discusses the efficacy of the bolted connections.

INTRODUCTION

In the past two to three decades, ferrocement has been successfully used in a variety of applications [3] such as for buildings, marine, agriculture, water supply etc.. Several different walling and roofing elements for use in housing [4-6] have been proposed and their behavior adequately reported. To exploit the full potential of this material, successful attempts have been made in Cuba and some other countries to undertake industrialized multistory low cost housing construction using precast ferrocement elements. In the absence of any reported scientific study on the behavior of either joints or the complete assembled ferrocement building, the guide lines available for similar construction in concrete are generally followed. However, in view of the fact that ferrocement elements are comparatively very thin, the applicability of such guidelines needs to be examined by investigating comprehensively the behavior of a total ferrocement building assembled from precast elements with different types of connections to ascertain their load carrying capacity with respect to both the strength and the serviceability. Such a study would help in the development of a rational basis for the analysis and design of precast ferrocement buildings.

With the ever increasing cost of the building materials, and the availability of relatively cheap low skilled manual labor in the developing countries like India, the stage is well set to exploit the industrialized construction using precast ferrocement elements in a big way to meet out the huge requirement of low cost housing. In a cost comparison study [2] at Roorkee it was found that for the construction of a three story hostel building, the solution with the precast ferrocement construction provides a relative economy of 20% and 40% over the traditional load bearing construction in masonry and the reinforced concrete framed building with masonry infill respectively.

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In view of the above needs to exploit ferrocement and the existing gaps in the scientific data about its behavior, an experimental research program has been undertaken to investigate the behavior of different types of connections including that of the assembled two story precast ferrocement model building. This paper presents the experimental load deflection results for the model building and its comparison with the theoretically predicted deflections using finite element method.

MODEL BUILDING MODULES

Keeping in view the basic requirements of prefabrication with regards to mass manufacture of the component elements for economy, multipurpose double 'T' units have been devised both as walling and roofing/flooring units in the construction of the model building. The building, 1.5 m x 1.5 m in plan, and two story high of 1.5 m each, (Fig. 1) has been assembled using 18 precast double 'T' elements. The floor and roof slab had a 100 mm projection outside the walling units to facilitate the bolted connections. Even though the basic element 'Type' is only one i.e. double 'T' shaped, three different forms of the elements were cast differing marginally with respect to the dimensions and with one or both ends finished to permit the cast-in-situ connection. The complete details of the two walling elements WE-I and WE-II and the floor/roof element FE are shown in Figs. 2 and 3 respectively. The wall element WE-I was cast 1500 mm long and 600 mm wide with 120 mm wire mesh and 75 mm skeletal bar projecting at both ends along the width. The wall element WE-II is similar to the wall element WE-I, the only difference is that its width is 675 mm and the wire mesh and the skeletal steel project at one end only. Both these elements are 25 mm thick and have four 15 mm diameter bolt holes at the shown locations. The floor/roof element FE, was cast 1700 mm long and 775 m wide with 120 mm wire mesh and 75 mm skeletal steel bar projecting at one end as shown in Fig.3(a). The ribs were cast 210 mm short along the length. The thickness of the element as well as that of the rib is 25 mm. The positions of eight bolt holes of 15 mm diameter for the connections are shown in the Fig.3(b).

On the basis of the study of different types of joints [7], a judicious combination of cast insitu connection with mesh overlap for all the vertical joints at floor/roof have been used in the assembly of the test building. Small pieces of cleat angle of size 75 mm x75 mm x 6 mm and 45 mm long have been used to facilitate floor/roof to wall connections. The chosen connections have the advantages of both the speedier errection as well as the required rigidity.

MATERIAL PROPERTIES

Cement and mortar of 1:2 proportion by weight and a water cement ratio of 0.45 has been used in casting of the elements. The structural properties of the mortar used is given in Table 1.

Table 1 Structural Properties of the Mortar

28 days compressive strength	=	22.9 MPa
Tensile strength	=	2.6 MPa
Modulus of elasticity	=	50.0 GPa

A square woven galvanized wire mesh of 22 G (wire diameter 0.71 mm and spacing 6.3 mm) and 6 mm diameter mild steel have been used for skeletal reinforcement. The yield stress, the ultimate stress and the Young's modulus of the two reinforcing materials are given in Table 2. Commercially

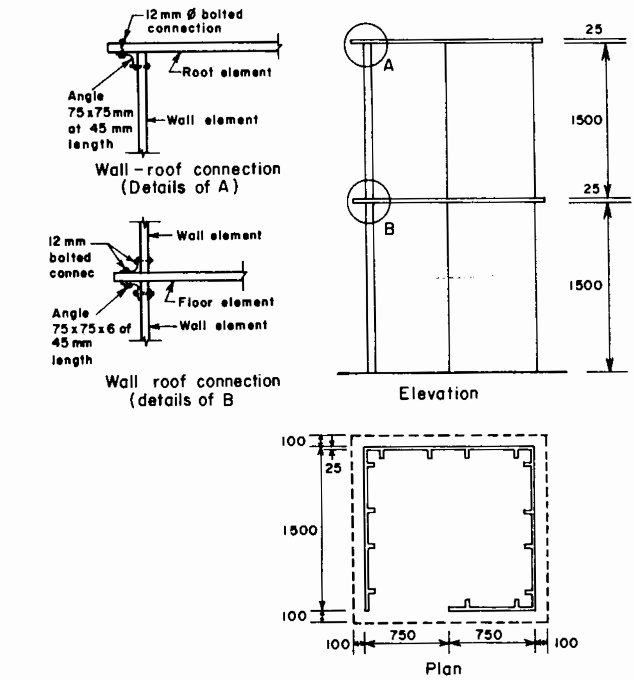

Fig.1. Model building dimensions and connection details.

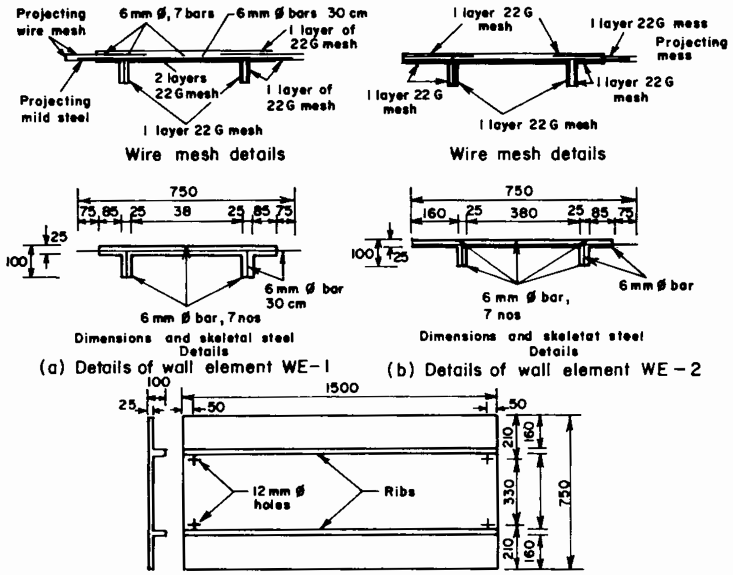

Fig.2. Details of wall elements WE-1 and WE-2.

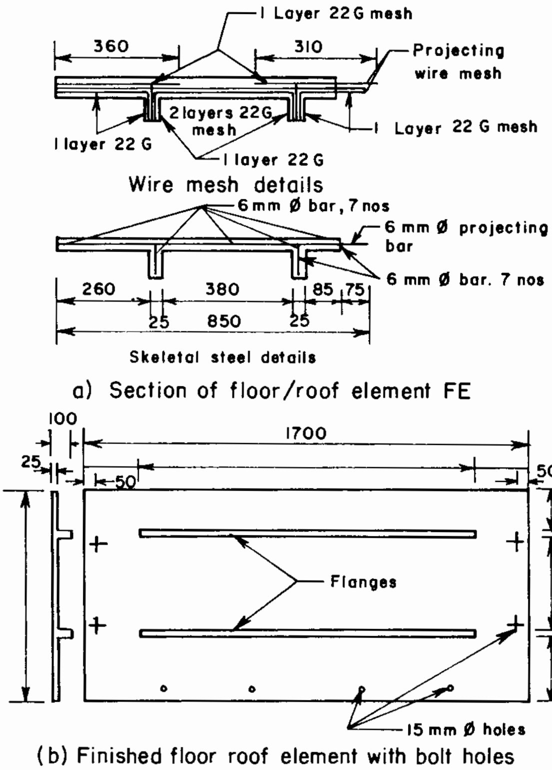

Fig.3. Details of floor/roof element FE.

available 12 mm diameter black bolt have been used for the bolted connections.

Table 2 Structural Properties of Reinforcing Material

	Wire mesh	Skeletal Steel
Yield stress (Mpa)	321.6	305.0
Ultimate strength (Mpa)	438.6	417.0
Modulus of elasticity (Gpa)	200.0	208.0

CASTING OF ELEMENTS

The reinforcement has been provided by considering the inplane bending of the wall elements under wind load and transverse bending of the roof elements under the vertical superimposed loads. All the elements have five skeletal steel bars in the flange and one in each rib, as shown. In the transverse direction, 6 mm diameter bars have been provided at 300 mm center to center. Two layers of 22 G square woven wire mesh have been provided in the flanges on the tension face in between the ribs whereas overhanging flange portions have one layer of the mesh on each face (Figs.2 and 3).

For casting wall/floor elements, the reinforcement cage was first prepared and then the elements cast over a level floor using a rectangular wooden mold 25 mm thick. The mortar was next poured into the mold, compacted by a surface vibrator after which mortar was plastered on the rib reinforcement. The specimens were than finished to the required dimensions. Provisions were made to leave bolt holes at the desired positions.

The complete casting of one element required 1/2 man day of a mason, 1/6 man day of the cage maker and one man day of an unskilled helper.

ASSEMBLY OF MODEL BUILDING

The building has been erected on a 1500 mm x 1500 mm mild steel angle frame security fixed to the bottom girder of the loading frame. The vertical legs of this angle iron frame had 14 holes of 15 mm diameter matching with the corresponding holes in the walling units.

The complete erection of the building was done manually. First of all, the seven walling elements of the first story were placed one by one in an erect vertical position, with the bottom ends firmly bolted to the base frame using 75 mm long and 12 mm diameter bolts and supported temporarily in the lateral direction from outside. The projecting wire mesh of the adjacent elements were overlapped by about 80 mm and tied firmly at few locations by using tying wires. Next the two first floor elements were placed in proper position over the already erected wall elements. The projecting wire mesh of the floor elements were lapped by 80 mm and tied well. The wall elements and the first floor elements were bolted by using two 75 mm long,12 mm diameter bolts and a 45 mm wide piece of mild steel angle section of 75 mm x 75 mm x 6 mm drilled with two holes of 15 mm diameter on both of its legs. The mild steel angles were placed between the floor and the wall panels on the external face of the building with bolt holes properly aligned. The joints between adjacent walling units and the two flooring units has been done cast-in-situ; where in the projecting wire mesh from the units were overlapped and plastered.

The second story was erected following the above procedure after the cast-in-situ joints in the first story had been cured for 7 days. The assembled structure is shown in Fig.1.

LOADING ARRANGEMENT

To simulate the actual loading the roof and the floor of the building were loaded with uniformly distributed vertical load. Over the roof, a 16 point loading was accomplished by using a hydraulic jack, 50 kN proving ring and a suitable arrangement of distribution girders. Since such an arrangement was not feasible for floor slab, it was decided to load it by using concrete cubes of 200 mm size having an average weight of 190 N each.

The lateral load was applied as line load across the full width of the roof and the floor slabs through two wooden logs of size 1700 mm x 100 mm x 100 mm placed against the edge of the slabs on the same side of the building. Two horizontal hydraulic jacks were used to apply horizontal loads by reacting against a prestressed masonry retaining wall of sufficient rigidity. The entire test set up is shown in Fig.4.

TESTING PROCEDURE AND INSTRUMENTATION

For measuring the deflections eighteen dial gages were arranged on front face, i.e. the face opposite to the laterally loaded face of the building. The dial gages were arranged at mid height of the third-fourth story height and floor/roof level in each story as shown in Fig.5.

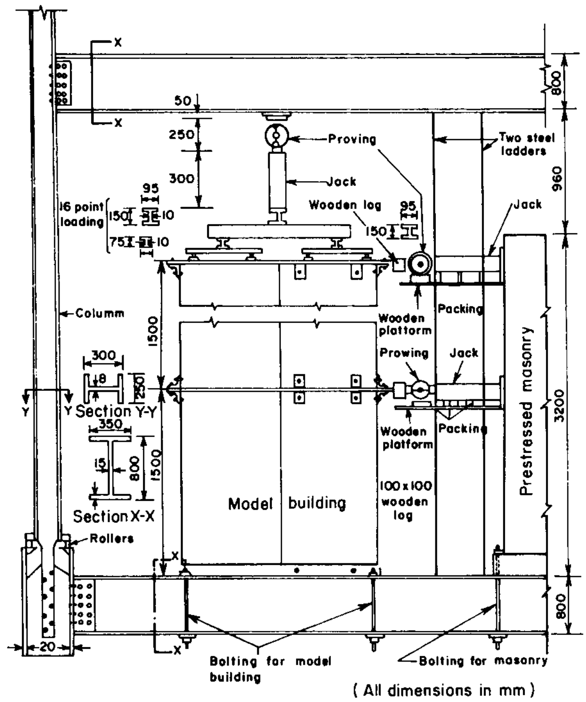

Fig.4. Testing arrangement of the model building.

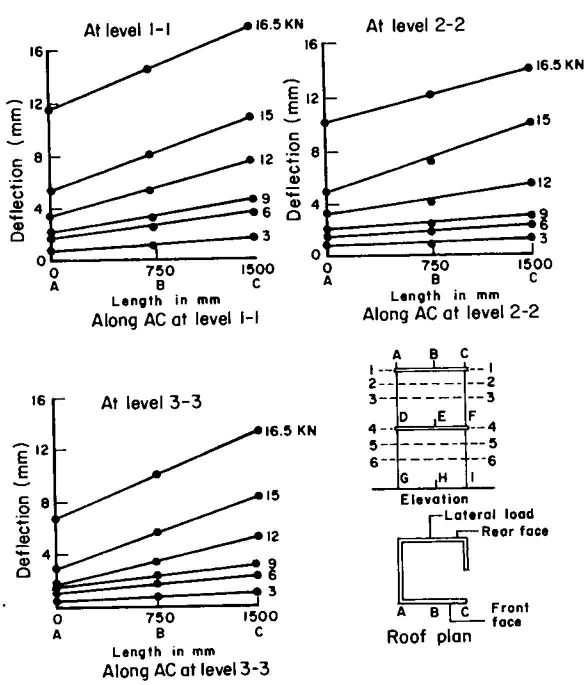

Fig.5. Lateral deflection of top stor y wall at different levels.

The building was tested under monotonically increasing vertical and lateral loads, keeping a constant of 4:1.5 proportion between the two loads in each step throughout. Since the lateral load at the first floor level was twice that at the roof level, this ratio was 2:1 between the vertical load and the lateral load on the first floor. These ratios have been fixed by actual floor/roof and wind load calculations for the model building.

In each load step, the loading procedure was as follows: First, the floor was loaded to 2000 N by placing concrete cubes; next the roof was loaded to 2000 N by operating the jack and finally the lateral loads were applied simultaneously at the roof and floor levels of 500 N and 1000 N value respectively. The whole system was watched for a while to see that the readings were fairly stable and then the dial gauge readings were noted. The same procedure was repeated for the second and subsequent load steps. The loading was stopped at the stage when further loading by cubes at the floor level due to the story height limitation was not possible. At that stage, the total load acting on the building was 44 kN vertical and 16.5 kN horizontal. These loads are greater than 2.5 times the normal design working loads.

TEST RESULTS

Figs. 5 and 6 show the variation of the lateral deflection of the horizontral grid lines on the front face of the building and Fig.7 shows the variation of lateral deflection along the vertical grid lines marked on the front face of the building at different lateral loads.

Fig.5 shows the plot of the lateral deflections of the horizontal grid lines 1-1, 2-2 and 3-3 at different lateral loads. It is clear that the points lying on the vertical edge CF have about 40% to 60% more deflection than the corresponding points on the edge AD, obviously due to the presence of the door opening on the resisting shear wall placed at the corner C. Further, the grid lines remain almost straight up to the full test load there by indicating that there is no transverse bending of the wall elements. The building has however, rotated in plan about its centre of rigidity due to torsional loading

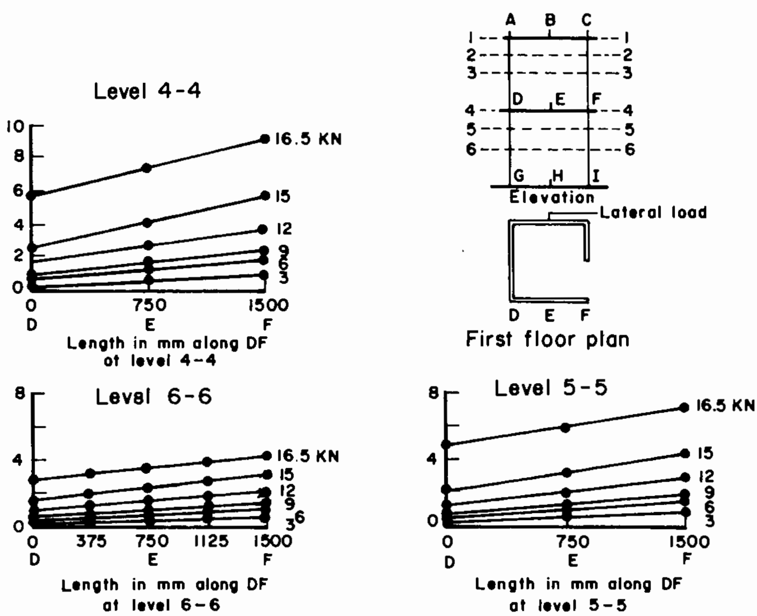

Fig.6. Lateral deflection of first stor y wall at different levels.

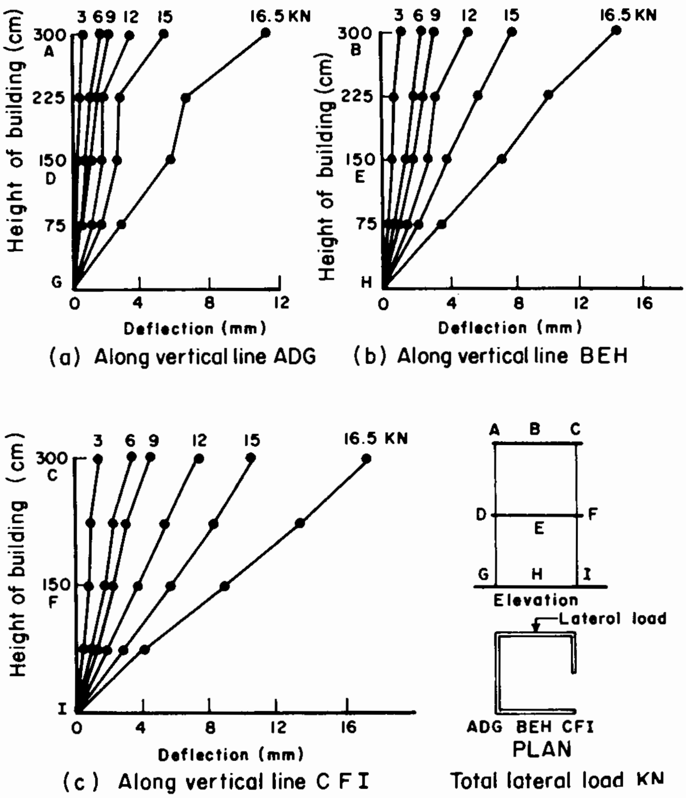

Fig.7. Lateral deflection of wall in the vertical plane.

resulting from assymmetric plan. Fig. 6 shows the deformatioins of the grid lines 4-4, 5-5 and 6-6 in the first story wall. The general trend of the results is similar to that described above for the second story wall.

Fig. 7 shows the deflections of the vertical edges ASG, BEH and CFI for different total lateral loads on the building. For lateral load up to 10.5 kN, the height versus lateral deflection curve is similar to that of a typical shear wall structure. There is no appreciable kink in the deflection curves at the first floor. For lateral load beyond 12 kN, the building exhibits non-linear behavior and the load deflection curve has sharp deviations at the first floor level and at the center of the top story as seen in Fig.7.

On the basis of the load deflection curves described above, it may be concluded that the building exhibited a linear behavior up to a lateral load of 10.5 kN, which is about one and a half times the normal design lateral load. Beyond this, the deflections increase rapidly till at a total lateral load 16.5 kN. This model building withstood a load over two and a half times the service loads without showing any crack anywhere, including even in the cast-in-situ joints. The ratio of the height to the maximum lateral deflection for the corner C of the model building is found to be 1/923 and 1/174 at the service load and the ultimate test load respectively.

THEORETICAL ANALYSIS

The theoretical analysis of the model building has been carried out using an existing program 'MAPWB" for the membrane analysis of the panel wall buildings. It has been seen in a previous study

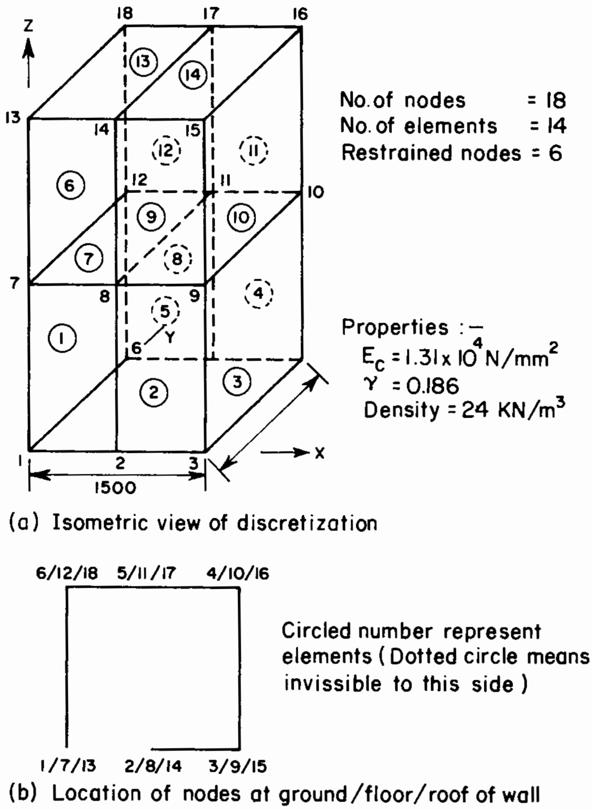

Fig.8. Details of discretization of model building.

[8] that a simple membrane type analysis using two dimensional elements is more efficient and economical than the flexural cum membrane type analysis, FCMAB, using four noded flat shell elements for the analysis of such buildings. The joints are assumed to be pinned along the edges in the membrane type analysis. The equivalent values of the Young's modulus Ec and the Poisson's ratio Yc for the ferrocement composite have been determined as 131 kN/mm^2 and 0.186 respectively and these have been used in the theoretical analysis.

This discritization of the building is shown in Fig.8, which has 14 elements and of nodes of which six are restrained. A linear analysis has been performed for one load step and the theoretical results extrapolated for comparison. The comparison of the theoretical and the test results up to a total lateral load of 12 kN is shown in Table 3. It is seen that the ratio of the experimental deflections to the theoretical deflections varied from about 6 in the first load step to about 13 at the lateral load of 12 kN, when the non-linearity has been first observed. This large difference in the test and the predicted values of deflections may be attributed to the flexibility of the bolted connections, as seen in an earlier study [7] of testing of bolted L and T type joints.

It is therefore, concluded that unless the bolted connections, which permit rigid body deflection of the entire building due to play in the bolt holes, are modelled properly in the discretization, the discripancies in the predicted values are bound to occur.

Table 3 Comparison of Theoretical and Experimental Deflection

Total lateral load (N)	Deflection							
	Floor level				Roof level			
Grid point	A - 4		E - 4		A - 1		C - 1	
	Theory	Test	Theory	Test	Theory	Test	Theory	Test
0	0	0	0	0	0	0	0	0
1500	0.014	0.080	0.026	0.23	0.035	0.23	0.074	0.45
3000	0.027	0.230	0.051	0.98	0.070	0.63	0.148	1.45
4500	0.041	0.470	0.077	1.38	0.105	1.30	0.222	2.41
6000	0.055	0.730	0.103	1.81	0.140	1.67	0.296	3.48
7500	0.069	0.910	0.128	2.10	0.175	2.07	0.370	4.04
9000	0.083	0.970	0.154	2.36	0.210	2.13	0.444	4.44
10500	0.096	1.090	0.154	2.36	0.245	2.30	0.518	5.31
12000	0.110	1.630	0.205	3.77	0.280	3.32	0.592	7.36

CONCLUSIONS

Based upon the results of the study of a two story precast ferrocement model building, the following broad conclusions are drawn:

1. Load carrying capacity of a multipurpose double *T* wall and roof/floor elements has been found to be extremely satisfactory. The building as a whole resisted over two and a half times the normal design loads.
2. No cracks have been noticed anywhere in the building including the cast-in-situ joints inspite of the elements being very thin and loaded much beyond the design load.
3. A judicious combination of the cast-in-situ connection with the mesh overlap for the vertical joints and flexible bolted connections at floor/roof levels has shown the viability of the assembly and the monolithic cellular action of the building to resist lateral loads.
4. The ratio of the height to the maximum lateral deflection at the normal design load is 932, though at the maximum test load of over two and a half times the normal design load, the ratio falls to 174. This confirms that the model building comprising thin ferrocement elements meets the serviceability requirement for the deflections satisfactorily.
5. The flexibility of the bolted connection imparts additional lateral deflections to the building and these increase with the increase in the lateral load.
6. Finally the results of the study confirms the feasibility of precast ferrocement construction for low cost multi-story buildings without any reservations.

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What is Ferrocement ?

Postscript to the Havana Symposium on Ferrocement - October 1991

D. Alexander *

The Havana Symposium was an event notable for the strong representation of delegates from Latin America which reflected a surprisingly diverse application of thin section cementitious composites in their countries. This term "cementitious composites' is used advisedly as the Symposium may also mark a watershed at which the boundaries of ferrocement became confused with reinforced cement mortars. As much of the work presented in the guise of ferrocement was cement mortar generally defined in sections ancillary to reinforced concrete codes, especially for water retaining structures.

The San Carlos Group of Brazil amongst others showed an ambivalence towards ferrocement and sought to redefine it to encompass reinforced cement mortar constructions in which they indubitably excel [1]. The common practice of this group and elsewhere in Latin America is to employ 75 mm and 150 mm plain weld mesh in 3 mm to 9 mm diameter wire sizes as the main resistive steel augmented by single and sometimes double layers of woven or weld mesh of intermediate size of crack attenuation. The motive for this approach lies in the inadequacy of fine mesh to provide suffficient area of steel at reasonable cost within the sections. It would take 3 to 5 layers of 21 gauge weld mesh to provide the same area of steel per 300 mm as could be obtained with a single 7 mm diameter rod.

It was not surprising that some discussion occurred amongst the delegates on the relevance to ferrocement of some of the applications presented.

It may therefore be timely that the *Journal of Ferrocement* provide a forum to reassent the position of ferrocement.

Originally ferrocement was conceived as a cementitious composite containing a high degree of dispersion of steel reinforcement, typically in the form of layers of fine mesh, which imparted certain unique performance characteristics to ferrocement.

Among these are:

- that it could be employed in relatively thin structural sections
- That the steel was protected against corrosion even with very thin covers (2mm-4mm)
- That there was fine dispersion of cracking quite unlike the coarse wide spaced cracks commonly observed in reinforced concrete structures.

In practice, the material durability was established first by Nervi and later in New Zealand often in severe marine conditions albeit with some failures due to the then incomplete understanding of the material. The protective mechanism provided by the cement rich low permeability mortars which was once thought to be peculiar to ferrocement is now commonly recognized across a broad spectrum of high performance concretes and mortar composites so that this is no longer unique to ferrocement.

In time rules were devised to guide the practice of ferrocement. These chiefly concerned the composition of the mortar and the fineness and dispersion of the steel reinforcement in the mortar. The steel reinforcement was quantified in terms of specific surface and volume fraction generally given as 2.7 to 7in ^{2} per in^3 (0.106 to 0.276 mm^2 / mm^3) and 7% to 20% by weight of steel in the composite

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respectively. Primarily the mortar composition and consistency controls the protection of the steel against corrosion while the amount and dispersion of steel reinforcement dictates the cracking regimes, although a secondary interaction occurs between these two effects. The mechanical properties obtained by these rules have been evaluated by B.K. Paul and R.P. Pama [2] and by the ACI committee 549 - *Guide to Ferrocement Construction.*

Reinforced cement mortars do not normally comply with the above definition of ferrocement as they usually lack the necessary subdivision and fineness of reinforced cement (in specific surface) that is necessary to ensure closely spaced crack regime typical of ferrocement. In fact the observation of the crack regime provides a simple way of discriminating between ferrocement and the other varieties which form the homologous series of reinforced cementatious composites and which include context could be defined as having crack spacing no greater than say 20 mm. Philosophically it may even be important to define ferrocement in terms of such an observable manifestation of its internal struction in the same way that stainless steel is identified by its manifest resistance to corrosion.

The reason why the achievement of fine dispersion of cracking has been traditionally important for ferrocement is that it permits the use of higher steel stresses within the serviceability range of the material prior to the formation of a 0.05 mm (0.002 in) crack width, as illustrated in the accompanying figure (Fig. 1) and thereby improves the efficiency of the material

This crack width is commonly considered a threshold below which the cracks do not significantly impair the protective function of the mortar covering the steel.

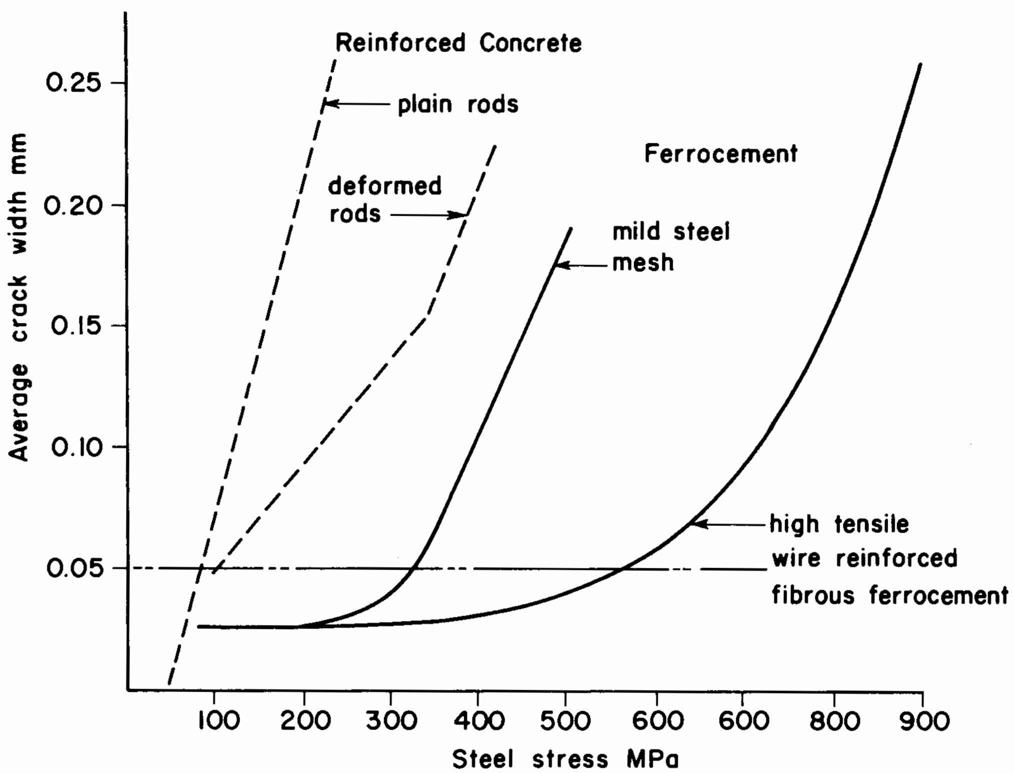

Fig. 1. Typical crack width response for ferrocement.

However, even with this efficiency the full potential for the commercialization of ferrocement has not been achieved and this may be due to its historical perception as a multi-mesh layered cementatious composite. The present ACI Committee 549 definition of ferrocement is not in fact restrictive but the textural content of the guide on ferrocement construction continues to emphasize the fine mesh multi-layer concepts.

It is well recognised that ferrocement in this form has several adverse features:

a. It is directionally undifferentiated in order to satisfy distribution criteria and close spacing within the matrix.

b. The interior mesh layers, also as a consequence of the distribution criteria, are increasingly under utilized in flexure,because of the lower strain within the section and reduced lever arms to the compression face.

c. Fine mesh severely limits the amount of steel area that can be placed in the outer layers to resist flexure. Also, because the meshes are almost universally costructed of mild steel because of manufacturing constraints, the crack serviceability limits are rapidly reached upon the onset of yield in the outer layers of mesh. As a result the flexural strength within the serviceable range of this form of ferrocement is exceeded by a range of competiitve materials including metal plate, fibreglass etc., except where labor costs in fabrication can be discounted.

d. Because of the above factors this form of ferrocement tends to be severely under reinforced in flexure and provides little scope or incentive to utilize the very high strength mortars (100 MPa-200 MPa) now in practice.

Therefore for many applications the continued use of multilayer fine mild steel wire mesh ferrocement is excessively restrictive in effect and expensive, both in terms of steel meshes and labor inputs.For example in the author's practice in marine applications a high tensile wire reinforced fibrous ferrocement [3] was developed to provide ultimate flexural strength comparable and exceeding that of mild steel plate on a weight for weight basis, to enhance impact resistance, and to fully utilize the permissable crack serviceability range i.e. a high performance ferrocement. (Table1 and Fig.2). In this material, which allows coarser and high strength wire (1500 MPa-2000 MPa) to be concentrated in effective outer layer locations, the dispersed fiber provides the fine subdivision needed to control cracking [4].

The modulus of elasticity (secant modulus) of the three HT wire panels up to the point of first visible crack was determined:-

Plain mortar / HT wire	- 16.87 x 10^3 MPa
14.5 mm EE fiber / HT wire	- 27.08 x 10^3 MPa
18 mm EE fiber / HT wire	- 21.22 x 10^3 MPa

Obviously, at the other end of the spectrum for housing construction and for that matter for the construction of 5000 gallon water tanks in New Zealand, the fine mesh dispersion of ferrocement is being discarded in favor of heavier rods and meshes more appropriate to reinforced cement mortars.

The question therefore remains are we being too restrictive in our definition of ferrocement - as the San Carlos Group maintains - and should there be a place for reinforced cement mortars within the general definition of ferrocement.

There is an argument that if a mortat composite is designed to remain uncracked with a satisfactory margin of safety the provisions to ensure a closely spaced crack distribution are no longer relevant, and therefore reinforced cement mortars that would otherwise comply with ferrocement criteria, except for specific surface, should have a place within the definition of ferrocement.

All panels with HT wire reinforcement failed in compression.

Table 1 Comparison of Flexural Stresses and Steel Stresses

Panel	Flexural stress at first appearance of moisture lines (MPa)	Flexural stress at at first visible crack (MPa)	Ultimate flexural strength (MPa)	Steel stress at first visible crack (MPa)	Steel stress at ultimate (MPa)	Concrete* compressive stress at ultimate (MPa)
a) Plain mortar	-	-	3.92	-	-	-
b) Fiber reinforcement 14.5 mm EE (5% wt.)	-	8.66	8.66	-	-	-
c) Fiber reinforcement 18 mm EE(5% wt.)	7.80	12.15	12.15	-	-	-
d) Plain mortar HT Wire	6.30	16.17	55.65	360	1251	102.1
e) 14.5 mm EE Ht Wire	8.75	37.40	62.87	790	1343	102.6
f) 18 mm EE HT Wire	11.69	32.72	47.53	780	1138	91.9

* All panels with HT wire reinforcement failed in compression.

Personally I would wish to see more research especially into bond effects (into which dilation theory should be introduced) and durability and corrosion resulting from the use of larger diameter steel reinforcement at wider reinforcement spacings within the uncracked and cracked range of the mortar before the definition of ferrocement was expanded but I expect that in one important property of ferrocement, that is in its ability to extend the allowable stress within a specified serviceability range, reinforced cement mortars fall short in accomplishment. Therefore envelop could impair the image of ferrocement, and it may be more clearly defined as a lower bound reinforced concrete which would afford its practitioners the opportunity to develop their own technology.

However, it is a fact that reinforced mortars may continue to be called ferrocement and that the public will continue to perceive them as being ferrocement.

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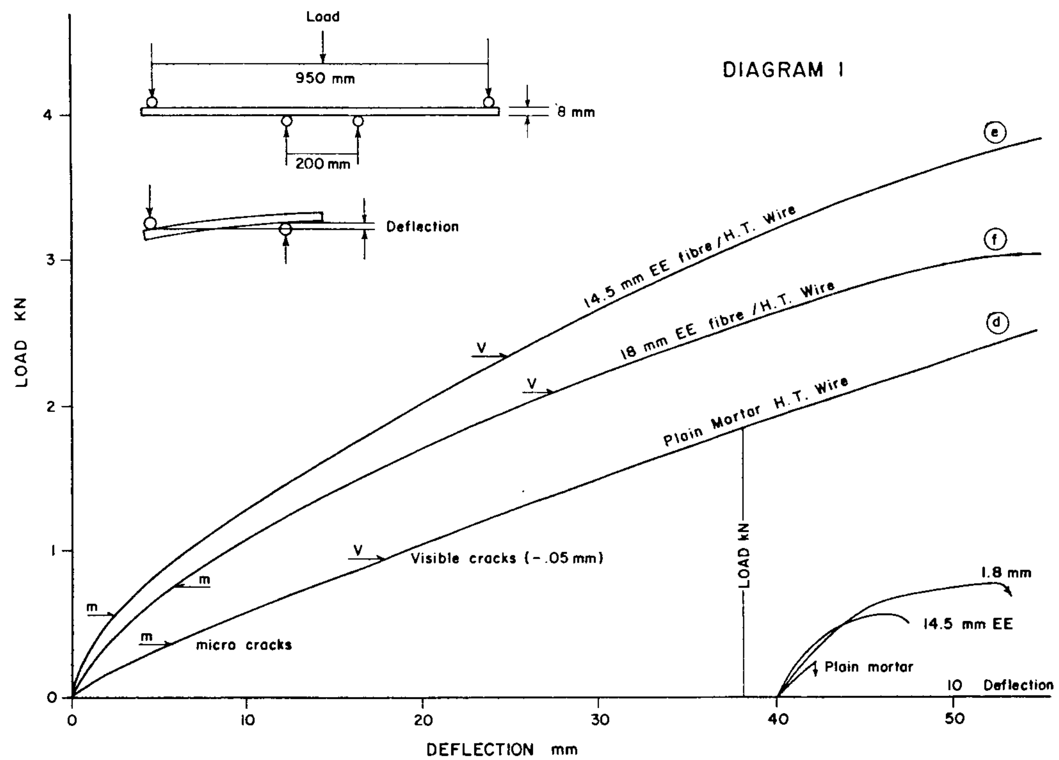

Fig. 2. Load - deflection curve.

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water tanks/hulls (structures)/slabs/costs

4313 A:1182/A:TA442 F492
Craig, R. Flexural behavior and design of reinforced fiber concrete members. *Fiber Reinforcment Concrete Properties and Application.* 517-563. American Concrete Institute.

beam (supports)/computer programs/ductility/fiber reinforced concretes/flexural strength/high strength/lightweight concretes/metal fibers/reinforced concrete/reinforcing steels/structural analysis/structural design/T-beams

Miscellaneous Notes

4306 A:1174
1962. Equipment for prestressed concrete concrete and constructional engineering. *Books on Concrete, Concrete Publications Ltd.*, London, U.K. Vol. LV11, No. 3, 133.

hulls/structures/ferrocement/boats

NEWS AND NOTES

AWARDS FOR IFIC

The **International Ferrocement Information Center** (IFIC) was awarded a Diploma for outstanding services to the ferrocement community. IFIC was recognized for its effective collection and dissemination of information; and for effective transfer of ferrocement technology.

Mrs. **Lilia R. Austriaco** was awarded a Diploma for international leadership in research, development and transfer of ferrocement technology. The award was also given in recognition of her outstanding involvement in the establishment and coordination of the Ferrocement Information Network (FIN).

The other awardees were:

- *ACI Committee 549 on Ferrocement*. For producing the "Guide for the Design, Construction and Repair of Ferrocement" ACI 549.IR.88.
- *Douglas Alexander*, Alexander and Associates, Consulting Engineering, Auckland, New Zealand. For the development of fiber reinforced-prestressed-ferrocement and for commercialization of ferrocement.
- *Joao Bento de Hanai*, Universidade Federal de Sao Carlos Brazil. For research and development of ferrocement in Brazil.
- *Antoine E. Naaman*, The University of Michigan, Ann Arbor, Michigan, U.S.A. For leadership in the preparation of the ACI Guide and in research and development of ferrocement.
- *Alfonso Olvera*, Universidad Autonoma, Mexico City, Mexico. For the development and dissemination of ferrocement in Mexico.
- *James P. Romualdi*, Professor of Civil Engineering, Carnegie-Mellon University, Pittsburg, Pennsylvania, U.S.A. For the develop-

Lilia Robles-Austriaco of IFIC receives diploma for international leadership in research, development and transfer of ferrocement technology.

Prof. Hugo Wainshtok Rivas receives his award.

ment of fiber reinforced composites and ferrocement.

- *Hugo Wainshtok Rivas*, Instituto Superuir Politecnico "Jose A. Echeverra" Havana, Cuba. For the development and dissemination of ferrocement in Cuba and for his dedicated effort to organize the Fourth International Symposium on Ferrocement.

The awards were presented jointly by the La Union Nacional Arquitectos Ingenieros de la Construccion de Cuba (UNAICC); the Organizing Committee of the 4th International Symposium on Ferrocement and the International Ferrocement Society (IFS) at Habana, Cuba on 25 October 1991.

IFIC NEWS

4th International Symposium on Ferrocement

The Fourth International Symposium on Ferrocement was held in Havana, Cuba on 22-25 October 1991. The symposium was attended by 250 participants from 36 countries.

Special lectures were presented on the main topics as follows:

* Use of Ferrocement in Cuba
  Hugo Wainshtok Rivas (Cuba)
* The Role of IFIC in Ferrocement Technology Transfer to Developing Countries
  Ricardo P. Pama (IFIC/AIT, Thailand)
* Ferrocement Houses : Current Situation and Prospects
  Alfonso Olvera (Mexico)
  Ricardo Carcano (Cuba)
* Marine Ferrocement : Current Situation and Prospects
  Carlos Llanes (Cuba)
* Ferrocement Ships : Building, Maintenance, and Durability
  D. J. Alexander (New Zealand)
  Jorge Cruz (Cuba)

The 146 technical papers were presented in three parallel technical commissions according to three general topics :

Commission No.1 Mechanical properties, Research and Development

Commission No.2 Ferrocement Applications; Standards and Codes and Building Technologies

Commission No.3 National Experiences, other applications, reservoir and swimming pool construction.

Technical visits allowed the participants to observe the ferrocement applications in Cuba. The participants were impressed with the development of the technology in prefabricated housing and swimming pool constructions during their visit to San Jose de Las Lajas ferrocement housing project and the Olympic swimming pool at the Higher Institute of Agricultural Sciences in Havana respectively. The visit to the Higher Politechnical Institute 'Jose Antonio Echeverria' was very interesting and informative. Dr Hugo Wainshtok Rivas presented all the completed and on-going researches on ferrocement.

The participants during one of the plenary session.

The participants at the prefabricated low-cost housing.

Prof. Dr. Ing. Hugo Wainshtok Rivas explains the researches on ferrocement at the Higher Politechnical Institute 'Jose Antonio Echeverria' to the participants.

Mrs. Lilia Robles-Austriaco of IFIC (fourth from right) and Lic. Norma C. Cardenas, Head, National Group of Scientific - Technical Information (third from right) with the directors of the ferrocement boatyards.

The participants being briefed on the manufacture of wire mesh.

The participants at the factory for prefabricated housing elements.

Ferrocement elements being tested at the Higher Politechnical Institute 'Jose Antonio Echeverria'.

Mrs. Robles-Austriaco with (L-R) Dr. Boris Mironkov, Russia, Dr. Y. Ohama, and Dr. Shirai, Japan.

During the closing ceremony, Mr. Paul Nedwell, the FIN United Kingdom coordinator invited all participants to attend the 5th International Symposium on Ferrocement in 1994. FIN United Kingdom based on the University of Manchester Institute of Science and Technology (UMIST) will host the 5th International symposium.

H.E. Mr. Jose Canete Alvarez, Minister of Industry, Materials, and Construction of Cuba declared the symposium closed.

BOTSWANA

Motorised Mesh-wire Machine

Rural Industrial Innovative Centre (RIIC) in Kenya has developed a motor-powered mesh-wire making machine to boost the local production of wire-mesh in Botswana. The machine is an alternative to the manually operated version. According to the chief engineer, small projects, Hans-Peter Zimmer, the machine will increase the production output since it operates faster than the manual version. The machine produces 2 mm galvanized diamond mesh-wire fence and is powered by a 220 V single phase electric motor.

The machine consists of a steel driving shaft, supported by two ball bearings which drive a blade and coil made of hardened steel. The machine will be marketed with different blades to ensure consistent performance with a variety of wire qualities. To minimize wear and tear of the blade bending-coil, lubrication and cooling facility are installed.

A no-volt-trip switch with overload relay for the motor and lubrication / cooling circuit is also installed to protect the motor from accidental overload.

Prototypes are now undergoing performance testing. On completion of the test trials the machine will be transferred to rural metal workshops through RIIC Technology Transfer Unit for mass production. Further information on prices and other technical data will be available as soon as the required testing is completed.

(News on Technology, Vol. 5, No 3, 1991)

CHINA

Application of Ferrocement in the Ship and Structural Engineering

The flexiport consists of two stack bridges, two caisson piers, two active bridge approaches, two ferrocement pontoons and the anchor system. The berth line of this flexiport is 85 m. This flexiport can be built fast, dismantled and transported. The main dimension of the ferrocement pontoons are as follow :

Length	= 40.00 m
Beam	= 10.00 m
Depth	= 2.60 m
Draught	= 1.20 m
Deck load	= 3.20 T / m^2

The vertical space-frame ferrocement bulkhead especially suited to be built on the mould. Its feature are light, prefabricated and inexpensive.

(Information from Mr. Zhu Yuankang, Professor ,Fujian Provincial Science andTechnology Research Institute of Communications, China)

Table 1 The Main Dimension and Characteristics of the Ferrocement Ship

Kind of Ship	Length (m)	Beam (m)	Depth (m)	Carrying capacity (T)	Main power (Kw)	Velocity (Kn)
Agricultural ship	7.80	1.76	0.44	1.80	2.20	5.20
	11.17	2.65	0.85	6.00	5.51	6.12
	16.60	3.40	1.35	20.00	17.65	7.75
Lighter	20.00	5.00	1.65	60.00		
	26.65	6.40	1.80	135.00		
Towboat	15.60	3.40	1.10		88.24	9.70
	19.75	5.40	1.76		183.82	10.05
Offshore cargoship	32.25	6.00	2.75	137.15	73.53	8.21
	41.33	7.40	3.70	333.17	226.47	9.76
	105.20	14.50	8.10	3024.00	1955.88	13.80
Fishing boat	20.26	5.00	1.50	30.00	88.24	9.37
	28.15	6.70	1.92	50.00	136.00	9.00
Pontoon	48.00	10.00	2.80	345.60		
	60.00	12.00	3.00	468.00		
Floating shears	32.00	8.00	2.40			
Waterworks Ship	72.00	16.00	3.60			

3000 T Class coastal assembly flexiport.

INDONESIA

Rural Development Activity

Dr. Ir. John B. Manga, head of Fluid Mechanics Laboratory, Faculty of Engineering, Hasanuddin University and 1978 participant of the IFIC training, is undertaking rural development with his student in Ujung Pandang. He conducted training for villagers to construct latrine, water jars, water tanks and water pipes. His priority project is water supply for the rural villagers.

Providing ferrocement water system for the people in Kabupaten Majine, a village 250 km from Ujung Pandang.

Training on ferrocement for academic industries, Ujung Pandang. Dr. Manga is third from left

The villagers constructing ferrocement container during the ferrocement training in the rural areas.

The students from the Faculty of Engineering, Hasanuddin University with Dr. Manga (3rd from right) on their way to conduct study on water supply needs of a village.

(Information and photographs from Dr. John B. Manga, Faculty of Engineering, Hasanuddin University, Kampus Baru Tamalanrea, Ujung Pandang, South Sulawasi, Indonesia.)

JAMAICA

Better Bricks from Bauxite Waste

Researchers from Jamaica and the university of Toronto have found, for the first time, a potential use for the millions of tons of thick red mud that is discharged as waste in alomina production. The new product will help dispose of the caustic waste and, at the same time could go a little way towards solving Jamaica's housing shortage.

With $ 301,000 financing from Canada's International Development Research Center (IDRC), the researchers have developed a technology to produce low cost bricks from red mud and non-commercial bauxite that are as strong as traditional concrete blocks, simpler to make and do not require costly energy for firing. Unlike concrete, explains Canadian project leader Dr. J. W. Smith, an engineer at the University of Toronto, the bricks harden through chemical bonding caused by an internal chemical reaction at normal tropical temperatures and therefore do not require firing using expensive imported oil.

A model brick house is now under construction at the Jamaica Bauxite Institute in Kingston and plans are underway for a seminar to train the the local people how to manufacture, and build with the bricks. In addition, the Construction Resource and Development Centre, a Jamaican NGO, will use the bricks in several community housing programs as part of their cyclone-resistant housing project, also funded by IDRC.

(Appropriate Technology, Vol. 18. No. 2, September 1991. Information from IDRC, P.O. Box 8500, Ottawa KIG 3H9, Canada.)

MALAYSIA

Ferrocement Garden

Dr. Zakaria Mohd. Amin, associate professor at the University Sains Malaysia and 1984 participant at the IFIC trainer's training, has constructed ferrocement garden furniture for his own garden. Dr. Zakaria has just formed a firm to produce ferrocement and related material products.

(Information from Dr. Zakaria Mohd. Amin, Pusat Pengajan Sains Kimia, University Sains Malaysia, 11800 Pulau Penang, Malaysia.)

Ferrocement garden

NEW ZEALAND

Joint Design Check-list

To assist joint designers in ensuring that the necessary information is available to the builder, the following checklist may be useful in reviewing design proposals :

Joint details : The plans should show full details of all joint types - including construction joints and joint-sealant requirements for all joints in the pavement. These details should indicate the required treatment at manholes and pits and procedures for avoiding acute angles in individual slabs or at the pavement perimeter.

Joint location : By using a suitable legend, the locations of all joints by type should be shown on a seperate drawing if necessary. Where options are made available, such as the variable spacing of the joints, notes should be included.

Reinforced slab : Where for reasons other than slab length, slabs are to be reinforced, those slabs should be identified on the plans.

Setting out of joints : Sufficient information should be included on the drawings or in supplementary sketches to allow joints to be located in the field to the required accuracy.

Joint construction : The details of specified or permitted procedures for the construction of all joint types should be included in the specification.

Construction sequence : Where a construction sequence is either planned or envisaged by the designer, and irrespective of whether or not it is stipulated that it be followed during construction, it is recommended that notes be provided for the information of the contractor. A pre-construction meeting can be useful in this regard.

(Reprinted from ' Australian Concrete Construction" June 1989. Source New Zealand Concrete Construction, April 1991.)

U. K.

European Specification on Sprayed Concrete

A draft specification for sprayed concrete has been completed by the Technical Committee- Sprayed Concrete, of EFNARC, the European Federation of National Association of Specialist Repair Contractors and Material Suppliers to the Construction Industry. The technical committee is made up of representatives from various European countries.

It is intended that the draft will be adopted as the European standard for sprayed concrete and embodied within the specification of the CEN standard on materials for the protection and repair of concrete structures, now being produced by CEN / TC104 / WG8.

The scope of the draft deals with the application of pneumatically placed concrete onto a surface. Application covers both wet and dry processes. Within the design and excecution of work, distinction is made between the following types of sprayed concrete- structural sprayed concrete; supporting sprayed concrete for mass rock and excavation support, surface improvement and repair.

The final agreed draft is expected to be published in the autumn. Further details can be obtained from : The Secretery, EFNARC, 241 high street, Aldershot, Hants, GU 11 1TJ. Tel : 0252 342072. Fax : 0252 333901.

Making High Quality Stabilized Soil Blocks

Certain products of Leicester has developed a strong, efficient, and easily operated machine for making building blocks from stabilized soil - the building material most widely used in developing countries.

Operating the "Elephant blockmaker" requires no special skills. It has been designed to produce a high output of densely compacted blocks at low cost, whether manually operated or powered by a diesel power-pack.

The machine is built for use on sites anywhere in the world and needs minimum, simple maintenance. The chassis has wheels with solid tyres for manoeuvrability on the site and jacks for stabilizing it during blockmaking.

Under manual operation the machine can produce 120 blocks an hour - sufficient for four squares meters of walling. By attaching a diesel hydraulic power pack the output can be increased to 200 or more blocks an hour.

The Elephant blockmaker

(British Overseas Development. No. 15, November 1991)

Coating will Fight Concrete Cancer

Formulated to combat concrete corrosion and carbonation, cryltane CF concrete coating system is available from the Research Laboratory for experimental Building (FEB).

The coating is compatible with concrete and most mineral substances and has good weathering characteristics, retaining its color in the most aggressive environments. It also protects against sulfur dioxide and carbon chloride.

(Construction Weekly, Vol. 3 No. 35, September 1991.)

Swell Remedy to Cover the Cracks

A self-sealing water proofing membrane that swells to heal hard-to-find cracks is available from Booth Engineering.

Swellite 1000 comprises a specially formulated mixture of velocity (a sodium bentinite high-swelling clay) and butyl rubber, bonded to high-density polyethylene. Water seeping through a crack activates the velocity / butyl layer, which within an hour forms an impermeable gel.

(Construction Weekly, Vol. 3, No. 32, 1991)

Plastic Additives to Prevent Cracks

Shrinkage cracking of pattern-impressed concrete can be prevented with the incorporation of a specially developed fiber, from Castle Building Products.

The micro-fine, mono-filament polypropylene fiber not only prevents cracking but is invisible in the finished surface.

(Construction Weekly, Vol. 3, No. 35, 1991)

ZAIRE

Training on Water Containers

The Armee du Salut Centre de Formation pour l' Artisanant Rural in Kinshasa 1, Zaire is conducting training course on ferrocement tanks and sand-cement jars. The project is directed towards rainwater storage in arid zone.

Seminar on ferrocement A' lemba, in Kasangulu, Zaire

Completing the Rain Water Cistern

(Information and photographs from Mr. Gracia Victor Matondo, Director, Armee du Salut Centre de Formation pour l' Artisanat Rural, B.P. 8636 Kinshasa 1, Zaire.)

International Ferrocement Society (IFS)

The birth of the International Ferrocement Society (IFS) was announced in Havana, Cuba on 25 October 1991 during the 4th International Symposium on Ferrocement by H.E. Mr. Jose Canete Alvarez, Minister of Industry, Materials and Construction of Cuba. Dr. Ricardo P. Pama, Vice-President for Development of the Asian Institute of Technology, Thailand was elected founding president. His mandate is to direct the activities of the IFS until the first general election. This election will be conducted as soon as IFS has 200 members or at the 5th International Symposium on Ferrocement in 1994 whichever comes first. Dr. Pama is internationally recognized for his work on ferrocement and was instrumental in setting up the International Ferrocement Information Center (IFIC).

The International Ferrocement Society (IFS) was founded to coordinate and to cater to the needs of practitioners, architects, engineers, and researchers on application, development and research on ferrocement with headquarters at the International Ferrocement Information Center at the Asian Institute of Technology, Bangkok, Thailand. Its aims are to unify experts, users, builders and manufacturers; to provide a forum for the exchange of ideas, enhance collaboration and cooperation; and also to promote the utilization of ferrocement.

The objectives of the Society shall be to promote the appropriate utilization of ferrocement as a construction material; to develop the full potentials of ferrocement specially in the interest of those whose needs are badly served by present day production methods; and to seek to unify testing procedures and design criteria for ferrocement.

In pursuance of this objective, the IFS shall:

- arrange periodic local meeting for discussion on ferrocement and related topics.
- arrange for conferences, symposia, seminars and lectures on ferrocement.
- adapt the *Journal of Ferrocement* as its official publication.
- cooperate with other international bodies involved with ferrocement.

The Governing council of IFS consists of the following:

Secretary : Lilia Robles-Austriaco (IFIC/AIT, Thailand)
Treasurer : Pichai Nimityongskul (SEC/AIT, Thailand)
Members : Douglas Alexander (New Zealand)
Kribanandan Gurusamy (Botswana)
B. de Hanai (Brazil)
Boris Mironkov (Russia)
Antoine Naaman (U.S.A.)
Paul Nedwell (U.K.)
Yoshihiko Ohama (Japan)
A. Olvera (Mexico)
Narayana Swamy (U.K.)
D.N. Trikha (India)
Hugo Wainshtok Rivas (Cuba)
Roberto Mattone (Italy)

The privileges of an IFS member are:

- A subscription to the *Journal of Ferrocement* which offers up-to-date information about ferrocement.
- An IFS discount card allowing members to take advantage of 20% discount to all IFIC publications

Dr. Ricardo P. Pama, IFS President (5th from right) and other participants with Mr. Jorge Mayo, Deputy President on Construction in the Executive Committee of the Government of the City of Havana (third from right).

Mr. Jose Canete Alvarez, Minister of Construction Material Industry (second from left) with (L-R) Mrs. Lilia Robles-Austriaco, IFS Secretary, Dr. Ricardo P. Pama, IFS president, Mr. Pedro Galeano, UNAIC president and Dr. Pichai Nimityongkil, IFS treasurer.

and services.

- Access to IFIC bibliographic database and reference collection.
- Priority right to participate in international symposia and other activities organized by IFS and IFIC in any part of the world at reduced fees.
- Membership listing in the *International Directory of Ferrocement Organizations and Experts.*
- Opportunities to work with IFS Committees.
- The right to vote in the election of IFS officers.
- A certificate of membership suitable for framing.
- A membership 'passport' numbered, signed, sealed, authenticated by the secretary and bearing member photograph.

For further information, contact: IFIC/AIT,G.P.O. Box 2754, Bangkok 10501, Thailand.Tel: 5160110-44 ext. 5864, Direct Line: 5245864. Fax: (66-2) 5162126, (66-2) 5245870.

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ABSTRACTS

FP 174 ASPECTS CONCERNING THE BEHAVIOR OF FERROCEMENT IN FLEXURE

KEY WORDS: beams (supports), bending, ductility, ferrocement, plates, reinforced concrete, ultimate moment

ABSTRACT: The ferrocement elements of plate and beam have a good behavior under working load due to the fact that the width of cracks appears to be very small than in the reinforced concrete. The good behavior at failure regarding the aspect of ductility and ultimate moment of the elements shows the capability of using ferrocement efficiently. The present paper presents some specific aspects concerning the behavior of ferrocement in a short time bending.

REFERENCE: Onet, Train; Magureanu, C.; and Vescan, V. 1992. Aspects concerning the behavior of ferrocement in Flexure. *Journal of Ferrocement* 22(1): 1-9

FP 175 DEFLECTION EVALUATION OF FERROCEMENT PLATES IN BENDING

KEY WORDS: bending, deflection, evaluation, ferrocement, plates (structural members), reinforced concrete, wire mesh

ABSTRACT: This paper presents some considerations about deflection evaluation of ferrocement plates in bending, when using large opening welded wire meshes.Flexion tests were made on forty specimens, with thickness of 15 mm to 35 mm and steel ratio of 100 kg/m^3 to 250 kg/m^3 of mortar. Square meshes (50 mm x 50 mm) and rectangular meshes (25 mm x 50 mm) with wire diameter of 2.5 m (f_{yk} = 600 MPa) were used.
Experimental plate deflections were compared with several theoretical formulations, mainly related to conventional reinforced concrete and ferrocement standards.

REFERENCE: Ballarin, A.W., and de Hanai, J.B. 1992. Deflection Evaluation of ferrocement plates in bending. *Journal of Ferrocement* 22(1): 11-16

FP 176 MATHEMATICAL MODE OF DETERMINATION OF CRITICAL CRACKING FORCE AT TENSION ZONE OF FERROCEMENT

KEY WORDS: ferrocement, hardening (materials), microcracks, reinforced concrete, setting, shrinkage

ABSTRACT: Generally, it has been established that during setting and hardening time of concrete the microcracks in the reinforced concrete-matrix composites have been formed, as a result of contact stresses on the contact surfaces "binder-aggregate" and as shrinkage effect and technological treatment. Cracks of this type are known in the literature as "structural microcracks". Exposing specimen of ferrocement to the action of axial tension it has been established that the structural microcracks have their direction being perpendicular to the tensile force and broadening themselves with the increase of the applied load.

REFERENCE : Walkus, R., and Gackowski, R. 1992. Mathematical Mode of Determination of critical cracking force at tension zone of ferrocement. *Journal of Ferrocement* 22(1): 17-26

FP 177 DURABILITY OF POLYMER FERROCEMENT

KEY WORDS: corrosion, durability, ferrocement, polymers

ABSTRACT: This paper discusses the durability of polymer-ferrocement in comparison with conventional ferrocement. The polymer-ferrocement, using a styrene-butadiene rubber latex , are prepared with various polymer-cement-ratios, and tested for accelerated carbonation, chloride ion penetration and accelerated corrosion. It is concluded from the test results that the carbonation resistance, chloride ion penetration resistance and corrosion-inhibiting property of the polymer-ferrocement are remarkably improved with increase in polymer-cement ratio.

REFERENCE: Ohama, Y., and Shirai, A. 1992. Durability of polymer ferrocement. *Journal of Ferrocement* 22(1): 27-34

FP 178 FERROCEMENT PREFABRICATED HOUSING: THE NEXT GENERATION

KEY WORDS: ferrocement, floors, housing, models, panels, prefabrication, walls

ABSTRACT: Ferrocement is a construction material ideally suitable for high levels of prefabrication. However, in order for ferrocement prefabricated products to successfully penetrate the housing sector, it should be demonstrated that high quality housing can be produced with ferrocement, in as an effective way as for low cost housing. This paper describes part of an ongoing investigation which attempts to address this concern. In a first study, published earlier, advanced manufacturing techniques were considered for the production of ferrocement housing units; this led to the development of a ferrocement housing system using standardized prefabricated U and box-shaped panels for the walls, floors, and roof of a typical house. It was assumed, in all cases, that joining of the panels can be properly achieved with bolted type connections. After a brief review of existing ferrocement housing systems (which address primarily the low-cost housing sector), this paper provides a brief summary of the progress achieved so far on the study of bolted connections. Two types of connection are identified, a shear-type and a moment-type connection. Test arrangement and test set-up designed to identify various failure modes are described. Typical load-deformation response curves are presented. It is hoped that experimental results will provide the basis for calibrating analytical models of the connections. Such models can be implemented in a computer program in order to investigate a large number of parameters, and eventually develop optimum connection configurations.

REFERENCE : Naaman, A.E., and Hammoud, H. 1992. Ferrocement prefabricated housing: the next generation. *Journal of Ferrocement* 22(1): 35-47

FP 179 STUDY OF A TWO STORY PRECAST FERROCEMENT MODEL BUILDING

KEY WORDS: buildings, ferrocement, floors, joints (junctions), models, roofs, serviceability, walls

ABSTRACT: Ferrocement is an ideal material for precast industrialized construction of low cost houses due to its high strength per unit weight . In some of the countries like Cuba [1], multi-story buildings using precast ferrocement elements have already been built successfully. However, ferrocement element being comparatively very thin, jointing needs special attention. Further, there is no reported scientific study on the behavior of ferrocement buildings as assembled from precast elements to ascertain their load carrying capacity particularly with respect to serviceability requirements.

This paper presents the results of an experimental study [2] of a two story precast ferrocement building under simulated vertical and lateral loads. The model building of size 1.5 m x 1.5 m in plan and 3.0 m high was assembled from precast elements in a two story construction. Similar ribbed elements have been used both as walling and floor/roofing elements. The vertical load was applied using concrete blocks on the first floor and through hydraulic jacks, keeping the ratio between the two loads constant through each load increment. The paper presents the deformations and the behavior of the building and discusses the efficacy of the bolted connections.

REFERENCE : Bhandari, N.M.; Trikha, D.N.; and Prakash, V.S. 1992. Study of a two story precast ferrocement model building. *Journal of Ferrocement* 22(1): 49-59

FP 180 WHAT IS FERROCEMENT ?

KEY WORDS: conferences, cracks, crack width, durability, ferrocement, specific surface, volume fraction

REFERENCE: Alexander, D. 1992. What is ferrocement ? *Journal of Ferrocement* 22(1): 61-65.

ABSTRACT :This paper is a postscript to the Havana Symposium on Ferrocement. Various aspects of ferrocement as cementitious composite and its diverse applications were discussed. Differences between ferrocement and reinforced cement mortar are highlighted. The problems of whether reinforced cement mortars is within the general definition of ferrocement is explored.

INTERNATIONAL MEETINGS

1-6 March 1992: **14th IABSE Congress on Civilization through Civil Engineering, New Delhi, India.** Contact: Mr. S.P. Chakrabarti, Secretary, Indian National Group of IABSE, IDA Building, Jamnagar House, Shahjahan Road, New Delhi-110011, India. Tel: 3716848, 386724.

24-26 March 1992: **3rd International Conference on Modern Techniques in Construction, Project & Engineering Management, Orchard Hotel, Singapore.** Contact : Mr. John S.Y. Tan, CI-Premier Pte Ltd, 150 Orchard Road # 07-14, Orchard Plaza, Singapore 0923. Tel: 7332922; Fax: 2353530.

3-15 April 1992: **Second National Concrete Engineering Conference, Chicago, U.S.A.** Contact: ACI Conference Register, American Concrete Institute, P.O. Box 19150, Detroit, Michigan 48219-0150, U.S.A. Tel: (313)532-2600, ext. 209. Fax: (313)533-4747.

3-8 May 1992 : **International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolonas in Concrete, Istanbul Turkey.** Contact : Mr. H.S. Wilson, P.O. Box 3065, Station C, Ottawa, Canada K1Y 4J3.

1-5 June 1992: **First International Conference on Fracture Mechanics of Concrete Structures, Colorado, U.S.A.** Contact : Mr. Marty Moser, Northwestern University, Evanston, Illinois 60208-3111, U. S. A. Tel : (708)491-4025. Fax : (708)467-1078.

7-10 July 1992 : **The Sixth International Conference on the Behavior of Offshore Structures, London, U.K.** Contact : Mr. Robert Gibbins, Boss 92 Secretariat, 2 Tavistock Place, London, U.K. WC 1H 9RA Tel : (071)837 6362. Fax : (071)837 0822.

13-15 July 1992 : **The International Symposium on Noteworthy Applications in Concrete Prefabrication. Singapore.** Contact : Mr. John S. Y. Tan, Symposium Director, CI-Premier Pte Ltd., 150 Orchard Road # 07-14, Orchard Plaza, Singapore 0923. Tel : 7332922. Fax : 2353530. Telex : RS 33205 FAIRCO.

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23-28 November 1992 : **9th International Congress on the Chemistry of Cement, New Delhi, India.** Contact : The 9th ICCC Secretariat, National Council for Cement and Building Materials, P.O. Box 3885, Andrews Gang, New Delhi 110049, India. Tel : 91-11-6440133, Telex : 031-66261 CRI IN. Telefax : 91-11-6468868.

27-30 October 1993 : **The Third Beijing International Symposium on Cement and Concrete. Beijing, China.** Contact : Mr. Wu Zhaoqi, Director of Cement Research Institute, China Building Materials Academy, Guanzhung, East Suburb, Beijing 100024, China. Tel : (86-01) 5761325. Fax : (86-01)5961713.

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B.K. Paul and R.P. Pama

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R.P. Pama and Opas Phromratanapongse

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Edited by R.P. Pama, Seng-Lip Lee and Noel D. Vietmeyer

This report is the product of the workshop "Introduction of Technologies in Asia - Ferrocement, A Case Study", jointly sponsored by the Asian Institute of Technology (AIT) and the U.S. National Academy of Sciences (NAS). Thirteen case studies on the 'State-of-the-Art' of ferrocement technology and applications in nine countries in Asia and Australia are presented. 106 pp., 59 illus.

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1. The complete manuscript should be written in English and the desired order of contents is Title, Abstract, List of Symbols, Main Text, Acknowledgements, References and Appendices. The Standard International System of Units (SI) should be used.

2. The manuscript should be typed on one side of the paper only (preferably 81/2" x 11" bond paper) with double spacing between lines and a 1 1/2 in. margin on the left.

3. Two copies of the manuscript and illustrations (one set original) should be sent to the Editor.

4. The title should be brief (maximum of 150 characters including blank in between words or other non-alphabetical characters) and followed by the author's name, affiliation and address.

5. The abstract should be brief, self-contained and explicit. The suggested length is about 150 words.

6. Internationally accepted standard symbols should be used. In the list of symbols Roman letters should precede Greek letters and upper case symbols should precede lower case.

7. Each reference should be numbered sequentially and these numbers should appear in square brackets [ ] in the text.

Typical examples are:

1. Broutman, L.J., and Krock, R.H. 1967. *Modern Composite Material.* London: Addison-Wesley Publishing Co.
2. Daranandana, N.; Sukapaddhanadhi, N.; and Disathien, P. 1969. Ferrocement for Construction of Fishing Vessels, Report No. 1, Applied Scientific Research Corporation of Thailand, Bangkok.
3. Naaman, A.E., and Shah, S.P. 1972. Tensile tests of ferrocement. *ACI Journal* 68(9): 693-698.
4. Raisinghani, M. 1972. Mechanical Properties of Ferrocement Slabs, M.Eng. Thesis, Asian Institute of Technology, Bangkok.

8. Graphs, charts, drawings, sketches and diagrams should be drawn in black ink on tracing or white drawing paper. Illustrations should preferably be drawn on 81/2" x 11" sheets. Photographs should be black and white prints on glossy paper and preferably 3 1/2 in. x 7 in. size.

9. Illustrations should be numbered consecutively and given proper legends and should be attached to the end of the manuscript.

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