Specialty Cellulose Fibers
for Cement Reinforcement

by K.D. Vinson and J.I. Daniel

Synopsis: This paper describes the investigation of a new range of cellulose fibers suited to the reinforcement of a Portland Cement matrix.

This investigation indicated that fibers selectively derived from high density summerwood are better suited for reinforcement than is the unmodified pulp which contains a large measure of fibers derived from springwood as well as summerwood.

Another cellulose fiber material, termed expanded fiber because of its finely fibrillated microstructure, was indicated to have potential as a processing aid. Expanded fiber displayed excellent suspending and retention properties and imparted relatively high uncracked strength to finished composites.

Overall, substantial performance differences were observed comparing tests on wet versus dry specimens and the long term durability was not evaluated. Despite these limitations, flexural stress/strain performance of the cellulose reinforced composites compared quite well to asbestos and glass fiber reinforced composites. The cellulose composites had substantially more ductility than asbestos cement; in this regard the load-deflection curve was similar to glass reinforced cement.

Keywords: asbestos; cellulose fibers; ductility; fiberboard; fibers; flexural strength; load-deflection curve; performance tests; portland cements; reinforcing materials
INTRODUCTION

Background

Asbestos for cement reinforcement consumed 1.5 million metric tons of fiber by early in this decade. Usage was principally for factory-made cement cladding panels and pipes produced in some 800-900 manufacturing units operating in virtually every country of the world. Consequently, the asbestos replacement activity during recent years has resulted in vast world-wide research into alternative cement reinforcement fibers.

Cellulose is widely regarded as offering one of the best cost/performance positions among the potential replacement fibers which include glass, carbon, aramid, acrylic, poly (vinyl alcohol), polypropylene, and others. Cellulose has been used for many years to some extent as an additive in the conventional asbestos cement industry; some of the asbestos cement replacement products utilize cellulose fibers as well. Cellulose fiber in these cases is sometimes used in small amounts, 1% or less by weight, in combination with the other fibers; in this role the cellulose contributes mainly processing benefits rather than reinforcement.

It is the purpose of this paper to report on an investigation designed to quantify the potential for cellulose fibers in general and, in particular, a new range of modified cellulose fibers to act as the sole reinforcement of a Portland Cement matrix.

Cellulose Fiber Preparation

It was a purpose of this investigation to focus on the effect of novel forms of cellulose fibers achievable by making modifications to the physical form of the fibers.

One of the modified fibers used in this study was made by a separation process by which a fraction composed predominantly of fibers of the summerwood type is created. Annual growth rings of coniferous (softwood) trees are comprised of low density, springwood, rings and high density, summerwood, rings. For illustration, the SEM micrographs in Figure 1 are two magnifications of a typical cross section of spruce, a Northern softwood. The thinner cell wall material of the springwood fibers compared to the summerwood fibers is clearly evident.
After the kraft pulping process, the fibers collapse into flat ribbons, with the springwood fibers being wider, but thinner than the summerwood fibers. Figure 2 is a photomicrograph of slash pine fibers after the pulping process. Slash pine is a Southern U.S. species and is the sole starting material for the cellulose fibers used in this investigation. Quantitatively, slash pine springwood fibers average 61 microns width at 12 microns cell wall thickness, while summerwood fibers average 37 microns width at 4 microns cell wall thickness.

Since there is little difference in the fiber length or in the density of the cell wall substance, a given mass of the summerwood fibers will contain about one-half as many fibers as a given mass of springwood fibers. However, the summerwood fibers have much higher fiber strength due directly to the greater mass of cell wall substance and indirectly to differences in the $S_2$ cell wall structure(2), in particular the average fibril angle(5).

The naturally occurring unweighted percentage of summerwood fibers in the slash pine pulp, designated SSK in the present investigation, is 55%. For the purposes of this study, a fractionated laboratory sample which contained 86% summerwood fibers was prepared (SSK-SUWD).

A second modified fiber type was prepared by reducing SSK fibers to a finely divided fibrillar material to examine the effect of extreme fine particle diameter on the reinforcement potential. Fibrillation is readily accomplished with naturally occurring cellulose fibers which are built-up of fibrils which can be separated and further sub-divided into smaller and smaller diameters owing to the high levels of molecular orientation. Indeed, a level of fibrillation is introduced by the action of beaters and refiners in common use by the paper industry to alter the drainage and bonding characteristics of paper pulps. A much higher level of fibrillation was used to prepare the modified fiber referred to as Expanded Fiber (EF) in this paper. In EF the fibrillation process was carried to extreme to render the constituent fibrils virtually completely separated.

The EF used in this investigation was prepared using SSK in a 1.5 liter horizontal media mill(3). A comparison of EF to SSK fiber prior to the fiber expansion process is illustrated by the micrographs in Figure 3, which display the fiber before and after the fibrillation process. The degree of fibrillation is best quantified by the tendency of an aqueous dispersion of EF to resist separating and settling from the aqueous medium. Specifically, the EF specimen in this investigation was fibrillated until a 0.1% dispersion of EF solids in water would settle to only 50% of its original volume upon quiescent standing for one hour.
Composites Preparation

Since fiber cement production technology based on the Hatschek process is widely available, it is highly desirable for a fiber to be compatible with this slurry dewatering method of composites formation(1). Consequently, we selected a laboratory, slurry dewatering process to investigate the potential of these fibers for reinforcement, essentially described as follows:

1. The cellulose fiber materials were dispersed with high speed mixing in an appropriate quantity of water for good dispersion. The solids content at this point varied from 1.75% to 4.6%.

2. Type 1 Portland Cement was added, with high speed agitation, to the fiber slurry in an amount necessary to give the desired final fiber content based on the weight of the dry ingredients.

3. The slurry was flocculated by adding an anionic polyacrylamide with gentle agitation, then poured into a mold fitted with an assembly of permeable screens, the finest being 100 mesh, to form the 15 inch (0.37 meter) square composite panels.

4. Excess water was removed through the screen using a vacuum dewatering device at 5 in (127 mm) mercury. Cement fines in varying amounts were visible in the drain water; this water was retrieved and the fines loss determined.

5. The moist cakes, which varied from 0.5 to 0.75 inch (13 to 19 mm) thick after dewatering, were pressed at 1200 psi (8300 kPa) for 3 minutes. Pressing was carried out in a hydraulic press designed to permit the excess water squeezed from the wet cake to be removed.

6. The procedures of steps 4 and 5 were adequate for all mixes except those containing EF only. In this case, the slurry would not dewater by vacuum and it was necessary to use the hydraulic press to remove all of the water. While dewatering in this way was a slow process, the maximum pressure of 1200 psi (8300 kPa) was maintained for only the 3 minute standard.

7. The composite boards after removing from the press were allowed to harden for four hours, then stored on edge and moist cured at 23°C and 100% relative humidity for seven days.

8. After moist cure, the boards were wet sawed into flexural test specimens of 2 X 12 X 0.5 inch (51 X 305 X 12.7 mm). These were stored at 23°C and 50% relative humidity until tested.
9. For each composite type, six specimens were tested dry and six specimens were tested wet. The dry specimens were tested as conditioned. The wet specimens were further prepared by soaking in water at $23^\circ C$ for 24 hours prior to testing.

10. All specimens were tested in third-point bending using a 0.09 in/min (2.3 mm/min) cross-head speed and a 10 inch (254 mm) free span. The resultant load versus deflection curve was used to compute the proportional elastic limit (PEL), the modulus of rupture (MOR), and the total flexural toughness. The PEL is calculated using the load value at the end of linearity of the load-deflection diagram and the MOR is calculated using the maximum load; these points on a typical load-deflection diagram are illustrated in Figure 7. The total toughness is the total area underneath the load-deflection diagram.

**RESULTS**

Fiber evaluations were normally completed in a series of three levels of addition, 4%, 8%, and 12% based on the weight of dry ingredients. A series of this type was prepared from each of the following types of fiber:

1. SSK fiber refined in a laboratory beater to 500 ml Canadian Standard Freeness (termed SSK-R)(4).

2. SSK-SUWD, unrefined.

3. SSK-SUWD, refined in a laboratory beater to a Canadian Standard Freeness of 500 ml (termed SSK-SUWD-R).

4. Expanded Fiber (termed EF).

During the course of the study, a dispersion benefit for composites prepared with the Expanded Fiber was recognized. To further investigate this finding a single composite was prepared at the level of 1.5% by weight EF, combined with 8% by weight of SSK-SUWD (termed SSK-SUWD/EF).

Hence, a total of thirteen sample types were prepared.

Figure 4 is a graphical representation of the flexural test results for PEL, Figure 5 for MOR, and Figure 6 for total toughness.

It should be pointed out that the specimens containing fiber types SSK-SUWD-R and SSK-SUWD/EF were tested at 49 days after preparation rather than the 28 day period common for the other specimens. Although all samples had the same 7 day moist cure period, the additional conditioning time could have affected the extent of cure of the matrix to a degree, although the effect should be negligible.
In addition to the flexural test results, some quantitative and qualitative observations were made relative to the processability of the fibers. For example, the fines retention varied considerably among the fiber types. Mixes containing SSK-R fibers had a 1.0%-1.5% loss of cement fines. With the SSK-SUWD fibers the loss increased markedly to 17%-18.5%. This system also had visibly poor distribution of fiber within the composites, especially when comparing one surface of the composite with the other surface. The loss of cement fines for the summerwood decreased markedly with refining to 1.8%-2.6%. The EF fiber mixes had a negligible fines loss. Finally, the mixed fiber system where the EF was blended with the unrefined summerwood (SSK-SUWD/EF) had a 3.8% fines loss.

Table 1 lists the water-cement ratios, the densities of the wet specimens, and the densities of the dry specimens.

CONCLUSIONS

1. The overall best performing fiber was the summerwood fiber type.

Although the unrefined version of the summerwood fiber is unacceptable from a processing standpoint, as illustrated from the poor fines retention and visibly poor dispersion in the composites, these processing difficulties with the unrefined summerwood were completely addressed by either the moderate refining to 500 ml CSF, or by blending the fiber with a small amount of the Expanded Fiber.

In both PEL and MOR, the refined summerwood version, and the summerwood/EF blend were the best performing fibers. In toughness, the refined summerwood fiber and the summerwood/EF blend were unsurpassed, and the unrefined version was clearly superior in this case.

In view of the lower surface area to volume ratio present in the summerwood fiber, this perhaps indicates that the specific bonding strength of cellulose to the cement matrix might be sufficient to approach the intrinsic fiber strength.

2. Expanded Fiber has potential use as a processing aid.

Composites containing Expanded Fiber possessed outstanding fines retention; as previously noted, the composites containing EF only had negligible fines loss. Although composites reinforced with EF lacked significant toughness, they did have excellent PEL. EF can thus be viewed as a matrix modifier which has a beneficial effect in suspending and retaining the solids materials in the slurry process.
3. While the relative performance among the various fiber types tested was essentially the same whether judged from wet testing or from dry testing, the absolute test values were quite different depending on the test condition.

Specifically, the PEL and MOR strengths were generally reduced by one-half in wet testing versus dry. Despite this reduction in load bearing ability, the total toughness values were actually observed to be about 40% greater when taken from the wet specimens compared to the dry specimens. Figure 7 is particularly illustrative of this point; it is a typical load deflection curve for the SSK-SUWD-R at 8% for both the dry and the wet condition. There are two possible explanations for this difference in behavior, wet compared to dry. First, the elastic modulus of cellulose is known to be reduced under wet condition compared to dry(5). Second, it is possible that hydrogen bonding between fiber and matrix is disrupted by the water directly resulting in more fiber pull-out.

4. The optimum fiber addition level depends upon the desired balance of properties; higher fiber levels yielding lower densities and higher toughness levels, but reduced PEL.

Generally, toughness goes up with addition of fiber. The only exception to this was the unrefined summerwood fiber which likely suffered from dispersion problems at the high, 12%, fiber level.

The PEL and MOR values were less predictable by fiber content. Wet PEL generally declined with higher fiber levels; dry PEL generally remained constant from 4-8% and declined at 12%. No uniform trends were evident with regard to MOR.

Water cement ratios increased and densities decreased with higher levels of fiber. These parameters depended more upon the fiber level of addition than upon the type of fiber.

5. Refining was shown to be highly beneficial, at least for the best performing fiber type, the summerwood fiber.

Refining improves the processability as illustrated by the summerwood fiber fines retention before and after refining; the 17%-18.5% fines loss decreased to 1.8%-2.6%. Further, refining greatly improved both PEL and MOR at some expense to toughness. This is probably a result of some fiber shortening and increasing in the fiber-to-matrix bonding with the mechanical surface modification caused by refining.
6. The best cellulose containing composite, compared to representative composites using the asbestos and glass fibers, were shown to have higher toughness than the asbestos cement at somewhat reduced load bearing ability; the load deflection diagram was similar to GFRC rather than asbestos cement.

Figure 8 is an assortment of load deflection curves intended to compare the best of the fibers tested in this investigation (SSK-SUWD-R) with asbestos cement(6) and with alkali resistant glass reinforced cement(7). Compared to asbestos cement, the cellulose cement composite panel possesses much greater energy absorbing quality at sharply reduced load bearing ability. In this regard, the characteristic curve for cellulose cement is more similar to that of GFRC.

RECOMMENDATIONS

The desirable properties of cellulose cement in view of the low cost of cellulose and its proven compatibility with the Hatschek process would indicate that there might be many applications for panel products based on cellulose fibers in general, with specific forms of cellulose capable of substantially improving composite properties.

No attempt was made in this investigation to evaluate the durability of the cellulose cement composites. A durability investigation should be completed.

The vast differences in properties of dry versus wet test specimens need to be taken into account by designers using the composites. A further modified fiber or system which would overcome these differences would be desirable.

ACKNOWLEDGMENTS

The cement reinforcement study was carried out in the laboratories of Construction Technology Laboratories in Skokie, Illinois; Mr. Eric D. Anderson of CTL assisted as a co-investigator on this project. Mr. Donald M. Schultz and Mr. Henry G. Russell of CTL, and Dr. Richard G. Fisher and Dr. B. Jerry L. Huff of Procter & Gamble Cellulose provided valuable input as members of the project steering committee. Dr. Arthur Tallentire of Vitrocem Consultancy Services, Manchester, assisted in the preparation of this paper.
REFERENCES


### TABLE 1--Summary of Specimen Properties After Pressing

<table>
<thead>
<tr>
<th>FIBER TYPE</th>
<th>FIBER CONTENT (% BY WEIGHT)</th>
<th>WATER-CEMENT RATIO</th>
<th>WET DENSITY ($\text{lb/ft}^3$)</th>
<th>DRY DENSITY ($\text{lb/ft}^3$)</th>
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<tbody>
<tr>
<td>SSK-R</td>
<td>4</td>
<td>0.33</td>
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<td>118</td>
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<td></td>
<td>8</td>
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<td></td>
<td>12</td>
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<td>113</td>
<td>98</td>
</tr>
<tr>
<td>SSK-SUWD</td>
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<td>117</td>
<td>109</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>0.57</td>
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<td></td>
<td>4</td>
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<td>129</td>
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</tr>
<tr>
<td>SSK-SUWD-R</td>
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<td></td>
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<td>132</td>
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<td>EF</td>
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<td>SSK-SUWD/EF</td>
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<td>117</td>
<td>109</td>
</tr>
</tbody>
</table>

SI equivalent: $1 \text{lb/ft}^3 = 16.05 \text{ kg/m}^3$
Fig. 1--Scanning electron micrographs of cross section of Spruce Wood at two magnifications; illustrating the cell wall differences between Springwood and Summerwood Tracheids
Fig. 2—Light micrograph illustrating Summerwood (dark lower fiber) and Springwood Fibers (two lighter upper fibers) after Kraft Pulping.
Fig. 3--Light micrographs of SSK fiber before expansion (above) and after expansion (below) illustrate the effect of the expanded fiber process.
**WET TESTS**

![Wet Test Graph]

**DRY TESTS**

![Dry Test Graph]

Fig. 4—Comparison of proportional elastic limit values of the several fiber types as a function of fiber loading by weight.
Fig. 5--Comparison of modulus of rupture values of the several fiber types as a function of fiber loading by weight.
Fig. 6—Comparison of total toughness values of the several fiber types as a function of fiber loading by weight
Fig. 7--Comparison of load/deflection behavior of specimens containing 8% SSK-SUWD-R, tested wet and dry
Fig. 8--Comparison of stress/deflection behavior of Asbestos Cement (1) and Alkali Resistant Glass Reinforced Cement (2) to the Refined Summerwood Cellulose Reinforced Cement (All tests apply to wet specimens.)
Performance of Non-Asbestos Fiber Cement Sheeting

by J.G. Keer

Synopsis

The production of a polypropylene-reinforced cement material marketed as an alternative to asbestos-cement is outlined. Typical tensile stress-strain curves of a number of alternative materials are compared with asbestos-cement. The load-deflection characteristics of corrugated sheets made from non-asbestos materials are also presented and discussed. The non-asbestos materials are generally much less brittle than asbestos-cement although they have a lower first-cracking strength. The pseudo-ductile behaviour exhibited, with multiple cracking before the ultimate load is reached, means that permissible loads in service must not be based solely on ultimate loads, but on cracking and possibly deflection criteria. Less well-defined stresses arising during installation and from restrained moisture movements, which may crack the non-asbestos materials, are likely to be critical for the effective performance of new sheeting materials.

Keywords: asbestos; cladding; cracking (fracturing); fiberboard; fiber reinforced concretes; performance; polypropylene fibers; polyvinyl alcohol; production methods; standards
Autobiographical Note

Dr Jeffrey G. Keer is a Senior Lecturer in the Department of Civil Engineering at the University of Surrey. He is a member of the Construction Materials Research Group, with particular interests in fibre cements and concretes and the durability of concrete structures. He has been a member of RILEM committee 49 TFR on Test Methods for Fibre Cements and is currently a member of RILEM committee 102 AFC on the Ageing of Fibre Cements.

INTRODUCTION

As sales of asbestos-cement have declined in Europe and North America in recent years, an extensive research and development effort has been devoted to producing alternative fibre cements to maintain the cement share of the roofing and cladding market. A number of alternative fibre cement materials are now marketed in Europe, although other cladding types have made severe inroads into the asbestos-cement market. In the U.K. for example, in 1975 about 40% of roof cladding for industrial buildings was asbestos-cement with less than 10% being steel sheeting. In 1986 the comparable figures were about 10% asbestos or fibre cement and about 65% steel (1).

Asbestos is appreciated as a remarkably good reinforcing fibre. The strength, stiffness, fineness and affinity for cement of the fibres contribute to the enhancement of the apparent tensile cracking strain of the cement matrix to values well in excess of 1,000 x 10^{-6}. Furthermore, the reinforcing effect is maintained over long periods of natural exposure. The manufacturing method ensures two-directional strength in sheets although there is some preferential (and beneficial) alignment in the direction of machine flow. More importantly in the manufacturing process, the asbestos fibres control slurry drainage ensuring that the water content is reduced without loss of cement. However, asbestos-cement is not a perfect fibre-cement cladding material; it is a brittle material and its lack of impact strength has been dangerous.

Since there were 208 major asbestos-cement sheet machine installations in Western Europe alone in 1979 (2), it is not surprising that many developments sought alternative reinforcement that would allow existing machinery to be used. This has resulted in composites in which the primary reinforcing polymer fibres (such as polyvinyl alcohol (PVA) fibres) must be combined with cellulose fibres or even with some asbestos (as in Japan) in order to provide the same control of slurry drainage during processing. Furthermore, the fibres must be short with the attendant problem of ensuring adequate bond so that reinforcement occurs.
A different approach has resulted from the development of the fibre cement, initially at the University of Surrey by Hannant et al., (3, 4) in which the reinforcing fibres are continuous opened networks of fibrillated polypropylene. The novel production technique required is outlined in this paper. The performance under load of this material and other alternatives to asbestos-cement are compared with asbestos cement. Possible problems associated with the performance and specification of the alternative materials are discussed.

PRODUCTION OF SHEETING CONTAINING CONTINUOUS POLYPROPYLENE NETWORKS

Fibrillated polypropylene film is normally produced in spools in the unopened form. In initial laboratory production, the film had to be held open to prevent springback and to allow the matrix to penetrate between fibrils. For commercial production a dimensionally stable, opened form is required, which can be fed from reels directly to the impregnation stage. This stable form is achieved by transversely stretching layers of fibrillated film and stabilising the network by thermal treatment at a temperature below that of the softening of the polymer (5). A pack of several layers of film can then be welded at the edges for subsequent use. Two-dimensional reinforcement of packs is obtained by a process which intersperses orthogonal networks between longitudinal networks (6). A typical network pack which results may be 1.2m wide, (48 in) containing eight layers longitudinally, four layers transversely and weighing 100g/m² (0.18 lb/yd²) (Fig 1). Stable opened one and two-directional networks are available commercially under the trade-name Retiflex (a registered trade name of Moplefan SpA). The film has a modulus of elasticity of about 16 GN/m² (2.3 x 10⁶ psi) and an ultimate tensile strength of about 550 MPa (80000 psi).

Two methods of production of flat or corrugated sheet are under development. The essential difference in the processes lies in the manner in which the two-dimensional polypropylene networks are impregnated by the cement matrix. In one process (Fig 2) (7), the cement matrix is deposited in an accurately controlled thin layer onto a porous, continuous belt by means of matrix laying boxes. The networks are fed and laid onto the matrix by means of special feeding and laying devices. The networks are impregnated by the cement paste by compacting and dewatering devices. A new layer of cement paste is then deposited onto the previously impregnated network and the cycle of operation repeated until the desired sheet thickness is obtained. For 6-6.5mm (0.25 in) thick sheet three or four layers are required depending on the type of network chosen. Additional dewatering follows, and the surface of the flat green sheet is pressed and finished with an upper porous
belt. The flat sheet is cut, corrugated if required and cured in a similar manner to asbestos-cement sheeting. The objective is to achieve production rates similar to those of asbestos-cement machinery.

In the second method (Fig 3) (8), the network pack is guided onto a moving felt band (which may be flat or corrugated in profile) and then impregnated by spraying the cement matrix onto it. A suction system enables excess water to be extracted from the sheet. Figure 3 shows three mesh impregnation stations. As in the previous process, the number can vary depending on the networks used and the thickness of sheet required. A compression device compacts the sheet, reducing its porosity and smoothing the surface. The sheet is then further processed as before. Sheets produced by Fibronit (8) have been on the market in Italy for more than one year.

The published range of cement matrices for both processes are similar in composition:

<table>
<thead>
<tr>
<th>Component</th>
<th>Range</th>
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<tbody>
<tr>
<td>Cement</td>
<td>100</td>
</tr>
<tr>
<td>Water</td>
<td>35-45</td>
</tr>
<tr>
<td>Fillers (fine silica sand, ground marble, pfa)</td>
<td>25-40</td>
</tr>
<tr>
<td>Workability aid</td>
<td>0.5-0.7</td>
</tr>
</tbody>
</table>

COMPARATIVE TESTS ON NON-ASBESTOS FIBRE CEMENTS

A test programme has been undertaken on alternative materials to asbestos-cement to establish material characteristics and performance under load of corrugated sheets. The materials tested were:

A: a compressed and autoclaved calcium silicate/cement product reinforced mainly with cellulose fibres and bought ex-stock.

B: a cement product reinforced by PVA and cellulose fibres, produced on modified Hatschek machinery and bought ex-stock.

C: a product similar to B but fully compressed for use as slates.

D: another cement product reinforced by cellulose and PVA fibres, made on modified Hatschek machinery and bought ex-stock.

E: the cement product reinforced by continuous polypropylene networks. The results presented here are from tests on hand-made laboratory sheet, not commercially produced sheet, although the behaviour is broadly similar. The
composite tested contained 5% by volume of polypropylene in the longitudinal direction parallel to the corrugations and about 3% by volume transversely. The matrix composition was similar to that given above.

F: for comparison, a typical asbestos-cement bought ex-stock.

The precise compositions of the commercially obtained materials A-D and F is unknown.

**Tensile stress-strain characteristics**

Tensile stress-strain curves are the most appropriate characteristics to assess the reinforcing effect, pseudo-ductility and energy absorption capability of the various products. It is known that products containing cellulose can be sensitive to the moisture condition at test, so products A to D were tested in wet and dry states. Specimens were 300mm long by 25mm wide by 5-8mm (12 in x 1 in x 0.25 in) thick strips and were tested in an Instron 1122 machine with strain recorded via a clip-on extensometer.

Typical tensile stress-strain curves for the various materials are shown in Figure 4. Table 1 summarises important properties, including density, water absorption and the volume of pores accessible to water. It can be seen from Figure 4 and Table 1 that the compressed materials A and C have a higher dry strength, a much lower strain to failure than the other alternative materials and a shape of curve similar to asbestos-cement. The effect of cellulose fibres is to reduce the wet strength of products A, B and C, but not of D. The unpressed, non-autoclaved materials B and D have a relatively low tensile strength but, as the next section indicates, give adequate performance in flexure as corrugated sheeting due to the increased ductility in comparison to asbestos-cement. Multiple cracking could only be seen in materials B, D and E and was very closely spaced for B and D, giving the impression of general disintegration of the matrix, possibly due to the angled fibres, although some discrete individual cracks could be seen under the microscope. Material E has a very high strain to failure compared to the other materials and was the only material to exhibit the stage following multiple cracking, when the continuous fibres alone are stretched as they slip through the matrix.

**Performance of corrugated sheets**

The load-deflection relationships of the corrugated sheets manufactured from the various materials are shown in Figures 5 and 6. Sheets of materials A, B and D were tested wet to simulate the worst conditions for these materials. The sheets in Figure 5 were
standard corrugated profiles (overall depth about 55mm (2.2 in)) tested over a span of 1.38m (54 in), the common span for general roofing in the U.K. The section moduli of the sheets are given on the Figure, together with the apparent flexural tensile stress, $\sigma_f$, at maximum load. The load is expressed as a uniformly distributed (u.d.) load per unit area of sheet between supports, and the deflection is the average value at midspan. The test procedure has been detailed elsewhere (4).

In Fig 5, the alternative materials are markedly more ductile than asbestos-cement (a-c). The a-c and material A sheets were the only sheets which actually broke into two pieces by a single crack propagating in a brittle fashion across the sheet with no multiple cracking apparent. The other sheets were unloaded at rather arbitrary deflections as the testing was carried out over a period of years as sheets were obtained. Certainly the material E sheets could have sustained much higher deformations without breaking in two. The load at the limit of proportionality (L.O.P.) can be determined in accordance with a standard procedure, such as that in Danish recommendations (9), for the sheets exhibiting pseudo-ductile behaviour. This load is in excess of 2.8 kN/m² (41.8 lb/ft²) for all sheets, a load which corresponds to a flexural tensile stress in the sheets of about 8MPa (1160 psi), which is approximately the load at which the materials crack in direct tension (Fig 4). Note that at the L.O.P. deflections are small and should not present a serviceability problem. The values of $\sigma_f$ at failure or maximum load reached in the test are close to the value of 15.7MPa (2280 psi), which has been a recommended value for asbestos-cement. These stresses, obtained by dividing the bending moment by the section modulus, do not, of course, represent real stresses because the position of the neutral axis changes in the post-cracking stage. All sheets withstood a load of 4 kN/m² (83.6 lb/ft²).

Fig 6 shows the performance of two sheet materials, B and D, with a deeper sheet profile (about 90mm (3.5 in)) designed to span over 1.8m (71 in), and tested over this span. The maximum load of the sheet material B is considerably greater than that of material D, although the apparent flexural tensile stress is lower. The higher load is the result of the more efficient profile shape of sheet B, which has a section modulus 60% greater than sheet D, although the cross-sectional area is only 5% greater.

Sheets of materials B, D and E, which multiply cracked but did not break, were soaked on the top surface after loading. Although cracks were very fine, lines of dampness quickly became visible on the underside of the sheets.
STANDARDS FOR NON-ASBESTOS FIBRE CEMENT SHEETING

Loading

A problem which has not yet been fully resolved is the specification of performance under load of the new fibre cements. Some recent proposals still refer to the measurement of a breaking strength or load only (10, 11). Since the alternative materials exhibit pseudo-ductile behaviour with the matrix possibly undergoing multiple cracking before the ultimate load is reached, permissible loads in service must not be based solely on ultimate values from tests (as with asbestos-cement) but on cracking and possibly deflection criteria. An obvious approach is to define the permissible uniformly distributed load $P_p$ at a particular span as:

$$P_p = \frac{\text{Load at L.O.P.}}{\gamma_1} \quad \text{or} \quad \frac{\text{Load at ultimate (max. load)}}{\gamma_2}$$

whichever is less, where $\gamma_1$ and $\gamma_2$ are safety factors and the load at L.O.P. could be assessed as in the Danish Standard (9). In fact there is a British Standard in existence, BS 5427 (12), for sheeting which already adopts this approach but which is not widely used in the fibre cement sheet industry. It suggests a $\gamma_2$ factor of 2 for sheets which fail by fracture and 5.0 for glass reinforced polyester sheets. Appropriate values for fibre cement sheets might be 2.5 for $\gamma_2$ and 1.25 for $\gamma_1$.

It is also important that concentrated loads are resisted without damaging sheets by matrix cracking. It is ironic that the high impact resistance of the alternative materials encourages the abandonment of traditional practice, such as the use of crawling boards, and more abuse of sheets during installation, although most sheet manufacturers will still recommend use of crawling boards for access. Concentrated loads can be a more demanding load criterion than u.d. loads. Fig 7 shows that the measured longitudinal strain distribution across an a-c sheet under an edge concentrated load of 0.9kN (200 lb), the usual concentrated load requirement (12), is more critical than that under a u.d. load of 2 kN/m$^2$ (42 lb/ft$^2$) or a total load of about 3kN (670 lb). While any cracks formed during installation do not allow water to drop through sheets and may heal autogenously (13), nonetheless they can be unsightly and undesirable on the underside, particularly when highlighted by absorbed moisture.

Moisture movements

One aspect of sheet performance which new standards are addressing (and a-c standards did not) are the significant stresses which may be induced in sheets due to wetting and differential
moisture movements. Draft ISO proposals for short corrugated sheets and flat sheets require sheets, fixed to a frame in standard fashion, to be subjected to a rain/heat cycling regime without showing signs of cracking or delamination (10, 11).

Fig 8 shows the deflection at the centre of corrugated sheets over a period of time during which the top surface of the initially dry sheets has been wetted with a saturated cloth. The sheets are free to deflect upwards, simply resting on supports under each corrugation at the sheet ends. Fig 8 shows results from two materials, B and E. The sheets deformed into the shape of an arch in the transverse direction, with little relative deflection along the line of any corrugation. The maximum deflections, for a wetting period of six hours, were 26mm (1 in) for the sheet B and about 16mm (0.6 in) for the sheet E. The wetting cycle in the draft ISO standard is 2 hours 55 minutes so that free, unrestrained deflection over this period would also be large. In Fig 8 deflections reduce as the sheet dries out and would also reduce eventually if the saturated cloth had remained in place, since the moisture gradients will reduce. The restraint of fixings on these free deformations induce significant stresses in sheets, of similar magnitude to those induced by service loads (14). Asbestos-cement behaves similarly and is known to crack in thunderstorms after long spells of dry weather. At the end of four wetting cycles of sheet B, microcracks had begun to appear even without external restraint of sheet deformation. Bearing in mind the higher cracking strength of asbestos-cement compared to the alternative materials, there is more work required in this area to relate laboratory assessments of moisture effects to satisfactory field performance.

One approach which has been considered to improve the performance of sheeting in some environments is the application of a waterproofing silane treatment to the surface of sheeting. This has three potential advantages:

(i) reduction in water absorption and hence, induced stresses due to restrained deformation. Fig 8 shows that for both materials, pretreatment with an alkylalkoxysilane product considerably reduces free deflection (in the sheet of material E to a negligible amount).

(ii) freeze-thaw durability can be enhanced (15).

(iii) treatments may be successful in preventing moisture penetration through very fine cracks, although work at Surrey indicates that this may not be a fruitful approach when cracks have formed post-treatment (16). The development of decorative coatings which are capable of crack-bridging and which limit moisture absorption into sheets is another area where improved sheeting performance in certain environments can be obtained.
Durability of new fibre cements

Appropriate methods of test for the durability of fibre reinforced cement products are currently being considered by ISO (TG 77) and RILEM (102 AFC) international committees. The problem is a complex one, because of the range of fibres involved and the variety of potentially degrading mechanisms. Also, with the introduction of new products, there is the need to have confidence in accelerated testing procedures whilst long-term results from natural exposures are being collected. One possible degradation mechanism of new fibre cements containing polymer fibres is oxidation of the polymer, which can lead to loss of performance, although antioxidants can be incorporated into the fibres. For the material containing polypropylene networks, accelerated tests on composite specimens at temperatures of 140°C, 120°C, 100°C and 80°C in air-circulating ovens have been carried out and specimens tested in tension after various exposure times. An Arrhenius plot is used to predict the lifetime at ambient temperatures, which has been estimated as well in excess of thirty years (17).

Real time tests, currently approaching ten year results, have also been carried out on polypropylene network reinforced specimens exposed to natural weathering, indoor exposure in laboratory air and underwater storage. The type of exposure had virtually no effect on composite strength and failure strain for times up to five years, and the cracking strength increases with age (Fig9)(17). At the present time, there is little published detailed data on the ageing of the other fibre cements referred to in this paper, although trade literature indicates design lives in excess of thirty years.

CONCLUDING REMARKS

1. Alternative fibre cements to asbestos-cement are more ductile than asbestos-cement, with different tensile stress-strain and load-deflection characteristics.

2. New international standards need to base permissible loads on loads at the limit of proportionality as well as on ultimate loads, because of the quasi-ductile nature of some alternative materials.

3. The alternative materials have lower cracking strengths than asbestos-cement and careful attention must be paid to the avoidance of cracking due to installation stresses and stresses due to restrained moisture movements. These stresses are less well-defined than those induced by normal design loading.
ACKNOWLEDGEMENTS

The author would like to thank John Keating, Rachel Scott, Dr David Hughes, Dr Armen Baroonian and Dr David Hannant for their contributions to this paper.

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<table>
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<th>MATERIAL</th>
<th>Elastic Modulus</th>
<th>Tensile strength</th>
<th>Failure strain</th>
<th>Relative density (saturated)</th>
<th>Water absorption</th>
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<td>B. PVA &amp; cellulose fibres</td>
<td>Wet 15.0</td>
<td>6.4</td>
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Fig. 1—Retiflex two-dimensional polypropylene networks
A Matrix laying boxes
B Network uncoilers
C Network embedding into matrix and compacting
D Final pressing
E Vacuum boxes

Fig. 2—Production process using continuous polypropylene networks laid onto matrix (7)

Fig. 3—Production process in which the matrix is sprayed onto continuous polypropylene networks (8)
Fig. 4—Comparative tensile stress-strain curves
A. calcium silicate matrix reinforced mainly with cellulose
B. PVA + cellulose reinforced cement
C. as B but fully compressed
D. another PVA and cellulose reinforced cement product
E. polypropylene-reinforced cement
F. asbestos-cement
Material E(1) 
\( \sigma_f = 14.4 \text{MPa} \)

Material E(2) 
\( \sigma_f = 13.3 \text{MPa} \)

Material A 
\( \sigma_f = 15.4 \text{MPa} \)

Material B 
\( \sigma_f = 15.6 \text{MPa} \)

Section moduli

- B: \( 82 \times 10^3 \text{mm}^3/\text{m} \)
- A: \( 71 \times 10^3 \text{mm}^3/\text{m} \)
- E(1): \( 99 \times 10^3 \text{mm}^3/\text{m} \)
- E(2): \( 77 \times 10^3 \text{mm}^3/\text{m} \)
- A-C: \( 90 \times 10^3 \text{mm}^3/\text{m} \)

Fig. 5--Load-deflection curves of various corrugated fibre-cement sheet materials, span 1.38m
A. calcium silicate matrix reinforced mainly with cellulose fibre
B. PVA + cellulose reinforced cement
E. polypropylene-reinforced cement
A-C. asbestos-cement
Thin-Section FRC and Ferrocement

Section moduli tension

Material B = $225 \times 10^3 \text{mm}^3/m$

$D = 139 \times 10^3 \text{mm}^3/m$

$\sigma_f = 8.7 \text{MPa}$

$\sigma_f = 10.4 \text{MPa}$

Fig. 6--Load-deflection curves of two deep profile corrugated fibre cement sheet materials, span 1.8m

B. PVA + cellulose reinforced cement type 1

D. PVA + cellulose reinforced cement type 2
Fig. 7—Comparison of strains induced in corrugated sheet by uniformly distributed load of 2kN/m² and concentrated load of 0.9 kN on edge corrugation, span 1.38m
Fig. 8--Deflections of corrugated sheets due to wetting of top surface
Fig. 9--Tensile stress-strain curves of specimens containing continuous polypropylene networks after five years natural weathering compared with one month under water
Fabrication and Properties for a New Carbon Fiber Reinforced Cement Product

by T. Ando, H. Sakai, K. Takahashi, T. Hoshijima, M. Awata, and S. Oka

Synopsis: The fabrication, properties and application of carbon fiber reinforced cement (CFRC) product made of coaltar pitch-based high-performance carbon fiber are presented. The experiments were conducted by mixing the chopped carbon fiber strands with cement and sand to obtain CFRC. The mixing test results revealed that this type of carbon fiber disperses quickly and uniformly in ordinary mortar. No special type of mixer is required. In order to optimize the characteristics of CFRC, experimental analysis was conducted on batches made in a mortar mixer regarding the fiber properties and mix proportion. The relationships of these parameters to the mechanical properties were examined. It was revealed that the parameters determining the apparent viscosity $F$ (flow index) of CFRC slurries are fiber diameter $\phi$, filament number $n$, specific surface area $S$ and fiber volume fraction $V_f$. It was also revealed that the parameters determining the strength of the hardened body were fiber tensile strength $TS$ and $V_f$. The flexural strength of the 20mm thick CFRC is about 3~4 times greater than that of plain mortar. This CFRC is also stronger and durable than other FRC under the same conditions. High productivity, lightweight and weatherability characterize this new CFRC. These characteristics being appreciated, precast CFRC products have been increasingly used in construction in Japan. Some detailed descriptions of the practical applications are also made.

Keywords: carbon fibers; cladding; durability; fiberboard; fiber reinforced concretes; flexural strength; mixers; mixing; production methods; tensile strength; tests; workability
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INTRODUCTION

This paper describes the carbon fiber optimization, properties and application of the new CFRC fabricated in an ordinary mortar mixer. Steel fiber (SF) and glass fiber (GF) have been used to reinforce cement matrices. The problems associated with steel fiber are that they are difficult to disperse and hard to trowel when finishing. In addition, rust staining of the concrete surface prevents extensive outdoor application. Glass fibers, on the other hand, have been known to lose tensile strength and ductility with aging(1). In this respect, carbon fiber offers more favorable properties especially in terms of thermal resistance, weatherability as well as mechanical properties. Attempts have been made to use carbon fiber for building and construction purposes in Japan. In 1987 lightweight CFRC curtain walls, 32,000 m² in all, were installed to a 37-story office building in Tokyo(2).
So-called "general-purpose pitch-based carbon fiber" (GP-CF) with low tensile strength and modulus had solely been used. These materials tend to form fiber balls on being mixed with cement matrix. Without a special type of mixer (Omni mixer), it has been impossible to uniformly disperse this type of fiber. "High-performance pitch-based carbon fiber" (HP-CF) developed recently can solve this problem. The authors recently succeeded for the first time in dispersing the chopped carbon fiber uniformly in cement matrices with an ordinary mortar mixer with less amount of dispersant (3,4). This has dramatically facilitated the production efficiency of CFRC products. Based on the experiments, the results of carbon fiber parameter analysis are also presented.

EXPERIMENTS

Materials and Mix Proportion

Two types of carbon fiber were used and they were both pitch-based. The general-purpose carbon fiber (GP-CF) was about 15 μ in diameter and 6mm in length. The average monofilament tensile strength and modulus are 60kg/mm² and 3 ton/mm² respectively. The high-performance carbon fiber was 17 μ in diameter and 18mm in length. The average monofilament tensile strength and modulus are 200kg/mm² and 20ton/mm² respectively. The cement, sand, water and additives are listed in Table 1. Early strength Portland cement was used. The sand was silicate sand finer than 0.5mm in diameter. The three types of additives used were air-entraining water-reducing agent, dispersant and defoamer. In Method A, the amount of dispersant used was 1.0% by weight, while it was 0.25% in Method B.

Batch Preparation

Fig.1. shows two types of mixing methods and time required in the experiments. In both cases, carbon fiber, cement and sand were dry-mixed at first. Water and admixtures were then added and mixed again. In Method A, 1/2 of sand was provided at the dry-mixing stage, another 1/2 being added at the wet-mixing stage. In Method B, on the other hand, the sand was added all at the same time. The mix proportion of the Method B using a 70 liter mixer was reproduced in a larger mortar mixer (750 liter). The flow index of CFRC slurry was determined by flow table test according to the current JIS-R 5201 "Test Method for Physical Properties of Cement."

Specimen Size and Preparation

CFRC slurry was cast into a wooden mold and was taken out of the mold 24 hours later. The specimens were 20mm thick, 40mm wide and 320mm long. These were subjected to 3 point flexural test. The specimens were cured in air at 20 °C, 60%RH for 7 days. The span length for flexural testing was 260mm.
Test Results

The test results are shown in Table 2. The fiber dispersion is equally uniform in both methods. No fiber balls are observed as shown in Fig.2. In Method A, however, 4 times greater amount of dispersant (carboxyl methyl cellulose) than in Method B was needed to achieve uniform fiber dispersion. Excessive addition of the dispersant hindered trowel finishability. In Method B, on the other hand, the trowel finishability was significantly improved by the lowered viscosity. Higher flow index (workability) was obtained by Method B. Moreover, it required shorter mixing time. These facts indicate the advantages of Method B in terms of production.

In order to examine the reproducibility of the test results, mixing was conducted by a larger mortar mixer with the capacity of 750 liter. It is considered that the result in Table 3. essentially reproduces the results in Table 2. in terms of workability and flexural strength, although a little longer mixing time was required. The difference in mixing time is mainly due to the difference of rotation rate: 46 rpm with the 70 liter mixer and 27 rpm with the 750 liter mixer. It is concluded that the difference in mixer capacity does not cause any special problems in reproducibility.

Significant findings can be summarized as follows:

1. Method B is superior to Method A in terms of trowel finishability and mixing time.

2. Though no conspicuous difference is observed in the fiber dispersion between the two mixing methods, the CFRC obtained by Method B is superior to Method A in the flexural strength by 57%. The difference is considered to be mainly due to the difference in the tensile strength of fibers.

3. No trouble was encountered to produce the CFRC by Method B with a larger mixer. Industrialized CFRC production has therefore become possible by Method B.

DATA ANALYSIS

The CFRC slurries made by the premix method should be workable enough to be shaped into various products. So far, the addition of various admixtures like superplasticizers, air-entraining agents, etc. have been attempted, as well as the adjustment of water cement ratio. However, the alternative approach optimizing the carbon fiber properties has not yet been given sufficient attention. The influence of carbon fiber properties on the workability and flexural strength is investigated by means of the regression analysis in order to optimize the carbon fiber and CFRC.
Flow Analysis

Typical relations between carbon fiber properties and flow index of CFRC slurry as a result of single regression analysis are shown in Fig.3. through Fig.7. The range of fiber parameters are chopped length $L = 3 \sim 25$ (mm), $\phi = 7 \sim 20$ ($\mu$), $TS = 60 \sim 300$ (kg/mm$^2$) and TM$=3\sim 24$(t/mm$^2$). The notation used is explained at the end of the text. It was revealed that carbon fiber properties highly correlated with slurry flow $F$ are $\phi$, TS, n and Vf. Each of these parameters will be considered in turn.

Fig.3. shows that fiber diameter $\phi$ is directly proportional to $F$. The decrease of $F$ is accompanied by the increase of TS as shown in Fig.4. This is mainly due to the general characteristics of the carbon fiber whose TS reduces with the increase of $\phi$. In other words, these carbon fiber properties (TS and $\phi$) have a strong interrelation. Single regression analysis with $F$, therefore, reveals the high correlation between TS and $F$. It is also indicated that TS can be replaced with $\phi$ as a result of multiple regression analyses as will be described later. Fig.5. and Fig.6. show the relationships between the fiber number $n$ and $F$, and the specific fiber surface area $S$ and $F$ respectively.

Among all these properties, the highest correlation with $F$ was obtained by $S$. This result can be explained as follows. It is clear that the thinner fiber has higher $S$ at the same Vf. The increased Vf of the same diameter and length of fiber, on the other hand, increases $S$, too. Therefore $\phi$ and Vf contribute to the reduction of $F$. As a result, $S$ presents the highest correlation with $F$. Fig.7. shows the monotonic $F$ reduction with the Vf increase. The Formula (1) is the typical result of multiple regression analyses of the relationships between the carbon fiber properties and $F$.

$$F = 0.098 \phi - 0.146 S - 5.31 Vf + 155.6 \quad -(1)$$

In this formula, the carbon fiber properties with little contribution to $F$ have been omitted. Among the interrelated carbon fiber properties, the ones with the higher contribution to $F$ have been adopted to the Formula (1). The formula is not contradictory with the results of single regression analysis between the fiber properties ($S$, $\phi$ and Vf) and $F$ in terms of partial regression coefficient. In addition, the multiple regression coefficient (R) is 0.954 and the coefficient of determination is 0.902. Both of these coefficients are high enough to represent $F$. It is considered that a sufficiently dependable formula is obtained as for the influence of the carbon fiber properties on the flow of CFRC slurry.

Analysis of Flexural Strength

Likewise, the relationships between the carbon fiber properties and the flexural strength of CFRC was analyzed by the regression method. Fig.8. through Fig.11. show the main results of sin-
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gle regression analysis. These figures show that the highly corre-
related properties with flexural strength are TS, TM, S and Vf. Fig.8. and Fig.9. indicate that the flexural strength of the hard-
ened body increases in direct proportion to both TS and TM and
that the level of contribution of these two are nearly equal. As
shown in Fig.12., TM and TS are highly interrelated (correlation
coefficient = 0.955). These two properties have therefore nearly
the same level of correlation with the flexural strength. Fig.10.
indicates the proportional increase of flexural strength with the
increase of S. This increase of S results from the increase of Vf
and the decrease of φ. Fig.11. indicates that the flexural
strength is almost directly proportional to Vf. Based on the
results of the multiple regression analyses of the influence of
the carbon fiber properties on the flexural strength, the Formula
(2) was deduced:

\[ \text{Flexural Strength} = 0.992L + 0.344TS + 22.202Vf + 47.7 \]

Fiber Efficiency Coefficient

Based on the results of both single and multiple regression
analyses, the fiber efficiency coefficient \( \alpha \) is defined by Vf
and TS, both of which strongly contribute to the actual flexural
strength addition to CFRC. Therefore,

\[ \sigma_b' = \alpha \times TS \times Vf \]

where \( \sigma_b' = \sigma_b' - [\sigma_b \text{ of plain mortar; } Vf=0\%] \).

As shown in Fig.13., \( \sigma_b' \) exhibits good correlation with
TS×Vf. Fig.14. shows the relationship between \( \sigma_b' \) and TS
with Vf as a parameter. It is indicated that \( \sigma_b' \) increases
with TS within the 2 to 4% Vf range. However, the maximum \( \sigma_b' \)
is reached where TS is between 200~250 kg/mm². This means that
the reinforcing efficiency of carbon fiber is optimized. It is
therefore considered that the improvement of flexural strength is
not always expected by simply increasing the tensile strength of
the fiber. Fig.15. summarizes the relationship between \( \alpha \) and
TS with a parameter of Vf. It is indicated that \( \alpha \) can be given as
follows:

\[
\begin{align*}
\alpha &= 0.479 + 0.631TS \times 10^{-3} - 4.887TS^2 \times 10^{-6} \quad (\text{at } Vf=2.0\%) \\
\alpha &= 0.302 + 1.337TS \times 10^{-3} - 5.380TS^2 \times 10^{-6} \quad (\text{at } Vf=3.0\%) \\
\alpha &= 0.328 - 0.025TS \times 10^{-3} - 0.328TS^2 \times 10^{-6} \quad (\text{at } Vf=4.0\%)
\end{align*}
\]

Summary

As a result of the regression analyses of the influence of
the fiber properties on CFRC, the following findings were
obtained:

1. The flow index of CFRC slurries are governed mainly by
φ, S and Vf. Poorer workability is expected with the decrease of
2. The flexural strength of the hardened body of CFRC is governed mainly by Vf and TS. The maximum strength is reached where TS is between 200 and 250 kg/mm². The fiber efficiency coefficient is reduced with the increase of Vf × TS.

**PROPERTIES OF THE NEW CFRC**

Practical properties of the new CFRC thus optimized are presented.

**Flexural Strength**

Fig.16. shows the typical stress–strain curve of 20mm thick CFRC (260mm span). The flexural strength of the thin-section CFRC is 200–210kg/cm² at Vf=3% and 160–170kg/cm² at Vf=2%. The failure mode of this specimen was non-catastrophic. This is about 3–4 times greater than that of plain mortar of the same dimensions. The toughness is also improved by 20 times or more.

**Weatherability**

Freezing and thawing resistance test -- No evidence of degradation was observed after 300 cycles of rapid freezing in air and thawing in water according to ASTM-C666-'77. As shown in Fig. 17., relative dynamic modulus of elasticity remains 95% or more. It is considered that the stress by freezing and thawing is alleviated by the uniformly-distributed carbon fiber and fine air cells in cement matrix.

Accelerated weatherability test -- 20mm thick specimens were laid in hot water of 85°C for 7 days and 28 days. The flexural strength test was conducted in ambient temperature after the immersion. As shown in Fig.18., the level of the flexural strength is nearly constant. This fact suggests that CFRC can maintain a high level of strength for a long period of time. Similar results have been obtained from the salt water spray test, long-term outdoor exposure and sunshine weatherometer irradiation tests.

**Noncombustibility**

The judgement of noncombustibility of building materials in Japan is currently based on the results of the two tests. The first one is the basic material test just like ASTM-E36-59T "Determining Noncombustibility of Elementary Materials." The second one is the surface burning test just like ISO TC-92 "Fire Propagation Test." The new CFRC (Vf=2%) successfully passed these two tests and has been approved "Noncombustible" by the Minister of Construction, Japan.
APPLICATION OF THE THIN-SECTION CFRC

Let us briefly describe some typical examples of the new CFRC products in Japan.

Roof Board

A CFRC board, 20mm thick, 600mm wide and 2300mm long with 60 mm thick lateral ribs on both sides, was molded by premix method. These boards were installed over an outdoor corridor as shown in Fig.19. As compared with ALC board installed alongside, the thin-section CFRC is lighter in unit area weight, stronger in flexure and less prone to corner damage by handling. More than 2 years have passed since they were installed. No evidence of cracking and deformation have been observed. On the contrary, an experimental cement mortar board reinforced with steel mesh began to suffer from transverse cracks and rainwater seepage within about a month after the installation.

Cladding Panel

A CFRC board, 15mm thick, 600mm wide and 2200mm long with 50 mm thick peripheral ribs, was molded. This was installed to the external wall of a plant building (Fig.20.). The surface was intentionally left unpainted in order to observe long-term changes by exposure. More than 4 years have passed and no evidence of deformation, cracking and rain water seepage has been observed.

Other Examples of Application

Carbon fiber reinforced polymer cement mortar -- suitable for existing concrete repair due to the good crack arrestability and weatherability.

Carbon fiber reinforced lightweight calcium silicate board -- non-combustible, asbestos-free timber substitute.

Lightweight CFRC curtain wall -- Kitakyushu Prince Hotel, Kitakyushu; Chigasaki Plant No.1 Building, MKC.,etc.

Road barrier -- Hanamizu Bridge on National Route No.1., Hiratsuka; Miyagi no Main Street Redevelopment, Sendai, etc.

CONCLUSION

The fabrication of new CFRC has been made possible by the development of the new high-performance carbon fiber (HP-CF) and composite technology. The product is weatherable and thermal resistant as well as excellent in mechanical properties. This production method is well suited to the mass production of CFRC at existing precast concrete plants without any addition of special facilities. This technology is also applicable to other inorganic
and polymeric matrices as well as cement matrix. It is anticipated that the new CFRC will be an excellent candidate material for the increasingly diversified and demanding applications of the construction industry worldwide.

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NOTATIONS

F: flow index (mm)
L: chopped length of carbon fiber (mm)
ϕ: carbon fiber diameter (μm)
TS: tensile strength of carbon fiber (kg/mm²)
TM: tensile modulus of carbon fiber (ton/mm²)
n: number of chopped carbon fiber in one cubic centimeter of CFRC (n/cm³)
S: specific surface area of chopped carbon fiber in one cubic centimeter of CFRC (cm²/cm³)
Vf: volume fraction of carbon fiber (%)
α: fiber efficiency coefficient
σ*: actual flexural strength addition by carbon fiber (kg/cm²)

REFERENCES


Table 1. Mix Proportion of CFRC
(by weight; in Method B)

<table>
<thead>
<tr>
<th>Component</th>
<th>Method B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Early Strength Portland Cement</td>
<td>100</td>
</tr>
<tr>
<td>Silicate Sand</td>
<td>50</td>
</tr>
<tr>
<td>Water</td>
<td>45</td>
</tr>
<tr>
<td>Air-entraining Water-reducing Agent</td>
<td>2.4</td>
</tr>
<tr>
<td>Dispersant</td>
<td>0.25*</td>
</tr>
<tr>
<td>Defoamer</td>
<td>0.1</td>
</tr>
</tbody>
</table>

* 1.0% in Method A

Table 2. Properties of CFRC

<table>
<thead>
<tr>
<th>Mixing Method</th>
<th>A</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fiber Dispersion</td>
<td>good</td>
<td>good</td>
</tr>
<tr>
<td>Trowel Finishability</td>
<td>poor</td>
<td>average</td>
</tr>
<tr>
<td>Mixing Time (minutes)</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>Flow Index (mm)</td>
<td>134</td>
<td>141</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Flexural Strength (kg/cm²)</td>
<td>140</td>
<td>220</td>
</tr>
</tbody>
</table>

Table 3. Properties of CFRC by a Larger Mixer

<table>
<thead>
<tr>
<th>Mortar Mixer (capacity)</th>
<th>750 liter</th>
<th>70 liter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fiber Dispersion</td>
<td>good</td>
<td>good</td>
</tr>
<tr>
<td>Trowel Finishability</td>
<td>average</td>
<td>average</td>
</tr>
<tr>
<td>Mixing Time (minutes)</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>Flow Index (mm)</td>
<td>143</td>
<td>141</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Flexural Strength (kg/cm²)</td>
<td>220</td>
<td>220</td>
</tr>
<tr>
<td>Dry Mixing</td>
<td>Method A</td>
<td>Method B</td>
</tr>
<tr>
<td>------------</td>
<td>----------</td>
<td>----------</td>
</tr>
<tr>
<td>Omni Mixer</td>
<td>1 min.</td>
<td>Mortar Mixer</td>
</tr>
<tr>
<td>1/2 Sand</td>
<td>GP-CF</td>
<td>Sand</td>
</tr>
<tr>
<td>Disperant</td>
<td></td>
<td>Disperant</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wet Mixing</th>
<th>Method A</th>
<th>Method B</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 min.</td>
<td>A.E.W.R. Agent</td>
<td>Water</td>
</tr>
<tr>
<td>4 min.</td>
<td>Defoamer</td>
<td>Defoamer</td>
</tr>
</tbody>
</table>

| Mixing Time | 8 minutes | 5 minutes |

**Fig. 1—Mixing methods and mixing time**
Note the uniform fiber distribution in both cases.

Fig. 2--Rapture face of CFRC specimens (microscopic magnification)
Fig. 3--Statistical relationship between $\phi$ and flow

Fig. 4--Statistical relationship between TS and flow ($V_f=2\%$)
Fig. 5--Statistical relationship between number of fiber and flow (Vf=2%)
Fig. 7--Statistical relationship between Vf and flow

Fig. 8--Statistical relationship between TS and $\sigma_b$
Fig. 9—Statistical relationship between TM and $\sigma_b$

Fig. 10—Statistical relationship between S and $\sigma_b$
Fig. 11--Statistical relationship between Vf and flow

Fig. 12--Statistical relationship between TM and TS
Fig. 13--Statistical relationship between Vf x TS and $\sigma_b$. 

Fig. 14--Statistical relationship between TS and $\sigma_b$. 
Fig. 15. Statistical Relationship between TS and $\sigma'_B/(V_f \times TS)$ (HPCF with $V_f = 2\% \sim 4\%$)

Fig. 16. Stress–Strain Curve of the New CFRC Specimens (span: 260mm)
Fig. 17--Freeze/thaw resistance of the new CFRC specimens
Fig. 18—Accelerated weatherability test of FRC specimens (in hot water: 85°C)
Fig. 19--The application of the new CFRC
(1) Roof board for an external corridor
Note the thickness of the ALC panel installed side by side

Fig. 20--The application of the new CFRC
(2) Cladding panel for an external wall
Here, FRC panels are intentionally unpainted for long-term observation.
Oriented Polyethylene Fibrous Pulp Reinforced Cement Composites

by D.M. Gale, A.H. Shah, and P. Balaguru

Synopsis: Researchers at Du Pont have developed a new form of fibrous polyethylene to replace asbestos fibers in asbestos-cement composites. This very fine, short, molecularly-oriented polyethylene pulp was tested in cement at various levels of incorporation and in combination with other fibers. Most of the initial investigation was focused on the pure cement matrix normally used for asbestos-cement products; however, this paper includes preliminary work with cast cement mortar matrices.

The polyethylene pulp can be used effectively for reinforcing cement. Flexural strengths can be increased by more than 200%. The pulp induces excellent ductility. Accelerated aging studies indicate that the pulp is durable in alkaline cement matrices.

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3, Professor of Civil Engineering, Rutgers, The State University of New Jersey

Keywords: asbestos; cellulose; cements; composite materials; durability; fibers; flexural strength; impact strength; mortars (material); polyethylene; portland cements; pulps; reinforcing materials
INTRODUCTION

Inhaled fibrous asbestos dust is believed responsible for fibrosis of the lungs (asbestosis), and two deadly forms of lung cancer (1). Concern for workers' health during the manufacture, and the ultimate disposal of asbestos-cement products, has caused the industry to actively seek environmentally safe substitutes. This is in spite of the fact that the major use of asbestos fibers since the turn of the century, worldwide, has been to reinforce cement. The asbestos-cement products, a variety of thin sheets and pipes, are highly useful to the building industry. Unfortunately, asbestos is a multifunctional ingredient when used in the manufacture of these products and so is difficult to replace.

In order to replace asbestos in cement systems, the three major functions of asbestos must be matched: reinforcement, cement retention (filter process aid) and durability. This multiplicity of functionality is partly possible because asbestos has both fiber and pulp characteristics. Fibers generally have a well-defined length and diameter, and if long and strong, provide reinforcement. In contrast, pulps are generally shorter, finer, more irregular, and have a broad distribution of lengths and...
diameters. This leads to good packing and retention of cement fines. Retention is required because the usual mode of manufacture for the asbestos-cement products involves the dewatering and accumulation of thin layers deposited on a moving felt from a dilute cement slurry, the well-known Hatschek process. Fibrous asbestos, as it is found in nature has mostly pulp characteristics, that is, short and irregular; but at the same time it has a fraction that is long and strong to provide reinforcement. Asbestos is durable in combination with Portland cement because it is chemically stable in alkaline environments.

Direct replacement of asbestos with a single substitute material has been difficult to achieve. Substitutes for asbestos in thin-section cement products include glass fiber, cellulose fibers and pulps, and synthetic fibers and pulps. Glass and the synthetic fibers provide excellent reinforcement, but are poor filter fibers. Coarse fibrillated polypropylene fibers, e.g., fifty to several hundred micrometers in diameter, found especially useful for reinforcing low slump concrete and mortar formulations, cannot be processed at all in the Hatschek system. Smaller diameter (10-20 micrometers), chopped "staple" fibers (4-12 mm long) cannot be processed alone because they do not retain cement fines. To gain processibility, these staple fibers are sometimes combined with cement-retaining pulps. Typical blend formulations use mixtures of polyvinyl alcohol (PVA) (2) or acrylic (PAN) (3) staple fibers together with cellulose pulps or a combination of cellulose and polyolefin pulps serving as filter aides.

For some applications, all-cellulose reinforcing/filter systems are used because they tend to be lower in cost. In the case of the all-cellulose asbestos replacement system, a mixture of cellulose fibers and pulps can be employed to gain both the needed reinforcement and filter characteristics. The problem, however, with using cellulose pulps as filter aides, and especially with the all-cellulose systems, is concern over long term durability. Good durability is very important since the most common uses of these cement sheet products require that structural components retain initial strength for many years, and they are frequently used for outdoor applications. Hence, researchers are still seeking new, non-cellulose fiber types for asbestos replacement.

Recently, researchers at Du Pont have developed a fibrous polyethylene pulp for asbestos replacement, which is marketed under the trademark Pulplus™. The objective is to match all the main features of asbestos in a single material. Since pulps are known to have good cement retention characteristics, testing consisted of measurement of composite flexural strength and toughness to establish reinforcement, and of accelerated aging to establish durability. Because the pulp consists of very fine, short filaments (film-fibrils), it is also finding application in injection molding, and in thin-section castings.
NEW REINFORCING PULP

Pulplus™ fibrous polyethylene pulp, consisting of very fine, short filaments or film-fibrils (4) was prepared by a special spinning and refining process. This pulp has a high level molecular orientation. During the final refining, the pulp is treated with a small amount of soluble polyvinyl alcohol resin to improve dispersibility in aqueous cement slurry.

The physical properties of the new pulp (Table 1) indicate that this material has strength and length/diameter ratios that might be useful for reinforcing the cement matrix. Property retention at elevated temperatures further suggests the pulp should be able to withstand moderate cure exotherms.

Zero span tensile strength is a measurement of the strength of a pulp obtained by measuring under special conditions the strength of a paper sheet made from that pulp. When failed in an Instron tensile tester, the jaws of the machine are placed as close together as possible, and hence the name "zero span." The strength measured under these conditions tends to reflect the intrinsic strength of the pulp filaments uncomplicated by paper quality characteristics. In order to obtain a high value, the pulp has to be molecularly oriented. The difference in strength behavior of oriented and non-oriented pulps can be seen in Table 2. Orientation increases the strength about threefold. Table 2 also presents comparative data relating to shrinkage and optical birefringence. Both of these are well known measures of molecular orientation.

Besides orientation, another important feature that is induced into oriented polyethylene pulps during manufacture is morphological fineness, Table 3. Finer pulps are better suited for trapping cement fines and so serve as better filter aides for the Hatschek process. Differences in fineness for various polyethylene pulps are most easily understood by comparing photomicrographs, Fig. 1. For further comparison, a photomicrograph of refined cellulose is included also. Interestingly, fineness cannot be measured quantitatively by the commonly used Brunnaeur-Emmett-Teller (B.E.T.) surface area because this method does not distinguish well between surface inside the pulps and external surface. Unoriented polyethylene pulps tend to have high void contents, which result in high surface area measured by the B.E.T. method, whereas, the oriented pulp tends to have low internal void content and hence lower B.E.T. surface area. For this reason, the Kajaani fineness measurement is used to quantitatively judge fineness differences, Table 3.
USE OF ORIENTED FIBROUS PULPS FOR REINFORCING CEMENT COMPOSITES

Because Pulplus™ oriented polyethylene pulp is made up of fibrous polyethylene that is strong, fine and relatively short, it has excellent potential for reinforcing thin, pure cement sheets. The initial investigation was focused on evaluating the mechanical properties of thin cement sheets reinforced with various amounts of pulp. Since commercial sheets are produced by the Hatschek process, a laboratory simulation of this process was used for specimen preparation. This section presents details of specimen preparation, the compositions used, and values measured for composite initial flexural strength and flexural toughness, as well as changes in these properties noted after accelerated aging exposure.

Since the Pulplus™ pulps have potential for other applications in the construction industry, research studies have been initiated to evaluate this material as a reinforcing agent for cast mortar composites. The primary variables studied are: fiber (pulp) content and cement to aggregate ratio. The reinforcing characteristics studied include flexural strength, flexural toughness and impact strength. Only preliminary results are presented in this paper.

THIN BEAMS (TILES) PREPARED SIMULATING THE HATSCHEK PROCESS

Thin tiles (0.5 cm thick) were used to judge the reinforcing contribution of Pulplus™ pulp. The primary testing was done in static flexure mode. Tests were also conducted to confirm that the ability of pulp to reinforce initially is not reduced with aging in the cement matrix. The primary test variable was pulp content which was varied from 2 to 7.4% by weight of the matrix. Tests were also conducted using a combination of Pulplus™ and longer (6 mm long) high tenacity polyacrylonitrile (PAN) or high tenacity polyvinyl alcohol (PVA) fibers. Cellulose fibers were included in some experiments.

Specimen Preparation

Type I Portland cement (200 gm) slurred in 1 liter of water was mixed with the appropriate amount of properly opened pulp in 5 liters of water and flocculated with 4 ml of a 0.1% solution of Dow Chemical Co. Separan* AP-273. The entire slurry was then cast into a sheet 33 cm X 33 cm in a standard Deckle box paper making mold and dewatered. The resulting sheet was then folded into four layers, trimmed to 15.2 X 15.2 cm and pressed in a mold for 30 min. at 11.6 MPa.
Flexural Testing

Specimens were cured 28 days and tested wet. Samples cut to 2.54 cm x 0.5 cm thick were tested in three point bending at a span of 7.6 cm. Load-deflection curves were recorded during the testing. Flexural toughness was taken as the integrated area under the load-deflection (stress-strain) curve until peak load (maximum stress) is reached. The average of 9-27 individual specimen tests are reported. Standard deviations were calculated and found to be within 15% of the average. This test is not a standard ASTM test; no standard test for asbestos-free cement composites existed at the time this study was made.

Durability Testing

Specimen samples of various sizes and 0.5 cm thick were subjected to 400 alternate cycles at 60° in a Weather-O-Meter durability tester: 2 hrs wet spray and then 2 hrs carbon-arc light/dry. Exposed samples were tested in flexure using the procedure and testing protocol described above.

Specimen samples were also tested for freeze-thaw durability in accordance with ASTM C 666.

RESULTS AND DISCUSSION

Flexural Strength and Flexural Toughness

The flexural strengths and toughness values using oriented and unoriented pulps are presented in Table 4. Results with tiles reinforced with mixtures of oriented polyethylene pulp and various short staple fibers are given in Table 5. The effect of fiber concentration is shown in Table 6. The following observations can be made:

- Orientation increases flexural strength and flexural toughness to a considerable extent, 159% and 2500%, respectively.
- Increase in pulp content consistently increases both the flexural strength and flexural toughness. Strength with 7.4% pulp incorporated was 150% of the strength at 2.9%, whereas toughness increased 1000%. The strength increase, however, levels off at about 6% fiber content.
- Flexural strengths up to 24 MPa can be obtained using oriented pulp alone. The strength is increased by more than 200% when compared to the unreinforced matrix.
Oriented polyethylene pulp can be successfully used in combination with other reinforcing (staple) fibers. Flexural strengths as high as 31 MPa were recorded for specimens reinforced with an oriented pulp/staple fiber combination.

**Effect of Aging on Flexural Performance**

Durability of pulps in the cement matrix was tested using accelerated aging with a cycled Weather-O-Meter, Table 6. Samples reinforced with oriented pulp retain both strength and toughness on accelerated aging, whereas the specimens reinforced with Western red cedar pulp loses both strength and toughness. The loss becomes higher with higher fiber contents. Composites reinforced with oriented pulp performed even better than the air-cured asbestos-cement system with regards to the retention of strength and toughness.

Oriented polyethylene pulp reinforced composites had excellent retention of flexural strength and toughness when subjected to 250 standard freeze-thaw cycles, Table 7. Unrefined Western red cedar pulp had good retention of properties, however, a refined wood pulp is reported to induce a 50% strength loss using this test (2). The ability to withstand freeze-thaw cycles is undoubtedly related to composite porosity, with the most porous structures failing the easiest. Refined cellulose pulps tend to form porous, lower density composite structures, which have difficulty with this test. Two commercial asbestos-containing cement composite samples and a refined cellulose-containing commercial sample had significant loss of flexural properties in our study.

**INJECTION MOLDING**

In addition to asbestos replacement in Hatschek systems, oriented polyethylene pulp is found to be useful in other cement-based systems. For example, in asbestos replacement for injection molded cement parts, the short length, cement retention capabilities and the ability to reinforce combine to make it uniquely suitable. The short length is useful in providing good packing during the critical molding/dewatering steps and helps avoid fissures.

**CASTING**

Primary variables were aggregate content and fibrous pulp content. Cement and sand ratios were varied from 1:0 to 1:3.5. The fiber (pulp) content was varied from 0 to 4% by weight. For specimens made using this casting process, a high range water-reducing admixture was used to improve the workability. ASTM type III cement and mortar sand were used for all the mixes.
Test Methods

Composites were tested for flexural strength, flexural toughness and impact strength. For the cast specimens, 1.3 cm thick and 5.1 cm wide beams were tested over a span of 14.7 cm, using middle third loading. Impact strength tests were conducted using 10 cm-diameter and 5.1 cm thick cylindrical discs. Smaller than usual dimensions were used because the matrix contained only sand.

Preliminary Results and Observations

- Regular cement and cement mortar mixes can be made up to a fiber content of 2% by weight without problems. High range water-reducing admixture is needed to improve the workability.
- Flexural strengths increase up to 20% for fiber contents higher than 1%.
- For fiber contents higher than 2%, the flexural specimens exhibit high ductility. At 4% fiber content, post peak behavior is almost horizontal.
- Impact strength increases considerably for fiber loadings of 2% and higher.

MECHANISM OF REINFORCEMENT FOR THE ORIENTED POLYETHYLENE PULP

Oriented polyethylene fibrous pulp provides effective reinforcement in spite of its short length and relatively low modulus. Tensile properties of pulps are difficult to measure, but the estimated tensile modulus is less than that of cement. In contrast, the tensile modulus of the high strength staple fibers used for reinforcement is higher than the modulus of cement. Pulp fractionation studies (Table 8) suggest that the critical length for reinforcement is about 0.6-1.0 mm. This is reasonable because small filament diameter allows for adequate l/d values, generally above 100. Also, the PVA-coating together with the fact that the pulp is quite flexible and so can assume a final conformation in more than one plane of the matrix, suggests good bonding to the matrix. In spite of low modulus, then, the pulp filaments can be assumed to reinforce with fiber yielding and pull-out as the main modes of energy absorption. This mechanism is consistent with the fact that the flexural strengths achieved well exceed the point of first crack formation, approximately 10-15 MPa. The load-deflection curve deviates from linearity at this point, Fig. 2.

Study of the variation of flexural strength with respect to fiber content indicates that the critical volume for reinforcement is at about 5 vol %. This value is about the point at which
one might expect to see significant overlap and some entanglement of the pulp filaments. Complete filling of the matrix by an ultrafine, partly entangled pulp would tend to markedly reduce paths for crack propagation. The pulp morphology, then, leaves open additional modes of energy absorption, namely disentanglement and micro crack-stopping owing to interdiction of growing micro cracks by pulp filaments.

CONCLUSIONS

Based on the information presented in this paper, the following conclusions can be drawn:

- Oriented polyethylene fibrous pulp can be used effectively to reinforce thin cement and cement mortar sections. Flexural strengths up to 24 MPa can be obtained using the pulp alone. In addition, the fibrous pulp induces excellent flexural toughness.

- Various fibers may be used along with the pulp to further augment its effectiveness.

- The fibrous pulp is stable in alkaline environments. Accelerated aging tests indicate that specimens reinforced with this pulp will retain both strength and toughness.

- The pulp is useful in many cement processes, including Hatschek, injection molding and regular casting. Pure cement or cement-aggregate matrices may be used effectively.

    Overall, the oriented polyethylene fibrous pulp is a highly useful material for reinforcement of various thin section cement products.

ACKNOWLEDGMENTS

The authors are indebted to Drs. S. S. Shelburne and J. R. Guckert for their help with this study.

REFERENCES


### TABLE 1

Properties of Oriented Polyethylene Pulp Suitable for Reinforcing Cementitious Systems

<table>
<thead>
<tr>
<th>BASE RESIN</th>
<th>Linear High Density Polyethylene</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>0.96 g/m³</td>
</tr>
<tr>
<td>Density, g/m³</td>
<td>134-138°C</td>
</tr>
<tr>
<td>Melting Point, °C</td>
<td>Inert²</td>
</tr>
<tr>
<td>Chemical Reactivity</td>
<td>1-20 Microns</td>
</tr>
<tr>
<td></td>
<td>2-4 M²/gm (BET, N₂)</td>
</tr>
<tr>
<td></td>
<td>3-4 sec/gm</td>
</tr>
<tr>
<td></td>
<td>Hydrophilic⁶</td>
</tr>
<tr>
<td>PULP</td>
<td>Hydrophilic⁶</td>
</tr>
<tr>
<td>Length, mm</td>
<td>1³</td>
</tr>
<tr>
<td>-- Retained on 10 mesh, %</td>
<td>&lt; 15⁴</td>
</tr>
<tr>
<td>-- Through 10 mesh on 100 mesh, %</td>
<td>&gt; 60⁴</td>
</tr>
<tr>
<td>-- Through 100 mesh, %</td>
<td>&lt; 30⁴</td>
</tr>
<tr>
<td>Diameter, Microns</td>
<td>1-20</td>
</tr>
<tr>
<td>Surface Area, M²/gm (BET, N₂)</td>
<td>2-4</td>
</tr>
<tr>
<td>Drainage Factor, sec/gm</td>
<td>3-4⁵</td>
</tr>
<tr>
<td>Surface Behavior</td>
<td>Hydrophilic⁶</td>
</tr>
<tr>
<td>Zero Span Strength, gm</td>
<td>22-25⁷</td>
</tr>
<tr>
<td>Strip Tensile Strength</td>
<td>&gt; 0.75⁸</td>
</tr>
</tbody>
</table>

**PROPERTY RETENTION AT HIGHER TEMPERATURES⁹**

<table>
<thead>
<tr>
<th>Property</th>
<th>@80°C</th>
<th>@100°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tenacity</td>
<td>100%</td>
<td>75%</td>
</tr>
<tr>
<td>Modulus</td>
<td>56%</td>
<td>50%</td>
</tr>
<tr>
<td>Work-to-Break</td>
<td>380%</td>
<td>230%</td>
</tr>
</tbody>
</table>

---

1. DSC peak at 10°C/min heating rate
2. Swells in chlorinated hydrocarbons
3. Tasman, TAPPI, Vol 55 #1, p. 136
4. TAPPI T33 OS75
5. Modified TAPPI 221 SU-72
6. Treated for wettability and adhesion
7. TAPPI 231 SU-70
8. Paper strip, 5-inch gage length, dry - lbs/in/oz/sq yd
9. Percent of room temperature filament properties on yarns spun from the base resin
TABLE 2

Effect of Orientation on Polyethylene Pulp Properties

<table>
<thead>
<tr>
<th>Polyethylene Pulp</th>
<th>Shrinkage Orientation</th>
<th>Optical Birefringence</th>
<th>Relative Zero Span Strength, gm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oriented</td>
<td>High</td>
<td>0.035</td>
<td>22.4</td>
</tr>
<tr>
<td>Unoriented &quot;A&quot;</td>
<td>None</td>
<td>0.020</td>
<td>8.3</td>
</tr>
<tr>
<td>Unoriented &quot;B&quot;</td>
<td>None</td>
<td>0.018</td>
<td>8.3</td>
</tr>
</tbody>
</table>

TABLE 3

Kajaani Fineness and B.E.T. Surface Area of Various Polyethylene Pulps

<table>
<thead>
<tr>
<th>Polyethylene Pulp</th>
<th>Fineness (m/mg)</th>
<th>B.E.T. Surface Area (sq. m/gm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oriented</td>
<td>6.6</td>
<td>2.5</td>
</tr>
<tr>
<td>Unoriented &quot;A&quot;</td>
<td>3.3</td>
<td>7.9</td>
</tr>
<tr>
<td>Unoriented &quot;B&quot;</td>
<td>3.3</td>
<td>7.9</td>
</tr>
</tbody>
</table>
### TABLE 4
Reinforcement of Ordinary Portland Cement with various Polyethylene Pulps

<table>
<thead>
<tr>
<th>Pulp Composition</th>
<th>WT%</th>
<th>Flexural Strength (MPa)</th>
<th>Toughness (KJ/sq m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oriented</td>
<td>6</td>
<td>22-24</td>
<td>1.5-2.0</td>
</tr>
<tr>
<td>Unoriented &quot;A&quot;</td>
<td>6</td>
<td>14-15</td>
<td>0.07</td>
</tr>
<tr>
<td>Oriented + Newspulp</td>
<td>2</td>
<td>18.1</td>
<td>0.31</td>
</tr>
<tr>
<td>Unoriented &quot;B&quot; + Newspulp</td>
<td>2</td>
<td>12.2</td>
<td>0.05</td>
</tr>
</tbody>
</table>

### TABLE 5
Cement Reinforcement with Mixtures of Oriented Polyethylene Pulp and various short staple Fibers

<table>
<thead>
<tr>
<th>Composition, WT%</th>
<th>Flexural St.</th>
<th>Toughness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oriented PE Pulp, 2.0 High Tenacity PAN, 1.0</td>
<td>20.9</td>
<td>0.23</td>
</tr>
<tr>
<td>Oriented PE Pulp, 2.5 High Tenacity PAN, 2.0</td>
<td>28.0</td>
<td>0.92</td>
</tr>
<tr>
<td>Oriented PE Pulp, 5.0 High Tenacity PAN, 2.5</td>
<td>31.2</td>
<td>2.39</td>
</tr>
<tr>
<td>Oriented PE Pulp, 2.0 High Tenacity PVA, 1.0</td>
<td>22.4</td>
<td>0.73</td>
</tr>
<tr>
<td>Oriented PE Pulp, 2.5 High Tenacity PVA, 2.0</td>
<td>23.7</td>
<td>2.44</td>
</tr>
<tr>
<td>Oriented PE Pulp, 5.0 High Tenacity PVA, 3.0</td>
<td>27.2</td>
<td>4.39</td>
</tr>
</tbody>
</table>
### TABLE 6

**Accelerated Aging Studies**

**400 HR Cycled Weather-o-meter**

<table>
<thead>
<tr>
<th>Pulp Type</th>
<th>Pulp Type</th>
<th>Initial Properties</th>
<th>Aged Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Flex. St.</td>
<td>Toughness</td>
</tr>
<tr>
<td>Orien. PE</td>
<td>2.9</td>
<td>15.3</td>
<td>0.2</td>
</tr>
<tr>
<td>WRC</td>
<td>2.9</td>
<td>15.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Orien. PE</td>
<td>3.9</td>
<td>18.1</td>
<td>0.7</td>
</tr>
<tr>
<td>WRC</td>
<td>3.9</td>
<td>14.8</td>
<td>0.2</td>
</tr>
<tr>
<td>Orien. PE</td>
<td>4.8</td>
<td>20.5</td>
<td>1.1</td>
</tr>
<tr>
<td>WRC</td>
<td>4.8</td>
<td>17.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Orien. PE</td>
<td>7.4</td>
<td>23.0</td>
<td>2.0</td>
</tr>
<tr>
<td>WRC</td>
<td>7.4</td>
<td>19.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

**Asbestos Controls**

<table>
<thead>
<tr>
<th></th>
<th>Flex. St.</th>
<th>Toughness</th>
<th>Flex. St.</th>
<th>Toughness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Commercial (Air Cure)</td>
<td>30.2</td>
<td>0.19</td>
<td>23.7</td>
<td>0.13</td>
</tr>
<tr>
<td>Commercial (Autoclaved)</td>
<td>14.1</td>
<td>0.06</td>
<td>14.7</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Orien. PE = Oriented Polyethylene Pulp

WRC = Western Red Cedar - Harmac K-10 Unrefined
### TABLE 7

250-Cycle Freeze-Thaw Studies using the ASTM C-666 Method

<table>
<thead>
<tr>
<th>Sample</th>
<th>Flex. St.</th>
<th>Toughness</th>
<th>Flex. St.</th>
<th>Toughness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Orient. PE Pulp, 6 WT% + OPC, 94 WT%</td>
<td>23.3</td>
<td>1.4</td>
<td>101</td>
<td>127</td>
</tr>
<tr>
<td>WRC, 6 WT% + OPC, 94%</td>
<td>18.0</td>
<td>0.6</td>
<td>126</td>
<td>98</td>
</tr>
<tr>
<td>Commercial Asbestos (air cure)</td>
<td>30.2</td>
<td>0.19</td>
<td>92</td>
<td>76</td>
</tr>
<tr>
<td>Commercial Asbestos (Autoclaved)</td>
<td>14.1</td>
<td>0.06</td>
<td>77</td>
<td>110</td>
</tr>
<tr>
<td>Commercial Cellulose (Autoclaved)</td>
<td>13.5</td>
<td>0.78</td>
<td>87</td>
<td>23</td>
</tr>
</tbody>
</table>

WRC = Western Red Cedar, unrefined  
OPC = Type I Portland Cement

### TABLE 8

Reinforcement with Bauer-McNett Fractions of Oriented Polyethylene Pulps at 6 wt% Incorporation

<table>
<thead>
<tr>
<th>Fraction</th>
<th>Flexural Strength</th>
<th>Toughness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Through 10 mesh and on 28 mesh</td>
<td>20.1</td>
<td>1.36</td>
</tr>
<tr>
<td>On 48 mesh</td>
<td>27.3</td>
<td>2.83</td>
</tr>
<tr>
<td>On 100 mesh</td>
<td>25.9</td>
<td>1.84</td>
</tr>
<tr>
<td>Fines</td>
<td>&lt; 20.0</td>
<td>&lt; 1.00</td>
</tr>
</tbody>
</table>
Fig. 1--Photomicrographs of Polyethylene and Cellulose Pulps
Thin-Section FRC and Ferrocement

YIELD STRESS: 15.2 MPa
YIELD STRAIN: 0.1%
MAX. STRESS: 26.0 MPa
MAX. STRAIN: 1.2%
MODULUS (2% PS): 15.5 GPa
TOUGHNESS: 2.3 kJ/m²

COMPOSITION: 6% BY WEIGHT OF ORIENTED POLYETHYLENE PULP,
94% TYPE I PORTLAND CEMENT;
PRESSED TO DENSITY OF 1.9 gm/cm³

Fig. 2--Load Deflection Curve for Type I Portland Cement Reinforced with Oriented Polyethylene Pulp
Development of Aramid Fiber Reinforced Cement Composites

by P. Soroushian, Z. Bayasi, and A. Khan

Abstract: A cementitious matrix capable of dispersing fibers using conventional mixing techniques was developed. The effects of reinforcing this matrix with different volume fractions (0% to 2%) of aramid fibers ranging in length from 1/8 in. (3mm) to 1/2 in. (12.7mm) on the composite material performance in the fresh and hardened states were assessed experimentally. The effects of matrix mix proportions on the fibrous material properties were also investigated. The test data generated in this study indicated that improvements in strength and toughness characteristics of cementitious materials can be achieved through aramid fiber reinforcement, with no need to use specialized manufacturing techniques.

Keywords: aramid; cements; fibers; impact strength; mixing; reinforcing materials; strength; tests; workability
Parviz Soroushian is an Associate Professor of Civil Engineering at Michigan State University. He received his Ph.D. in 1983 from Cornell University. His speciality is in concrete materials and technology. He has published several papers in this area, and is serving on a number of ACI, ASCE and TRB technical committees.

Ziad Bayasi is an Assistant Professor in the Department of Civil Engineering and Construction at Bradley University. He received his Ph.D. in 1989 from Michigan State University, and has several publications on concrete and fiber reinforced concrete materials and applications. He is also serving on the ACI committee on fiber reinforced concrete.

Ataullah Khan is a Ph.D. candidate in the Civil Engineering Department of Michigan State University from where he received his M.S. degree in 1988. He has worked as a structural engineer for three years after receiving B.S. from the N.E.D. Engineering University at Karachi, Pakistan.

INTRODUCTION

Cement-based materials suffer from a common deficit. They are brittle and weak under tensile stress systems. An effective technique to overcome this deficiency is the reinforcement with randomly oriented short fibers. Such fibers, especially when they are low in diameter and thus closely spaced, delay the catastrophic propagation of microcracks in cement-based materials, thereby increasing the strength and toughness of the material under tensile stresses. Following the propagation of microcracks, fibers bridge the resulting cracks and restrain their widening, thereby enhancing the post-peak ductility and energy absorption capacity of cement-based materials under tensile stress systems. Better bonding of fibers to the matrix, as far as it does not lead to fiber rupture prior to pull out, is advantageous at this stage of performance.

After about three decades of extensive research and development, the applications of fiber reinforced cementitious composites are now going through a period of rapid growth. Fibers such as carbon and cellulose are finding new applications in cement-based materials, and each fiber type seems to have its own optimum applications in cement and cement products.

The research reported here has been concerned with the development of aramide fiber reinforced cement composites, using conventional manufacturing techniques and cementitious matrices compactible with aramide fibers.
ARAIMD FIBERS

The aramid fiber used in this study was KEVLAR, which is a member of the family of aromatic polyamide fibers. They have high strengths and elastic moduli, and are also resistant to corrosion. They are chemically and mechanically stable at wide temperature ranges. The high strength of KEVLAR fibers may be attributed to their highly oriented crystalline nature with high molecular weight.

KEVLAR fibers are commercially available in several different types. The one used in this investigation was KEVLAR 49. Some physical and chemical properties of this fiber are shown in Table 1.

The aramid fibers used in this study have a linear tensile stress-strain relationship up to failure. In compression, however, their behavior becomes nonlinear at strains above 0.3%. The fibers have a good thermal stability, retaining a high percentage of their properties when exposed for long periods to temperatures as high as 355 deg. F (180 deg. C). At a cryogenic temperature as low as -320 deg. F (-196 deg. C), the fibers show essentially no embrittlement or degradation. They also have excellent thermal dimensional stability with a slightly negative coefficient of thermal expansion. Some information indicating the excellent chemical resistance of KEVLAR 49 (except in strong acids) are presented in Table 2.

BACKGROUND

Aramid fiber reinforced cement composites have been manufactured in the past generally by the spray-suction and slurry-dewatering techniques. In the spray-suction manufacturing technique (originally developed for glass fiber reinforced cement), an atomized slurry of cement and air stream containing fibers are directed simultaneously onto the flat surface of a mold. The spray head travels transverse to the mold to cover it uniformly with a mixture of slurry and fiber at a depth of 3/8 in. (10mm). The resulting fibrous cement sheet is then dewatered and troweled flat. Curing can be achieved in a moist environment or in autoclave.

In the slurry-dewatering process of manufacturing aramid fiber reinforced cement, a fiber-cement slurry is mixed and placed inside molds. The slurry is then dewatered (by the application of vacuum) and pressed to a relatively high density. Thin sheets manufactured by the slurry-dewatering technique have been generally moist cured or autoclaved.
Test results reported by Majumdar and Laws, Walton and Majdar are indicative of the desirable tensile and flexural strength and toughness characteristics, fatigue life, and weathering resistance of aramid fibre reinforce cement composites manufactured by the spray-suction and slurry-de-watering techniques (or by simply hand placing the fibres between layers of cementitious matrix).

The study reported herein has been concerned with the development of cementitious matrices that are more compatible with Aramid fibres (as far as the bonding to and protection of fibres are concerned) and possess desirable fibre dispersability characteristics (required for manufacturing the composite material by conventional mixing techniques). Manufacturing techniques were also established for the developed fibrous material, and the effects of the Aramid fiber reinforcement at variable fibre volume fraction and length on the composite material performance were assessed experimentally. The effects of some matrix mix variables on the composite material performance were also investigated.

EXPERIMENTAL PROGRAM

Uniform dispersion of aramid fibers in cement matrices is vital to the development of cement composites which effectively take advantage of the desirable reinforcement properties of fibers. Problems with the uniform dispersion of aramid fibres (which typically have relatively low diameters and high aspect ratios) have led in the past to the use of specialized manufacturing techniques for the production of aramid fiber reinforced cement composites. The approach followed in this investigation was to refine the cementitious matrix such that conventional mixing procedures, with an optimized mixing sequence, could be used to uniformly disperse the fibres in cement-based materials. For this purpose, a fraction of cement was substituted with a fine pozzolan (silica fume). Silica fume, when mixed with water, produces a cohesive mixture which is capable of breaking fiber balls and coating individual fibres, thereby facilitating the uniform dispersion of fibres. In the mix sequence established in this study, following the dispersions of fibres in a silica fume paste, the remaining mixed constituents were added to the mixture in order to achieve a fresh mix with improved workability which hardens to a composite material with desirable performance characteristics.

The use of silica fume in cementitious matrices also improves the bonding of the matrix to aramid fibres. The fine silica fume particles fill the voids in the interface zone and, through their pozzolanic reactivity, improve the structure of the interfacial zone. The high pozzolanic reactivity of the silica fume also leads to reduced alkalinity of cementitious matrices. This could enhance the long-term durability characteristics of aramid fibre reinforce ce-
ment composites at elevated temperatures. Cementitious matrices incorporating silica fumes are also less permeable and thus reduce the access of water soluble aggressives to aramid fibers in severe exposure conditions. One should, however, be aware of the potentially negative effects of silica fumes on the workability characteristics of fresh cementitious mixtures. Silica fume tends to make the mix cohesive with reduced flowability. This might require certain adjustments in mix proportions of cementitious matrices.

Neat cementitious pastes were used in this study as the matrices incorporating different volume fractions of aramid fibers with different lengths. The matrix mix proportions were adjusted in order to ensure convenient dispersion of fibers (using a conventional mortar mixer) and achievement of desirable fresh mix workability characteristics. The sequence and rate of addition of different mix constituents to the mixer were also established.

The matrix mix proportions used in this study as well as the corresponding fiber volume fractions and lengths are introduced in Table 3. The cementitious matrix consists of type I Portland cement and silica fume. The physical and chemical properties of the silica fume used in this study are presented in Table 4. (15) The superplasticizer had naphthalene formaldehyde sulfonate as its active ingredient, (16) and the fiber type as mentioned earlier, was KEVLAR 49.

A mortar mixture was used to manufacture aramid fibre reinforced cement in this investigation. The sequence and rate of addition of different mix constituents to the mixer were as follows:

1. Add silica fume and 2/3 of the water and superplasticizer, start the mixer at low speed and mix for 1 minute (until a uniform mixture is produced);

2. Add the fibers gradually (to avoid balling) as the mixer is running;

3. Add the remainder of water and superplasticizer;

4. Charge the mixer with cement;

5. Continue mixing at low speed for about 1 minute, then stop the mixer and wait for 30 seconds;

6. Change the mixing speed to medium, and mix for another 1 minute.

The fresh aramid fiber reinforced cement mixtures were characterized by the flow table test. (8) The specimens manufactured through molding and external vibrations for mechanical characterization of hardened ara-
mid fiber reinforced cement are described below (all specimens, after demolding at 24 hrs, were air cured until the test age of 28 days):

1. 3X6 in. (75X150mm) cylindrical compression (ASTM C-873) test specimens (two for each mix);

2. 1.5X1.5X6.5 in (38x38x165mm) prismatic flexure (ASTM C-1018) test specimens (three for each mix, with center point loading on a span of 4.8 in., 122mm).

3. Briquet tension test specimens with 1 in (25mm) square critical section (three for each mix); and

4. 6 in. (150mm) diameter by 2.5 in. (64 mm ) high cylindrical impact test specimens (11) (three for each mix).

EXPERIMENTAL RESULTS

Flow table test results for fresh aramid fibre reinforced cement mixtures are presented in Fig. 1. Fig. 1.a indicates that higher volume fractions and lengths result in reduced flow of fibrous cement. The increase in silica fume content is observed in Fig. 1.b to negatively influence the flowability of fresh mix. Fig. 1.c shows the increase in flow at higher superplasticizer dosages. The increase in water content is observed in Fig. 1.d to have desirable effects on the flowability of fresh aramid fibre reinforced cement mixtures.

The flexural strength test results present in Fig. 2.a are indicative of important gains in flexural strength resulting from the addition of aramid fibres. For 1/8 in. (3mm) fibers, the increase in flexural strength continues even at volume fractions above 1%. For longer fibers, however, the damage to workability at volume fractions above 1% seems to negatively influence the flexural strength of cementitious composites.

The flexural energy absorption test results are shown in Fig. 2.b. Flexural energy absorption is defined in this study as the area underneath the flexural load- deflection curve up to a deflection twice the value at peak load. The longer fibers (1/2 in., 13.0mm) give the best energy absorption at 1% fiber volume fraction. At 2% fiber content, however, the shorter fiber (1/8 in., 3 mm) which produce fresh mixes with better workability characteristics seem to give superior energy absorption capacities.

The effects of silica fume and water contents on the flexural performance of aramid fiber reinforced cement with 1% volume fraction of 1/4 in. (6mm) long fibers are shown in Fig. 3. The increase in silica fume content is ob-
served in Fig. 3. a to increase the flexural strength but negatively influence the energy absorption capacity of the composite material. This drop in energy absorption could be illustrated by the increase in fiber-to-matrix bond strength at high silica fume contents which could encourage fiber rupture (instead of pullout), thus reducing the frictional energy absorption associated with fiber pull-out. The increase in water-binder ratio is observed in Fig. 3.b to reduce the flexural strength but have positive effects on the flexural energy absorption capacity of aramid fiber reinforced cement.

Fig. 4 shows the tensile strength test results for plain and fibrous cements with different volume functions of 1/2 in. (13mm) aramid fibers. Higher fiber volume fractions obviously lead to cementitious composites with higher tensile strengths.

The increase in fiber volume fractions is observed in Fig. 5 to slightly increase the compressive strength and energy absorption capacity (area underneath the compressive stress-strain curve) of cementitious materials.

The increase in silica fume-binder ratio is observed in Fig. 6 to increase the compressive strength of aramid fiber reinforced cements incorporating 1% volume fraction of 1/4 in. (6mm) fiber.

The impact strength test results presented in Fig. 7 are indicative of substantial improvements in the impact resistance of cementitious materials resulting from aramid fiber reinforcement. At 2% fiber volume fraction, the improvements in impact resistance are more than two orders of magnitude when compared with plain cement.

**SUMMARY AND CONCLUSIONS**

Cementitious matrices were proportioned for convenient dispersion of aramid fibers using regular mixing techniques. The effects of aramid fiber reinforcement at fiber volume fractions ranging from 0% to 2% and length from 1/8 in., (3mm) to 1/2 in., (13mm) and matrix mix proportions on the fresh mix workability and hardened material mechanical characteristics were assessed experimentally. The following conclusions could be derived from the experimental data generated in this study:

1. The combination of silica fume (for fiber dispersability) and superplasticizer (for workability) in cementitious pastes can help in the manufacture of aramid fiber reinforced cement composites using a regular mortar mixer.

2. The flowability of fresh fibrous mixtures tends to decrease with increasing fiber length and volume fraction, and silica fume content. The increase in
superplasticizer dosage and water content of the mix can help in improving the workability of fresh fibrous mixtures.

3. The increase in fiber volume fraction leads to important gains in flexural strength and energy absorption capacity of cementitious materials. For longer fibers, however, there is a limit (below 2%) on volume fraction beyond which the damage to fresh mix workability tends to adversely influence the hardened material performance.

4. Higher silica fume contents and lower water contents tend to produce aramid fiber reinforced cement composites with higher flexural strengths but reduced energy absorption capacities.

5. The increase in aramid fiber volume fraction results in slightly increased compressive strength of cementitious materials.

6. High silica fume contents (silica fume-binder ratios of about 0.29) produce aramid fiber reinforced cement composites with sharply increased compressive strengths.

7. Substantial improvements in the impact resistance of cementitious materials can be achieved through reinforcement with aramid fibers.

ACKNOWLEDGMENTS

The research reported herein was sponsored by the Research Excellence Fund of the state of Michigan, and the Civil Engineering Department of Michigan State University. The KEVLAR fibers were provided by E.I. du pont de Nemours & Company (Inc.), the superplasticizer by W.R. Grace & Company, and the silica fume by Elkem Materials. These contributions are gratefully acknowledged.

The authors are also thankful to Paul G. Riewald, Sr. Research Associate at the Fibers and Composites Development Center of E.I. du pont de Nemours & Company for his valuable contributions to this work.

A patent (pending) filed by Michigan State University on KEVLAR Fiber Reinforced Cement Composites (U.S. Dept. of Commerce serial No. 174.207) covers the mix proportioning and manufacturing techniques described in this article.
REFERENCE


5. Soroushian, P. and Bayasi, Z. (editors), "Fiber Reinforced Concrete: Design and Applications," Proceedings, MSU Concrete Technology Seminars -1, Michigan State University, East Lansing, February 1987,


Table 1. Properties of Kevlar 49 Filaments

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>12.1 microns</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>1.44</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>18,000 ksi</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>525 ksi</td>
</tr>
<tr>
<td>Elongation</td>
<td>2 to 2.6 %</td>
</tr>
<tr>
<td>Coefficient of Thermal Expansion (in fiber axis)</td>
<td>-1.1x10^{-6}/deg.F</td>
</tr>
<tr>
<td>Equilibrium Moisture Level (@ 72 deg. F and 50% Rh)</td>
<td>3.5 to 4.5 %</td>
</tr>
</tbody>
</table>
Table 2. Chemical Resistance of KEVLAR 49.

(% Initial strength obtained after 100 hrs of exposure)

<table>
<thead>
<tr>
<th>Chemicals</th>
<th>% Initial Strength</th>
</tr>
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<tbody>
<tr>
<td><strong>Acids</strong></td>
<td></td>
</tr>
<tr>
<td>Acetic (99.7%)</td>
<td>100</td>
</tr>
<tr>
<td>Hydrochloric (37%)</td>
<td>36</td>
</tr>
<tr>
<td>Nitric</td>
<td>13</td>
</tr>
<tr>
<td><strong>Bases</strong></td>
<td></td>
</tr>
<tr>
<td>Sodium Hydroxide (40%)</td>
<td>97</td>
</tr>
<tr>
<td>Ammonium Hydroxide</td>
<td>92 (1000 hrs)</td>
</tr>
<tr>
<td><strong>Other Chemicals</strong></td>
<td></td>
</tr>
<tr>
<td>Gasoline</td>
<td>100 (1000 hrs)</td>
</tr>
<tr>
<td>Salt Water</td>
<td>99</td>
</tr>
<tr>
<td>Boiling Water</td>
<td>88</td>
</tr>
</tbody>
</table>
Table 3. Mix Proportions and Fiber Reinforcement Properties

<table>
<thead>
<tr>
<th>Cement :Water:Silica fume: Superplasticizer</th>
<th>$V_f$ (%)</th>
<th>$L_f$ (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>0.8</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.5</td>
</tr>
<tr>
<td>1: 0.45: 0.29: 0.041</td>
<td>1.0</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.125</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0.125</td>
</tr>
<tr>
<td>1: 0.46: 0.20: 0.037</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1: 0.46: 0.41: 0.043</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1: 0.44: 0.30: 0.032</td>
<td>1.0</td>
<td>0.25</td>
</tr>
<tr>
<td>1: 0.47: 0.30: 0.048</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1: 0.39: 0.30: 0.040</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1: 0.52: 0.30: 0.040</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Ratios by weight (except for fiber volume fraction):
$L_f$ = fiber length
$V_f$ = fiber volume fraction
1 in. = 25.4 mm
Table 4. Physical and Chemical Properties of Silica Fume.

<table>
<thead>
<tr>
<th>Physical Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity</td>
<td>2.3</td>
</tr>
<tr>
<td>Bulk Density</td>
<td>14 lb/cu. ft. (225 kg/cu.m.)</td>
</tr>
<tr>
<td>Specific Surface area</td>
<td>$14 \times 10^6$ in $/ (16(200,00 \text{cm}^2/g)$</td>
</tr>
<tr>
<td>Average Particle Size</td>
<td>$6 \times 10^{-5}$ in. (0.14 microns)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Chemical Composition (by weight)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO$_2$</td>
<td>96.5 %</td>
</tr>
<tr>
<td>C</td>
<td>1.4 %</td>
</tr>
<tr>
<td>Fe$_2$O$_3$</td>
<td>0.15 %</td>
</tr>
<tr>
<td>MgO</td>
<td>0.20 %</td>
</tr>
<tr>
<td>Al$_2$O$_3$</td>
<td>0.15 %</td>
</tr>
<tr>
<td>K$_2$O</td>
<td>0.04 %</td>
</tr>
<tr>
<td>Na$_2$O</td>
<td>0.20 %</td>
</tr>
</tbody>
</table>
Fig. 1(a)--Effects of fiber volume fraction and length

Fig. 1(b)--Effect of silica fume content ($V_f=1\%$, $L_f=1/4$ in., 6mm)

Fig. 1--Flow table test results
Fig. 1(c) -- Effect of superplasticizer content ($V_f=1\%$, $L_f=1/4$ in., 6mm)

Fig. 1(d) -- Effect of water content ($V_f=1\%$, $L_f=1/4$ in., 6mm)

Fig. 1(cont'd) -- Flow table test results
Fig. 2--Effects of fiber reinforcement on flexural performance
(1 psi=0.00694 Mpa; 1 in.=25.4 mm)
Fig. 3--Effects of matrix proportions on flexural performance 
($V_f=1\%$; $L_f=1/4$ in., 6mm; 1psi=0.00694 Mpa)
Fig. 4—Tensile strength test results
($L_f=1/2$ in., 13 mm; $1$ psi = $0.00694$ Mpa)
Fig. 5(a)—Compressive strength

Fig. 5(b)—Compressive energy absorption

Fig. 5—Effects of fiber reinforcement on compressive performance (1 Ksi = 6.9 Mpa)
Fig. 6—Effects of silica fume content on compressive strength
\(V_f = 1\%, L_f = 1/4\) in., 6mm; 1Ksi = 6.9 Mpa

Fig. 7—Effects of aramid fiber reinforcement on impact resistance
(1 in. = 25.4 mm)
Reinforcement of Cement-Based Materials with Cellulose Fibers

by P. Soroushian and S. Marikunte

**Synopsis:** A brief review of the literature on cellulose fiber reinforced cement is presented followed by the results of an experimental study concerned with the effects of mechanical and chemical pulps on the performance characteristics of neat cement paste in the fresh and hardened states. The mix proportions and manufacturing techniques used in this study for the production of cellulose-cement composites are reviewed. The air content, setting time and drop in workability with time are compared for plain cementitious materials and those reinforced with 1% and 2% mass fractions of mechanical and chemical pulps. The flexural and compressive strength and toughness characteristics, impact resistance, specific gravity, and water absorption capacity of plain and fibrous materials are also compared. Effects of moisture content on the flexural performance of plain cementitious materials and those reinforced with mechanical pulp are also discussed.

**Keywords:** cellulose fibers; cements; composite materials; flexural strength; impact strength; pulps; reinforcing materials; strength
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**INTRODUCTION**

World use of hydraulic cements is close to one billion tons per year[1] and along with steel, they are the most important construction materials used today.

Cement-based materials suffer from one common shortcoming; they fail in a brittle manner under tensile stresses or impact loads, i.e. lack of resistance to the propagation of cracks. Figure 1 typically shows the brittleness of cement when compared with ductile metals[2].

The use of short randomly distributed fibers is an effective method of strengthening brittle materials against cracking under stress. Broadly, the reason why weak, brittle materials are made tougher by very small additions of fiber is that cracks are deflected in the presence of fibers and as a consequence, the toughness or ductility is dramatically increased.

**CELLULOSE FIBERS**

Figure 2 briefly illustrates the structure of wood. If a piece of lumber is considered, it may have defects (knots, cracks etc.), by selection, a piece of clear wood (near macro defect-free) could be obtained with a tensile strength of say 70 MPa (9.31 Ksi)[1]. However, single fiber which consti-
tutes the reinforcing unit of bulk wood, has been tested and found to have tensile strengths greater than 700 MPa (93.1 Ksi)[1]. If one considers cellulose as the basic molecule which makes up the fiber, and if one could express the strength of the chemical bonds which make up the structure of cellulose in terms of tensile strength, an even greater value of around 7000 MPa (931 Ksi) would be recorded[1].

Natural fibers have a long history of being used for reinforcing purposes. In addition to cellulose fibers in the form of paper pulp, saw dust and chips, jute, coir, elephant grass and henequen as well as sisal and bamboo in the form of whole or fragmented chips have been used as reinforcement[3]. Despite this long history and the existence of well-established cellulosic materials such as wood, plywood, paper and paper laminates, recent developments in cement composites have been concerned primarily with fibers such as glass, asbestos, carbon, polypropylene and steel.

Wood fibers are highly cost efficient. They require less production energy than glass, steel and other fibers conventionally used for the reinforcement of cement.

Worldwide, the asbestos cement sheet industry has been searching for an alternative reinforcing fiber (owing to the health risk associated with the use of asbestos, and its limited supply and rising costs)[4]. Cellulose fibers derived from softwood and hardwood, being fairly strong and stiff as well as cheap and plentiful with low energy demand during manufacture, appear to be strong contenders as reinforcing fibers in cement-based materials.

Trees serve as the major raw material for cellulose fibers. The trees harvested for the production of cellulose fibers are known commercially as "softwoods" and "hardwoods." Among commercial trees, softwoods are the source of so-called "long fibers." The unbroken cellulose fibers in important softwoods range in length from about 2.5 mm (0.098 in.) up to 7 mm (0.28 in.), but the vast majority of these fibers average in length between 3 and 5 mm (0.12 and 0.20 in.)(5). Even within the same tree species, fiber lengths can vary considerably. Softwood cellulose fibers have widths, or diameters, that range from about 15 to 80 microns (30 to 45 microns for most softwoods).

Hardwoods yield cellulose fibers that, on an average, are about 1/3 to 1/2 the length and about 1/2 the width of softwood fibers. Cellulose fibers produced from hardwood also have higher fines content when compared with those obtained from softwood.

Figure 3 provides information on the geometry and appearance of the major fiber types in softwoods and hardwoods. All diagrams in this figure are at
the same magnification to show the relative sizes of these elements.

The cells in their natural arrangement in solid softwoods and hardwoods are bonded together by a layer of amorphous cementing material. It is this bonding that must be broken in the cellulose fiber production (pulping) process, by either chemical or mechanical means.

Pulping processes are classified as either chemical, semi-chemical, or mechanical[6]. This classification refers to the nature of the defiberization process. In the mechanical pulping process the cells are separated by frictional forces often aided by steam pressure. In the chemical process the wood cells are separated from one another primarily by dissolving and removing the natural bonding agent. Semi-chemical processes use a combination of both chemical reactions and mechanical power. Chemical pulps also called kraft pulps are commonly used in the production of book papers and writing papers, while mechanical pulps are regularly used for the manufacture of news print.

The major chemical components of wood are cellulose, hemicellulose, lignin and a very small fraction of so-called extractives. Table 1 shows an average analysis of softwoods and hardwoods.

**CELLULOSE FIBER REINFORCED CEMENT COMPOSITES**

The basic constituents of cellulose fiber-cement composites are cellulose fiber and cementitious binder. Fine aggregates (e.g. ground silica) may also be used in cellulose-cement composites[7].

The manufacturing procedure generally used for cellulose fiber reinforced cement composites can be categorized as either molding or slurry-dewatering. The molding procedure is similar to that used in the construction of conventional mortars. In slurry-dewatering approach, first a slurry (with relatively high water content) of the mix constituents is produced, and then the extra water is removed through the application of vacuum and pressure in order to compact the composite material.

Coutts,[8, 9] Andonian[10] andMorissey[11] et. al. have reported the results of flexural tests on cement composites reinforced with unbeaten Pinus Radiata kraft fibers (Canadian Standard Freeness of 700). The tested specimens, after a typical pre-curing period of 24 hours inside their molds, were either air-cured in an atmosphere with 50% relative humidity and 22 deg. C (68 deg. F), or were autoclaved for 8 hours at 0.86 MPa (124 Psi) steam pressure, and then exposed to air. All tests were performed in an environ-
ment with 50% RH. The flexural strength test results for slurry-dewatered cement pastes and mortars (silica/cement=1.0) cured in air and autoclave are presented in Figure 4. There is clearly an increase in flexural strength with increasing fiber content. The effects of cellulose fibers on flexural strength seem to depend on the nature of the matrix and curing conditions. While the air-cured and autoclaved mortars gave rather comparable flexural strengths, the air-cured cements are observed to have higher flexural strengths. The maximum flexural strength in any case seems to be achieved at a fiber mass fraction of about 8%. Cellulose cement is observed to reach flexural strengths exceeding 30 MPa (4320 Psi).

Fracture toughness (defined as the area under the flexural load-deflection curve) is a material property which may be as desirable as strength of stiffness in application to building products. Figure 5 shows the measured values of toughness for slurry-dewatered cement pastes and mortars (silica/cement=1.0) reinforced with different mass fractions of P. Radiata kraft pulp and cured in different conditions (all test results were obtained at 50% RH). There is clearly a significant increase in fracture toughness with increasing cellulose fiber content in cementitious materials. There seems to be little difference in fracture toughness of air-cured cements and autoclaved mortars reinforced with comparable fractions of P. Radiata kraft pulp. Air-cured fibrous cements, produced relatively high toughness values.

It is important to ensure that the improvements in material properties of cement achieved through cellulose fiber reinforcement would be retained over a long time period in actual exposure conditions. In particular, one should be careful about the affinity of wood fibers to moisture, their durability in the alkaline environment of cement, and the possibility of biological attack. As far as the biological attack is concerned no evidence is available to indicate that natural fibers can be decomposed biologically when used in cement materials.

Kraft pulps are the dominant wood fibers types used in cement-based materials. These fibers have minimum lignin contents and, noting the susceptibility of lignin to alkaline attack, have not developed durability problems in past applications[12]. Mechanical pulps, which contain higher lignin and hemicellulose contents than kraft pulps, may require attention for preventing the dissolving of lignin in the alkaline environment of cement.

Cellulose fibers, when applied to cementitious matrices, can produce composites with performance/cost ratios comparable to those obtained with asbestos. Due to the health hazards associated with asbestos, cellulose fibers are being seriously considered for the production of asbestos-free cement products. Cellulose fiber reinforced cement has found commercial applications in the manufacture of flat and corrugated sheets. These thin
sheets are currently used in the interior of buildings. They also have the potential for exterior applications once their durability characteristics in harsh environments are better understood. Non-pressure pipes can also be manufactured with cellulose-cement composites.

The increased fracture toughness of cellulose fiber reinforced cement suggests that they could be valuable in areas of application in which resistance to impact is a noted advantage. Renders and walls in locations of high usage, (e.g. schools, shops and factories) are examples of such areas. Successful use of cellulose fiber reinforced cement for the manufacture of cable pits has also been reported[4].

EXPERIMENTAL PROGRAM

Molded cellulose fiber reinforced cements were manufactured using different cellulose fiber types and mass fractions, and were mechanically characterized at different moisture contents.

The cellulose fibers used in this investigation were: Southern Softwood Kraft (SSK)[13], Northern Hardwood Kraft (NHK)[14], and Mechanical Pulp (1000L)[15]. Some key properties of these cellulose fibers are presented in Table 2. The properties presented in Table 2 include CSF (Canadian Standard Freeness), which presents a measure for the level of beating. An important consideration in different applications of cellulose fibers is the refinement of fibers through mechanical beating which leads to the fibrillation of fibers. The beaten fibers have exposed fibrils on their surfaces, which help in the development of mechanical bonding and also tend to prevent the loss of cement particles during the suction stage of slurry-dewatering. A common measure of the degree of beating is the Canadian Standard Freeness (CSF) of a suspension containing the fibers. Freeness is a measure of fibrillation of fibers; a smaller CSF is indicative of a higher beating (refinement) level. An important consideration in the beating of cellulose fibers is to optimize the beating level such that the fiber-to-matrix bond is improved to the point where gains in strength characteristics of the composite are not accompanied with substantial losses in toughness characteristics.

The cementitious matrices used in this study were neat cement paste consisting simply of regular type I Portland cement and tap water. The fiber mass fractions and matrix mix proportions are given in Table 3. The water content was adjusted (increased with fiber content) in order to maintain the fresh mix workability at a reasonably practical level represented by a flow of 65±5% at minute after mixing. The values of flow at 1 minute are also pre-
The kraft pulp (SSK and NHK) are generally shipped in the form of relatively compact sheets, and has to be disintegrated in water using a mortar mixer (at 450 revolutions per minute) before being added to the mixture; otherwise a uniform dispersion of fibers inside the cementitious paste can not be achieved conveniently. No disintegration prior to mixing was necessary for the 1000L mechanical pulp.

The mixing procedure for the manufacture of cellulose fiber reinforced cement in a regular mortar mixer was as follows: (1) add cement and 70 percent of water, and mix at low speed (140 RPM) for about 1 minute or until a uniform mixture is achieved; (2) gradually add the fibers and the remainder of water in to the mixture as the mixer is running at low speed (over a period of 2-5 minutes depending on the fiber content), taking care that no fiber balls are formed; and (3) turn the mixer speed to medium (285 RPM) and mix for 1 minute, stop the mixer and wait for 1 minute, and then finalize the process by mixing at high speed (450 RPM) for another minute.

The fresh fibrous cement mixtures were tested for: (1) flow (ASTM C-230) at 1 min., 5 min. and 10 min. after the mixing process; (2) air content of hydraulic cement mortar (ASTM C-185); and (3) setting time by penetration resistance (ASTM C-403).

For each mix molded specimens were manufactured for flexure (ASTM C-1018 and JCI-SF4), compression (ASTM C-873 and JCI-SF5) and impact tests in the hardened state. The void content, specific gravity and water absorption of the hardened materials were also assessed using the broken flexure specimens (ASTM C-642). The flexural specimens were prisms with 38.1 mm (1.5 in.) square cross section and total length of 152.4 mm (6 in.), tested by 4-point loading on a span of 114.3 mm (4.5 in.). The compression test specimens were cylinders 76.2 mm (3 in.) in diameter and 152.4 mm (6 in.) high. The cylindrical impact specimens were 152.4 mm (6 in.) in diameter and 63.5 mm (2.5 in.) high. Three replicated flexure, compression and impact test specimens were manufactured and tested for each mix.

All the fibrous specimens were compacted through external vibration, and were kept inside their molds underneath a wet burlap covered with plastic sheet for 24 hours. They were then demolded and moist cured for 5 days before being air cured in a regular laboratory environment until the test age of 28 days. In the case of 1000L specimens, in order to investigate the effects of moisture content on mechanical properties, extra series of flexure specimens were manufactured, and were either oven dried at 105 deg. C (221 deg. F) for 24 hours, or immersed in water for 48 hours, both starting at 28 days of age, prior to the performance of flexural tests.
In the flexure and compression tests, both load and deflection were monitored throughout the test in order to obtain complete load-deformation relationship.

**EXPERIMENTAL RESULTS**

This section presents the effects of cellulose fiber reinforcement on the fresh mix and hardened material properties of cement using the test data generated in this investigation.

Figure 6(a), 6(b) and 6(c) show the drop in workability with time for cementitious matrices reinforced with different mass fractions of mechanical pulp, softwood kraft pulp and hardwood kraft pulp, respectively. These figures show that the drop in workability with time is comparable in fibrous and plain mixtures.

Figure 7 shows the effects of cellulose fiber reinforcement on the initial and final setting times of cementitious matrices. There is a tendency in setting time to increase in the presence of mechanical pulp. Some constituents in mechanical pulp could cause this tendency by playing the role of set retarders. Kraft pulps only slightly increase the final setting time of the matrix. It should be emphasized that fibrous mixtures had higher water contents than the plain matrix.

The effects of cellulose fiber reinforcement on air content of fresh cementitious matrices are shown in Figure 8. An increase in air content is observed to result from the application of cellulose fibers to cement. This might indicate that the fibrous mixtures are not as compactable as the plain matrix (in spite of the fact that they all have comparable flows).

Figures 9(a) and 9(b) present the flexural strength and fracture toughness test results respectively. A combination of ASTM C-1018 and JCI-SF4 was used for performing the test. Flexural toughness according to JCI-SF4 is defined[16] as the area under the flexural load-deflection curve up to a deflection of 0.762 mm (0.03 in.), which is the span length divided by 150. There are marked improvements in the flexural strength and toughness of cement in the presence of cellulose fibers. Kraft pulps (SSK softwood and NHK hardwood) seem to be more effective than the 1000L mechanical pulp in enhancing the flexural performance of cement. The softwood kraft pulps at 2% mass fraction are observed to produce flexural strengths about 5 times that of the plain matrix. Improvements in flexural toughness are even more significant.
The compressive strength and toughness results are shown in Figures 10(a) and 10(b), respectively. A combination of ASTM C-873 and JCI-SF5 was used for performing the test. According to JCI-SF5 compressive toughness is defined[16] as the area underneath the compressive load-deflection curve up to a strain at 0.0075. Cellulose fibers are observed to reduce the compressive strength and toughness of cementitious materials. This effect is relatively small and, considering the major improvements in flexural performance, it is not a major factor in application of cellulose fiber reinforced cement to thin-sheet products which are typically subjected to flexure and impact loads.

Effects of cellulose fiber reinforcement on the first crack and ultimate impact strengths of cement are presented in Figures 11(a) and 11(b), respectively. Impact resistance represents the number of blows by a standard hammer (10 lb. mass with a drop height of 457.2 mm, 18 in.) required for cracking and failure of the test specimen[17]. Figure 11 indicates that tremendous improvements in the impact resistance of cement can be achieved through cellulose fiber reinforcement. Kraft pulps are much more effective than the mechanical pulp in this regard.

Results of tests on the void content, specific gravity and water absorption of hardened plain and fibrous cementitious materials are shown in Figure 12. The hardened material void contents are observed in Figure 12(a) to be comparable with those of fresh material (Figure 8) and tend to increase with increasing fiber content. While the increase in fiber mass fraction from 0% to 1% results in a relatively sharp increase in void content, the increase in fiber content from 1% to 2% caused a relatively small increase in void content. Figure 12(b) shows that the increase in void content resulting from cellulose fiber reinforcement leads to a drop in the specific gravity of the material. Water absorption is observed in Figure 12(c) to increase with increasing fiber content.

An important concern in the use of cellulose fiber reinforced cement is related to the effects of moisture on cellulose fibers and their bonding to cementitious matrices. In order to study the effects of moisture on the properties of cellulose fiber reinforced cement, flexural tests were performed on specimens with different moisture contents. All the specimens were compacted through external vibration, and were kept inside molds underneath a wet burlap covered with plastic sheet for 24 hours. They were then demolded and moist cured for 5 days before being air cured in a regular laboratory environment (at about 50±10% RH and 22±3 deg. C, 72±5 deg. F, temperature) until the test age of 28 days. A series of standard conditions for testing were established for studying the moisture effects on the 28 day old specimens. Samples were conditioned in the following environments: (a) in the
laboratory with relative humidity of 50±10\% and 22±3 \textdegree\text{C} (72±5 \textdegree\text{F}) temperature; (b) in an oven at 116 \textdegree\text{C} (241 \textdegree\text{F}) for 24 hours and then cooled in the atmosphere; and (c) in water for 48 hours with excess water being removed with a cloth prior to testing. The flexural strength test results for different conditioning cases are compared in Figure 13. From the limited test data presented in Figure 13, it may be concluded that the presence of mechanical pulps tends to increase the sensitivity of cementitious materials to moisture. Fibrous specimens immersed in water for 48 hours tend to have more pronounced drops in flexural strength when compared with plain cementitious specimens.

SUMMARY AND CONCLUSIONS

A brief background was presented on cellulose fibers and their application to cement. The results of an experimental study concerned with the effects of different mechanical and chemical pulps on the fresh mix performance, and mechanical and physical properties of hardened cementitious materials were presented. The matrix used in this study was neat cement paste and its water content was increased with increasing fiber content in order to maintain the workability comparable in different mixtures. The molding manufacturing technique used for the production of plain and fibrous specimens. The fiber mass fractions were 0\%, 1\% and 2\%. The test data produced in this investigation indicate that:

1. Cellulose fibers have relatively small effects on the drop in workability of cementitious materials with time, but setting time tends to increase in the presence of mechanical pulp (kraft pulps only slightly increase the final setting time);

2. The fresh and hardened material air content of cementitious materials are comparable and tend to increase (at a decreasing rate) with increasing fiber content.

3. Major improvements (up to 500\% at a fiber mass fraction of 2\%) in the flexural strength of cement can be reached by the use of kraft pulps. The increase in fracture toughness is even more significant. Mechanical pulps are less effective than kraft pulps in increasing flexural strength and fracture toughness.

4. There is a gradual tendency for the compressive strength and toughness of cementitious composites to drop with increasing cellulose fiber content.

5. Considerable improvements in the impact resistance of cementitious ma-
terials can be reached through reinforcement of the material with kraft pulp. Some improvements in impact resistance can also be achieved by mechanical pulps.

6. Cellulose fiber reinforcement results in a decrease in specific gravity and increase in water absorption of cementitious materials.

7. In the presence of the mechanical pulp, cementitious materials tend to be more sensitive to moisture effects, with the flexural strength and toughness of fibrous materials being more adversely influenced by immersion in water (when compared with plain cementitious materials).

ACKNOWLEDGMENTS

Financial support for the performance of this research project was provided by the U. S. Department of Agriculture (Wood Utilization Research Program in Eastern Hardwood and the Research Excellence Fund of the State of Michigan. The authors are also thankful to the Department of Forestry and the Composite Materials and Structure Center of Michigan State University for their technical contributions to this project. The fibers used in this project were provided by the American Fillers and Abrasives Inc., and The Procter and Gamble Cellulose Company. The technical support of the concrete laboratory personnel at Michigan State University, especially those of Mr. Siavosh Ravanbaksh, are gratefully acknowledged.

REFERENCES


Table 1. Average Chemical Composition of Softwoods and Hardwoods [6].

<table>
<thead>
<tr>
<th>Components</th>
<th>Softwoods</th>
<th>Hardwoods</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cellulose</td>
<td>42 ±2</td>
<td>45 ±2</td>
</tr>
<tr>
<td>Hemicellulose</td>
<td>27 ±2</td>
<td>30 ±5</td>
</tr>
<tr>
<td>Lignin</td>
<td>28 ±3</td>
<td>20 ±4</td>
</tr>
<tr>
<td>Extractives</td>
<td>3 ±2</td>
<td>5 ±3</td>
</tr>
</tbody>
</table>

Table 2. Properties of Cellulose Fibers [13, 14, 15].

<table>
<thead>
<tr>
<th>Manufacturer</th>
<th>Brand Name</th>
<th>Type</th>
<th>Species</th>
<th>Avg. Length</th>
<th>CSF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Am, Fillers</td>
<td>1000L</td>
<td>Mech.</td>
<td>Softwood</td>
<td>8.0 mm</td>
<td>-</td>
</tr>
<tr>
<td>Procter &amp; Gamble</td>
<td>SSK</td>
<td>Kraft</td>
<td>Softwood</td>
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<td>700</td>
</tr>
<tr>
<td>Cellulose</td>
<td>NHK</td>
<td>Kraft</td>
<td>Hardwood</td>
<td>0.9 mm</td>
<td>500</td>
</tr>
</tbody>
</table>
### Table 3. Mix Proportions

<table>
<thead>
<tr>
<th>Fiber Type</th>
<th>Fiber Mass Fraction (%)</th>
<th>Water-Cement Ratio</th>
<th>C:W:F by Weight</th>
<th>Flow (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0.28</td>
<td>10:2.8:0</td>
<td>66</td>
</tr>
<tr>
<td>SSK</td>
<td>1</td>
<td>0.35</td>
<td>10:3.5:0.14</td>
<td>65</td>
</tr>
<tr>
<td>SSK</td>
<td>2</td>
<td>0.40</td>
<td>10:3.5:0.28</td>
<td>62</td>
</tr>
<tr>
<td>NHK</td>
<td>1</td>
<td>0.35</td>
<td>10:3.5:0.14</td>
<td>63</td>
</tr>
<tr>
<td>NHK</td>
<td>2</td>
<td>0.40</td>
<td>10:3.5:0.28</td>
<td>62</td>
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<tr>
<td>1000L</td>
<td>1</td>
<td>0.35</td>
<td>10:3.5:0.14</td>
<td>74</td>
</tr>
<tr>
<td>1000L</td>
<td>2</td>
<td>0.40</td>
<td>10:3.5:0.28</td>
<td>66</td>
</tr>
</tbody>
</table>
Fig. 1--Typical load-deflection curves for brittle and ductile materials[2]

Fig. 2--Schematic representation of the substructure of a tree[1]
Fig. 3—Diagrams of major fiber types in softwoods and hardwoods[5]

Fig. 4—Flexural strength versus fiber mass fraction for cements and mortars reinforced with unbeaten P. Radiata Kraft Pulp[8]
Fig. 5—Fracture toughness versus fiber mass fraction for cements and mortars reinforced with unbeaten *P. Radiata* Kraft Pulp

Fig. 6--Flow table test results at different time intervals after mixing
Fig. 6(b)--Softwood Kraft pulp

Fig. 6(c)--Hardwood Kraft pulp

Fig. 6(cont'd)--Flow table test results at different time intervals after mixing
Fig. 7(a) -- Initial set

Fig. 7(b) -- Final set

Fig. 7 -- Setting time test results
Fig. 8--Fresh mix air content
Fig. 9(a)—Flexural strength

Fig. 9(b)—Flexural toughness

Fig. 9—Flexure test results
Fig. 10(a) -- Compressive strength

Fig. 10(b) -- Compressive toughness

Fig. 10 -- Compression test results
Fig. 11(a)--First crack

Fig. 11(b)--Failure

Fig. 11--Impact test results
Fig. 12(a)—Void content

Fig. 12(b)—Specific gravity

Fig. 12(c)—Water absorption

Fig. 12—Hardened material void content, specific gravity and water absorption
Fig. 13--Effects of moisture content on flexural strength of plain and fibrous cements
Plastic Shrinkage and Permeability in Polypropylene Reinforced Mortar

by M.A. Sanjuan, B. Bacle, A. Moragues, and C. Andrade

Synopsis: Fibres are added to concrete to improve several of its properties. The ability of polypropylene fibres to modify different characteristics of concrete is controversial. This paper presents results on the influence of adding polypropylene fibres (0.1% - 0.2% by volume) on mortar permeability and plastic shrinkage. The influence of adding polypropylene fibres on the early stages of shrinkage is studied with 120 x 15 x 3 cm. specimens. These were fabricated in mortar and then held in a chamber with controlled temperature and ventilation. The specimens have a special geometry in order to enable the shrinkage measurement in the plastic state, and the influence of this on mortar cracking. The variables studied were: water/cement ratio, sand/cement ratio and fibre content.

In addition, the ability of fibre concrete to absorb water and its permeability to \( \text{CO}_2 \) were tested. Water absorption was measured in accordance with Frech standard NF B 10.502. Carbonation was studied by introducing fibre mortar specimens in a chamber saturated with \( \text{CO}_2 \) and comparing the results with natural carbonation.

The results show that the addition of fibre reduces plastic shrinkage when compared with the same type of mortar without fibres. Concerning water absorption, it is reduced when water/cement ratio is about 0.5, however, when the water/cement ratio is higher than 0.5 this behaviour is reversed and the fibre mortar is more water absorbent. Accelerated and natural carbonation show that \( \text{CO}_2 \) diffusion increases in mortar with the highest amount of fibres.

Keywords: absorption; carbon dioxide; diffusion; fiber reinforced concretes; fresh concretes; mortars (material); permeability; plastic shrinkage; polypropylene fibers
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M. A. Sanjuan, graduate in Chemistry at the Complutense University in Madrid. He prepared his doctorate in Engineering Chemistry at the "Eduardo Torroja" Institute.

**INTRODUCTION**

Fibre-reinforced concrete has been developed to improve several properties of plain concrete (1). Since polypropylene fibre concrete has a relatively good response against restraint in the early stages of shrinkage (2, 3), this paper discusses the shrinkage behaviour of polypropylene fibre-reinforced mortar specimens with different cement/sand and water/cement ratios and fibre contents. Due to the close relationship with material porosity and, therefore, its facility to allow diffusion to take place, water absorption tests were carried out with different mortars in order to obtain more information on behaviour during shrinkage. This study was completed with CO$_2$ measurements inside the mortars subjected to natural and forced carbonation.

**EXPERIMENTAL**

**Apparatus**

Experimental equipment was designed to measure plastic state shrinkage in mortars and concrete (Fig.1). It consists of a chamber in which air flow speed and temperature are held constant, inside which are two specimens (20 x 150 x 1,200mm.). Shrinkage is measured with extensometers located on steel plates and connected to other steel plates by a steel rod.
Accelerated carbonation of the specimens is obtained in a chamber with an atmosphere saturated with CO$_2$ to 60% R. H. (4). The CO$_2$ advance line is indicated with phenolphthalein.

Materials

An ordinary Portland cement was used. The sand used complied with Spanish Standard RC-75. The commercial polypropylene fibres used in these experiments had a rectangular cross section and were 19 mm. long. The specific gravity, modulus of elasticity and tensile strength of the polypropylene fibres is 0.90 - 3.5 kN/mm$^2$ and 0.56 - 0.77 kN/mm$^2$, respectively.

Preparation of specimens and procedure

The specimens used for studying shrinkage in the plastic state (30 x 150 x 1,200) are fabricated in accordance with the proportions shown in Table 1. Once the extensometers are located on these specimens, the chamber is closed and the interior wind speed and temperature conditions are set. The shrinkage reading (mm/m) is measured over time.

The specimens employed for water absorption (40 x 40 x 160 mm) are fabricated in accordance with Spanish Standard UNE-80-101-84, and were cured in a chamber at 90% R.H. for 28 days. A cement/sand ratio of 1/3 was used, together with different water/cement ratios (0.5 and 0.6) and fibre proportion (0%, 0.1% and 0.3%). The water absorption test is performed in accordance with French Standard NF B 10.502.

Proportioning of the specimens (50 x 50 x 50 mm) used for studying CO$_2$ diffusion is shown in Table 2. These were submerged in water and cured for 28 days, and then kept for 90 days in laboratory ambient conditions (20°C and 60% R.H.). Then a series of specimens (series A) were kept for a further 90 days under the same conditions, while another series (series B) were subjected to accelerated carbonation.

RESULTS AND DISCUSSION

Plastic Shrinkage

Fig. 2 shows a characteristic trend obtained in the shrinkage tests. Three different stages are observed: 1) slight expansion due to the settling of layers on the mortar; 2) strong mortar shrinkage; 3) loss of plastic state.

The addition of polypropylene fibres reduces plastic shrinkage (Fig.3a). This reduction depends on the mortar proportioning
It was confirmed that for the same cement/sand ratio of 1/3, the reduction in shrinkage on adding fibres (represented by the gradient of the straight lines in Figure 3a) is similar for different water/cement ratios (W/C = 0.45 and 0.50.

With mortars that are richer in cement (C/S = 1/2), the polypropylene fibres act more effectively, reducing plastic shrinkage. This is because the material is less porous and the contact area is therefore greater between the fibre and the mortar.

Figures 3b and 3c show that the increase in shrinkage is similar in mortars with 0.1% fibres by volume, when the water/cement ratio rises from 0.4 to 0.6 (C/S = 1/2) and the cement/sand ratio increases from 1/9 to 1/0.8 (sand/cement = 0.5).

In Figure 3c it can be seen that the addition of 0.1% fibre to mortars with high cement/sand ratios (1/2 – 1/0.8) and with a water/cement ratio of 0.50, may be more favourable to plastic shrinkage than in the case of fibre-less mortars with a water/cement ratio of 0.55. This may be due to a greater capacity of the material to favour evaporation and to allow water to pass through it.

Water Absorption

The degree of water absorption by mortars with polypropylene fibres cured for 28 days, is given by the different slopes of the straight lines in Figure 7.

Using a cement/sand ratio of 1/3 and a water/cement ratio of 0.5, an increase in fibre content decreases water absorption. However, on using a water/cement ratio of 0.6, fibre addition favours water absorption (Fig. 8). This is presumably due to the fact that in very porous mortar, fibre can act as a link between pores. This behaviour is maintained throughout the 90 days.

CO₂ Diffusion

Figure 4 shows the degree of CO₂ penetration in the mortars studied. It is confirmed that, as expected, the carbonation of mortars weak in cement is much greater than in cement-rich mortars.

The rate and form of carbonation in fibre-less mortars and with 0.1% by volume of polypropylene fibres is similar; however, with 1% fibres, CO₂ diffusion to the interior of the mortar is much greater, and the diffusion is not uniform. It is interpreted that this is due to the fact that as large amounts of fibres were used it was not properly distributed, forming clumps providing preferential CO₂ diffusion paths.

With accelerated carbonation the different types of 1/3 mortar reach a similar level of carbonation (Figure 6). In contrast, in 1/1 mortars, this level is only reached by specimens
with a 1% fibre content, having similar carbonation readings for mortars without fibres and with 0.1% polypropylene fibre (Fig. 5)

It is considered that this behaviour could be due to the formation of different types of pores in each case.

CONCLUSIONS

The results obtained in this paper may be summarised as follows:

1.- The fibres studied reduce plastic shrinkage of the mortars. The extent of their influence depends on the proportion of fibres and on the water/cement and cement/sand ratios.

2.- Water/cement ratios of 0.6 reverse shrinkage behaviour, cancelling out the beneficial effect of the fibres.

3.- Very high fibre contents (1% by volume) favour mortar carbonation. Contents of 0.1% do not modify the penetration rate.

ACKNOWLEDGEMENTS

The authors wish to thank HISPANO-QUIMICA, S.A. for their kind cooperation with this study.

REFERENCES


5.- Kraai, P.P. "Fibermesh Concrete Engineering Data". Report K003-082283. San José (California, USA), 1985.
Table 1 - Mix Proportion of Polypropylene-fibre Reinforced Mortar

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Table 2 - Mix Proportion of Polypropylene-fibre Reinforced Mortar

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Fig. 1--Experimental equipment. A: fan; B: heating coils; C and D: temperature gauge and controller; E and F: moulds; G: extensometers; H: hygrometer
Fig. 2--Characteristic trend obtained in each plastic shrinkage test

$\text{SHRINKAGE (1E-2 = )}$

$\text{SPECIMEN N. 1}$

$\text{SPECIMEN N. 2}$

$c/e = 1/2$

$w/c = 0.52$

$\text{fibre = 0.14v.}$

$\text{TIME (min. )}$
Fig. 3--Plastic shrinkage of mortar, as a function of: a) fibre content; b) W/C ratio; c) C/S ratio
Fig. 4 -- CO₂ diffusion inside the mortars. Series A: natural carbonation. Series B: accelerated carbonation.
Fig. 5 -- Accelerated carbonation in 1/1 mortars with fibres

Fig. 6 -- Accelerated carbonation in 1/3 mortars with fibres
Fig. 7--Water absorption in mortar cured for 28 days (W/C = 0.5)

Fig. 8--Water absorption in mortar cured for 28 days (W/C = 0.6)
Interaction Between Fibers and the Matrix in Glass Fiber Reinforced Concrete

by B. Mobasher and S.P. Shah

Synopsis:
Traditionally, the first cracking strain of plain matrix is used as the material property in the fiber reinforced cement based composites. It is used to indicate the tensile strength, and thus termination of the contribution of the matrix phase. In the presence of high volume fraction of fibers, formation of the first crack does not necessarily lead to the fracture instability, thus matrix is able to carry increasing loads. The strength of the matrix is thus dependant on the type, volume fraction, bond, and strength of the fibers.

This paper investigates the tensile stress strain response of cement paste in the presence glass fibers. A test procedure is described which can characterize the toughening effect of various fiber types on the matrix properties.

Keywords: cement pastes; composite materials; cracking(fracturing); durability; fiber reinforced concretes; glass fibers; stiffness; strength; stress-strain relationships; tension tests
Dr. Barzin Mobasher is a member of Technical Staff at the Research and Development section of USG Corporation, Libertyville, IL. His research interests include fiber reinforced concrete, nonlinear fracture mechanics, and durability of concrete. He was the recipient of ACI student fellowship in 1983, and the Federal Highway Administration fellowship award in 1985.

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He has been extensively involved with constitutive relationship, failure and fracture of concrete, nonlinear fracture mechanics applied to rock, fiber reinforced concrete, high strength concrete, impact and impulsive loading, and behavior of concrete structures subjected to seismic excitation. Dr. Shah has published over two hundred and fifty papers in journals, proceedings and has edited ten books.

He is currently chairman of the American Concrete Institute’s Committee on Fiber Reinforced Concrete, Chairman of RILEM Committee on Fracture of Concrete, and Vice Chairman of the Society of Experimental Mechanics Committee on Fracture of Rock and Concrete.

Professor Shah received the ACI Anderson award in 1989, the Thompson award from the American Society of Testing and Materials in 1983 and the Gold Medal from RILEM in 1980. He is a fellow of ACI and is currently on the Board of Editors of four international journals. He was a NATO Senior Visiting Scientist to France in 1986, a member of an advisory committee to the Danish Government to evaluate Danish research groups in the field of concrete in 1986, and has received the Alexander Von Humboldt Fellowship Award for Distinguished Senior Scientist in 1989.

INTRODUCTION

It is well known that the single most limitation in a wider applicability of cement based materials are their notch sensitivity and low fracture toughness values (1). The brittleness of these materials can be significantly improved by means of incorporating fibers of various kinds. Due to difficulties in mixing and casting, in conventional construction it is common to use low volume concentration of fibers. The mode of failure in these composites is governed by the formation of a single crack and by localization of the deformation field at or around it. The eventual debonding, and pull-out of the fibers bridging the macrocrack, contributes to the absorption of energy at points remote to the crack tip, thus resulting in the
toughening of the composite (2). It is usually assumed that after the formation of the first crack, the matrix does not contribute significantly. This is perhaps true for the low volume concentration of the fibers (i.e. less than 1% steel fibers).

It is possible that when volume fraction of fibers is large, the fibers can prevent the microcracks from a catastrophic propagation. In that case, the matrix can contribute to carry increasing stresses until a new microcrack is formed. Thus, if fibers can delay localization of the microcracks, they can enhance the tensile properties of the matrix. For large volume of steel (3), and polypropylene fibers (4)(5), researchers have reported essentially linear stress-strain for values of strain much larger than the unreinforced matrix cracking strain. This may indicate that matrix properties are enhanced by the fibers.

To examine whether continuous glass fibers alter the properties of matrix was the goal of this research. Aligned glass fiber-cement matrix composites were subjected to uniaxial tension. Composites with different volume fraction of fibers are studied within a strain range of up to 0.5% strain.

EXPERIMENTAL PROGRAM

Specimen Preparation

In order to study the behavior of matrix as an independent phase, the fiber contribution to the composite must be accounted for. The fiber properties were computed by preparation of fiber-epoxy specimens with varying fiber contents and tested under the same conditions.

Uniaxial glass fiber reinforced composites were manufactured using both cement and epoxy as the matrix phase. The specimen dimensions were 7" x 1" (177.8 x 25.4 mm), and the thickness varied from 0.15" to 0.2" (3.8 to 5.1 mm). A relatively low viscosity epoxy was used (Buehler brand Isopropyldenediphenol epichlorohydrin resin). The fiber used was Owens-Corning alkali resistant (AR-glass) glass fiber strands containing 204 fiber filaments. In order to study the aging effects on the glass fibers, several roving of glass fibers were subjected to accelerated aging in a constant temperature calcium hydroxide solution at 50°C; they were subsequently used in specimen preparation.

Cementitious composites were manufactured using a type I portland cement which was fairly ground using a mortar and pestle. The water-cement ratio of the mixture was 0.35. A superplasticizer with a dosage of 0.05 oz/lbs of cement was also used to aid in workability.

Figure 1 represents the set-up used in the casting of the
Each strand was run length-wise across a mold six inches long. To insure proper alignment of the fibers, weights were tied to the ends of the fibers and suspended. Fibers were distributed evenly across the width, and along the specimen thickness were covered by 1/16" of matrix on either side. Once the matrix was poured over the fibers, a metal plate was placed across the top of the mold and weight was added until the excess matrix had been squeezed out. Specimens were cast individually and allowed to cure for 24 hours. After curing, the specimens were removed and the excess glass and matrix were trimmed off. The cement based specimens were cured in 100% RH continuously until the time of test. All specimens were tested at 7 days of age.

In all 15 specimens were cast with varying fiber volume concentrations. Six specimens were cast with epoxy, and the remaining with cement matrix as shown in table 1. The estimated volume concentrations used were between two and ten percent.

**Mechanical Testing**

All specimens were tested in direct tension using a closed loop servovalve controlled MTS testing machine (20 kips/89 KN capacity). Frictional wedge loaded grips were used. The average response of two LVDT's (Linear Variable Differential Transducers) was used as the control parameter. The transducers used had a range of ± 0.05 in (1.27 mm). They were mounted across a gage length of 3" (76.2 mm) using a knife edge assembly, shown in figure 2. A constant strain rate of 0.01333% ε/min was used throughout the study. Uniaxial strain was also measured by means of thin foil strain gages of 1 mm gage length for several specimens. The load, elongation, and strain responses were acquired using a digital oscilloscope.

**TEST RESULTS** and **DISCUSSION**

**Glass Fiber - Epoxy Specimens**

Figure 3 represents the load elongation response of glass-epoxy composites. The effect of fiber volume fraction on increasing the longitudinal stiffness of the composite is quite evident. The stiffness of the composite was measured from the load vs. elongation behavior. Using test results for various volume fractions, and assuming that the load carried by the composite is linear with respect to the stress in the fibers, matrix, and the volume fractions of each, one can compute the stiffness, and thus the contribution of the each phase to the overall composite, i.e.:

\[
\text{computed based on the LVDT's response.}
\]
subscripts \( m \), \( c \), and \( f \) represent the matrix, composite, and the fibers respectively. Assuming that no voids are present, one can represent the matrix volume fraction as:

\[
V_m = 1 - V_f = \frac{t - A_f N}{t}
\]

\( N \) represents the number of the fiber rovings per unit length, \( A_f \) the area of a roving, and \( t \) the specimen thickness. Note that this formulation uses a parallel (iso-strain) approach, i.e., the strain in the matrix, fibers, and the composite are identical, thus resulting in a linear dependance of the composite stiffness on the fiber volume fraction. In the case that the matrix behaves nonlinear (i.e. due to formation of microcracks), the stress in matrix as a function of the composite strain can be defined as:

\[
\sigma_m(\epsilon_c) = \frac{t}{t - A_f N} \left[ \sigma_c(\epsilon_c) - \frac{A_f N}{t} E_f \epsilon_c \right]
\]

Equation 2 is derived based on the assumption that the total load carried by the composite is the summation of the load carried by the matrix, and the fibers. This macroscopic averaging approach is dependant on the degree of microcracking. As the crack spacing decreases, more reliable results can be obtained by this approach.

The computed stiffness of the composite is plotted vs. the volume fraction of fibers as shown in figure 4. The intercept is the modulus of the plain epoxy which is computed to be about 523 Ksi. This value compares favorably with the value of 541 Ksi obtained experimentally for plain epoxy. Furthermore, the slope of linear curve represents the difference between the moduli of the fiber and the plain epoxy. The Young's modulus of the fiber is thus computed to be equal to 11509 Ksi.

Using this value of Young's modulus, one can compute the stress strain response of the matrix according to equation 2. Figure 5 shows the computed, as well as the experimental response for plain epoxy. It is observed that the response of plain epoxy can be accurately computed from that of the composite specimens. Note that the computed stress strain of the epoxy is independent of the fiber volume fractions and that the calculated response matched the experimentally observed stress-strain curve.
Glass Fiber-Cement Paste Composites

Figure 6 represents the load vs. elongation of a composite containing 5% volume fraction of fibers. It is observed that the behavior of the composite is initially linear with a stiffness close to the value predicted by the rule of mixtures. It is interesting to note that matrix cracking as indicated by the drop in the load carrying capacity, occurs over a large volume of strain and while the load is increasing. This is in contrast with the assumptions of Aveston, Cooper, and Kelly that multiple cracking occurs at a relatively constant value of load (6).

Figure 7 represents the initial response of a specimen with $V_f = 4\%$. Response of a strain gage of 1 mm gage length is also plotted versus the overall strain computed from the LVDT's response. The strain values computed from both transducers is identical during the linear elastic portion. Cracking initiates in the specimen at around 100 microstrains. LVDT's response which averages strain and crack opening increases monotonically. The strain gage which measures the local strain, indicates an increase in strain to magnitudes as high as 260 $\mu$str (more than twice the first cracking strain). Thus matrix is continuously carrying load. The matrix reaches the ultimate strength, and enters the softening region which is indicated by a decrease in the load carried by the matrix. Furthermore, at an overall strain of 1000 $\mu$str, it is observed that matrix does carry in excess of 150 $\mu$str.

Application of the aforementioned technique in order to compute the cement matrix response from the glass fiber-cement composites requires the assumptions of no slip, and perfect bond between the glass fibers and cement. It should be noted that it was not possible to impregnate every filament of a glass fiber strand with fresh cement paste. As a result not all the filaments of strands will be strained to the same extent. Thus the assumption of a perfect bond between the phases will overestimate the fiber contribution and underestimate the matrix's (efficiency factor for strand effect = 1; see Proctor(7) for example).

The average stress strain response of matrix was computed using equation 2 for $V_f = 4, 5$, and 6% and is shown in figure 8. An important characteristic shown in figure 8 is the non-uniqueness of the cement matrix constitutive response as compared to that of epoxy. There is an enhancement in the stress strain response of the matrix as the volume of fibers increases. It must be noted that these specimens were manufactured in 3 layers of matrix-fiber-matrix using a simple sandwich mechanism. No efforts were made to decrease the density by removing the voids, applying compacting pressures, or evenly distributing the fibers throughout the entire matrix. As a result, some specimens failed by splitting in the direction perpendicular to the fibers. It is likely that with
an improved fabrication technique the composite can carry higher failure loads than reported here.

Enhancement of cement matrix properties with large volume of fibers has also been observed for fibrillated polypropylene fibers as shown in figure 9. The polypropylene composites were manufactured through a pulltrusion process (3), (5), resulting in a low void density, and uniform fiber dispersion. The ultimate strength of the matrix is shown to be significantly affected by the fiber volume fraction, type, specific surface area, and degree of dispersion, and the processing technique. Although the glass fiber specimens did not attain the degree of fiber dispersion obtained in polypropylene specimens, the increase in ultimate tensile strength of matrix can still be observed. The post-peak behavior of the matrix is also significantly enhanced since at 0.2% strain the glass fiber composites carry roughly 2 MPa.

It is hence expected that through more elaborate specimen preparation techniques, one can achieve significant increase both in the strength, and ductility of the cement based composites. The extent of improvement is likely to depend on the volume of the fibers, the type of matrix, and the nature of bond between the fiber and the matrix.

The physical interpretation of this strengthening mechanism could be attributed to the inherent property of cement based materials which exhibit a stable microcracking region prior to the unstable propagation of cracks. The interaction of fibers in closing, and thus resisting the propagation of the cracks could result in the observed macroscopic strengthening.

From an experimental point of view, observation, and a quantitative characterization of microcrack formation in polypropylene fiber cement based composites has been attempted by means of fluorescent optical microscopy (5). Formation, interaction, and propagation of microcracks in similar specimens has also been studied by means of LASER holographic interferometry (8).

The theoretical modelling of the problem can be achieved through an R-curve type formulation to the response of plain materials. The goal of this approach is to use the existing models for fracture of cement based materials to develop the relationship between the fracture toughness and the crack length (9). Role of fibers on the fracture of matrix can be modelled by means of the traction across the crack surfaces. Using single fiber pull-out test results, effect of these forces can be considered by means of Reduction of the stress intensity factor, and closure of crack surfaces. These effects can be implicitly included into the matrix fracture criterion, and thus obtaining theoretical matrix strength in the presence of the fibers (10).
The aging of glass fibers in the presence of highly alkaline pore solution of cement matrix has been discussed significantly in detail elsewhere. There are two mechanisms which are believed to be operative. These are: the CH⁻ attack of glass fibers and the embrittlement of glass fiber strand due to filling up of the interstitial pores. A conventional test to evaluate the aging effects is strand in cement test (SIC) which consists of testing of a single strand of fiber embedded in a cement paste. Uncoupling of the two mechanisms discussed above, is not possible to be achieved by means of the SIC test. The reduction of fiber strength due to CH⁻ attack can however be studied by the methodology developed in this report.

The glass-epoxy test specimens discussed earlier were loaded to ultimate failure. Figure 10 represents the plot of composite strength as a function of fiber volume concentration; it is shown that the composite strength increases as a function of fiber volume concentration. Note that the difficulties encountered in testing of a single strand can be overcome by embedding the glass fibers in the epoxy. Assuming that both phases are linearly elastic, combined with a further assumption that the failure of the composite occurs at the ultimate strain of fibers' (ε_{fu} < E_{emu}), one can use the rule of mixtures to obtain the ultimate strain of the fibers.

\[
\epsilon_{fu} = \frac{\sigma_c}{E_m V_m + E_f V_f}
\]

(3)

Where \(\sigma_c\) represents the strength of the composite, \(E_{emu}\), and \(\epsilon_{fu}\) represent the ultimate strain of matrix and fibers respectively. Using this approach, the strain to failure of the AR-glass fibers is obtained to be 1.52 ± 0.4%. Note that it was observed that plain epoxy had an ultimate strain of \(\epsilon_{emu}=1.7\%\) which justifies the assumption made earlier. Using a limited number of tests, glass fibers were subjected to aging at 50°C in saturated CH⁻ solution for 30 days. Fiber epoxy specimens were then cast using the aged fibers. By testing the fibers independently, the strength reduction due to aging can be independently accounted for.

Figure 11 represents the normalized stiffness and strength of the composites studied. It is noted that the stiffness of the fibers does not significantly change due to aging, while there is a reduction of roughly 70% in strength of fibers due to aging, this reduction can significantly affect the ability of fibers in crack stabilization, thus resulting in the overall aging response.
Based on the results discussed above, it is expected that at different loading levels different mechanisms are present. In order to characterize the role of fibers, an understanding of how the fibers affect matrix properties is needed. Such understanding can be achieved by means of an in-depth quantitative study of the microcrack evolution and a micromechanical approach to the problem which is currently underway at Northwestern University.

CONCLUSIONS

Development of a test method to evaluate the response of individual phases in a composite material are discussed. Tensile response of cement paste in the presence of glass fibers was shown to be dependant on the fiber type and volume. The role of fibers in acting as crack arresters thus increasing the toughness and apparent strength of the composite were also investigated. Strength degradation of the glass fibers in the presence of calcium hydroxide can also be measured.

ACKNOWLEDGEMENTS

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REFERENCES


Table 1. Summary of Experiments

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Fig. 1—The specimen preparation set-up
Fig. 2--Schematics of the specimen and the elongation measurement set-up
Fig. 3--Load-elongation response of glass-epoxy composites

Fig. 4--Stiffness of the composite vs. the volume fraction of fibers
Fig. 5--The computed, and the experimental response for plain epoxy

Stress–Strain for Epoxy

Calculated by:

\[ E_c = E_t V_f + E_m V_m \]

\[ E_t = 11.5 \times 10^6 \text{ psi} \]
Fig. 6--Load vs. elongation of a composite with 5% volume fraction of fibers.
Fig. 7--Initial response of a specimen with $V_f = 4\%$

Local strain
(gage length = 0.039 in, 1 mm)

Overall strain,
gage length = 3 in

Specimen 4830, $V_f = 4\%$

$E_c = 5.2 \times 10^6$ psi (initial)
Matrix Contribution to the composite
Under Assumptions of
1) Rule Of Mixtures,
2) Perfect Fiber Matrix Bond

Fig. 8—Average stress strain response of
matrix for $V_f = 4\%, 5\%, \text{and } 6\%$
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Design Considerations for GFRC Facades

by R.G. Oesterle, D.M. Schultz, and J.D. Glikin

Synopsis: Thin-walled glass fiber reinforced concrete (GFRC) panels are used as facade systems for commercial structures. Wind load and gravity load are primary load cases typically considered in panel design. However, since the GFRC skin is relatively thin, it responds rapidly to thermal and moisture variations. Therefore, minimizing restraint of the GFRC skin movement under varying environmental conditions and/or determination of stresses resulting from restrained movement are also primary considerations in GFRC facade panel design.

This paper addresses concepts for design of GFRC panels including material behavior, design strengths, and loading combinations. Discussions of load conditions and recommended design considerations are presented for the effects of manufacturing, handling, and erection loading, gravity loading, wind loading, and loading due to external and internal restraint of moisture and thermal movements.

This paper is based on the authors' experiences during their involvement in the design process for several new GFRC installations along with observations made and lessons learned in evaluation of GFRC facade failures.

Keywords: cladding; composite construction (concrete and steel); cracking (fracturing); facings; fiber reinforced concretes; glass fibers; loads (forces); panels; restraints; shrinkage; structural design; temperature; tensile strength; wind pressure
INTRODUCTION

Lightweight glass fiber reinforced concrete (GFRC) panels have been used in the United States to create architecturally pleasing building facade systems for commercial structures. Current design of the GFRC panels in the United States incorporates a rigid, lightweight steel stud framework supporting a thin GFRC skin. Figure 1 illustrates a section through a typical GFRC facade panel. Vertical and horizontal loads are transferred from the skin to the steel stud system through the use of flex-anchors and gravity anchors. Figure 2 shows a typical flex-anchor connection. The flex-anchor is welded to the steel stud, and connected to the GFRC skin.
through use of a bonding pad of GFRC material that is compacted around the extended foot of the flex-anchor. Figure 2 also illustrates another feature of the GFRC skin. That is, commonly, a face mix or architectural finish is used on the exterior surface of the skin. The GFRC is typically on the order of 1/2-in. thick. The face mix is typically 1/8 to 3/8-in. thick.

Gravity anchors can be fabricated in a number of ways. Figure 3 shows a typical gravity anchor formed by use of a diagonal rod in conjunction with a flex-anchor and a common bonding pad to form a truss.

Since the GFRC skin is relatively thin as compared to normal reinforced or prestressed concrete, it responds more rapidly to thermal and moisture variations. Therefore, the primary concept of design of GFRC facade panels using flex-anchors and gravity anchors is to allow free movement of the skin under varying environmental conditions. The anchoring system has to allow for free movement while still providing sufficient strength to carry the wind loads, seismic loads, and dead loads through the skin to the anchors and to the steel stud framing system. A satisfactory design for the skin and connections must eliminate excessive external or internal restraint of skin movement.

This paper presents concepts for design of GFRC facades including discussion of the fundamental material behavior of GFRC that must be considered in design. The paper also presents information on design strengths for the material and connections, various loading conditions, and some of the sources of problems that should be considered in design of GFRC panels.

CONCEPTS OF DESIGN

Material Behavior

Flexural behavior -- Typical stress-strain plots for flexural coupons are illustrated in Fig. 4. Figure 4(a) illustrates a flexural test result for GFRC at an early age. The response is essentially linear up to a stress that is noted as early flexural yield (EFY). The EFY is also called the proportional elastic limit (PEL) or the limit of proportionality (LOP). At an early age, the material is quite ductile and will demonstrate increasing stress and strain beyond yield up to ultimate strength indicated by early flexural ultimate (EFU) in Fig. 4(a). The EFU is also commonly called the modulus of rupture (MOR).

Even though at an early age the material has significant strength and deformation capacity beyond the EFY, the present design concepts for GFRC only utilize the linear portion of the
stress-strain response up to the EFY. This approach is similar to the design approach for other materials such as steel or reinforced concrete. However, the flexural test results in Fig. 4(b) for "aged" GFRC show how GFRC differs from these other construction materials. Flexural tests of aged GFRC material indicate that the GFRC loses a significant portion of its ultimate strength and ductility with aging [1], [2]. Figure 5 illustrates the typical flexural strength versus time relationship for GFRC. With continued outdoor exposure in most climates, the ultimate strength decreases and, in the "aged" condition, will approximately equal the yield strength. In this "aged" condition, GFRC is a brittle material. Design concepts must consider the brittle nature and lowered strength of the aged material.

Figures 4 and 5 relate to the flexural capacity of GFRC. The flexural strength is a calculated strength determined from test results assuming a linear stress-strain relationship and a linear strain gradient through the thickness of the flexural coupon. In reality, the stress-strain relationship is not linear and the stress calculated from the flexural test is not a true indicator of the direct tensile capacity of GFRC material.

Direct tensile behavior -- Results of testing carried out to measure a direct tensile strength of GFRC material with no strain gradient through the thickness are shown by the lower line in Fig. 6. The ETY indicated in Fig. 6 is the early tensile yield and the ETU indicate the early tensile ultimate for direct tension. These terms are also frequently called the bend over point (BOP) and the ultimate tensile strength (UTS). As shown in Fig. 6, testing indicates that the direct tensile strength is significantly lower than the flexural tensile strength. This is an important consideration in the design of GFRC. The designer must consider the appropriate measure of strength for the type of stress condition under consideration.

Moisture movement -- A significant material behavior of GFRC that must be anticipated in design is the moisture movement that occurs in GFRC. The moisture movement characteristics of GFRC are illustrated in Fig. 7. Since GFRC has a high ratio of volume of cement paste to volume of aggregate, the amount of shrinkage expected may be two to four times the shrinkage expected in normal concrete. Because the GFRC material is thin, a large portion of the initial shrinkage can be expected to occur in a relative short time. Also, GFRC can recover approximately two-thirds of the initial shrinkage. This shrinkage recovery is called reversible shrinkage. A facade panel on a building will be continually expanding and contracting with environmental moisture changes. As a result, these movements must be carefully addressed in the analysis and design. Examples of design considerations for effects of
external and internal restraint of moisture movements are presented in a following section of this paper.

Thermal movement -- A facade panel will also be continually expanding and contracting in response to temperature changes. The panel skin can be subjected to temperatures ranging from a low well below freezing to a high of approximately 140°F with exposure to direct sunlight. Data published in the current PCI Recommended Practice [3] indicate the coefficient of thermal expansion typically might range from 6 to 9 millionths per °F. This is somewhat higher than the range of coefficients of thermal expansion for normal concrete. Also, there is some evidence indicating that the coefficient of thermal expansion for GFRC material, being a high cement paste content material, may be significantly higher than 9 millionths per °F. The coefficient of thermal expansion for hardened cement paste can be approximately two times the values stated for GFRC in the PCI Recommended Practice depending on the moisture content within the cement paste [4]. However, there is a complex interaction of shrinkage strains and thermal strains when an exposed panel undergoes temperature changes. Therefore, the specific value that should be used for design of GFRC is a subject for further research. Until further data become available, the use of at least 12.0 millionths per °F has been recommended based on test data published by BRE [5]. The test data was measured on large panels subjected to continuous natural weathering in the United Kingdom. It should be noted that the BRE coefficient was developed based on test data of GFRC panels with a higher cement/sand ratio than is currently being used in the United States and, therefore, should provide conservative results.

Both thermal and moisture movements are very critical when the GFRC material is combined with a face mix material that has properties significantly different than the GFRC material. Differential movement problems associated with the interaction of face mix and GFRC will be discussed further under "internal restraint" in this paper.

Design Strengths

Flexural strength -- The primary equation presented in the PCI Recommended Practice [3] to determine the design flexural strength \( f_u \) is as follows:

\[
\begin{align*}
  f_u &= \phi f' u \\
  f' u &= \text{lesser of:} \\ \\
  &= f_y (1 - tV_y)/0.9 \\
  &= 1/3 f_u (1 - tV_u)/0.9 \\
  &= 1300 \text{ psi}
\end{align*}
\]
$f_u'$ = assumed aged ultimate flexural strength for design purposes. It is taken as that value which, statistically, no more than 1% of the tests will fall more than 10% below.

$f_{yr}$ = average 28-day flexural yield strength of 20 consecutive tests (of 6 specimens each)

$f_{ur}$ = average 28-day flexural ultimate strength of 20 consecutive tests (of 6 specimens each)

$t$ = Students $t$, a statistical value to allow for the number of tests expected to fall below $f_u'$. It varies substantially with the number of tests. The value is 2.539 for the recommended 20 tests (19 degrees of freedom)

$V_y, V_u = \text{coefficient of variation of the yield and ultimate test values: } V_y = \sigma_y/f_{yr} \text{ and } V_u = \sigma_u/f_{ur}$

$\sigma_y, \sigma_u = \text{sample standard deviation (n-1) of the yield and ultimate tests}$

$\phi$ = strength reduction factor = 0.67

The value of $f_u$ is the design strength to use for situations involving out-of-plane bending with a strain gradient through the thickness of the material. Typical values for $f_u$ are on the order of 400 to 800 psi.

The strength reduction factor ($\phi = 0.67$) is intended to cover, among other things, the variation in material dimensions and inaccuracies in design equations. It should be pointed out that the level of safety afforded by use of a $\phi$ factor of 0.67 can be easily diminished by poor quality control of material thickness alone. Since the material is relatively thin, a small error in fabrication thickness can result in a large change in flexural strength. The difference between the section modulus of 1/2-in. thick material vs. 3/8-in. thick material is 44%. Therefore, the $\phi$ factor is already overcome if the material is fabricated 1/8 in. too thin. The GFRC material should be given a minus zero tolerance on the design thickness.

**Direct tensile strength** -- As illustrated in Fig. 6, the direct tensile strength of GFRC material is lower than the flexural strength. An additional 0.4 factor is included in the PCI Recommended Practice [3] for limiting direct tensile stresses associated with in-plane shear, the 0.4 factor should also be used to determine the design strength of situations where direct tension governs. A typical value for $f_u$ for direct tension is on the order of 200 to 300 psi.
Connection strength -- Another design strength to consider is the strength of the connections. The critical strength is generally the pull-off or axial strength of the flex-anchor bonding-pad connection for resistance of the negative or "suction" wind load. It is recommended that the design strength of connections be based on tests of aged specimens [3]. The amount of strength reduction in connections subjected to aging can vary as a result of manufacturing procedures. It has been CTL's experience that the strength reduction can range from approximately 30% to 50% of the unaged strength. Therefore, it is recommended that the design strength be based on tests of representative aged specimens as fabricated by the intended manufacturer of the panels. Since the failure of aged specimens is brittle, it is recommended that the design strength be based on a minimum factor of safety of 4 with respect to the average aged bonding pad strength. However, if the coefficient of variation of bonding pad strength is relatively large, consideration should be given to using a larger factor of safety. The appropriate factor of safety to use, as a function of the coefficient of variation for the connection strength, is an area for further research.

Load Combinations

The design concept presented in the PCI Recommended Practice [3] is a factored load concept. The main loading combination to be considered as a minimum for the GFRC panels includes dead load, wind or earthquake load, and moisture or thermal load in accordance with the following factored load combination presented in the PCI Recommended Practice [3]:

$$0.75[1.4D] + 1.7 (\text{greater of } L, W \text{ or } 1.1E) + 1.6 (\text{greater of } M \text{ or } T)$$

where:

- \(D\) = dead load
- \(E\) = earthquake load
- \(L\) = live load
- \(M\) = self-straining forces and effects arising from contraction or expansion due to moisture changes
- \(T\) = self-straining forces and effects arising from contraction or expansion due to temperature changes
- \(W\) = wind load

The factor for moisture and temperature change is greater than that for normal concrete construction due to the uncertainties in values and calculation procedures and the greater potential effect if underestimated.
It should be emphasized that moisture and thermal loading must be considered as primary loading conditions for GFRC panels in that these two load conditions can be the major sources of stress.

Considerations for Brittle Behavior

It should be emphasized that the aged GFRC material is a brittle material. Therefore, the overall design philosophy for GFRC includes an attempt to maintain total stresses below the "aged" cracking strength of the material. Typical civil engineering materials such as steel, reinforced concrete, or prestressed concrete, possess ductility. Although ductility is not usually directly considered in the design process, it is implicitly considered through the use of simplified analyses for the structure and for the connections within the structure. Some of these simplified design procedures inherently consider that the material has the capacity for plastic strain at stress concentration points to facilitate redistribution of loading accompanying ductile yield in local regions. "Aged" GFRC is a brittle material and should be expected to have little capacity for redistribution of loads. Therefore, the brittle nature must be considered in design for handling, erecting and supporting the material in-place.

LOADING CONDITIONS

Manufacturing, Handling, and Erection

The critical loading conditions for a GFRC facade panel may occur during demolding of the panel. GFRC is generally sprayed-up in horizontal molds, then the frame with the flex-anchors is attached to the fresh GFRC skin with bonding pads and the whole panel is left to cure. The following day, the panel is stripped or demolded by applying a preload to the lifting hardware. The panel may separate easily from the mold under the preload. However, it is not uncommon for panel to require additional load to release from the mold. It has been CTL's observation that manufacturers may jar the panel free by hitting one of the legs of the lifting slings or by prying on a corner of the steel frame. As the panel releases from the mold, the panel may literally jump out of the mold. Therefore, there are some dynamic forces or impact loads to be considered during demolding. Also, it should be considered that the strength at this very early age of both the GFRC skin and the flex-anchor bonding pads is only one-half to two-thirds of the 28-day strength.

The magnitude of the dynamic forces can be minimized by controlling the preload placed on the lifting slings. If the preload is limited to approximately 125% of the dead load on the panel, the maximum effective load on the panel from the
Dynamic shock of release from the mold is on the order of 2.5 times the dead load.

In addition to minimizing stresses in the GFRC material, stresses in the steel frame during demolding, handling, and erection must be considered in the design process. Lifting and handling procedures should be analyzed and specified by the designer to minimize the level of stresses and deflections in the GFRC panel skin and steel frame.

Gravity Loading

The GFRC skin can be supported by two methods. One method is to use gravity anchors, which are relatively rigid connections, in combination with flex-anchors as shown in Fig. 1. The other method is to use only flex-anchors to support the dead load of the skin. An advantage of using gravity anchors is that the gravity anchors are generally placed near the bottom of the skin which tends to induce a slight compression into the GFRC material above the level of the gravity anchors. A slight compression in the skin helps to compensate for vertical tensile stresses which may develop from other loading conditions. Use of gravity anchors also allows the use of more flexible flex anchors.

A potential problem associated with gravity anchors, however, is that they are rigidly fixed points within the GFRC skin. Therefore, care must be taken to avoid restraint of potential skin movement towards the fixed points. Also, with the fixed points at the gravity anchors at the bottom of the panel, the deformations at the top of a tall panel due to the accumulated strain in the skin can be large.

With use of only flex-anchors to support gravity load, the point of zero movement in the skin for thermal or moisture changes is the centroid of the flex anchor resistance, generally located at the center of the panel. Therefore, total vertical movement at the extreme edge of the panel is reduced as compared to a panel with gravity anchors at the bottom. However, use of only flex-anchors to support gravity loads in larger, heavier panels requires relatively stiff flex-anchors which may produce excessive restraint of moisture and thermal related movements.

Another consideration for gravity load is the effect of offsets in the section. In flat panels, the gravity loads are transmitted through the plane of the skin to the gravity anchors. This places a slight compression in the skin which helps compensate for tensile stresses from other loading conditions. However, in the situation illustrated in Fig. 8(a), the gravity load on the upper half of the panel has to be carried through the offset in the geometry through bending in the GFRC skin. Excessive bending stresses can be
produced by this type of situation and must be considered in design. The bending stresses can be partially relieved by use of internal diagonal braces as shown in Fig. 8(b). However, bending stresses associated with the modified load path must still be considered in design.

Another potential problem with gravity loads is the use of multiple layers of gravity anchors. Gravity anchors are relatively fixed points on the GFRC skin. Therefore, there should only be one level of gravity anchors within the skin. A potential problem is illustrated in Fig. 9 with a panel designed to fit around an opening producing a re-entrant corner in the bottom left of the panel. A designer might locate the gravity anchors for the left side of the panel above this opening and at a different level than the anchors on the right side of the panel. With the two levels of gravity anchors, there will be some restraint of the skin in the region of the re-entrant corner and therefore, a potential for cracking. The gravity anchors within a panel should all be located along the same horizontal level within the panel.

Wind Loading

The capacity of the GFRC skin to resist pressure from wind loading is generally considered using a flat plate analysis assuming the skin to span between the flex-anchors similar to a floor slab spanning between columns. The wind load analysis must consider both inward (or positive) pressure and outward (or negative) pressure. The designer should be attentive to the boundary conditions of the GFRC skin considering both typical interior spans and exterior spans near edges of the panels or near openings. Also the effects of cantilevers must be considered.

A potential source of a long cantilever is the use of vertical and horizontal returns on the panels. Figure 10 illustrates the support of a horizontal return on the top of a panel. The wind load on the skin acts normal to the surface of the panels. Therefore, the wind load on returns is normal to the surface of the returns. Without additional support as shown in Fig. 10, returns frequently constitute very long cantilever conditions. Therefore, additional support has to be provided. However, when providing this support, the potential for restraint of skin movement must be considered. Use of an anchor as shown in Fig. 10(a) will restrain the connection point very rigidly and induce significant bending stresses in the return as the skin wants to shrink towards the gravity anchors at the bottom of the panel. Therefore, use of flexible anchors to support the return, as shown on 10(b), or use of internal brace as shown on 10(c) is recommended. Use of a flexible anchor requires balance between the support strength to resist the wind load and the flexibility to allow movement.
in the skin. Use of the internal brace requires consideration of the bending stresses associated with the load path.

Another consideration for wind load is the effects of the relative stiffness of the stud system. Figure 11 illustrates a support frame for a GFRC panel. The dots in Fig. 11 indicate the connection points of the frame to the building. It is not unusual to provide a stronger vertical member to carry the reaction loads at the connection points to the building. In the framework in Fig. 11, the two vertical members with connecting points are structural tubes whereas the remainder of the vertical members are cold-formed studs. The horizontal members at the top and bottom are cold-formed track. Because the vertical tube members are connected to the building and because these members are significantly stiffer than the other vertical studs, the frame does not provide uniform support to the skin.

Figure 12 illustrates the resulting bending stresses determined from an elastic finite element analysis of the 3/4-in. thick skin connected through flex-anchors to the frame shown in Fig. 11. Figure 12 shows a plot of stress contours for bending stress in the horizontal direction. It can be seen from these stress contours that the skin, even though not considered in design, is tending to span between the flex-anchors on the stiff tubes with significantly less support from flex-anchors on the flexible cold-formed studs. This behavior significantly increases the bending stress in the GFRC skin. It also adds significantly to the axial load being carried by flex-anchors on the stiffer members. This behavior, which is typically not considered explicitly in normal design procedures, must be recognized by the designer since it can significantly deplete the conservatism provided by use of the \( \phi \) factor for skin stress design and the safety factor for design of connections.

The designer can minimize the additional stresses from this type of behavior by using a more uniformly stiff framing system to support the GFRC material. The PCI Recommended Practice [3] indicates the frame should be designed to limit maximum deflections to a value of \( L/360 \). However, depending upon the span and skin thickness, a deflection of \( L/360 \) corresponds to additional skin stresses of approximately 100 to 300 psi. Therefore, consideration should be given to stiffer steel framing systems.

**Moisture and Thermal Movement**

The effects of moisture change and temperature change in the GFRC skin are very similar. The skin will expand or contract with moisture changes and with thermal changes. Either external restraint or internal restraint to the volume change will result in stresses.
External Restraint -- An example of external restraint to the GFRC skin movement is shown in Fig. 13. With offsets in the skin geometry, there is a potential that the designer will use a relatively short flex-anchor to attach the skin to the steel frame as illustrated in Fig. 13. A short flex-anchor will act as a fairly rigid connection to the skin. Therefore, for the situation illustrated in Fig. 13, the skin between the short flex-anchor and the gravity anchor at the bottom of the panel is restrained producing additional bending stresses in the offset region of the skin.

Another example of external restraint is the support of returns discussed earlier and illustrated in Fig. 10. Volume change within the GFRC skin will create movement relative to a zero reference point located at the line of gravity anchors near the bottom of the panel. Therefore, returns along panel edges, must be provided with relatively flexible supports.

A third example of external restraint is illustrated in Fig. 14. This figure shows an elevation view of an L-shaped panel. The centroid of the flex-anchors connected to the skin is shown on Fig. 14a. As the skin shortens with shrinkage or thermal strains, the flex-anchors will resist the movement. The flex-anchors farthest from the centroid of the movements will exert the highest force on the GFRC skin. The concept of flex-anchors is to limit these forces with relatively flexible anchors. However, with a large number of flex-anchors, forces within the skin accumulate to significant levels and depending upon the shape of the panel, these in-plane forces can produce high stresses.

Consider the resultant of the flex-anchor forces acting on the outstanding vertical leg of the panel shown in Fig. 14b. The resultant force produces an in-plane moment within the skin that can lead to high tensile stresses and possible cracking at the re-entrant corner of this panel.

It should be noted that the previous two examples (Fig. 10 and Fig. 13) of stress resulting from external restraint relate primarily to flexural stresses with out-of-plane bending of the GFRC skin. Therefore, the flexural design strength should be used to evaluate those stresses. However, for the case shown in Fig. 14, the stress induced by restraint is an in-plane stress with little or no gradient through the thickness of the material. In this case, the lower design strength for direct tension should be used.

Another example of external restraint that produces direct tension in panels is simply the use of tall panels with a large number of flex anchors. With tall panels, relatively high tensile stresses can accumulate near the fixed line of the gravity anchors along the bottom of the panel. Another consideration for a tall panel is the amount of cyclic
deformations that can occur in the flex-anchors near the top of the panel. As the panel skin undergoes volume change as a result of an environmental change, there is a potential that the flex-anchors near the edges of tall panels will undergo reverse cyclic yielding of the steel flex-anchor rod which may result in a low cycle fatigue problem. The dimensions of panels should be limited to minimize the potential for high stresses due to accumulated flex-anchor restraint and to eliminate the potential for reverse cyclic yielding of flex-anchors.

Internal restraint -- An example of internal restraint is illustrated in Fig. 15. This figure shows the result of an elastic finite element analysis for a GFRC panel having a vertical return along one edge. The analysis considered a differential temperature within the skin of the panel simulating the effects of radiant solar heating on the large face of the panel with the vertical return being in the shade. A typical temperature differential of 35°F between the face and the vertical return of the panel was used in this analysis. The face of the panel will expand with increased temperature and the return portion of the panel will resist this expansion. Therefore, the skin is restrained internally by its own shape. Figure 15 shows the contour plot of the stresses in the vertical direction. The highest tensile stress occurring at the corner of the panel on the face of the return is on the order of 600 psi. It should be noted that this is a stress in the plane of the GFRC material with little or no gradient through the thickness. Therefore, the tensile stresses resulting from this loading situation should be compared with the design strength for direct tension.

Another major source of internal restraint to moisture and thermal movements is illustrated in Fig. 16. Figure 16(a) consists of a composite piece of panel with the GFRC skin connected to the face mix. If the two materials were not connected and if the GFRC had different properties than those of the face mix, the two materials would move different amounts in response to moisture or temperature change as illustrated in Fig. 16(b). However, as shown in Fig. 16(c), the two materials are connected and internal restraint is produced. If the materials were maintained flat, as illustrated in Fig. 16(c), the GFRC would be in direct tension and the face mix would be in direct compression.

Because of the internal couple associated with the restrained stresses, the composite material will bow if not restrained externally. Bowing relieves some of the internal stress associated with the differential movement of the two materials. However, there still are some residual stresses retained within the material in its bowed shape. Also, the material is restrained from bowing by the steel stud system through the connecting flex-anchors. The restraint from the
steel stud system holds the GFRC panel in a relatively flat condition. Even though restrained bowing might appear to induce a flexural load on the skin, the actual stress induced is relatively uniform through the thickness of the GFRC and face mix. Therefore, the design strength for direct tension should be considered in this situation. The level of stress from restraint bowing can be calculated using equilibrium and compatibility relationships. Also it should be noted that creep in the materials will relieve some of the stresses produced from internal restraint. The amount of stress relief due to creep under internal restraint conditions is a topic for further research. However, a solution for reducing effects of differential movement between the GFRC and face mix is to use a face mix with material properties similar to those of the GFRC and to keep the face mix as thin as possible.

CONCLUSIONS

Although GFRC facade panels might be considered to be relatively simple structures, the design of GFRC facade panel considering all the potential sources of high tensile stress can be a very complex problem. It is recommended that, in order to keep the design problems simple, the panels should be kept simple. The GFRC in the panels should be predominantly flat pieces that are kept relatively small using real joints in the skin. During the design process, the designer should attempt to visualize all of the potential movements that the skin will undergo with shrinkage, shrinkage recovery, and thermal changes, and visualize the potential restraints to these movements. The designer should then eliminate or minimize the restraints by providing joints or flexibility to the support system.

Also, the designer, manufacturer, and building owner must realize that, because GFRC loses strength and ductility with "aging," the use of GFRC facade panels is still a relatively new and unproven concept with a limited performance data base. The complete viability of the GFRC panel concept will not be demonstrated until a sufficient data base of buildings with "aged" panels demonstrates good performance. It was the intent that the material presented in this paper will help minimize the potential for misuse of GFRC panels.

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Fig. 1 GFRC Facade Panel

Fig. 2 Flex-Anchor Connection
Fig. 3 Gravity Anchor

Fig. 4 Flexural Behavior
Fig. 5 Flexural Strength Versus Time

Fig. 6 Comparison of Flexural Strength and Direct Tensile Strength
Fig. 7 Moisture Movement Characteristics of GFRC
Fig. 8 Effect of Offset in Skin Geometry on Gravity Load Stresses
Fig. 9  Panel With Multiple Layers of Gravity Anchors

Fig. 10  Support of Panel Returns
Fig. 11 Support Frame for GFRC Skin

Fig. 12 Bending Stress Contour With Wind Load Due to Variable Stiffness of Support Frame
Fig. 13 Example of External Restraint of GFRC Skin Movement
a) Flex-Anchor Forces Resisting Shrinkage or Thermal Movements

b) Resultant of Flex-Anchors Forced on Outstanding Leg May Produce Cracking

Fig. 14 External Restraint of Flex-Anchors on L-Shaped Skin
Panel Skin Exposed to Radiant Solar Heating

Panel Return in Shade

High Direct Tensile Stress in Return

High Tensile Stress

High Compression Stress

Fig. 15 Stress Contours Resulting From Internally Restrained Differential Thermal Expansion
Fig. 16 Restraint Due to Differential Strain in GFRC/Face Mix Composite Skin
Manufacture and Installation of GFRC Facades

by N.W. Hanson, J.J. Roller, J.I. Daniel, and T.L. Weinmann

Synopsis: Thin walled, non-load bearing exterior building facade panels of Glass Fiber Reinforced Concrete (GFRC) are manufactured by the spray-up process. Controlled factory conditions with strict attention to quality control are essential to help assure manufacture of a high quality product. Furthermore, careful attention to installation and erection procedures cannot be overlooked.

This paper describes the authors' experiences during their involvement in several major GFRC facade installations. Observations made during successful GFRC panel applications and lessons learned in evaluation of GFRC facade failures have formed the basis for development of an effective Quality Control/Quality Assurance (QC/QA) program that has been successfully implemented.

The paper addresses QC/QA aspects of panel manufacture and installation that go beyond guidelines given in the PCI Recommended Practice. Methodologies presented in this paper will be a valuable tool for owners, designers, manufacturers, and contractors participating in the manufacture and installation of GFRC facades.

Keywords: cladding; facings; fiber reinforced concretes; glass fibers; installing; manufacturing; panels; quality assurance; quality control
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INTRODUCTION

In the United States, Glass Fiber Reinforced Concrete (GFRC) facade panels are used as an alternative to conventional precast concrete panels. Facade panels consist of a thin GFRC shell supported by a steel frame assembly. Connections between the GFRC shell and the steel frame are typically made using flexible steel anchors that are welded to the steel frame at one end and bonded to the GFRC shell at the other end via a GFRC bonding pad. A photograph of the front and back of a typical GFRC facade panel is shown in Fig. 1.

The lightweight of GFRC panels makes them particularly desirable and attractive to architects. A lightweight facade is an obvious advantage from a building design standpoint. However, the thin shell of the GFRC panel (typically 3/4 to 1-in. thick) makes them particularly sensitive to errors in design, manufacture, and installation.

It is a recognized and accepted phenomenon that GFRC, when exposed to the outdoor environment, will lose strength and strain capacity with time at a rate that is environment dependent. Under exterior exposure conditions, the strength of a GFRC composite will eventually decrease to the strength level of the cement matrix. Essentially, all glass fiber reinforcing qualities are lost, as illustrated in Fig. 2. The time required for this loss in strength and ductility will vary from 20 to 30 years in cooler, dryer climates such as Canada to as little as 5 to 8 years in warmer, more humid climates such as Florida.

Although it is the aged strength of the composite which is used in panel design, unanticipated excessive stresses resulting in early cracking of the GFRC skin could develop as a result of design, manufacture or installation errors. Early cracking may not become apparent until the panel becomes aged.

The increasing use of GFRC facade panels warrants a close look at what is needed to help ensure a durable and attractive product. The Precast/Prestressed Concrete Institute (PCI) "Recommended Practice for Glass Fiber Reinforced Concrete Panels" (1) provides guidelines for the design, manufacture and erection of GFRC facade panels. However, special attention must be given to certain quality control/quality assurance (QC/QA) aspects. The PCI recommended practice touches upon many of the important QC/QA requirements but does not provide specific details of how to implement or apply them. This paper addresses quality control/quality assurance requirements relating to panel manufacture and installation. A related paper addressing design considerations for GFRC has been prepared by others (2).
QUALITY CONTROL/QUALITY ASSURANCE PROGRAM

Quality Control/Quality Assurance Manual

To help assure overall quality of GFRC facade panels, it is necessary for manufacturers to develop strict quality control procedures. These procedures should be precisely documented in a plant QC/QA Manual. A copy of the manual should be submitted to every worker involved in panel manufacture. It is necessary that each worker be fully aware of his or her own responsibilities and understand the importance of doing the job right.

In general, the QC/QA Manual should include everything that the PCI Recommended Practice includes, but in much greater detail. The PCI Recommended Practice addresses quality control aspects of panel manufacture in very general terms. The QC/QA Manual must state specifics regarding quality control, answering not only the questions of "how" but also "why." The QC/QA Manual should also include the quality assurance (QA) aspects of panel production. The manual should cover both the responsibilities and the authority of the plant QA inspector.

Quality Control Responsibility

In general, Quality control is the responsibility of everyone involved in panel manufacture. Although responsibility may ultimately lie with the plant manager or foreman, this responsibility must be delegated so that workers are held accountable for the tasks they perform.

Quality Assurance Responsibility

Even when panels are designed appropriately, special attention must be given during manufacture and installation to aid in assuring a high quality finished product from both an appearance and performance standpoint. Product quality assurance basically involves inspection and monitoring during panel production and installation. Quality assurance inspection is intended to catch errors of design, manufacture and installation that could compromise the intended performance of the finished installed product. Inspection information can be used in establishing trends to evaluate the overall consistency of the product and to identify areas where problems exist.

The manufacturer's quality control personnel (the workers) should be expected to prevent most errors of manufacture but cannot be expected to notice subtle errors or omissions that might cause future problems. An experienced engineer inspector with a sensitivity to the nature of GFRC is required to fulfill the necessary responsibilities associated with product quality assurance. The quality assurance inspector should be able to
effectively interact with the design engineers as well as the production personnel. A working knowledge of all facets of panel design, manufacture and installation is essential. The qualifications of QA inspector should not be compromised.

In order for the QA inspector to be able to function effectively, an appropriate level of authority must be inherent to the position. Specifically, errors or omissions discovered by the QA inspector must be resolved to the satisfaction of the QA inspector in a timely manner. Ideally, the QA inspector should report to the company president. The QA inspector's qualifications should be reviewed by the building owner for approval. In certain instances, it may be prudent for the building owner to hire third party, independent QA services. This provides a second level of QA to help ensure the successful design, manufacture, and installation of the GFRC panels.

QUALITY CONTROL ASPECTS OF PANEL MANUFACTURE

Fabrication of GFRC/Facing Shell

Materials -- The PCI Recommended Practice suggests that constituent materials conform to certain specifications. In addition to this criteria, it is also important that individual constituent materials come from the same supply source throughout the entire job to help ensure a reasonable level of product consistency. Slurry consistency is an extremely important factor with respect to obtaining a reasonably consistent fiber content throughout panel production. Therefore, it is important that the QC/QA Manual address the parameters that affect slurry consistency. Cement is a primary consideration with respect to slurry consistency since different types and brands of cement will produce noticeably different slurry consistencies. Consistency of the sand with respect to color and moisture content is also important in developing a consistent slurry. Variations in mixing water temperature also have a noticeable effect on slurry consistency.

Compatibility of materials is something that also should be addressed in the QC/QA Manual. In many instances, it may be necessary to experiment with mix designs prior to commencing with the panel production to verify material compatibility. Tests of various mix designs to determine differential volumetric changes due to moisture and temperature variations, specifically between face mixes or facing materials and the GFRC backing mix, should be conducted to provide input for panel design as well as providing a basis for establishing actual mix designs.

Facing -- Manufacture of a panel starts with spraying a slurry mist coat or placing a thin layer of concrete against the form surface. The slurry mist coat or concrete layer makes
up the face mix which will be the exposed outer surface of the finished panel. The primary concern with the face mix is uniformity and thickness. Thickness control of this first unreinforced layer is important to ensure that sufficient material is available for sandblasting or other surface treatments. However, it is critically important that the facing thickness be controlled and uniform since the thickness of the GFRC skin, to be subsequently applied on top of the facing, will be determined based on a measurement of the total thickness of the shell (GFRC plus face mix thicknesses). A non-uniform facing layer will result in non-uniform and possibly an insufficient GFRC skin thickness. Special attention to facing thickness is needed at false joints (reveals) and at corners to ensure adequate thickness of the GFRC skin so that corners are sufficiently strong. A section of the GFRC/facing shell at a corner and a false joint, illustrating the need for special attention at these locations, is shown in Fig. 3.

GFRC skin — The GFRC backing mix is typically applied using the manual spray-up procedure. During the manual spray-up procedure, a cement, sand, and water slurry is simultaneously sprayed with chopped glass fiber strands directly onto the form surface or pre-placed facing material. The PCI Recommended Practice suggests that the GFRC backing mix be sprayed in a criss-cross pattern so that uniform thickness and distribution of glass fiber and cement matrix are achieved during the application process. However, the orientation of the hand-held spray gun is also an important factor that affects composite uniformity and fiber distribution. The spray gun should ideally be oriented at 90°(± 30°) to the mold surface and should be kept a distance of approximately 2 to 4 ft away from the mold surface at all times. Spray-up according to these guidelines will result in a more uniform fiber distribution and composite consistency.

The GFRC backing mix is typically applied in layers. After application of each layer, the composite is compacted using special rollers that encapsulate the glass fibers into the matrix. Poor composite compaction will result in air voids and reduced strength properties. Therefore, it is important that the compaction procedure be performed correctly. The thickness of each GFRC layer has a lot to do with the effectiveness of the compaction procedure. The thickness of individual GFRC layers should not exceed 1/4 in. (25.4mm). Layers that are thicker than 1/4 in. (25.4mm) cannot be adequately compacted without using excessive pressure. Excessive rolling pressure causes material to be displaced (pushed away) resulting in potentially substandard GFRC thickness. Special care must be taken at the location of returns, reveals and false joints to prevent displacement of sprayed GFRC material during compaction. Vertical returns (as oriented in the form) should
always be rolled with an upward motion to oppose the tendency of the sprayed material to slide downward under its own weight.

As noted earlier, special rollers are necessary to compact the composite and encapsulate the glass fibers in the cement matrix. These rollers should be ribbed so that they press the fibers into the cement matrix. Rollers with flat surfaces cannot achieve this objective and therefore should not be used.

**Verification of fiber content** -- Glass fiber content of the GFRC skin can vary greatly if spray operators fail to make routine equipment adjustments or conduct periodic tests to verify fiber content. One potential cause of fiber content variation is inconsistencies in the cement slurry. Inconsistencies in the slurry can be caused by many different factors. Many of these factors can be controlled by the manufacturer. However, factors such as ambient conditions are often beyond control of the manufacturer.

Consistency of slurry mixes can be checked by performing a slump test. If slump test results indicate that the mix consistency varies significantly from mix to mix, chemical admixtures should be used to make consistency adjustments. However, in production, routine use of a slump test may be impractical. Therefore, some manufacturers have installed ammeters on their mixers to provide some indication of slurry consistency. An ammeter can provide an indication of slurry consistency during the mixing operation by measuring the "pull" on the mixer motor. Typically, chemical admixtures are used during mixing to achieve consistent ammeter readings from batch to batch.

Other potential causes of fiber content variation are changes in the spray-up equipment itself. Initial equipment adjustments (or calibrations) are established based on results of fiber flow rate tests and slurry flow rate tests which should be conducted at the start of each production shift. However, additional adjustments may be necessary if changes in the supply air pressure occur or if the equipment has been sitting idle for a period of time. Supply air pressure should be monitored regularly during each shift and spray-up equipment should be cleaned thoroughly when allowed to sit idle for substantial periods of time after use. Anytime the spray-up equipment is cleaned or if there is a noticeable change in the supply air pressure, flow rate tests should be conducted and equipment adjustments should be made if necessary to achieve the target fiber content.

**Quality control** -- Quality control testing is required to ensure that properties and quality of the GFRC skin consistently meet design and specification requirements. Quality control test specimens are obtained from test boards which are fabricated at the same time as are the panels. These
test boards are intended to be representative of the GFRC skin and therefore must be manufactured using the same equipment, the same personnel, and the same overall workmanship as the panel product.

Test boards should be fabricated during each work shift for each spray-up unit used. Test boards should be fabricated large enough to incorporate two wash-out test specimens, six 2x12 in. (51x305 mm) flexural test specimens, and one 12x12 in. (305x305 mm) flex-anchor or gravity anchor pull-off test specimen. The test boards should be fabricated at a different time each day so that they represent the full range of production conditions and so they do not become part of a routine sequence of events. Test boards should be cured and stored under conditions identical to those used for the panels.

The wash-out test, flexural tests, and anchor pull-off tests should be conducted in strict accordance with the procedures outlined in Appendix A of the PCI Recommended Practice. Failure to conduct QC tests according to these procedures can result in significant error. For instance, the authors have observed that many manufacturers do not conduct the wash-out test with an appropriate degree of accuracy. Most of the inaccuracies observed to date were the result of using a weighing scale with an inappropriate degree of accuracy or using improper methods of drying out the glass fiber. In a typical wash-out specimen, failure to measure weights to the nearest 0.1 g (0.004 oz) can result in significant error in the final glass fiber weight percentage obtained. For the sake of saving time, some manufacturers do not use heat convection ovens to dry out the glass fibers. Microwave ovens cannot dry out the glass fibers sufficiently to provide an accurate result. The glass fibers must be free of moisture and residual sand particles prior to weighing. Baskets containing dried glass fiber should be repeatedly tapped on a table top to remove residual sand particles that were not removed during the actual slurry washout.

In general, quality control tests are only as good as the methods used to conduct the tests. If conducted correctly, results from these tests provide valuable information which can be used to establish trends for evaluating product quality. Test results should be documented on a daily basis and plotted so that they can be continuously compared with specified values and results from previous days of production.

Fabrication of the Steel Frame

The steel frame is designed to support the dead weight of the GFRC/facing shell and to transfer wind and seismic loads to the building structure. A typical steel frame consists of horizontal runners, vertical studs and steel flex-anchors. These members are assembled (usually welded) according to the
shop drawings and associated specifications. When assembling the frame, it is important that the workers pay close attention to both the location and orientation of each member and do not deviate from what is indicated by the shop drawings. Variations in location and orientation of frame members often result in excessive restraint of the shell which could be detrimental to the future serviceability of the panel.

Connections Between GFRC and Steel Frame

The anchors connected to the GFRC skin are designed such that they impose minimal restraint to dimensional changes of the GFRC/facing shell while still maintaining enough vertical and horizontal stiffness to transfer the panel service loads. The relatively thin shell responds quickly to changes in temperature and moisture. Fluctuations in temperature and moisture result in significant dimensional changes in the GFRC/facing shell. For this reason, it is important that the actual location and orientation of anchors agree closely with the design drawings.

A typical connection between the GFRC/facing shell and the steel frame is shown in Fig. 4. During panel manufacture, the steel frame, including the attached L-shaped flex-anchors, is positioned above the GFRC skin while it is still in a plastic (unhardened) state. When placing the frame, it is important that the GFRC is not damaged. Therefore, care must be taken so as not to drag the frame on the GFRC. Frame anchors should never actually contact the GFRC. The weight of the frame would cause damage to the unhardened GFRC. The PCI Recommended Practice suggests that a gap ranging from 1/8 to 3/8 in. (3 to 10 mm) should be left between the anchors and the GFRC surface. Any inadvertent damage that occurs to the GFRC during frame placement should be repaired immediately.

Immediately following placement of the frame, GFRC bonding pads are placed over the foot of each anchor and integrated into the GFRC skin. The GFRC used for the bonding pads should be the same as that used for the skin. Thickness of the bonding pad over the top of the flex-anchor should be at least equal to the specified thickness of the GFRC skin or a minimum of 1/2 in., whichever is greater. Details of a good bonding pad are shown in Fig. 5. Note that the bonding pad covers the entire foot of the flex-anchor exclusive of the heel, which should remain uncovered. The effective area of the bonding pad (effective length x effective width) should be approximately 18 to 32 sq in. (116 to 206 x 103 mm2) according to the PCI Recommended Practice.

Panel Identification Markings

Each finished panel should be clearly marked with the date of fabrication and an identification designation. These markings
Panel markings make it possible to tie quality control data to specific panels. They also facilitate documentation of panel production, panel inspection, and panel repair.

**Panel Handling Procedures**

Each manufacturer should develop specific panel handling procedures. These procedures should include demolding/lifting procedures, storage procedures, and procedures involved in preparing panels for transport. Lifting points should be determined by design engineers and specified on the shop drawings.

Significant panel stresses are often produced during demolding. Proper care must be taken to minimize demolding stresses and keep them within allowable limits. Allowable demolding forces should be determined by the design engineer and specified on shop drawings. Forces required for panel demolding should be monitored using a dynamometer or load cell which can be attached to the lifting hardware. In general, total lifting load should be kept below 1.2 to 1.3 times the total panel dead weight.

A general rule of thumb regarding panel storage and transport preparation procedures is to not subject the GFRC/facing shell to any loads. Panels should always be supported and tied down by the steel frame and should never be stacked. Panel handling procedures should also consider maintaining the clean architectural surface of the panel.

**QUALITY ASSURANCE (QA) ASPECTS OF PANEL MANUFACTURE**

Quality assurance inspections are intended to catch errors in manufacture, defects and damage, as well as obvious design errors. Prior to commencing with panel inspection, specific criteria for panel acceptance/rejection should be developed. Acceptance/rejection criteria should be developed based on guidelines from the designer, shop drawings and specifications, and reference to past experience. Each panel should be evaluated based on the established criteria. Quality assurance inspection includes random inspection of the various manufacturing-related tasks, monitoring quality control test data, and inspection of the finished product. The QA inspector reserves the right to accept a panel, reject a panel pending repair, or unconditionally reject a panel based on the established criteria. Early detection of errors reduces the number of panels needing field repair or modification. Therefore, it is in the manufacturer's interest to have the QA inspector look at every panel on each day of production.
Random Inspection of Manufacturing - Related Tasks

Random QA inspection of various manufacturing procedures should be conducted to monitor quality and consistency of workmanship. When production schedules become tight, workers may compromise the overall quality of workmanship for the sake of meeting the production schedule. At times like these, the role of the QA inspector becomes especially critical.

The QA inspector needs to be concerned with all manufacturing-related tasks. The inspector also must constantly provide the quality control personnel with feedback, whether feedback is positive or negative. It is important that the QA and QC personnel interact with each other.

Monitoring QC Data

During spray-up, test boards of the GFRC skin should be fabricated for subsequent quality control tests. Test specimens for fiber wash-out tests, flexural strength tests and flex-anchor pull-off tests should be obtained from each test board. Methods and equipment used to conduct QC tests should be approved by the QA inspector. Results from the various quality control tests should be constantly monitored by the quality assurance inspector. Records of these tests should be plotted daily and trends discussed with the plant QC personnel. Remember, everybody involved in panel manufacture is part of the product quality control. In many instances, results of the QC tests can be used as criteria for either accepting or rejecting completed panels.

Inspection of the Finished Product

In many instances, panels are not manufactured exactly as indicated on the shop drawings. However, variations from the shop drawings may be acceptable and, at times, are necessary. Therefore, it is critical that QA inspection be performed by an individual who is capable of exercising sound engineering judgment. Qualifications of the QA inspector have been discussed earlier in this paper.

Acceptance/rejection criteria for QA inspection of the finished panel product may vary from one manufacturer to another as well as from one project to another. Therefore, it would not be prudent to discuss specific criteria. The objective of this paper is to discuss specific aspects that need to be considered when conducting an as-built evaluation of a finished panel. Important aspects of inspection are discussed with respect to various panel components in the following paragraphs.

GFRC/facing shell -- The GFRC skin and the facing should be thoroughly inspected for damage. Cracks discovered in the GFRC skin or facing are generally cause for unconditional rejection.
of the panel. A single visible panel crack will often be accompanied by microcracks that cannot be seen with the unaided eye. The glass fibers function to effectively distribute stress-related cracking. Therefore, if the GFRC is visibly cracked at one location, there will most certainly be several other cracks nearby that may be invisible to the unaided eye. As a GFRC skin ages, reinforcing qualities provided by the glass fibers are gradually lost. Therefore, undetectable matrix cracks in the young GFRC open up and may potentially become apparent in the aged GFRC panel. For this reason, any clear-through crack existing in the GFRC of a new panel should be sufficient cause for unconditional rejection of the panel. Furthermore, cracks detected in the facing at a young age most probably are arrested at the GFRC/facing interface due to the effective reinforcing qualities of the GFRC. However, as the GFRC ages, these facing cracks may reflect through the GFRC to become clear-through cracks in the GFRC/facing shell. In summary, the GFRC/facing shell should never be allowed to crack at any stage throughout production, handling, erection, or while in service on the building.

Steel stud frame -- The steel stud frame is relied upon to transfer panel loads to the structural frame of the building. All loads from the panel shell are transferred to the steel stud frame through the flex-anchors. When inspecting the steel stud frame, it is important to make sure that the frame does not contact the shell at any point. Contact between the panel shell and the steel stud frame will impose restraint to movements of the shell that will occur due to normal and expected moisture or temperature changes. Excessive restraint imposed on the panel shell could result in stresses of sufficient magnitude to cause cracking. Therefore, it is important that this situation be corrected if the QA inspector determines that the imposed restraint is significant.

It is also important that thin members of the steel stud frame be adequately protected against corrosion. Special attention should be given to welds of light gauge galvanized members. Welds to material of this sort alter the corrosion resistance of the material throughout the entire thickness of the metal. Therefore, the metal should be painted on the welded side as well as the side opposite the weld. All welds to light gauge metal members should also be inspected to make sure that the welder has not burned through the base metal.

Anchors -- Panel shell loads are transferred to the steel stud frame through various types of flexible anchors. An as-built evaluation of each finished panel with respect to shell support and restraint conditions should be conducted. Evaluation of support and restraint conditions is critical with respect to minimizing stresses in the GFRC skin resulting from volumetric (dimensional) changes and service loads. Panel design should have taken into account stresses resulting from
dimensional changes and service loads. However, during the design stage, it is sometimes difficult to visualize the effects of shell support and restraint conditions for every panel configuration. Therefore, it becomes important for an individual with good engineering judgment and familiarity with GFRC design concepts to evaluate the as-built panel product. That individual should be the QA inspector. The QA inspector can provide an objective evaluation of each panel that might not be achieved if the design engineer performed the inspection.

Support and restraint of the panel shell is provided by anchors that are connected to the GFRC skin using bonding pads. Each type of anchor functions to resist a specific type of panel loading. Every panel should have flex-anchors to resist wind loads and gravity anchors to support the dead weight (gravity load) of the panel shell. In some cases, for small panels, the flex-anchors may also support the gravity load. Panels may also require seismic anchors to provide horizontal stiffness for earthquake loading.

Flex-anchors typically consist of L-shaped bars which are welded to the steel stud frame along one leg and connected to the GFRC skin using a GFRC manufactured bonding pad formed over the other leg. Flex-anchors are generally designed to have sufficient axial stiffness to transfer wind loads to the frame without buckling. However, the length of these anchors must also be sufficient to allow the anchor to flex and accommodate movement due to dimensional changes in the shell. If flex-anchors are not flexible enough to accommodate this in-plane movement, restraint forces will be imposed upon the shell which may result in excessive stresses in the GFRC skin. Flex-anchors usually consist of L-shaped circular bars which meet the GFRC skin at a right angle. A typical flex-anchor is shown in Fig. 6.

Gravity anchors support the dead weight of the shell. They are designed to be stiff in the vertical direction and flexible in the horizontal direction. Gravity anchors provide restraint to vertical dimension change movements of the shell. Therefore gravity anchors should all be located along a single horizontal row, preferably near the bottom of the panel such that panel stresses are in compression due to the dead weight of the panel shell. Locating all the gravity anchors along one horizontal row creates a reference line. Vertical dimension change movements of the shell occur relative to this line which serves as a zero reference. A gravity anchor usually consists of either an L-shaped circular bar that meets the GFRC skin at an angle at a flex-anchor location or a flat rectangular bar which has a vertical stiffness that is greater than that of a flex-anchor. Typical gravity anchors are shown in Fig. 7.

Seismic anchors are designed to be stiff in the horizontal direction and flexible in the vertical direction. Seismic
anchors are sometimes necessary when earthquake loading must be considered in design and the flex-anchors are not stiff enough to prevent significant horizontal displacement of the shell relative to the panel frame. Because seismic anchors will impose restraint to horizontal dimension change movements of the shell, they should all be located along a single vertical row, preferably near the middle of the panel. Locating the seismic anchors along one vertical row creates a reference line. Horizontal volume change movements of the shell occur relative to this line which serves as a zero reference. A seismic anchor usually consists of an L-shaped circular bar that meets the GFRC skin at an angle with respect to horizontal. A typical seismic anchor is shown in Fig. 8. A flat rectangular bar, similar to that shown in Fig. 7, may also be used to provide stiffness in the horizontal direction.

The overall stiffness of an anchor is a function of size, length and orientation. As part of panel design, a relationship has been established between these factors for the various types of anchors. However, quite often the panel is not manufactured exactly as the designer intended. This is particularly true with respect to the length and orientation of the anchors.

As indicated in Fig. 9, dimensional changes in the shell take place with reference to points of maximum horizontal and vertical stiffness. Therefore, the farther an anchor is away from the zero reference, the more flexible that anchor must be to accommodate the dimension change without inducing excessive restraint to the shell.

As described earlier, anchors that are not oriented perpendicular to the shell tend to restrain shell movement. With respect to gravity anchors and seismic anchors, the restraint is intentional. However, if more than one zero reference line is created in a given direction due to an improperly oriented flex-anchor or significantly bent flex-anchor, panel length changes between multiple reference lines may be great enough to cause the shell to crack. Therefore, it is important for the QA inspector to identify the zero reference lines for each panel during inspection and verify that there is in fact only one reference line for each direction of expected dimensional change (horizontal and vertical).

Anchor leg length is another factor that has a substantial effect on stiffness. The effective length of an anchor leg often depends on the direction of the shell length change movement. As indicated in Fig. 10, the effective length of the anchor shown is substantially less for movement to the left since the anchor is oriented on the right side of the frame member and bears against it.
Early drying shrinkage-related length changes start taking place immediately after fabrication and are the first dimensional changes that the panel shell will experience. A significant amount of the length change that occurs as a result of early drying shrinkage is irreversible. That is, the panel shell will not return to its original dimensions even when exposed to a wet environment. Generally, anchors should be oriented on the side of the frame member that corresponds to the direction of anticipated length change movement due to early drying shrinkage. Also, flex-anchor feet should be oriented such that toes point toward the zero reference line to allow for early drying shrinkage length changes of the shell. The idea is to allow the foot to potentially slip out of the formed bonding pad or bend away from the stud in response to length change associated with early drying shrinkage. Some manufacturers have used plastic sleeves to debond the foot and allow this slippage. If panel anchors are oriented appropriately, horizontal dimensional changes in the shell resulting from early drying shrinkage will take place independent of the anchor if slip sleeves are used, or will tend to bend the anchor away from the stud. Since most of this initial displacement is irreversible, the anchors will be "set up" for subsequent dimensional changes in either direction.

It may not always be possible to orient the anchors appropriately with respect to the zero reference. When such situations arise, the anchor can be pre-bent slightly so that the anchor will not bear against the frame member as indicated in Fig. 10. However, bending the anchor should never be allowed after the bonding pad has been applied. Therefore, if an anchor is identified as being mispositioned after application of the bonding pad, the panel should be rejected until the anchor is repaired to provide the necessary function.

As indicated in Fig. 11, anchors could potentially provide greater resistance to horizontal volume change movements than to vertical movements. The potential greater horizontal stiffness of the anchor is due to the end conditions created by the bonding pad. For horizontal volume change movements, interaction between the foot of the anchor and the bonding pad makes the anchor function as fixed at both ends. For vertical volume change movement, the foot of round anchors can rotate within the bonding pad, thus the anchor functions fixed at one end and pinned at the other. The situation illustrated in Fig. 11 is not necessarily typical. Some manufacturers use a sleeve around the foot that allows the foot of the anchor to both slide and rotate within the bonding pad. However, it is important that the QA inspector has a clear understanding of the various conditions that affect the required anchor length and orientation.

When evaluating the effective length of anchors, special attention is required at panel returns. As indicated in
Fig. 12, often special anchors are required at returns in order to provide sufficient support for the return without restraining shell dimension change movements. Returns add a third dimension to the panel skin which means that the anchors that support the return must accommodate volume change movements in two different planes. Therefore, special anchors are often required to support returns as indicated in Fig. 12.

**GFRC Bonding Pads** -- GFRC bonding pads should also be inspected for damage as well as for overall quality. Bonding pad damage can take on several different forms and is sometimes difficult to detect. Each bonding pad should be examined closely since cracks cannot always be easily seen. Usually cracks are found at the interface between the bonding pad and the GFRC skin. However, there are several factors that could cause other failure modes to occur as indicated in Fig. 13. Panels with cracked bonding pads should be rejected until the pad has been repaired using approved methods.

**Reinspection After Defects are Repaired** -- Panels which were found to incorporate defects during the initial inspection must be rechecked to ensure that defects have been properly repaired. A simple recording system can help to verify that all original defects are repaired before the panels leave the yard. Compact yard storage may obscure panel markings that are usually on the inside surface of the shell. Therefore, during the initial inspection, all panels should receive an extra marking on the steel frame by the QA inspector to aid in later identification.

**QUALITY CONTROL/QUALITY ASSURANCE ASPECTS OF PANEL INSTALLATION**

Panel erectors are responsible for quality control with respect to panel installation. However, the panel erection schedule is often more demanding than the manufacturing schedule. Therefore, quality control aspects are often not given adequate attention during the panel erection. For this reason, QA inspection of the final installed panel is of paramount importance.

**Inspection of the Installed Finished Product**

There are several reasons for the QA inspector to review each panel on the building. Since the time when the panels were initially inspected in the plant, panels may have been damaged in handling or have been field modified to fit. Also, panel defects may not have been noticed in previous inspection or panels with un repaired defects may have reached the building site. Therefore, each panel should be reviewed again by the QA inspector using the same concepts as used for plant inspection, and additional acceptance/rejection criteria to cover installed conditions.
Good records will help the QA inspector keep track of installed panels. A simple documentation system should be used to indicate panels that have been inspected as well as any defects that were found. Documentation should include such things as panel location, panel mark number, manufacture date and inspection date. This information will help track panels from the manufacturing plant to the site as well as aid repair verification.

As mentioned earlier, installed panels should be thoroughly inspected using the same criteria as were previously used during in-plant inspection as well as additional criteria pertaining specifically to installation. These additional criteria relate to connections between the panel and the building, evaluation of any existing field modifications, and panel placement.

Connections of the panel frame to the building must be inspected and compared with those specified on the erection drawings. Factors including size, location and length of welds or push/pull connections should be evaluated. Again, the QA inspector must be able to exercise sound engineering judgment since field modifications and other variations are often encountered with respect to panel connections.

Judgment of the QA inspector is also often required to evaluate other types of panel modifications. Sometimes shims or attachments are added or anchors are cut during erection to help position the panel for final connection to the building. Whatever the modification may be, the QA inspector must evaluate each case and either accept or reject the panel based on engineering judgment. It is not always practical or possible to manufacture and install a panel in exact accordance to design details. However, when modifications must be made, an experienced QA inspector should always review such changes.

The QA inspector should clearly mark any defects found. Following inspection, each panel should also be marked in some way to indicate that the panel has been either accepted or rejected. These marks help the repair crew to find defective panels and help the QA inspector on his reinspection after repair. Routine and regular reporting of the inspection and repair status should be made to the owner and to the manufacturer. Also, routine reporting of comments on design defects should be made to the designer to keep errors or omissions from recurring.

CONCLUSIONS

The purpose of this paper is to share with the GFRC industry, as well as owners, architects and engineers, information that the authors have learned regarding quality control/quality assurance aspects of GFRC panel manufacture and installation.
ACKNOWLEDGEMENTS

Preparation of this paper was sponsored by the Portland Cement Association. The opinions and findings expressed or implied in this paper are those of the authors. They are not necessarily those of the Portland Cement Association.

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Fig. 1 Typical GFRC Facade Panel
Fig. 2 Reduction of Strength Due to Aging

EFU - Early Flexural Ultimate
EFY - Early Flexural Yield
AFU - Aged Flexural Ultimate
AFY - Aged Flexural Yield
Fig. 3 Section of GFRC/Facing Shell at Corner and False Joint
Fig. 4 Typical Flex-Anchor/GFRC Skin Connection
Fig. 5 Details of a Bonding Pad
Fig. 6 Typical Flex-Anchor Prior to Bonding Pad Installation
Fig. 7 Schematic of Typical Gravity Anchors
Feet of Both Anchors are placed in Same Bonding Pad

Horizontal Section (Plan View) at Seismic Anchor

Fig. 8 Schematic of Typical Seismic Anchor
Fig. 9 Zero Reference for Volume Change Movements
PROBLEM WITH WRONG-SIDE PLACEMENT OF FLEX-ANCHOR

ONE ALTERNATIVE FOR WRONG-SIDE PLACEMENT OF FLEX-ANCHOR

Fig. 10 Flex-Anchor Placement Considerations
Thin-Section FRC and Ferrocement

For Horizontal Movement Relative to Frame - Flex-Anchor is Fixed at Both Ends

Weld

Frame Member (Tube or Stud)

Flexible Anchor Length

VIEW FROM TOP EDGE OF PANEL

For Vertical Movement Relative to Frame - Flex-Anchor is Fixed at one end.

Face Material

GFRP Skin

Bonding Pad of GFRP

Flex-Anchor Free to Rotate

VIEW FROM SIDE OF PANEL

Fig. 11 Flex-Anchor Considerations
Fig. 12 Flex-Anchor on Returns
Fig. 13 Modes of Failure on Bonding Pads
Improvement of the Durability of GFRC by Silica Fume Treatments

by A. Bentur

Synopsis: Treatments of AR glass fibres in silica fume slurry prior to their incorporation in cementitious matrix was found to be an effective means for improving the durability performance of GFRC composites. The improvement was found to be dependent on the extent of penetration of the silica fume particles into the spaces between the filaments during the slurry treatment. In a glass fibre fabric, which was heavily coated with polymer, the penetration was hindered and therefore the advantage offered by the silica fume treatment was not as great as in continuous glass fibre strands which were more readily wetted by the slurry.

Keywords: durability; fiber reinforced concretes; glass fibers; performance; silica fume; slurries
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INTRODUCTION

Glass fibre reinforced concrete (GFRC) composite is being increasingly used as a thin sheet material for production of building components of various shapes. The use of this material is limited mainly to non-structural applications, because of its tendency to lose part of its tensile (flexural) strength and most of its toughness after prolonged exposure to humid environment. At present this limitation is accounted for by the design procedures of the components, which take into consideration the expected change in mechanical properties over time. Yet, this state of affairs is not entirely satisfactory, and there is an active research and development effort to improve the long term performance of this composite.

Two processes can lead to the reduction in properties in this composite: (a) Microstructural changes over time which lead to the growth of dense CH* crystals around the glass filaments causing a tight bond to develop, which results in embrittlement, and (b) Chemical effect in which the glass fibres become corroded in the highly alkaline cementitious matrix. The first mechanism is believed to be the more significant one in composites with alkali-resistant (AR) glass fibres, while the second one is much more important in E-glass composite. The efforts to improve the durability of GFRC are based on developing means by which one or both of these processes (microstructural and chemical) can be suppressed. This includes the production of glass fibres of improved alkali resistivity (e.g. reference (5)), treatment of the glass fibre surface to reduce its affinity to CH (e.g. reference (6)) and modification of the matrix to reduce its alkalinity and CH content (e.g. references (7)-(9)).

* Cement chemistry notation is used
C=CaO, S=SiO₂, A=Al₂O₃, H=H₂O.
A new approach was recently developed, which might be considered intermediate between surface treatment and matrix modification. It involves treating the glass fibre strands in a silica fume slurry, prior to their incorporation in the matrix \[10,11,12\]. This method is based on the special structure of the reinforcing unit, which is a strand consisting of an assembly of about 200 filaments, that are closely spaced (Fig. 1a). This structure is maintained in the composite, since the filaments are not separated from each other in the production. During immersion of the strand in the slurry, the small silica fume particles \(0.1 \mu m\) in diameter) fill the spaces between the filaments (Fig. 1b), and they are thus positioned in the place where they are most needed, i.e. at the fibre surface. Here, their high pozzolanic activity can be most effectively used, to prevent formation of CH crystals at the interface and they may also provide some reduction in the alkalinity of the matrix in the vicinity of the fibres. This treatment was found to be effective in improving the preservation of strength and toughness, compared to only modest improvement achieved when silica fume was used only as partial substitution of the portland cement in the matrix \(11\).

In the previous work, the silica fume treated specimens were prepared by dipping individual strands or rovings, and then laying them in the composite, between layers of paste matrix.

The present paper describes an extension of this study, to evaluate a preparation method which could be used in a full scale operation and not limited to laboratory tests only. It is based on the use of fabrics of AR glass fibres, which are dipped in the silica fume slurry and then hand-laid in the composite. The evaluation of the composite included the comparison of the aging performance of GFRC prepared with continuous strands and with fabrics. Specimens with and without treated fibres were tested.

EXPERIMENTAL

All the glass fibres used in the present work were of the AR type. The fabric was a product of Nippon Electric Glass Co., Japan, labelled type TD 5x5, with a density of 0.145 kg/m\(^2\). Its structure is shown in Fig. 2. The continuous strand was AR glass produced by Owens Corning, USA.

ASTM Type I portland cement produced by Nesher, Israel, was used. The silica fume slurry was a product of Elkem Chemicals, containing about 50% water.

The treatment in silica fume was based on dipping the strands or the fabric for 10 minutes in the slurry followed by air drying for an additional 15 minutes.

The composites were prepared by hand lay-up of the fibres, using a special mold and spacers. The matrix between the fibre layers was a 0.35 w/c ratio paste.
The composites with continuous strands consisted of 2 layers of strands and 3 layers of paste, each with thickness of 3mm. The total thickness of the composite was about 10mm, and it was prepared as bars, 110mm long and 20mm wide. The fibre content was about 1.5% by volume.

The composites with fabric reinforcement consisted of 4 layers of fabrics, and 5 layers of paste, each with a thickness of 1.5mm. The total thickness of the composite was about 10mm and it was prepared as slabs, 110mm long and 150mm wide. The fibre content was about 2.5% by volume.

The specimens were demolded after one day and then kept continuously in lime water at 20°C until 28 days. At 14 days, the fabric reinforced slabs were cut into strips of 20mm wide to obtain, in this case too, bars with a width of 20mm and length of 110mm.

At 28 days, the composites were exposed to accelerated aging conditions, in lime water at 50°C, for periods up to 9 months. The mechanical properties of the composites, before and after accelerated aging were determined by means of flexural test. Four point loading at a span of 90mm was applied, and the load-deflection curve was recorded. Flexural strength was calculated and the toughness was evaluated as the area under the load-deflection curve, to the point where the load dropped to 75% of its maximum. This area will be referred to as work of fracture. The values reported are the average of at least 6 specimens, with the coefficient of variations being 5–15% for strength, and 10–20% for work of fracture.

RESULTS AND DISCUSSION

The effects of the silica fume treatment on the performance in accelerated aging is shown in Fig. 3 for the composite with continuous strand reinforcement, and in Fig. 4 for the composite with fabric reinforcement.

The treatment with silica fume increased the durability performance in both of the glass systems (strands and fabric), but the improvement seemed to be more dramatic in the strand reinforcement (Fig. 3). Here, the silica fume eliminated strength reduction and enabled preservation of considerable toughness even after 5 months of accelerated aging. It should be noted that in the control most of the toughness was lost within one month of accelerated aging, whereas in the treated composite there seems to be stabilization after about one month, with the work of fracture remaining constant at a level of approximately 50% of the initial toughness.

In the case of the fabric reinforcement (Fig. 4), the treatment with silica fume slowed down the rate of reduction in flexural strength and work of fracture; however, after 6 months of aging the
values in both composites (control and silica fume curves in Fig. 4) reached the same levels, and stayed constant thereafter.

For a valid comparison of the influence of the silica fume treatments in both reinforcing systems, one should account for the differences in the properties of the composites prior to aging. The fabric system has a larger content of fibres (2.5% vs. 1.5%); yet half of them are oriented in the transverse direction. Thus, in the main longitudinal direction, both composites are reinforced with fibre contents that are different by only 20%. This accounts for the same order of magnitude of flexural strength and work of fracture values prior to aging, which is in the range of 17.5 to 21 MPa for flexural strength, and 800 to 1400 Nmm for work of fracture. Yet, the differences, in each of these ranges, are not negligible. In order to normalize for these variations, the flexural strength and work of fracture were plotted as the values relative to the properties prior to aging (Fig. 5).

The curves in Fig. 5 indicate several important characteristics:

1. The aging in the untreated (control) fabric reinforced composite is significantly slower than that of the untreated (control) strand reinforced system. This is particularly evident when the work of fracture is considered: After prolonged aging, 30% of the initial toughness was retained in the fabric system, which showed considerable pull-out and post-cracking load bearing capacity (Fig. 6). In contrast to that, the strand reinforced composite (control) became brittle within 1 to 3 months of accelerated aging (Fig. 7).

2. The treatment with silica fume enhanced the strength retention in the strand reinforced composite to a greater extent than in the fabric reinforced system (Fig. 5). When the work of fracture is considered, the silica fume had a different influence on the shape of the curves, but eventually, after 5 months of accelerated aging, the toughness retention was similar in the fabric and strand system, with both having 40 to 50% retention (Fig. 5). Yet, the increase in toughness, when comparing the systems with and without silica fume treatment, is greater in the strand reinforced composite, as can be readily seen from Fig. 5, and the load deflection curves in Fig. 7: In the strand system, the silica fume changed the behavior after aging from essentially a brittle one, to a composite with significant post-cracking load-bearing capacity. In the fabric reinforced composite, considerable post-cracking performance after aging was evident even without silica fume treatment, and the role of the treatment was to enhance this performance modestly.

The differences in the effectiveness of the silica fume in the two reinforcing systems may be associated with the ability of the tiny silica fume particles to penetrate between the filaments. In the strand system this occurred readily, with effective impregnation of the small spaces separating between the filaments in the strand.
However, it seems that this penetration is hindered in the fabric reinforcement. Microscopical observations indicate that, in this case, the strands were heavily coated with polymeric material (Fig. 8). This coating seemed to be particularly thick at the intersection of longitudinal and transverse fibres (Fig. 8a,b) suggesting that its purpose is to stabilize the fabric. However, even in zones away from the fibres intersection, some polymeric material could be seen between the filaments (Fig. 8c). Observations of the fibres after immersing in the silica fume slurry showed accumulation of silica fume around the strand, but much less penetration of this material in-between the filaments (Fig. 9), compared to the massive infiltration in the strands (Fig. 1b). The limited penetration may be due to physical constraints induced by the polymer (occupying the space between the filaments) and possibly to changes in the surface properties, reducing the wetting characteristics of the fibres with respect to the silica fume slurry.

It was suggested that the effectiveness of the silica fume is closely linked with its ability to penetrate in-between the filaments and to be positioned at the fibre surface. Therefore, silica fume replacement in the matrix only was inefficient, while the slurry treatment of the fibres was extremely effective in enhancing durability. This hypothesis may account for the observation that in the fabric reinforced composite the silica fume treatment was not as effective as in the strand reinforcement, because of the influence of the polymer coating in the fabric, to limit silica fume penetration during slurry immersion. On the other hand, the presence of the polymer coating, with perhaps protective effect, may account for the improved durability of the control fabric composite relative to the control strand composite.

CONCLUSIONS

1. Treatment of AR glass fibres in silica fume slurry, prior to the incorporation of the fibres in the composite, was shown to be an effective means for improving the durability performance of GFRC composites.

2. The effectiveness of the silica fume treatment was found to be greater in the strand reinforcement. This was attributed to the observation that, in this system, the silica fume particles were able to penetrate into the spaces between the filaments during the immersion treatment, whereas in the fabric reinforcement the presence of polymer coating hindered this penetration. Thus, the surface treatment of the glass fibres is an important factor which must be taken into account in such treatments.

ACKNOWLEDGEMENTS

The author would like to acknowledge the interest of Henry J. Molloy in this work, and his help in providing the AR glass fiber fabric.
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Fig. 1: Glass Fibre Strands
(a) A portion of a strand prior to silica fume treatment.
(b) Penetration of silica fume particles into the spaces between the filaments in the strand, after treatment in silica fume slurry.
Fig. 2: The structure of the AR glass fabric.
Fig. 3: Effect of accelerated aging in 50°C water on the flexural properties of GFRC composites reinforced with continuous AR strands which were untreated (control) or treated with silica fume slurry (silica fume).
Fig. 4: Effect of accelerated aging in 50°C water on the flexural properties of GFRC composites reinforced with AR fabric, which was untreated (control) or treated in silica fume slurry (silica fume).
Fig. 5: Effect of accelerated aging in 50°C water on the relative flexural properties of the strand and fabric reinforced composites with silica fume and without silica fume (control) treatments.
Fig. 6: Effect of silica fume treatment on the load-deflection curve of fabric reinforced composites after 6 months of accelerated aging.

Fig. 7: Effect of silica fume treatment on the load-deflection curve of strand reinforced composites, after 3 months of accelerated aging.
Fig. 8: Microscopical observation of the fabric reinforcement prior to the treatment with silica fume.
(a)(b) Optical observations at the interaction of perpendicular fibres, showing thick polymer impregnation.
(c) - next page.
Fig. 8 (c): SEM observation away from the intersection, showing thinner polymer film between filaments.
Fig. 9: SEM observations of the fabric reinforcement after treatment in silica fume slurry, showing accumulation of silica fume around the strand, but much less penetration of silica fume into spaces between the filaments.
Glass Fiber Reinforced Cement in Mining Applications

by I.R.K. Greig

Synopsis: The inherent lightweight, toughness, low permeability, smooth surface finish and resistance to shrinkage cracking have all contributed to GFRC being an attractive alternative to traditional materials in the following areas of mining:

1. Stabilisation of rock tunnels by in-situ spraying of thin skins.

2. Construction of ventilation stoping walls both by a surface bonding technique and as a direct substitute for simple lime and sand mortars.

3. Fire protection of timber packs by lightweight GFRC renders with improved adhesion and impact strength.

4. Manufacture of drainage channels which are lighter in weight than their concrete counterparts and tougher than the asbestos cement alternatives.

5. Production of permanent formwork which is lighter in weight and has a better surface finish than concrete and is much more efficient than the use of temporary shuttering.

Keywords: drains; fiber reinforced concretes; formwork(construction); glass fibers; mines (excavations); protective coatings; tunnel lining; tunnels
Mr. I.R.K. Greig has been associated with GFRC for 13 years. He is the author or co-author of several patents relating to various process and product developments in fibre reinforced cement based materials. Mr. Greig is R & D Manager of Pilkington Reinforcements Ltd, the company which pioneered GFRC.

TUNNEL LINING

INTRODUCTION

Having had no previous experience in this area, our initial approach as research scientists was to carry out a detailed theoretical analysis with a view to designing appropriate lining systems. In attempting to calculate the stresses generated in a rock tunnel so many variables were identified that little progress was made. Discussions with people in the tunnelling industry confused the issue rather than clarified it. It was obvious that there were conflicting views coupled in some cases with a lack of logic and in others with a worrying degree of ignorance. After careful assessment of the accumulated information it became clear that in the majority of cases, the requirement was not for a massive structural support, but for a thin protective skin.

The stability of a rock tunnel depends upon the performance of the rock arch, or "ground arch" as it is more usually called. Stresses resulting from the overload are equalised throughout the arch and are mainly compressive. Most movement occurs in the period immediately following the excavation, but smaller movements continue for some time afterwards. Many rocks deteriorate surprisingly rapidly on exposure to air and undergo oxidative spalling which, along with changing stress levels, water seepage and the effects of blasting, can lead to progressive failure in the immediate vicinity of the tunnel perimeter. This type of failure usually begins with the formation of small cracks which gradually develop into larger cracks and which finally intersect to form slabs. If small pieces of rock are allowed to fall out, the slabs will become displaced and eventually collapse.

Traditional Methods of Support

The traditional methods of support are very labour intensive and consist of either steel ring beams with timber sets, or rock bolts with wire lacing and steel mesh. Neither method prevents decay of the rock but instead provides support as slabs become displaced.

More recently many mines have used conventional shotcrete applied 3 - 4 inches thick in combination with rock bolts and steel mesh. Although there is a commonly held belief that the thicker the concrete lining, the stronger it will be, this is
a misconception. The most important factors are adhesion to the rock surface and penetration of cracks. The reasons for this are well described in a paper by Sem (1).

The GFRC Alternative

Glass fibres can be used to advantage in any standard concrete mix, but for thin skin applications one obviously uses fine aggregates. Fibre addition is usually in the range of 0.5 - 1.0 % of the dry solids and can be carried out in situ or alternatively preblended in a dry ready mix. Although GFRC has better flexural and impact strengths than ordinary concrete it is probably the ability of the glass fibres to control shrinkage cracking which is the most important property. This allows the use of cement rich formulations and fine aggregates which produce skins with much lower porosity than normal concrete. It also results in effective penetration of cracks and excellent adhesion to the rock surface. Another important factor is the strain to failure of the GFRC which is at its highest during the period when maximum movements are occurring around the tunnel perimeter. It is difficult to predict how long tunnels treated in this manner will remain effectively stabilised but experience to date is that they outlive those supported in the traditional manner.

Application can be carried out using standard wet or dry shotcreting machines and equipment of the type used for spraying external renders and internal plaster finishes.

Trial Applications

It was agreed that prior to carrying out any trials in the confines of an underground tunnel it would be wise to gain experience in operating the equipment above ground. To this end, a team of five people spent a week in a large quarry familiarising themselves with the spraying of GFRC formulations. Parameters such as adhesion to the rock, penetration of cracks and rebound levels were compared for a wide range of rock surfaces and under a variety of operating conditions. The machine used was of the type commonly employed in the U.S.A. for the spray application of external renders. It was a more or less self contained unit with its own mixer, holding vessel and pump unit. The basic formulation sprayed was 1 : 1 sand to cement containing 1 % by weight of ½ inch chopped AR glass strands, delivered to site as a dry bagged premix. After some initial problems with blocked pipes and spray nozzles 0.1 % carboxymethyl cellulose was added as an antisegregation agent along with 0.01 % polyethylene oxide as an internal lubricating agent. This resulted in trouble free pumping and spraying through a 300 feet length of hose for continuous periods of up to 4 hours. As a final training exercise, a 30 feet section of tunnel linking two parts of the quarry was sprayed.
The first underground trial was carried out in a tin mine in the southwest of England. Two sections of tunnel were selected for spraying: a footwall drive 500 feet below the surface, which was described as "typical" and a lode drive 700 feet below the surface which was very wet and delaminating badly; this latter was described as "difficult". Spraying was carried out with a machine of the type commonly used in Europe for the application of gypsum plasters to internal walls. It was a much more compact machine than the one used in the quarry trials, and more suited to the narrow tunnels of this mine. The GFRC was supplied as a 1:1 sand to cement dry bagged premix containing 1% by weight of ½ inch ARG chopped strands carboxymethyl cellulose and polyethylene oxide were also incorporated at the dry blending stage at 0.1% and 0.01% respectively. Spraying of the walls was carried out at a water to solids ratio of 0.17 to 1 and the roof areas at 0.16 to 1. For the wettest areas in level 7 the carboxymethyl cellulose level was doubled to improve adhesion to the rock. In both sections the nominal overall thickness of the GFRC skin was ⅛ inch. The following comments and conclusions were supplied by the mining company after inspection by their engineers some 15 months later:

1. For best results, the application should be carried out as early as possible after excavation and to damp rock, but not that running with water.

2. A much thinner application is required than for normal shotcrete.

3. There is very little rebound compared to normal shotcrete and wastage levels are negligible.

4. The application seals the rock face and eliminates the type of problems associated with absorption of atmospheric moisture.

5. It provides an integral shell which significantly assists in the maintenance of the structural integrity of a tunnel excavation in this type of rock.

6. It improves ventilation by providing smoother walls and eliminating timber or steel set supports.

At the time of this inspection, it was interesting to see the extent of degradation which had occurred in the adjacent sections of level 5 where conventional support methods had been used. The treated section in level 7 was also performing satisfactorily although there had been considerable rock falls in adjacent untreated sections. These latter sections became so dangerous that the whole area was closed for safety reasons some 6 months later.
The thin skin approach was next pursued in South Africa where applications were in the main by dry shotcreting. Trial sections of tunnel were successfully stabilised in the Ellsberg, Vaal Reefs, President Steyn and President Brandt mines.

During the President Steyn Trial a 300 feet section of haulage tunnel was sprayed with \( \frac{1}{2} \) inch GFRC in 2 hours, compared to 60 feet with conventional shotcrete in 4 hours. This latter was spraying time only and did not include the time for fixing the steel mesh.

In the President Brandt mine, a haulage tunnel section 1 mile vertically below the surface and described as a real problem area was again successfully stabilised by a \( \frac{1}{2} \) inch GFRC skin.

As a result of these demonstrations, several mines adopted the system on a commercial basis. Cost savings, not only from the speed of application and reduction in material used, but also from the reduction in transportation of material underground. This latter is particularly important in deep level mining where cage time is at a premium.

VENTILATION STOPING WALLS

INTRODUCTION

A common mining system, particularly in coal mining, is that termed "pillar and chamber". As the excavations advance and material is removed, pillars are left at regular intervals to support the roof. Networks of chambers can extend for many miles and it is a major operation to provide fresh air to the working faces. This is done by building walls between the rock pillars to create corridors along which air is blown by huge fans.

Traditionally, these walls have been built from mortared hollow concrete blockwork and rendered with a simple sand and lime mortar. They usually extend to within 2 pillars of the working faces and so are subjected to quite severe shockwaves from blasting. It is quite common for conventional renders to fall off and allow leakage of air due to the pressure drop across the walls. In some mines as little as 30% of the air leaving the fans actually reaches the working faces.

GFRC Systems

GFRC has been promoted in two ways:

1. As an improved render for sealing conventionally built block walls.
2. In the construction of surface bonded dry stacked block walls.

The two alternative systems are offered, because in some cases only low grade blocks are available and/or the standard of labour is so poor that it is both impractical and dangerous to build dry stacked walls. Material is usually supplied as a dry bagged premix, similar to that used for tunnel lining, and can be applied by either hand trowelling or spraying. The choice of system will depend upon the conditions present or the equipment available. Where good quality blocks are available and there is a well trained wall building team, the surface bonding system is recommended. Only the bottom layer of blocks is set in mortar, to create a level base, the remainder are dry stacked with a false pillar in the middle. The wall is constructed to half height then rendered to a thickness of approximately 1/8 inch on each side either by hand trowelling or spraying. After this has hardened, the dry stacking is completed, except for a V section which is left open in the top centre. This is to allow compression waves from blasting to pass through the wall whilst its strength is developing. Wooden wedges are often used at roof height to help stabilise the wall in its early life. The remaining V is filled in some 24 hours or so later.

Typical air leakage rates at an air pressure equivalent to 8 inches of water are:

1. For an uncoated block 700 CFM per 100 square feet.
2. For a block coated on one side with a lime/sand mortar 200 CFM per 100 square feet.
3. For a block rendered on one side with a GFRC mortar 0.175 CFM per 100 square feet.
4. For a block rendered on both sides with a GFRC mortar 0.075 CFM per 100 square feet.

Fire Protection of Timber Packs

Timber packs are used to provide temporary support and to give warning of pressure build up in unstable areas within the mined stope, i.e. the veins from which ore is removed. In deep level mining, these stope are often so hot that timber dries out to a stage where spontaneous combustion and serious fires can result. Traditionally fire protection was provided by coating the packs with mud, but this tended to drop off under compression or the slightest impact. More recently low density mortars based on cement and vermiculite have been used and although an improvement over mud, these still have rather poor adhesion to the timber and low impact resistance. A lightweight GFRC mortar based on cement/vermiculite with 1% by weight of ½ inch chopped AR glass strands and organic flow aids provides a coating with
both good adhesion and satisfactory impact resistance. The material can be applied by either hand trowelling or spraying; the latter gives better penetration.

**Drainage Channels**

It is common practice in mining to transport effluent water along simple, open, gravity drainage systems.Traditionally these have consisted of half round concrete gulleys 2 inches thick and 3 feet long. Although once in place, these units perform entirely satisfactorily, they are heavy and present handling problems, particularly in deep level mining. Asbestos cement alternatives have been tried and did, of course, have a weight advantage over concrete. However, breakage rates were so high that their use proved uneconomic. GFRC units have been used successfully for some years now in a number of gold mines. They offer the combined advantages of low weight, good impact resistance and a smooth surface which helps flow.

The main method of manufacture has been by spraying flat sheets of GFRC and cutting them into strips which are then moulded by hand using simple wooden formers. A more sophisticated method is to use reciprocating spray guns mounted on traverse units and travelling moulds, a technique which has also been used successfully in the manufacture of large irrigation channels.

**Permanent Formwork**

Wherever concrete is cast, it is usual to provide some type of formwork, either permanent or temporary. Underground working conditions are such that the assembly of temporary shuttering is difficult and the end result is often cast concrete with a poor surface finish. This may at first seem unimportant for underground applications, but as mentioned earlier smooth surfaces improve ventilation and in the case of coal mines, reduce the build up of potentially explosive dusts. The use of permanent formwork offers several obvious advantages although heavy precast units present their own problems in transportation and handling. GFRC is an ideal material for permanent formwork in that it is light in weight whilst strong enough to withstand the inevitable rough handling it will receive. It can be produced in a wide variety of simple or complex shapes and forms a good bond with cast concrete to give a smooth integral skin. This skin also has a low permeability and gives excellent protection to any steel reinforcement which may be present.

In order to maximise the advantages and cost savings, any GFRC system must be properly designed. The units should be of such size and shape that they do not present handling and transportation problems within the mine. Complex fixings should be avoided and the whole system kept as simple as possible. It should also, as far as possible, fit into standard working practices otherwise its advantages are likely to be overlooked.
Overall Conclusions

In each of the areas discussed GFRC offers advantages over traditional methods and materials; also over some newer alternatives.

In the case of tunnel lining, the advantages are numerous and include: lower transportation and handling costs, speedier application, more effective sealing, and a smoother end product resulting in improved ventilation.

In the construction of ventilation stopings greater efficiencies can be achieved in supplying clean air to the working faces.

Fire protective coatings for timber packs have much improved adhesion and greater impact strength than commonly available alternatives.

GFRC drainage channels show cost savings in transport and handling when compared with the available alternatives.

GFRC permanent formwork has much to offer, again in reduced transportation and handling costs, plus in general, a far superior end product.

The enthusiasm with which these new materials and technologies have found acceptance has varied widely around the world. For example, the European industry seems particularly conservative with a "the thicker it is the better" philosophy. However, other countries such as the U.S.A. and South Africa have, as in many other areas, been much less resistant to change and have taken advantage of these new technologies.

References

Fig. 1--Practice spraying on quarry wall
Fig. 2—Spraying in Level 5 English tin mine
Fig. 3—Badly decayed rock in conventionally supported area 18 months after excavation (English tin mine)

Fig. 4—Sprayed roof section in South African gold mine
Fig. 5--Schematic of pillar and chamber mining

Fig. 6--Block wall constructed to half height
Fig. 7—Wall to full height except for "V" gap in middle
Fig. 8--Spraying timber pack with low density fire protective G.F.R.C.
Strength Properties of Steel Fiber Reinforced Concrete in Marine Environment

by N.C. Kothari

Synopsis: Strength properties of steel fibre reinforced concrete and plain concrete specimens subjected to normal atmospheric exposure and accelerated cyclic testing in marine environment were examined. The concrete mix design consisted of cement : sand : aggregate in ratio of 1 : 1.96 : 3.01 with water-cement ratio of 0.6. The steel fibres, 10 mm in length were added in volume of 0.0, 0.6 and 1.2 percent of the mix. Strength properties – compressive, flexural and tensile strength of the concrete specimens containing steel fibres showed considerable improvement to those obtained in the plain concrete exposed to the normal atmospheric condition. Both steel fibre reinforced and plain concrete specimens subjected to accelerated cyclic testing at 60°C, 24-hour cycle in marine environment showed that the addition of fibres provided considerable improvement in strength properties. However, corrosion of the fibres was observed at or near the surface, and continued to worsen after 20 cycles. Specimens with 1.2 volume percent of steel fibres exhibited the largest increase in compressive and flexural strength in both test conditions, normal atmospheric and accelerated cyclic testing.

Keywords: compressive strength; concretes; corrosion; cracking (fracturing); fiber reinforced concretes; flexural strength; marine atmospheres; mechanical properties; metal fibers; seawater; tensile strength
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INTRODUCTION

Concrete, a low cost material with adequate compressive strength is the most widely used construction material in the world. The principal disadvantages of plain concrete are low tensile and impact strengths. These properties can be improved by incorporating steel reinforcing bars or steel fibres. The effectiveness of steel fibre reinforcement in providing improved tensile, interfacial bonds and impact strength including crack control under various stresses (tensile, shear, etc.) is well established1-9.

Romauldi and Batson9 indicated that the presence of discrete steel fibres substantially increased the tensile strength of the concrete. However, the primary function of the fibres was to arrest cracking by raising cracking stress which is inversely proportional to the square root of the decrease in effective fibre spacing. The advantages and the improvement in properties of concrete containing 2 vol. percent steel fibres are given in Table 1.

Thus, the superior properties of steel fibre concrete (Table 1) has led to a gradual increase in its use as a material for industrial flooring, pavements and overlay at airport runways and highways. Properties such as tensile strength, bond stresses, moisture migration (permeability), workability, etc. are affected by the fibre characteristics – aspect ratio, volume fraction and the nature of bonding. Furthermore, the permeability property (moisture and chemical ions migration) is very important if it is used in a marine environment.

Since the 19th century, concrete has become indispensable for coastal and offshore marine construction. Today about 8000 million tonnes of concrete is produced annually and a substantial
proportion of this is used for coastal and marine structures. However, there is ample evidence that the harsh marine exposure, especially in tidal and splash zones, causes rapid deterioration and destruction of steel reinforced concrete structures. Damage to the reinforced concrete structures is related to the inability of concrete to accommodate tensile stresses developed due to the oxidation of reinforcing steel. However, the use of fibre-reinforced concrete in marine structures is perhaps the greatest unexplored area.

**DETERIORATION OF CONCRETE IN A MARINE ENVIRONMENT**

Deterioration of reinforced concrete in a marine environment is a problem that continued to concern the civil and material engineering profession. Concrete is both porous and complex in composition and, over a period of time in a marine environment, chloride and sulphate ions present can react with certain compounds in concrete. Sulphate ions reacting with tricalcium aluminate (CA$_3$Al$_2$O$_6$ known as C$_3$A) produce calcium-alumino-sulphate, ettringite, while chloride ions in seawater also react with C$_3$A resulting in chloroaluminates. Both of these reactions are expansive in nature causing concrete to crack. Furthermore, the breakdown of passive film of calcium hydroxide in concrete due to the ingress of chloride ions tends to cause corrosion of reinforcing steel. The rate and severity of this corrosion attack is influenced by many factors which are given in Table 2.

Crack development and propagation during the service life of the structures is the chief cause of this deterioration. The main sources of cracking are excessive deflections under static and dynamic loading, impact due to wave action and vessel contact, frost action, alternate wetting and drying, corrosion of reinforcing steel and chemical reactions (alkali-silica). It is widely accepted that cracks are the chief avenues for migration of corrosive ions chlorides, oxygen, etc. to the reinforcement causing steel to corrode. Since steel fibres can effectively reduce the cracking under service load, there is a good possibility of extending the life of concrete structures in coastal and marine environments by the use of steel fibre reinforced concrete. At present a large number of concrete products strengthened by different fibres are available and the steel fibre reinforced concrete shows great potential for becoming an
economically viable substitute for concrete (plain and steel bar reinforced concrete). In some marine areas, steel fibre concrete offers improved flexural and impact strength and crack restraint. However, long term durability of the steel fibre reinforced concrete in a marine environment is still open to question.

This work was carried out to determine the suitability of steel-fibre reinforced concrete in coastal and marine environments utilizing accelerated testing technique. The paper also discusses flexural, tensile and compressive strength properties of steel fibre reinforced concrete subjected to atmospheric and accelerated marine environmental conditions.

EXPERIMENTAL DETAILS

Although the physical and mechanical properties of steel fibre-reinforced concrete are well established, there has been little published on its long-term performance. This is understandable, as a long time is normally required for such research. This can be shortened by using an accelerated testing technique. However, fibre-reinforced concrete has been the subject of very few, if any, investigations utilising this method.

Plain steel fibres treated with CCl₄ (degreasing compound), diameter 0.30 mm and length 10 mm, type 'A' Portland cement, sand 2.5 mm and coarse aggregate passing 10.0 mm were used to make all the concrete test samples. Steel fibre volumes of 0.0%, 0.6% and 1.2% were used. The concrete matrix for the plain and fibre reinforcement was the same throughout the experimental program and consisted of a cement : sand : aggregate in ratio of 1 : 1.96 : 3.01 with water cement ratio of 0.6. All mixes were made in a bowl-type mixer. A special mixing procedure was used to obtain uniform steel fibre distribution in the concrete matrix. The coarse aggregate, sand and cement were mixed for two minutes, followed by steel fibre dispersion manually passing through a 10 mm sieve. Water was added and the mixer was further operated for three minutes. This approach helped in ensuring random orientation of fibres and prevented any fibre balling.

A total of 150 concrete specimens (plain concrete, concrete containing 0.6 vol. percent and 1.2 vol. percent steel fibres, now known as 0.6 Vf and 1.2% Vf) were cast. These specimens
comprised of 60 beams, 500 x 75 x 75 mm; 60 compression test cubes, 75 mm square and 30 direct tensile specimens, gauge length 200 mm with a cross-section 50 mm square. After casting, the specimens were cured for 72 hours in a fog room kept at 25°C and 100% RH. After curing the specimens were exposed to either atmospheric or accelerated testing conditions. The environmental testing was limited to 60 days.

ENVIRONMENTAL TEST CONDITIONS

Atmospheric Testing
Specimens were placed on a wooden rack in the open air, fully exposed to the weather. They were given a regular morning spray of seawater. The duration of testing was kept to 60 days.

Accelerated Testing
The accelerated testing used here was similar to that reported by Nishibayashi\textsuperscript{12}. Specimens were subjected to repeated cycles of immersion in air-saturated flowing seawater for 24 hours and oven drying at 60°C for 24 hours. The testing was limited to 30 cycles over 60 days. Nishibayashi\textsuperscript{12} conducted his tests over 200 cycles and compared his results with Gjørv's\textsuperscript{13} long-term 30-year concrete durability study in seawater and concluded that his 200 cycles correspond to approximately 30 years exposure or his one cycle in accelerated test conditions was equivalent of about 55 days. This test period of 30 cycles in accelerated conditions over 60 days was equivalent to approximately 5 years normal exposure.

All flexural, tensile and compressive tests were carried out on the Avery Universal Testing Machine at a loading rate of 1.5 KN/min.

RESULTS AND DISCUSSIONS

Appearance of Specimens
The specimens showed considerable change in appearance, the most obvious effect of the accelerated testing.
Plain concrete beams were darker, more mottled and had a wide spread area of scaling and crazing when compared to the beams that had undergone atmospheric exposure (Figure 1). A more dramatic difference in appearance was seen in beams containing steel fibres (Figure 2). Corrosion of the fibres at or near the surface was observed after completion of the first cycle and continued to worsen over the test period. Figure 3 shows the severe rusting of individual fibres after 30 cycles. Similar surface corrosion and appearance changes were observed in compression and direct tension specimens. In comparison, fibres near the surface of beams exposed to atmospheric conditions showed no rusting apart from a very minor instance after 60 days exposure.

MECHANICAL PROPERTIES

Flexural, tensile and compressive strength properties of concrete specimens with and without steel fibre reinforcement are given in Tables 3 and 4 and Figures 4 to 7.

Flexural Strength

Load-deflection curves for atmospheric and accelerated test conditions are given in Figure 4, while ultimate flexural strength results are given in Table 3.

The addition of fibres appeared to increase the first crack strength (Figure 4) and was more pronounced in specimens containing 1.2% Vf of fibres.

The plain fibre beams seemed to continue carrying a significant proportion of the ultimate flexural load after the first failure. However, there was a gradual increase in load carrying capacity as the deflection increased. However the end point record indicated 70% load carrying capacity at the total failure.

The fibre reinforced concrete beams showed higher ultimate flexural strength in comparison to that of plain concrete beams (Table 4). The increase in the ultimate flexural strength was much greater for specimens that underwent accelerated testing (Table 4). This seemed to be the result of rapid curing.
Considerable differences in crack initiation, distribution and propagation were seen in concrete beams specimens with and without steel fibres. The cracks in the beams containing fibre reinforcement were much finer compared to those seen in the plain concrete beams (Figure 5) tested in atmospheric conditions. The crack spacing in the beam containing fibres was approximately one-half of that in the plain beam.

**Direct Tensile Strength**

Results showed very little differences in the ultimate tensile strength properties between the plain concrete containing 0.6\% Vf and 1.2\% Vf fibres (Table 3). However, the addition of increasing volume fraction of fibres appeared to have a detrimental effect. The low tensile strengths in the fibre reinforced concrete specimens seemed to be the result of fibre distribution in the tensile specimens. The uneven distribution of fibres has also caused uneven crack development. Cracks developed at the face containing the least fibres, and propagated across to the side of the specimen containing the greater number of fibres (Figure 6).

**Ultimate Compressive Strength**

Table 4 and Figure 7 showed the compressive strength properties of the concrete specimens with and without steel fibre reinforcement exposed to various environmental test conditions.

For atmospheric exposure conditions, the addition of fibres did not show any significant strength increases. The 60 days exposure showed that the strength increase was 100 percent to that of curing conditions. The specimens containing 1.2\% Vf fibres showed some increase in the compressive strength (Table 4).

Increase in compressive strength was more significant for specimens exposed to the accelerated test conditions. The strength also increased with increasing fibre content at each stage of the test period (Table 4 and Figure 7).

The increase in the fibre contact had reduced the spacing between fibres allowing faster load transfer and support by the adjacent fibres. This resulted in higher compressive strength in concrete specimens containing 1.2\% Vf fibres.
Further work will be continued to determine crack initiation, spacing and propagation in concrete specimens with and without steel fibre and normal bar reinforcement in various exposure conditions.

CONCLUSIONS

- The concrete specimens subjected to the accelerated testing showed dramatic deterioration in the external surface appearance over those that underwent normal atmospheric exposure. The deterioration was particularly severe in the case of 1.2% Vf fibre reinforced specimens.

- The specimens exposed to accelerated conditions showed an increase in flexural and compressive strength. However no significant increase in tensile strength properties of specimens with and without fibre reinforcement.

- The fibre addition reduced crack spacing significantly.

- The fibre reinforced concrete specimens showed higher surface fibre corrosion. However, fibres at a depth of greater than 1mm from the external surface were not affected by corrosion.

- The fine cracks developed on the fibre reinforced concrete specimens showed a crack-sealing tendency by electrochemically precipitated products during the accelerated testing indicating a decrease in the permeability properties of concrete.

REFERENCES


TABLE 1 — Advantages and improvements in properties of plain concrete by the addition of 2 volume percent steel fibres.

<table>
<thead>
<tr>
<th>Advantages</th>
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<tbody>
<tr>
<td>- Greater resistance to cracking</td>
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<tr>
<td>- Increase in fatigue life</td>
</tr>
<tr>
<td>- Improved resistance to thermal shock</td>
</tr>
<tr>
<td>- Allow production of a thinner section for a given design</td>
</tr>
<tr>
<td>- Higher production rate</td>
</tr>
<tr>
<td>- Less maintenance</td>
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<tr>
<td>- Uniform (isotropic) properties</td>
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</tbody>
</table>

Percent Improvements over Plain Concrete

<table>
<thead>
<tr>
<th>Property</th>
<th>Percent Improvement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexural strength (first crack)</td>
<td>50</td>
</tr>
<tr>
<td>Ultimate modulus of rupture strength</td>
<td>100</td>
</tr>
<tr>
<td>Ultimate compressive strength</td>
<td>15</td>
</tr>
<tr>
<td>Ultimate shear strength</td>
<td>75</td>
</tr>
<tr>
<td>Fatigue endurance limit</td>
<td>225</td>
</tr>
<tr>
<td>Impact resistance</td>
<td>325</td>
</tr>
<tr>
<td>Abrasion resistance</td>
<td>200</td>
</tr>
<tr>
<td>Thermal spalling resistance</td>
<td>300</td>
</tr>
<tr>
<td>Freeze-thaw durability</td>
<td>200</td>
</tr>
</tbody>
</table>
TABLE 2 — Principal factors governing corrosion of reinforcing steel in concrete\textsuperscript{11}.

- Presence of cracks in concrete cover
- Environmental conditions: temperature, humidity, etc.
- Composition of chloride and oxygen ions
- Permeability of concrete
- Resistivity of concrete
TABLE 3 — Flexural and direct tensile strength properties of concrete specimens after 60 days atmospheric or 30 cycles accelerated exposures*.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Strength Properties, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>60 days Atmospheric Exposure</td>
</tr>
<tr>
<td>Plain concrete</td>
<td>5.25</td>
</tr>
<tr>
<td>0.6% Vf fibre reinforced concrete</td>
<td>6.89</td>
</tr>
<tr>
<td>1.2% Vf fibre reinforced concrete</td>
<td>8.39</td>
</tr>
</tbody>
</table>

Flexural Strength, MPa**

Direct Tensile Strength, MPa

<table>
<thead>
<tr>
<th>Specimen</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain concrete</td>
<td>1.88</td>
</tr>
<tr>
<td>0.6% Vf fibre reinforced concrete</td>
<td>1.91</td>
</tr>
<tr>
<td>1.2% Vf fibre reinforced concrete</td>
<td>1.71</td>
</tr>
</tbody>
</table>

* Average of two specimens

** $10^3$ psi = 6.895 MPa
TABLE 4 — Compressive strength of plain and steel fibre reinforced concrete specimens at various exposure conditions*.

<table>
<thead>
<tr>
<th>Exposure condition</th>
<th>Plain concrete</th>
<th>0.6% reinforced concrete</th>
<th>1.2% Vf reinforced concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Atmospheric Exposure</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>After curing</td>
<td>16.5</td>
<td>15.5</td>
<td>15.3</td>
</tr>
<tr>
<td>10 days</td>
<td>23.6</td>
<td>22.2</td>
<td>22.8</td>
</tr>
<tr>
<td>20 days</td>
<td>26.7</td>
<td>26.4</td>
<td>27.8</td>
</tr>
<tr>
<td>40 days</td>
<td>32.2</td>
<td>32.7</td>
<td>33.3</td>
</tr>
<tr>
<td>60 days</td>
<td>34.8</td>
<td>35.3</td>
<td>27.3</td>
</tr>
<tr>
<td>Accelerated Cyclic Exposure, 60°C</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>After curing</td>
<td>16.5</td>
<td>15.5</td>
<td>15.3</td>
</tr>
<tr>
<td>5 cycles</td>
<td>37.0</td>
<td>37.8</td>
<td>38.9</td>
</tr>
<tr>
<td>10 cycles</td>
<td>39.4</td>
<td>40.3</td>
<td>42.5</td>
</tr>
<tr>
<td>20 cycles</td>
<td>40.9</td>
<td>42.1</td>
<td>44.7</td>
</tr>
<tr>
<td>30 cycles</td>
<td>43.2</td>
<td>44.6</td>
<td>49.8</td>
</tr>
</tbody>
</table>

* Average of three specimens

** $10^3$ psi = 6.895 MPa
(a) 60 days atmospheric exposure.

(b) 30 cycles accelerated exposure.

Figure 1 — Plain concrete beam after 30 cycles or 60 days exposure.
Figure 2 — Steel fibre reinforced beam (1.2% Vf) after 30 cycles or 60 days exposure.
Figure 3 — Steel fibres showing extensive corrosion of individual fibres after 30 cycles accelerated exposure in concrete beam containing 1.2% volume of fibres.

Figure 4 — Flexural load – deflection curve of concrete specimens with and without fibre reinforcement after 60 days or 30 cycles exposure.
Figure 5 — Crack distribution in concrete beam specimens with and without fibre reinforcement. 10X.

(a) Plain concrete beam  
(b) 0.6% Vf fibre reinforced concrete beam

Figure 6 — Showing distribution of fibres in the direct tensile strength specimens.

(a) Fibre distribution in tensile specimen  
(b) Crack formation
Figure 7 — Effect of exposure conditions on compressive strength of concrete specimens with and without fibre reinforcement.
Properties of Sandwich Beams with Thin Layer of Steel Fiber Reinforced Mortar

by M. Rahimi and H.T. Cao

Synopsis: Flexural behaviour of sandwich beams reinforced with thin layers of steel-fibre reinforced mortar was studied in this investigation. The effect of variations in thickness of the reinforced layer on the modulus of rupture, Young's modulus and toughness of the member was investigated.

This investigation considered one single specimen size with fibre reinforced mortar using one fibre geometry and content. Steel fibres with 0.6 x 0.3 mm cross-section and 18 mm long were used. The specimens were cast in 100 x 100 x 350 mm moulds. Eight series of sandwich beams with different thicknesses of the reinforced layer were tested.

Experimental results indicated that sandwich beams can have strength and toughness comparable to fully fibre reinforced beams. The minimum thickness of the fibre reinforced layer required to impart ductile behaviour to the sandwich beam was found to be about one-sixth of the beam depth.

Keywords: beams (supports); bending; flexural strength; layered system; metal fibres; modulus of elasticity; mortars (material); sandwich structures; thickness
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INTRODUCTION

The concept of partially reinforcing brittle materials with fibres has been understood for quite some time. However, since the first published report of partially steel-fibre reinforced mortar by Rahimi & Kesler (9) in 1979, little research (5,7,8,10) has been found in the literature.

Because of the high performance and low cost of partially steel-fibre reinforced mortar, there is potential for using this type of material in a number of engineering applications. The applications could include the use of steel-fibre reinforced mortar or concrete (FRC) in the manufacture of sandwich panels, composite construction where a strong outer layer is required over a weak core, and repair and/or strengthening of concrete structural members. However, further testing and additional data are needed before the full potential of partially fibre reinforced mortar is realized.

This investigation presents part of a research program on the behaviour of composite materials incorporating the use of partially reinforced mortar. The flexural behaviour of sandwich beams reinforced with a thin layer of steel-fibre mortar are investigated. The purpose of this study is to describe the performance and to determine some parameters for the optimum use of a thin layer of steel-fibre reinforced mortar.

ANALYTICAL CONSIDERATIONS

Fibre Orientation

The effectiveness of fibre reinforcement is enhanced when a large proportion of the fibres is aligned to the maximum tensile direction of the section. In the case of short fibres, the effective length of the fibre is related to the length of the fibre and the spacing of planes within which it is confined (9).

In the case of randomly placed fibres, the orientation factor is given by Equation (1):

$$
\eta = \frac{\int_0^\beta \int_0^\phi \cos \theta \cdot \cos \phi \cdot d\phi \cdot d\theta}{\int_0^\beta \int_0^\phi \phi \cdot d\phi \cdot d\theta}
$$

(1)
where:
\[
\Theta = \sin^{-1}\left(\frac{h}{l}\right) < \frac{\pi}{2}
\]
\[
\Phi = \sin^{-1}\left(\frac{b}{l}\right) < \frac{\pi}{2}
\]

and \(l, h\) and \(b\) are fibre length, thickness and width of the reinforced layer respectively.

\(\Theta\) and \(\Phi\) are angular coordinates of fibre orientation.

The effect of the ratio of reinforced depth, \(h\), to fibre length, \(l\), on the orientation factor is shown in Figure 1. From this figure it can be seen that when the thickness of the reinforced layer is less than about 2/3 of the fibre length, the orientation factor (hence, the effectiveness of the reinforcement) only varies within 18 percent of the total possible variation, whereas there is a significant reduction in the orientation factor when the thickness of the reinforced layer is greater than 2/3 of the fibre length.

The strength of a partially reinforced mortar is therefore expected to follow the same tendency as that of fibre orientation factor. This has been reported previously (9).

**Flexural Toughness**

Toughness, or energy absorption, of concrete is increased considerably by the addition of fibres. The most convenient method of evaluating toughness is by means of the areas obtained under the load-deflection curves. This was the suggested approach recommended by ACI Committee 544 (1,2). ACI Committee 544 defined the toughness index as the measure of the amount of energy required to deflect the fibre reinforced concrete beam used in the modulus of rupture test by a given amount compared to the energy required to bring the fibre beam to the point of first crack.

The value of this index depends on the specified deflection criteria used. Various indices have been proposed by ASTM C1018-85 and are defined as 15, 110, and 130. The indices are computed by dividing the area under the load-deflection curve up to a deflection of respectively 3.0, 5.5 and 15.5 times the first crack deflection by the area up to the first crack. In addition, the index 1075 is also defined by ASTM C1018-85 as the ratio of the area up to a deflection of 0.075 inches (1.9 mm) to the area up to first crack.

Plain concrete fails after cracking, without further significant load carrying capabilities. This is a brittle failure and the toughness indices are 1.0. The use of fibre reinforcement generally increases the toughness indices and impart a ductile behaviour. However toughness indices of fibre reinforced sandwich beams varies greatly depending on the position of the crack, the volume fraction, type, distribution and orientation of the fibres.

**Young's Modulus**

It is often assumed that when a composite material is stressed, no slipping occurs at the fibre-matrix interface. In such a case, Young's modulus of a fibre reinforced concrete, according to the theory of mixtures, is given by Equation (2) :
or:

\[ E_c = f(\eta) \]

where \( E_c, E_m \) and \( E_f \) are the moduli of elasticity of fibre reinforced concrete, matrix and steel fibre respectively; \( V_m \) and \( V_f \) are the matrix volume and fibre volume fractions respectively; and \( \eta \) is the orientation factor, assuming the length efficiency factor for a particular fibre is constant. The orientation factor \( \eta \) is a function of the thickness and width of the beam and the fibre length.

In the case of a fibre reinforced sandwich beam, the modulus of elasticity of the beam is influenced mainly by modulus of elasticity of the thin layer of FRC, which is in turn dependent on the orientation factor. With a constant fibre volume fraction, the only variables that will affect the modulus of elasticity of the beam are the fibre length and thickness of the reinforced layer.

The modulus of elasticity of a fibre reinforced sandwich beam \( E_{sb} \), can therefore be expressed as:

\[ E_{sb} = f(\eta) \]

Figure 2 shows the theoretical relative Young's modulus \( E_{sb}/E_{ff} \) as a function of reinforced depth over total depth of sandwich beams. In this figure \( E_{sb} \) and \( E_{ff} \) are Young's modulus of fibre reinforced sandwich beam and fully fibre reinforced beam respectively.

It can be seen that the relative variation in Young's modulus due to the FRC layer has a similar trend as shown in Figure 1 when the thickness of the FRC is less than fibre length. There is no expected increase in Young's modulus afterwards when the thickness of the FRC is greater than fibre length.

**EXPERIMENTAL INVESTIGATION**

**Materials**

Ordinary Portland Cement type A was used in the sample preparation. The aggregate was a glacial sand that passed a 6.7 mm sieve. The steel fibres were 0.6 x 0.3 mm cross section and 18 mm long. No additives, such as fly ash, were used in mix proportioning.

**Mortar Proportions**

It is noted that paste content, aggregate grading and content, and water-cement ratio can differ considerably for plain and fibre reinforced concretes. In this investigation, efforts were made to minimize the differences between the plain and fibre reinforced mortars. The proportions of cement and aggregate were the same for both the plain and reinforced mortars. The water-cement ratio was 0.4 for both the plain mortar and the reinforced mortar. The slump was 80 ± 15 mm for plain mortar and the aggregate-cement ratio was 1.5 for all plain and reinforced mixes. The fibre content of the reinforced mortar used was 1.5% by volume. The mix proportions and properties are given in Table I.
Fabrication and Testing of Specimens

The specimens were cast in 100 x 100 x 350 mm moulds. All the specimens for each series were cast from a single batch. The mortars were consolidated by vibration. After 24 hours the specimens were removed from the moulds and stored in a fog room at 23 degrees Celsius until tested in flexure in a saturated condition at an age of 28 days.

A compression testing machine with a capacity of 100KN and an accuracy of ±0.5% was used. The specimens were loaded at the third points on a 300 mm span. Load was applied continuously until rupture at the constant rate of 1MPa/min for all specimens and controlled by strain rate. The testing procedures were in accordance with ASTM C 1018-85 and carried out at the Research House laboratory (Koukourou & partners) in Adelaide (Australia).

Experimental Program

Table 2 shows the eight types of specimens tested in this investigation. For each type, three beams were made and tested. Type 1 and type 8 specimens were control samples for unreinforced and fully fibre reinforced conditions respectively. In specimen types 2, 3, and 4 the reinforced thickness was less than the fibre length of 18 mm and in specimen types 5, 6 and 7 the reinforced thickness was greater than the fibre length.

Except for specimen type 7 (where the ratio of thickness of reinforced layer to depth of beam is 0.5), other reinforced specimen types had reinforcement on the compression face as well as the tension face (sandwich).

Both plain mortar and fibre reinforced mortar were prepared together. A weighed amount of fibre reinforced mortar was placed in the bottom of the mould and a weighed amount of plain mortar was cast over it. The same weighed amount of fibre reinforced mortar was then cast on the top.

Three measuring devices were used to take account of localised deformation due to concrete crushing at the beam supports and loading points. They were measuring displacements of midspan, over the beam supports and movable plate of the testing machine.

RESULTS AND DISCUSSION

The computed modulus of rupture and composite tensile modulus of elasticity, determine from the deflection measurements and the single beam theory, are included in Table 2. The results are plotted in Figures 3,4 in a non-dimensional graphs. In these figures \( R_{rb} \) and \( R_{rr} \) denote the moduli of rupture and \( E_{rb} \) and \( E_{rr} \) the moduli of elasticity for sandwich and fully fibre reinforced beams respectively.

Clearly, as expected, any thickness of fibre reinforced concrete on the tensile surface increases the strength of the beam compared to that of an unreinforced beam.

The most interesting feature observed was that if the thickness of the reinforced layer is about one-sixth of the depth of the beam, the flexural strength and composite tensile modulus of elasticity are higher than those for a fully fibre reinforced beam (Figures 3,4).
The effect of fibre orientation was clearly demonstrated by the results obtained. The strength of some beams, having a relatively thin reinforced layer, were greater than the beam reinforced throughout with randomly oriented fibres. This was due to the favourable orientation adopted by the fibres in the relatively thin FRC layers during placing. The results obtained in this investigation are in accordance with previously published data on partially fibre reinforced concrete (9).

The results of the toughness indices ratio are calculated and given in Table 3. The toughness indices used in this investigation are according to ASTM C1018-85. The beams in which the thickness of the reinforced layers were less than one-sixth of the depth of the beam failed in a brittle fashion. The toughness indices for these beams were assumed to be 1.0.

The relative toughness indices of sandwich beam to fully fibre reinforced beam are presented in Figure 5. For simplicity, the following notations were used in the graphical presentation:

R5, R10, R30 and R075 where, for example, R5 is the ratio of \((I5)_{sb}/(I5)_{ff}\); subscripts sb and ff denote sandwich beam and fully fibre reinforced beams respectively.

The main deductions from Figure 5 are:

1. When the thickness of the reinforced layer was less than 20% of the beam (for the fibre used in this study), the ductility of the sandwich beam was negligible.

2. The relative toughness indices of the sandwich beam increased almost in a linear fashion with the FRC thickness.

3. When the FRC thickness was about one-third of the beam thickness, the sandwich beam had a better ductility than the fully fibre reinforced beam.

4. The "optimum" toughness was achieved when the beam was partially reinforced to half of the beam thickness (specimen type 7). The toughness index II0 of this specimen type was more than double the value for a fully reinforced beam. This seems to indicate that as far as toughness is concerned, partial reinforcement on the compression side is somewhat redundant.

Figure 6 illustrates another comparison of beam toughness. In this case, the ratio of relative toughness indices are plotted.

Generally the fully reinforced beam shows a better load carrying capacity than the partially reinforced beams after first crack. When the reinforced layer was greater than about one-sixth of the beam thickness, the post-cracking load carrying capacity of the sandwich beam was comparable to that of the fully fibre reinforced beam.

In considering the combination of strength, toughness and post-cracking load carrying capacity, the results obtained indicated that partial fibre reinforcement to a thickness of about one-sixth of the beam depth appeared to be the "optimum" with regard to mechanical properties of sandwich beams.

Figure 7 presents typical load-deflection curves of the specimens. The specimen number is marked on the relevant curve.
CONCLUSIONS

This study indicates the following, for the specimen size, fibre geometry, and fibre content used:

1. Beams with a reinforced thickness less than one-tenth the depth of the members were weaker than a fully reinforced beam.

2. Beams with a thin reinforced layer of about one-sixth the depth of the member have 25%-30% greater flexural strength than a fully reinforced beam.

3. The moduli of elasticity of beams with a reinforced thickness of about one-sixth the depth of the member are approximately 12%-15% higher than a fully reinforced beam.

4. Beams with a reinforced thickness less than one-tenth the depth of the member exhibited brittle failures.

5. Beams with a reinforced thickness more than one-sixth the depth of the member exhibited ductile failures and post-cracking load carrying capacity increased as the thickness of the reinforced layer increased.

RECOMMENDATIONS

It was demonstrated in this investigation that sandwich beams incorporating thin layers of FRC can have better properties than fully fibre reinforced mortars. It seems that sandwich beams can also be cost competitive, that is, precast sandwich panels with lighter core material suitable for one or two storeys residential development. Further research is required before the full potential of this material can be realized. At the moment, the effects of aggregate size and grading, specimen size, fibre geometry and fibre content are still not fully understood. Further theoretical treatment and full size testing of panels are also needed.

ACKNOWLEDGEMENT

The authors are grateful to the technical staff of the Research House, Adelaide, Australia for their assistance in the experimental work in this investigation. In particular, they would like to express their thanks to Mr. Peter Bayetto and Mr. Peter Koukourou, the Directors of Research House and Koukourou & Partners. The fibres were provided by Fibresteel Australia.
NOTATION

1 fibre length
h thickness of FRC layer
b width of FRC layer
d depth of beam
\( \Theta, \Phi \) angular coordinates of fibre orientation
\( \eta \) orientation factor
I\(_5\), I\(_{10}\), I\(_{30}\) toughness indices according to ASTM C1018-85
E\(_c\), E\(_m\), E\(_f\) modulus of elasticity of FRC, matrix and fibre respectively
V\(_m\), V\(_f\) volume fraction of matrix, fibre
E\(_{sb}\), E\(_{ff}\) modulus of elasticity of sandwich beam, fully fibre reinforced mortar
R\(_{sb}\), R\(_{ff}\) modulus of rupture of sandwich beam, fully fibre reinforced mortar
R\(_5\), R\(_{10}\), R\(_{30}\) and R\(_{075}\) relative toughness indices of sandwich beams to fully fibre reinforced mortar, e.g., R\(_5\=(I_5)_{sb}/(I_5)_{ff}\) and so on.

REFERENCES


Table 1 Mortar Mix Proportions and Properties

<table>
<thead>
<tr>
<th>Mix</th>
<th>Water/Cement Ratio</th>
<th>Aggregate/Cement Ratio</th>
<th>Compressive Strength * (MPa)</th>
<th>Splitting Tensile Strength ** (MPa)</th>
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<tr>
<td>Plain Mortar</td>
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<tr>
<td>FRC</td>
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<td>8.8</td>
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* Based on 14 tests  ** Based on 11 tests

Table 2 Flexural Tests Results

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<th>h/d</th>
<th>Modulus of Rupture (MPa)</th>
<th>Modulus of Young's Modulus (GPa)</th>
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Table 3 Toughness Ratio

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<th>Type of Specimen</th>
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<th>I_{30}/I_{10}</th>
<th>I_{30}/I_{15}</th>
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Fig. 1 Orientation factor as a function of the thickness of reinforced layer

Fig. 2 Theoretical relative Young's Modulus $E_{ab}/E_{ff}$
Fig. 3 Relative Modulus of Rupture $R_{sb}/R_{ff}$

Fig. 4 Relative Young's Modulus $E_{sb}/E_{ff}$
Fig. 5 Relative Toughness Indices \((I_{ab})/(I_{tt})\)

Fig. 6 Ratio of Relative Toughness Indices
Fig. 7 Typical Load-Deflection Curves
Structural Behavior of Thin SFRC and Ferro-Fibro Overlays

by S.K. Kaushik, R.M. Vasan, P.N. Godbole, D.C. Goel, and S.K. Khanna

Synopsis: The paper reports on the performance of Semi-full scale pavement and overlay slabs under static loads. The test results of 60mm SFRC pavement slabs having 0.5% fibres by volume have been presented under different loading and subgrade conditions. The test results of 100mm PCC (plain cement concrete) pavement slab resting over a well compacted subgrade have also been presented. The performance of 20mm Ferro-fibro overlay cast over 60mm cracked SFRC pavement has been reported and compared with a 40mm SFRC overlay slab cast over 60mm SFRC pavement. The experimental results of semi-full scale overlay and pavement slabs have been validated by Infinite element analysis, a numerical technique developed for the analysis of unlimited domain of a layered system consisting of an overlay, pavement and subgrade of known properties. A comparative study has been presented with respect to Ferro-fibro and SFRC overlays.

Keywords: compacting; concrete pavements; concrete slabs; ferrocement; fiber reinforced concretes; layered system; loads (forces); metal fibers; performance tests; plain concrete; plate load tests; resurfacing; static loads; subgrades; welded wire fabric
Professor Kaushik, teaches in the areas of Structural Materials and Engineering at University of Roorkee, India, and is a Chartered Engineer and has served as a consultant on Complex Buildings, Bridges, Airport pavements and overlays in India. He has authored 60 research papers and reports in addition to guiding 10 doctoral thesis.

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Mr. Goel received his Bachelor and Master of Engineering Degree from University of Roorkee. His research efforts are in the area of Structural analysis and design, Finite element analysis, Currently he is Assistant Executive Engineer at Central Public Works Department, New Delhi, India.

Prof. Khanna obtained his Ph.D. Degree from University of Roorkee, India. His areas of research are Transportation Engineering, Highway and Airfield pavements. He is the author of a textbook on Highway Engineering. Currently he is Secretary, University Grants Commission, New Delhi.

INTRODUCTION

Numerous investigations (1,2,3) have established that, relative to conventional concrete, the introduction of 0.5 to 1.5% of steel fibres by volume brings about significant improvements in flexural strength, flexural toughness, impact resistance and fatigue endurance of fibrous concrete overlays and pavements. Fibre composites have been found to be promising materials for pavement construction and as overlays on existing pavement slabs and bridgedeck slabs. The composite matrix possesses superior crack arrest properties, greater ductility and distinct post cracking behaviour. It offers improved pavement life for a given slab thickness or significantly reduced overlay or pavement thickness for equivalent performance. Closely spaced, uniformly dispersed discrete steel fibres when added to concrete provide a crack arrest mechanism. The composite matrix
can be used to advantage in strengthening highway pavements when thin (thickness less than 0.05 times the length) overlays are required at certain locations such as road intersections where the thickness can not be increased beyond certain limits. The combination of plain concrete, ferrocement and random fibres results into a composite matrix which may be termed ferro-fibrocrete.

The use of the composite matrix has been reported (5) to optimize the strong points and suppress the weak points of the individual constituents forming the matrix. Ferro-fibrocrete possesses the advantage of reducing the severity of cracks due to the multiple cracking mechanism exhibited(6) by the matrix. The presence of the wire mesh in mortar layers (Ferrocement) plays a significant role in preventing reflection cracking at the surface which has been reported(7) to be present in the case of SFRC overlays over plain cement concrete pavements.

Semi-fullscale thin sections of ferro-fibro overlay, cast over thin SFRC pavement and SFRC overlay over cracked SFRC pavement overlying compacted subgrade and subjected to static plate load tests at central, edge and corner loading conditions showed good performance. Thin SFRC pavement slab resting over subgrade and tested under central, corner and edge loading conditions exhibited excellent results.

**Experimental Investigation**

Semi-fullscale pavement and overlay slabs were cast in 1.8m x 1.8m panels. A 20mm thick ferro-fibro overlay (FFO) consisting of 1:2 cement sand mortar was cast over a cracked 60mm SFRC pavement slab ($SFP_{1}$). Two layers of 0.456 mm dia galvanised woven wire mesh, size 10mm x 10mm and steel fibres, 0.5% by volume were used in the ferro-fibro mix. The total mesh reinforcement was 1.92% by volume (155 kg per cubic meter of mortar) of mortar. The overlay was fully bonded to the underlying pavement. Hooked end steel fibres having 0.456 mm dia and aspect ratio 80 were
used in the ferro-fibro overlay as well as SFRC pavement. The 60 mm thick SFRC pavement slabs (SFP₁ & SFP₂) and 40 mm thick SFRC overlay (SFO) were cast over a well compacted subgrade and 60 mm cracked pavement slab (SFP₂) respectively. The SFRC mix was designed by the ACI (8) method. The pavement and overlay sections were subjected to static plate load tests using a 300 mm dia plate under central, edge and corner loading conditions with a reaction frame having a 220 kN capacity. The properties of the mixes used in the investigations are given in Table 1. The characteristics of pavements and overlays are given in Table 2. The properties of steel fibres are given in Table 3.

**Performance of Overlays and Pavement Slabs**

Comparative performance of thin Ferro-fibro overlay (FFO), and thin SFRC pavement slabs SFP₁, SFP₂ under central, edge and corner loading conditions is given in Table 4. Test results of static plate load tests showing deflections and strains with applied loads are shown in Figures 1 and 2.

**FERRO-FIBRO AND SFRC OVERLAYS**

The 20 mm thick ferro-fibro overlay (FFO) resting over 60 mm failed SFRC pavement (SFP₁) exhibited significant improvements in load carrying capacity at first crack and ultimate stage. The composite overlay sustained a load of 200 kN under a central loading condition without any signs of cracking. Under edge loading conditions, it sustained a load of 160 kN at first crack with a crack width 0.01 mm. On subsequent application of load, a second crack was observed at 180 kN load with a 0.07 mm crack width. The Ferro-fibro overlay was tested up to a maximum of 200 kN load. The crack width at this load was 0.10 mm. Under corner loading condition, the first crack occurred at the 80 kN load followed by the second, third and fourth cracks at 90 kN, 100 kN and 140 kN loads respectively. The observed crack width was 0.10 mm at the 100 kN load and 0.20 mm at the 140 kN load. The 40 mm SFRC overlay (SFO) fully
bonded with 60 mm cracked SFRC pavement (SFP₂) sustained a load of 200 kN under central loading condition without occurrence of any crack. Under edge loading condition, it sustained a load of 120 kN at first crack. On subsequent application of load a second crack was observed in the Ferro-fibro overlay (FFO) at the 170 kN load with a 0.090 mm crack width. Under corner loading condition, the SFRC overlay sustained 110 kN load with a crack width 0.15 mm.

**SFRC PAVEMENTS**

The 60 mm thick SFRC pavement SFP₁ sustained a load of 100 kN at first crack and failed at a 150 kN load under central loading condition. Under edge and corner loading conditions the occurrence of the first crack was observed at 90 kN and 70 kN loads respectively. The 60 mm SFRC pavement SFP₂ supported by the weaker subgrade failed at a 70 kN load under central loading condition and could not be tested under edge and corner loading conditions.

**CRACK PROPAGATION**

The crack patterns for the Ferro-fibro overlay slabs (FFO) and SFRC overlay (SFO) are shown in Figures 3 and 4 respectively. Crack patterns of the SFRC pavement SFP₁ and SFP₂ are shown in Figures 5 and 6 respectively. In the Ferro-fibro overlay no cracking was observed up to a 200 kN load under central loading condition. The Ferro-fibro overlay also exhibited a significant improvement in load carrying capacity under edge and corner loading conditions displaying lower crack widths at loads much beyond the axle loads normally encountered (80-100 kN) in highway pavements. The Ferro-fibro overlay exhibited a peculiar cracking behaviour under edge and corner loading conditions. The crack initiation in the Ferro-fibro overlay took place at much higher loads than in the SFRC overlay and SFRC pavements. The combined action of steel fibres and the ferrocement in the composite matrix resulted in delayed formation of cracks and reduced crack widths under edge and corner loading conditions. The fibre inclusion in the composite matrix
helped maintain the cracks tightly closed due to crack arrest properties of the SFRC mixes. Ferro-fibro combination in the composite matrix exhibited multiple cracking in the Ferro-fibro overlay SFRC pavement combination. Due to multiple cracking the cracks were distributed over a larger surface area and the crack widths remained low (0.05 mm - 0.2 mm). In view of significant improvement in load carrying capacity, reduced crack widths and crack propagation, thin SFRC pavements and overlays are quite advantageous for highway pavements. Ferro-fibro overlay is a potentially advantageous composite matrix having relatively superior performance to conventional concrete. It offers the possibility of laying overlay sections in very small thicknesses (20mm in the present case). This is an important requirement for strengthening of existing pavements and in special applications like bridge deck slabs. The significant points with respect to performance of Ferro-fibro overlays and SFRC overlays and SFRC pavements are as follows:

(a) A 20mm Ferro-fibro overlay section (FFO) exhibited a significantly higher load carrying capacity than 40mm SFRC overlay (SFO). Absence of cracking under the central loading condition at a 200 kN load and delayed initiation of first crack demonstrated the structural adequacy of the composite ferro-fibro matrix (Ferro-fibrocrete) in bearing relatively heavier loads with significantly reduced damage.

(b) Ferro-fibro overlay exhibited multiple cracking mechanism and low crack widths with cracks distributed over a larger surface area of the pavement.

(c) SFRC overlays are potentially advantageous in strengthening existing rigid pavements.

(d) SFRC pavement slabs offer the possibility of placement directly on strong subgrades. For thin SFRC pavement sections on weaker subgrades (modulus of subgrade reaction less than 0.06 N/mm$^3$) the provision of sub-base or base becomes necessary.
FINITE ELEMENT ANALYSIS

A finite element software incorporating numerically integrated finite and infinite elements has been employed to evaluate the strains and deflections in pavements and overlays under central loading conditions. The application of the method to the layered pavement system analysis has been reported (9,10) by several investigators. The infinite element mesh for the three layered pavement system consisting of ferro-fibro overlay over SFRC slab is shown in Figure 7. The various parameters considered in the analysis are: the modulus of elasticity of the overlay slab, pavement and subgrade; Poisson's ratio of the pavement layers and; the thickness of overlay and pavement slab. In the analysis, infinite elements with \((1/r^n)\) type decay, where \(r\) is the coordinate direction extending to infinity and \(n\) is the degree of decay \((n = 0.5 \text{ and } 1)\), considering axisymmetric problem (11) have been employed. The computerised results are tabulated in Table 5 and compared with the results obtained from semi-fullscale field investigations of ferro-fibro overlays, SFRC pavements and SFRC overlay of known properties. A comparison of the experimental and analytical values of deflections at 41 kN and 80 kN loads indicates that the ratios of experimental versus analytical values lie between 0.89 and 1.90 respectively. The ratios of observed versus analytical strains lie between 0.69 and 1.63. It can therefore, be concluded that infinite element formulation can be employed in the analysis of stresses, strains and deflections in the layered system consisting of SFRC pavements and overlays and composite ferro-fibro overlay SFRC pavement combinations, from the known material characteristics of the layered pavement system. The infinite element method is potentially advantageous due to its simplicity and requires nearly half the computer time as compared to the finite element analysis.

CONCLUSIONS

Ferro-fibrocrete reinforced with steel fibres (SFR-FFC) is a potentially advantageous composite material which can be employed
in thin pavement and overlay sections. The material possesses the major advantages of higher load carrying capacity, reduced pavement thickness requirements, delayed initiation of cracks, and superior crack arrest properties. The peculiar multiple cracking mechanism has been found effective in reducing the severity of cracks. The absence of cracking of the overlay surface at a 200 kN load clearly demonstrates the role played by the ferro-fibrocrete composite in preventing reflection cracking. At such a high load, the 60 mm thick base pavement is expected to have failed. The results of the experimental investigation establish that thin SFRC pavement sections placed directly over well compacted strong subgrades have adequate load carrying capacity and thin SFRC overlay sections exhibit a significant improvement in the load carrying capacity of cracked pavements showing at the same time superior crack arrest properties even at higher loads. The significant improvements in structural performance exhibited by Ferro-fibro overlays, SFRC overlays and SFRC pavements under satisfactory subgrade conditions lead to the following conclusions:

(1) The performance of semi-fullscale overlay and pavement sections holds good under satisfactory subgrade conditions for testing under static loads only.

(a) The 20 mm Ferro-fibro overlay exhibited an excellent performance showing that the maximum observed deflections were less than 6 mm at twice the axle load. The absence of cracking at loads far greater than twice the allowable loads and capacity to bear higher strains establishes the structural superiority of the composite over SFRC pavements. A factor of safety 2 is available under central, edge and corner loading conditions.

(b) SFRC overlays over cracked SFRC pavements exhibit satisfactory performance under all the three loading conditions and a factor of safety of more than 2 is
available. SFRC overlays are recommended for strengthening existing pavements.

(c) The 60mm SFRC pavements placed directly over subgrade exhibit satisfactory performance under central and edge loading conditions. The pavement showed first crack at a 70 kN load under corner loading condition. Thin SFRC pavement sections over favourable subgrade conditions are expected to give superior performance by further increasing the fibre contents from 0.5% to fibre volume fractions of 1.2-1.25%.

(2) A 60 mm SFRC pavement section laid directly over a weaker subgrade showed satisfactory performance under central loading condition only. Provision of subbase/base is necessary for thin SFRC pavements over weaker subgrades.

(3) Finite element software incorporating numerically integrated finite and infinite elements may be employed in layered pavement system analysis with considerable saving in computer time.

Recommendations

As an extension of the investigation reported, the following additional studies need to be undertaken to establish the potential for use of Ferro-fibre and SFRC composites for highway pavements and overlays:

(1) A larger number of full scale experiments should be undertaken to establish performance under actual field conditions under varying subgrade and traffic conditions.

(2) Laboratory investigations should be undertaken at fibres contents of 1.2 to 1.25%.

(3) Further studies should be undertaken to assess the actual field performance of composite pavements.
The effect of different base courses on the performance of composite pavements needs to be studied.

REFERENCES


8. Recommended Practice for Selecting Proportions for Normal and Heavy Weight Concrete, (1977), ACI 211-1-77, American Concrete Institute, Detroit, Michigan, U.S.A.


TABLE 1
Mix Design for SFRC Pavement and Ferro-Fibre/SFRC Overlay

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Type of Slab**</th>
<th>*Weight of materials per cu.m. of concrete (kg)</th>
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<td>Water</td>
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<td>1.</td>
<td>20mm Ferro-Fibro Overlay (SFO) over 60mm SFRC pavement (SFP₁)</td>
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<tr>
<td>2.</td>
<td>40mm SFRC overlay (SFO) over 60mm cracked pavement(SFP₂)</td>
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<tr>
<td>3.</td>
<td>60mm SFRC pavement (SFP₁, SFP₂)</td>
<td>250</td>
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</tbody>
</table>

* Fineness Modulus of Fine Aggregate (2.43),
  Fineness Modulus of Coarse Aggregate (5.99)
** Slabs Sl.No. 1, - volume of mesh 1.92%
  All the slabs Sl.No., 2,3 contain 0.5% fibre volume

TABLE 2
Characteristics of PCC/SFRC Pavements and Ferro-Fibre Overlay

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Type</th>
<th>Pavement/Overlay</th>
<th>Subgrade</th>
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<td>Modulus of elasticity* GPa</td>
<td>Poisson's Ratio*</td>
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<td>SFRC Overlay (SFO)</td>
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<td>SFRC Pavements (SFP₁, SFP₂)</td>
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<td>PCC Pavement (SPC)</td>
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* Cylindrical test specimens 150mm x 300mm, Universal testing machine
** Cylindrical test specimens 150mm x 300mm, Triaxial test.
### TABLE 3

**Properties of Steel Fibres**

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<tr>
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<th>Value</th>
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<td>Gauge</td>
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<td>Average diameter, mm</td>
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<td>Young's modulus of elasticity, GPa x $10^5$</td>
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<tr>
<td>6.</td>
<td>Yield strength, GPa x $10^4$</td>
<td>3.13</td>
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<td>7.</td>
<td>Ultimate strength, GPa x $10^4$</td>
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<tr>
<td>8.</td>
<td>Ultimate bond stress between fibres and concrete, GPa</td>
<td>152.5</td>
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*Steel fibres obtained from National Council of Building Materials, New Delhi, India.*
### Table 4
Comparative Performance of Thin SFRC Pavement and Ferro-Fibro/SFRC Overlay Slabs Under Static Plate Load Tests

<table>
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<th>Sl.</th>
<th>Pavement Type</th>
<th>Modulus of Subgrade Reaction N/mm$^3$</th>
<th>First Crack Load kN</th>
<th>Defln. Crack Defln. Crack Load kN</th>
<th>Ultimate Crack Defln. Width mm</th>
<th>Loading Condition</th>
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<td>Centre</td>
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<td></td>
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<td>-</td>
<td>-</td>
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<td>-</td>
<td>-</td>
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<td>-do-</td>
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<td>-</td>
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<td>8.41</td>
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* Central loading condition
** Infinite element method
Fig. 1—Deflections in pavement and overlays

Fig. 2—Strains in pavement and overlays
Fig. 3—Crack pattern of ferro fibro overlay (FFO) over SFRC pavement (SFP)

Fig. 4—Crack pattern of SFRC overlay (SFO) over SFRC pavement (SFP)
Fig. 5—Crack pattern of SFRC pavement (SFP)

Fig. 6—Crack pattern of SFRC pavement (SFP)
Fig. 7 — Infinite element mesh (3) for ferro-fibro overlay (FFO) over SFRC pavement (SFP)
Deformation Characteristics of Ferrocement Elements Under Tension

by T.P. Tassios and V. Karaouli

Synopsis: A simplified analytical procedure is proposed to predict stress-strain diagram of ferrocement composites under tension. A fracture mechanics approach is used to predict the load "at first cracking". Results of a limited experimental investigation are also shown, used to evaluate the analytical model. The drastic role of curing is also evidenced experimentally.

Keywords: bond (concrete to reinforcement); cracking (fracturing); curing; deformation; ferrocement; stress-strain diagram; tension
INTRODUCTION

The stiffness of ferrocement (FC) elements under tension, depending on their degree of cracking, is a basic property for the design of ferrocement structures. On the other hand, crack width in ferrocement elements is a critical parameter as its prediction is necessary to assess the risk of corrosion under certain environments, which might be very high for thin structures (small covers).

All this information is directly or indirectly included in the stress strain diagram of the composite under tension.

The purpose of this study is to contribute to the prediction of stress strain relationships of ferrocement elements subjected to uniaxial tension.

ANALYTICAL MODELLING

A simplified analytical model is described here, aiming at predicting the full stress-strain diagram of FC under tension. It is believed that, despite its several inadequacies, such a model may be useful as a better framework for experimental findings; thus, subsequent calibrations or corrections of the model can be more easily made.

In what follows a step-by-step procedure is adopted, as needed for a clearly non-linear phenomenon.
Stress At First Crack

In ferrocement (FC) composites under tension, the definition of such a stress-level cannot be based on the appearance of a macro-crack; well before it, the force/elongation diagram deviates from its initial linearity when several micro-cracks of only few hundredths of a millimeter develop. Besides, such small widths of microcracks are addressed by Fracture Mechanics: Only a pseudo-separation $w_0$ takes place, and we may still count for residual (post-peak) tensile stresses $\sigma_{ct}$ of the matrix (see Fig. 1), whereas, thanks to these microcracks, the participation of increased steel stresses is secured, mobilised by a pull-out slip $s = w_0/2$ (see Fig. 2).

Thus, thanks to its very small crack widths, the matrix of FC continues to contribute to the resistance even after nominal cracking, and the total forces developed in the composite are higher than those developed when just $f_{ct}$ was first reached.

In reality, these synergetic mechanisms are rather complicated; a theoretical assessment of a limit-value of $w_0$ after which a significant increase of elongations occurs, is not easy. In the present simplified model, a tentative value of the order of $w_0 = 0.01$ mm was used; the mobilisation of the corresponding steel-to-matrix bond $\tau_m$ was also evaluated.

Several other $w_0, \tau_m$ pairs might have been examined for the purpose. However, in this paper we were guided by the practical concept of a 20% margin of stress redistribution; besides, a calibration of the composite stress at first crack was carried out by means of experimental results of other authors.

More specifically, as a nominal "first-cracking" limit, the following width of fictitious crack is taken, roughly corresponding to a residual stress response $\sigma_{ct} = 0.8 f_{ct}$ (Fig. 1):

$$w_0 = \frac{1}{4} G_f/f_{ct}$$

(1)

where $G_f$ is the fracture energy of the cementitious matrix (i.e. the area of the post-peak stress-displacement diagram)

and $f_{ct}$ denotes the tensile strength of the matrix

Consider now a wire across such a fictitious crack; a bonding length $l_b$ is sought on both sides, with stress distribution as shown in Fig. 2. Only the local steel stress increment $\Delta \sigma_{sr}$ is illustrated, as
produced by the fictitious crack (in addition to the initial value $\sigma_{SO}$ before "cracking").

Along the pullout length, an almost triangular bond stress distribution may be found; thus, an average bond stress value $\tau_m = \tau_u/2$ will be used, where $\tau_u$ denotes the mobilised maximum local bond stress.

Considering a local bond/slip curve (Fig. 3) for local slip values around 0.001d (d being the steel bar diameter), a rough linearisation is possible, i.e.

$$\tau_u = k_\tau \frac{w_0}{8}$$  \hfill (1a)

where the "effective" slip has been taken equal to the fourth of the front-end slip (see Fig. 2).

The bond stiffness modulus $k_\tau$, for the slip values considered, may be taken as

$$k_\tau = \frac{k_0}{\sqrt{d}} f_{ct}, \quad k_0 \approx 1200 \text{ [Nmm]} \hfill (2)$$

This expression is based on experimental findings of Shima et al., 1987 (1)(p. 155) on deformed bars; for meshes an equal $k_0$-value may be taken, whereas for simple smooth wires $k_0 \approx 600$ and $k_0 >> 1200$ for expanded metal.

Subsequently (see notation in Fig. 2),

$$\tau_m = \frac{1}{2} \tau_u = \frac{1}{16} \left( \frac{k_0}{\sqrt{d}} f_{ct} \right) w_0$$  \hfill (3)

Equilibrium and compatibility conditions along the bonding length $l_b$ under pullout:

$$\Delta \sigma_{sr} \frac{\pi d^2}{4} = l_b \tau_m \pi d \hfill (4)$$

$$\frac{w_0}{2} = \frac{1}{2} \frac{\Delta \sigma_{sr}}{E_s} l_b$$  \hfill (5)

where $E_s$ denotes the modulus of elasticity of steel.

Combining Equs 3, 4 and 5, one finds
In reality, bonding lengths increase with front-end slip, but here a specific order of magnitude of such a slip was considered.

Similarly,

\[
\Delta \sigma_{sr} = \frac{1}{2} \omega \sqrt{\frac{E_s k_{o}}{f_{ct}}} \frac{d}{3/4} = \frac{1}{8} G_f \sqrt{\frac{E_s k_{o}}{f_{ct}}} \frac{d}{3/4}
\]

The actual steel stress \( \sigma_{sr} \) at first crack should now be found, taking also into account its initial value due to the elongation of the FC tie outside the fictitious crack under the stress \( f_{ct} \) which led to the first crack:

\[
\sigma_{sr} = \frac{E_s}{E_c} (0.8 \min_{ct}) + \frac{G_f}{8} \sqrt{\frac{E_s k_{o}}{\min_{ct}}} \frac{d}{3/4} \min_{ct} \left( \frac{E_s}{E_c} + \frac{1}{\rho} \right) \tag{8}
\]

where \( \min_{ct} \) (\( \approx 0.7 f_{ctm} \)) denotes the lowest characteristic value of \( f_{ct} \) (\( f_{ctm} \) being the mean value, see Fig. 4b)

and \( \rho = A_s/A_c \), is the steel ratio in the loading direction.

The global elongation of the FC tie at the moment of first cracking may be estimated by means of

\[
\varepsilon_{sm,r} = \min_{ct}/E_c
\]

In reality, a somehow higher deformation is expected at this stage, because some plastic strain of concrete occurs before reaching the peak of its local tensile stress.

However, if microcracking appears early at this stage (i.e. if the second part of Equ. 8 governs), a larger average strain should be taken into account (comp. Equ. 21): 

\[
\varepsilon'_{sm,r} = \frac{\min_{ct}}{E_c} + \frac{1}{2\lambda} \frac{d}{\ell} \frac{\min_{ct}}{E_s} \frac{1}{\rho^2}
\]

where \( \lambda = \tau_u/f_{ct} \) (see Equ. 16a).
**Transition cracking**

Beyond the tensile stress level of $\sigma_{sr}$ and although crack widths are still within the range shown in Fig. 1, the tensile response of the matrix is so small and uncertain that for the needs of this model it will be practically assumed that macro-cracking is rapidly developing (Fig. 4a). Because of the non-uniformity of the matrix, first cracks correspond to lower $f_{ct}$-values ($\min f_{ct}$), later cracks correspond to higher $f_{ct}$-values based on a scattering band (Fig. 4b) of available tensile strengths.

Steel stress for the initially non-cracked areas is

$$\sigma_s = \frac{E_s}{E_c} \sigma_c = \alpha \sigma_c$$  \hspace{1cm} \text{where} \hspace{0.5cm} \alpha = \frac{E_s}{E_c} \hspace{4.5cm} (10)$$

Equilibrium of internal/external forces

$$\sigma_s A_s = \frac{A_s}{\rho} \sigma_c + \sigma_{so} A_s$$ \hspace{2cm} (11)

where $\rho = A_s / A_c$

Hence $\sigma_c = \sigma_s / (\alpha + \frac{1}{\rho})$ \hspace{3.5cm} (12)

$$\sigma_s - \sigma_{so} = \sigma_s / (1 + \alpha \rho) \hspace{4cm} (13)$$

At the moment of the next crack, $\sigma_{sr} = f_{ct} (\alpha + \frac{1}{\rho})$ \hspace{3.5cm} (14)

Equilibrium along the "transfer length", $e$, at the moment of next crack:

$$(\sigma_s - \sigma_{so}) A_s = \frac{2}{3} \tau_u \nu d$$ \hspace{2.5cm} \text{or, by means of Equs 13 and 14,}$$

$$e = \frac{3}{8} \frac{1}{\rho} \left( \frac{f_{ct}}{\tau_u} \right) d \hspace{4cm} (15)$$

For the macro-cracking considered in this paragraph, ($w$=0.05 to 0.10 mm of $s_{eff}$=0.01 to 0.03 mm or $s/d$=0.020), the full bond strength is mobilised. Therefore for the deformed bars or smooth wire-meshes (bond capacity improved by means of the transverse wires),

$$\frac{f_{ct}}{\tau_u} = \frac{1}{3} \hspace{4cm} (16)$$
However, depending on the bond properties of each particular composite, it is convenient to write:

\[
\frac{f_{ct}}{\tau_u} = \frac{1}{\lambda} \tag{16a}
\]

Subsequently, Equ. 15 becomes

\[
e = \frac{3}{8\lambda} \frac{d}{\rho} \tag{17}
\]

or

\[
a_{lim} \approx 2e = \frac{3}{4\lambda} \frac{d}{\rho} \tag{18}
\]

However, this is only an average value of crack spacing; the variability of tensile and bonding properties of the matrix produce a variability of this spacing.

Total number of cracks

\[
n = \frac{l}{a_{lim}} = \frac{4}{3} \lambda \frac{d}{\rho} \tag{19}
\]

Each time a new crack appears, an additional elongation is produced, corresponding to the following average strain for the entire tie:

\[
\Delta\varepsilon_{sm,i} = \frac{2e}{L} \frac{2}{3} \left( \sigma_s - \sigma_s^{0} \right) \frac{1}{E_s} \tag{20}
\]

Using Equs 13 and 17,

\[
(\Delta\varepsilon_{sm})_i = \frac{1}{2\lambda} \frac{d}{L} \frac{f_{ct,i}}{E_s} \tag{21}
\]

Each time, this additional strain corresponds to a stress level (see Equ. 14)

\[
\sigma_{sr,i} = f_{ct,i} (\alpha + \frac{1}{\rho}) \tag{21a}
\]

Both, \(\sigma_{sr,i}\) and \(\Delta\varepsilon_{sm,i}\), are governed by the respective tensile strength \(f_{ct,i}\) gradually appearing out of the scattering band illustrated in Fig. 4.b. Therefore, for each one of the "n" cracks to appear (see Equ. 19), a gradually higher \(f_{ct}\)-value applies, up to a "reasonable" maximum. Obviously, this is only a pseudo-probabilistic approach to a much more complicated problem. For this oversimplified solution, it is suggested that the additional total strain of this
stage be calculated combining Equs 19 and 21,

$$\sum \Delta \varepsilon_{\text{sm},i} = n \Delta \varepsilon_{\text{sm},i} = \frac{2}{3} \frac{f_{\text{ctm}}}{E_s}$$

(22)

using the mean value of $f_{\text{ct}}$. This total strain (see Fig. 6) corresponds to the interval between the crack stress $\sigma_{r,n}$ at the end of the transition stage (as estimated by Equ. 14 putting $f_{\text{ct}} = 1.3f_{\text{ctm}}$), i.e.

$$\sigma_{r,n} = 1.3f_{\text{ctm}} (\alpha + \frac{1}{\rho})$$

(23)

Generalised cracking stage

Now that transfer zones touch each other (Fig. 4a), there might be still a possibility for some further cracking. However, for simplicity (and especially for the case of long ties) we will neglect such a possibility; only further widening of existing cracks will be examined.

The situation after the appearance of the last crack is illustrated in Fig. 5(\*). The new steel stress distribution $\sigma_s'$ (with a rather low stress difference $\Delta \sigma_s$) reflects the small area under the bond stress diagram $t'$; unable to produce large tensile stresses ($\sigma_s'$) to the matrix.

It is worth noting that the steel stress inducing the last crack (denoted as $\sigma_{s1}$ on Fig. 5) has the value of $\sigma_{sr,n}$ discussed in the previous paragraph.

Up to the moment of just after the formation of the last crack, the overall elongation has been considered in the previous paragraph.

Now, a further increase in the external load (leading to higher steel stresses $\sigma_s > \sigma_{s1} = \sigma_{sr,n}$), will introduce further elongation, equal to the normalised area KLMN of Fig. 5. Thus,

$$\Delta \varepsilon_{\text{sm}} = (\sigma_s - \sigma_{sr,n})/E_s$$

(24)

where $\sigma_{sr,n}$ from Equ. 23.

\footnote{For further details and a respective computer program, see Tassios et al., 1981 (2).}
In this approximation, it has been assumed that after the final cracking, the steel-stress distribution remains identical and moves upwards (KL→NM) remaining parallel to itself. In reality this is not true; the tension stiffening effect may also increase for a while (Δο' > Δο") since there is still a possibility to mobilise higher bond stresses (τ" > τ' in Fig. 5). However, this is only temporary (bond is degrading for higher slip values) and it is not numerically very important.

Besides above the limit of linearity of steel (f_{sa} ≈ 0.75f_{sy}), all tension stiffening vanishes rapidly (Fig. 6).

**Crack widths**

Neglecting the extensibility of concrete,

\[ w_{cr} = \varepsilon_{sm} a_{lim} \]  \hspace{1cm} (25)

where \( \varepsilon_{sm} \) is taken from \( \sigma_s/\varepsilon_{sm} \) diagram (e.g. Fig. 6) and \( a_{lim} \) from Equ. 18.

For the estimation of the maximum crack width, two kinds of variabilities should be considered:

a) The variability of crack spacings (which is produced because of the variability of tensile strength of the matrix and the bonding properties of the composite).

b) The different history of generation of each crack's family (see Yannopoulos et al., 1989, (3)).

To account for such variabilities, an empirical factor equal to 2 is normally used. Thus,

\[ \max w_{cr} = 2 \varepsilon_{sm} a_{lim} \]  \hspace{1cm} (26)

It is worth noting that under these conditions, closed formulae for crack width prediction seem to be hardly applicable in general cases.

**CALIBRATION**

The assessment of the stress of the composite at first crack has a particular interest for the design of ferrocement objects and structures. This "stress at first crack" was therefore repeatedly investigated and was directly related to the specific surface ratio
was successfully used to this end. Experimental work by Shah et al. [1971, (5), 1972 (6)] has clearly confirmed such a relationship. Using these experimental findings, (Fig. 8), as compiled by Paul et al., 1978 (7), it would be possible to make a first checking of the validity of the approach presented in this paper. To this end, the following input data were used:

- $\minf_{ct} (\approx 0.7f_{ctm}) = 2.1 \text{N/mm}^2$ (approximate ordinate of experimental findings for $S_L=0$), or $f_{ctm}=3.0\text{N/mm}^2$
- $E_s = 2.1 \times 10^5 \text{N/mm}^2$
- $k = 1200 \text{[N/mm]}$ (from Equ. 2)
- $\alpha = E_s/E_c \sim 4$ (roughly estimated value)
- $\rho \sim 4\%$ (an average value of the steel ratio used in the relevant experimental investigations).
- $G_f \sim 0.08 \text{N/mm}$. This value is deduced from Wittmann et al., 1988 (8) by extrapolating $G_f$-values up to the case of maximum grain size of 2 mm.

From the definition of $(\sigma_{FM})_{cr}$

$$(\sigma_{FM})_{cr} = \left( A_s \sigma_{sr} + A_c \minf_{ct} \right)/A_c$$

$$= \rho \sigma_{sr} + 0.7f_{ctm}$$

and using Equ. 8, one finds

$$(\sigma_{FM})_{cr} = 0.7(1+0.8\rho)f_{ctm} + \left[ \frac{G_f}{32} \sqrt{\frac{E_s k_o}{0.7f_{ctm}}} d^{1/4} \right] \left( \frac{4\rho}{d} \right)$$

or, for $0.5<d<2.0 \text{ mm}$ and with an error lower than $\pm 15\%$,

$$(\sigma_{FM})_{cr} \approx 0.7f_{ctm} + \left[ \frac{G_f}{32} \sqrt{\frac{E_s k_o}{0.7f_{ctm}}} \right] S_L$$

Putting in Equ. 30 the input data corresponding to the experimental investigation used, it is finally found

$$(\sigma_{FM})_{cr} \approx 2.4 + 26.5S_L$$
a straight line plotted on Fig. 8. Despite the roughness of the numerical input data used for this checking, it can be said that the analytic prediction does not contradict the experimental results of the investigations considered. However, further calibrations are needed in order to select the most appropriate combinations of the nominal value $w_0$ of the fictitious crack-width and the corresponding bond-factor $k_0$ (see Fig. 1 and Fig. 3) after appropriate linearisation.

EXPERIMENTAL INVESTIGATION

A limited experimental program was conducted consisting of two groups of tests. Specimens measuring 300x50x20 mm, reinforced with different types of mesh wires (woven or welded), were subjected to concentric tensile tests. Two curing conditions namely "wet" and "dry" were used.

All specimens had the same mortar matrix with mix proportions of 1:3:0.5 (cement:sand:water, by weight), exhibiting compression strength $f_{cc} = 20.40$ MPa and tensile strength $f_{ct} = 2.40$ MPa. The maximum aggregate size was 2.8 mm (crushed limestone sand).

Cross-sections and other characteristics of the test specimen are summarised in Table 1.

Test results and discussion

Fig. 9 and Fig. 10 show the mean values of average strains ($\Delta l/l$) for each external load imposed; only the experimental results of wet cured specimens are shown in these Figures.

On the same Figures, the predictions of the previously described simplified model are also shown (the numerical results of the application of the model are summarised in Table 2).

From the comparison of experimental and analytical findings, it is difficult to derive a conclusion of a more general validity. However, it seems that the fracture mechanics approach proposed for the prediction of $\sigma_s$ at first cracking may offer satisfactory solutions for FC reinforced with small diameter wire-meshes. Nevertheless, further analytical work is needed for the transition cracking stage.

On the other hand, Fig. 11 shows the dramatic consequences of dry curing in this experimental inve-
stigation. Micro-precracking has taken place which, among other consequences, "consumed" a considerable part of $f_{ct}$ and of the fracture energy $G_f$, bringing down the value of $\sigma_{Sr}$ (whereas the final tensile strength of the composite remained unaffected). The beneficial role of higher steel percentages becomes evident from Fig. 11.

CONCLUSIONS

Crack widths in Ferrocement composites (near their serviceability stage) are of the same order of magnitude as the fictitious cracks foreseen by Fracture Mechanics before full separation of the matrix under tension. Thus, the force-elongation diagrams of such composites, reinforced with small diameter wire-meshes ($d \approx 1$ mm) and well cured, lie considerably higher than those predicted by the tension-stiffening theory valid for conventional reinforced concrete.

This paper is an attempt to model this fact, be it in an oversimplified way. Analytical refinements and further calibration versus available experimental results are needed.

However, it is believed that the analytical modeling presented in this paper offers some advantages in predicting the effects:

- of the properties of the cementitious matrix, i.e. its composition (plain, fiber-reinforced, polymer modified), its maximum grain size and the rate of loading, all reflected in the respective fracture energy value $G_f$.

- of the bond properties of the reinforcement (e.g. the differences between woven mesh and expanded metal), expressed by the bond factor $k_0$ and the "bond vs. tensile strength" ratio $\lambda$. 
REFERENCES


### NOTATION

**Latin letters**

- $a$: crack spacing
- $A_c$: cross section of the cementitious matrix
- $A_s$: cross section of steel reinforcement in the loading direction
- $d$: steel bar or wire diameter
- $e$: bond transfer length
- $E_c$: modulus of elasticity of cementitious matrix
- $E_s$: modulus of elasticity of steel
- $f_{ct}$: tensile strength of the cementitious matrix
- $f_{ct}^{\text{inf}}$: lower fractile characteristic value of $f_{ct}$
- $f_{ct}^{\text{m}}$: mean value of tensile strengths of the cementitious matrix
- $G_f$: fracture energy of the cementitious matrix
- $k_o$: bond factor within a linearisation field
- $k_r$: bond stiffness modulus within a linearisation field
- $l$: length of the composite element under tension
- $l_b$: bonding length
- $n$: total number of cracks
- $s$: local slip (steel-to-matrix)
- $S_L$: specific surface of reinforcement
- $w, w_{cr}$: crack width
- $w_0$: fictitious microcrack width corresponding to the nominal "first crack" state of FC
Greek letters

\( \alpha = \frac{E_s}{E_c} \) \hspace{1em} \text{the modular ratio}

\( \varepsilon_s \) \hspace{1em} \text{steel strain}

\( \varepsilon_{sm} \) \hspace{1em} \text{average steel strain (equal to the normalised global elongation of the FC tie)}

\( \varepsilon_{sm,r} \) \hspace{1em} \text{average steel strain at "first cracking}

\( \varepsilon_{sm,n} \) \hspace{1em} \text{average steel strain at the end of the transition stage of cracking}

\( \lambda = \frac{\tau_u}{f_{ct}} \)

\( \rho = \frac{A_s}{A_c} \) \hspace{1em} \text{steel ratio}

\( \sigma_c \) \hspace{1em} (or \( \sigma_{ct} \)) \hspace{1em} \text{tensile stress of the cementitious matrix}

\( \sigma_s \) \hspace{1em} \text{steel stress}

\( \sigma_{s,a} \) \hspace{1em} \text{steel stress level at linearity limit}

\( \sigma_{sr} \) \hspace{1em} \text{steel stress at "first crack"}

\( \sigma_{sr,n} \) \hspace{1em} \text{steel stress at the end of the transition stage of cracking}

\( \tau_m \) \hspace{1em} \text{average bond stress}

\( \tau_u \) \hspace{1em} \text{ultimate local bond stress}

Conversion Note

1 N = 0.2248 lb

1 mm = 0.0394 in

1 N/mm² = 145.0 psi
Table 1: Characteristics of the FC specimens tested

<table>
<thead>
<tr>
<th>Series</th>
<th>Number of specimens</th>
<th>Mesh</th>
<th>Diameter (mm)</th>
<th>Cross section (mm²)</th>
<th>Size of mesh (mm)</th>
<th>Number of layers</th>
<th>Volume fraction ( \phi = \frac{\text{As}}{\text{Ac}} )</th>
<th>Specific surface (mm)</th>
<th>( f_{sy} )</th>
<th>( f_{tu} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>4</td>
<td>Woven</td>
<td>1.0</td>
<td>7.85</td>
<td>747</td>
<td></td>
<td>0.80</td>
<td>0.031</td>
<td>450.0</td>
<td>522.0</td>
</tr>
<tr>
<td>II</td>
<td>4</td>
<td>&quot;</td>
<td>1.0</td>
<td>15.71</td>
<td>747</td>
<td></td>
<td>1.60</td>
<td>0.062</td>
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<tr>
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<td>0.094</td>
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<tr>
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<td>541.0</td>
</tr>
<tr>
<td>V</td>
<td>4</td>
<td>&quot;</td>
<td>2.0</td>
<td>25.13</td>
<td>230</td>
<td></td>
<td>2.51</td>
<td>0.050</td>
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</tr>
</tbody>
</table>

Table 2: Numerical application of the analytical model (stresses in MPa, strains %)

<table>
<thead>
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<th>Experimental series</th>
<th>Characteristic points</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>( \sigma_{sr} )</td>
<td></td>
</tr>
<tr>
<td>( \varepsilon_{sm,r} )</td>
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</tr>
<tr>
<td>( \sigma_{sr,n} )</td>
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<td>( \sigma_{sa} )</td>
<td></td>
</tr>
<tr>
<td>( \varepsilon_{sm,q} )</td>
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</tr>
<tr>
<td>I</td>
<td>259*</td>
</tr>
<tr>
<td>II</td>
<td>149</td>
</tr>
<tr>
<td>III</td>
<td>149</td>
</tr>
<tr>
<td>IV</td>
<td>169*</td>
</tr>
<tr>
<td>V</td>
<td>92*</td>
</tr>
</tbody>
</table>

(*) In this case, Equ. 8 yields \( \sigma_{sr} \) values lower than \( f_{ct} (\alpha + \frac{1}{\rho}) \) which then is taken as real \( \sigma_{sr} \).

(**) In the previous case, instead of \( \min f_{ct}/E = 0.07\% \), the average strain is taken from Equ. 9a.
Fig. 1: Residual tensile stress corresponding to a fictitious crack width $w_0$ of the order of 0.01 mm.

Fig. 2: Pullout of a wire at the location of a fictitious crack.
For \( \frac{s}{d} < 4 \cdot 10^{-3} \)

\[
\tau \approx 1.2 \sqrt{\frac{s}{d}} \cdot 10^{-3} f_{ct}
\]

Fig. 3: Local bond "\( \tau \)" is mobilised by a local slip "\( s \)" of a steel bar of diameter "\( d \)". (Based on Shima et al., 1987, for deformed steel bars). A rough linearisation is made near \( s=0.001 \text{ mm for } d \geq 1 \text{ mm} \)

Fig. 4: Transition cracking up to a limit spacing \( a_{lim} \) of cracks. No further cracks appear for higher \( \sigma_s \)-values.
Fig. 5: Distribution of $\sigma_s$, $\sigma_c$, $\tau$, $s$ just before the generation of the last crack at point B; just after that, the distributions $\sigma_s'$, $\sigma_c'$, $\tau'$, $s'$ apply. Further increase of external load leads to the set of distributions $\sigma_s''$, $\sigma_c''$, $\tau''$, $s''$. The interplay between slip values $s$ and the mobilised bond stresses $\tau$ is also shown.
**Fig. 6**: Summing up of the proposed model for FC elements under axial tension
Fig. 7: Numerical examples for FC composites with wire meshes $d=1\,\text{mm}$, S400. Full lines describe the proposed FC model. Point lines illustrate the conventional RC model, for large diameter bars.
Fig. 8: Composite stress at first crack as a function of the specific surface of reinforcement; experimental results [Ref. 5 and 6] and analytical prediction (full line, Equs 8, 30, 31)
Fig. 9: Ferrocement composites (woven meshes d=1 mm, $f_{sy}=450$ MPa, under tension ($l=300$ mm); experimental versus analytical results
Fig. 10: Ferrocement composites (welded mesh d=2 mm, $f_{sy} = 477$ MPa) under tension (l=300 mm); experimental versus analytical findings
Fig. 11: Experimental stress-strain diagrams of FC specimens as described in Fig. 9, cured a) wet or b) dry.
Flexural Behavior of Thin Fiber Reinforced and Ferrocement Sheets

by R.N. Swamy and M.W. Hussin

Synopsis: This paper presents comprehensive test data on the flexural strength, deflection and cracking behavior of thin sheets of 6 to 13mm thickness reinforced with a wide range of reinforcing elements. Two different sizes of sheets were generally tested under four point loading, and in the case of glass fibers, a further small laboratory scale test specimen was also tested. Five different types of reinforcing elements were used: steel fibers, welded steel mesh without and with steel fibers, two types of woven polypropylene fabrics and glass fibers. The matrix was designed for durability and high workability with low water-binder ratio and a superplasticizer. In addition, 50% to 70% of the portland cement was replaced by fly ash. Extensive test data are presented and compared in terms of limit of proportionality, modulus of rupture and cracking. It is shown that a wide range of reinforcement elements can be successfully used for thin sheet applications, and that the performance characteristics of thin sheets are very much a function of the type, geometry and volume fraction of the reinforcement.

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Keywords: composite materials; cracking (fracturing); deflection; ferrocement; fiberboard; flexural strength; glass fibers; metal fibers; polypropylene fibers; portland cements; reinforcing materials; welded wire fabric
INTRODUCTION

Asbestos cement has been an outstandingly successful fiber cement sheet material, extensively used all over the world, because of its excellent properties, very favorable dimensional and chemical stability of the fibers, and the strong interaction between the fibers and the cement paste matrix. However, the dangers to health associated with the handling of the fibers and fabrication of its products has focussed attention on alternative reinforcing elements, and several studies have been conducted during the last decade to find asbestos substitutes (1).

Although extensive research has been carried out during the last two to three decades on fibers such as steel and glass, and more recently, on a wide range of synthetic mineral and organic fibers as well as natural fibers (1-4), there is no agreed consensus of opinion as to the most acceptable fiber or combination of fibers to replace asbestos. It is clear from the evidence of technical performance that there is probably no single fiber or combination of fibers that will fulfill their intended function for all thin sheet applications such as cladding, roofing, thin wall elements, storage containers and concrete boats. The properties of the matrix are also important since the fiber morphology and surface structure have a major influence on the porosity, structure and stability of the composite. Again it may well be that the cement matrix needs to be modified to suit the type of fiber, and no single matrix type may serve successfully all fiber composite combinations and applications.

One of the difficulties in assessing and evaluating the performance characteristics of thin sheets is the lack of comparative data on the role on the fiber in strengthening and controlling cracking of the composite, and in influencing the durability aspects of the composite. The aim of the present paper is to provide comprehensive comparative data on the flexural strength, deflection and cracking behavior of thin sheets of 6 to 13mm thickness reinforced with a wide range of reinforcing elements. The fibers studied in the project are steel, welded steel mesh without and with steel fibers, two types of woven polypropylene fabrics and a combination of glass fibers with a much improved alkali resistance and cellulose fibers. The matrix constituents were varied to suit the type of reinforcing element.

EXPERIMENTAL PROGRAM

The test program was designed to yield performance data on thin sheets based on cement matrices modified to suit a wide range of reinforcing materials. Five series of tests were carried
out as follows:

Series A: Steel fiber reinforced sheets.
Series B: Ferrocement sheets made from welded steel mesh without and with steel fibers.
Series E: Glass reinforced cement (GRC) with some cellulose fibers.

Test Specimen Geometry

The specimen geometry was chosen in order to reflect the size of prototypes generally fabricated in factories as well as the size of specimens used in laboratory tests. To establish the link between the two, two sizes of test specimens were used for all the series, namely, 1000 x 300 x 10 mm thickness and 500 x 100 x 10mm. In addition, for tests on GRC in Series E, additional specimens of size 150 x 50 x 6mm were also tested. All the specimens were tested under four point loading. The effective spans for the three sizes of specimens were respectively 900mm, 400mm and 135mm giving span-depth ratios of 90, 40 and 22.5 respectively.

Test Details

The experimental program consisted of casting and testing five series, each containing several sets of specimens. The specimens in each series had identical matrix. The specimen and reinforcement details are shown in Table 1. It should be noted that specimen sizes 1000 x 300 x 10 mm thickness and 500 x 100 x 10mm were identical in all respects except for their size. The large specimens are prefixed by the letter W, while the letter F denotes the smaller size specimens.

Series A -- Normal weight mortar matrix was used in this series. The reinforcement consisted of high tensile strength brass coated duiform steel fibers 40mm x 0.5m, with a yield strength of 90-1000 MPa and a fracture strength of about 2230 MPa. Three volume percentages, namely, 3%, 4% and 5% were used.
Series B -- Series B consisted of test specimens made with normal weight mortar matrix as in Series A reinforced with welded and galvanised steel wire meshes (conventional ferrocement) or with different sizes and layers of meshes and the brass coated duiform steel fibers described above. The ferrocement sections had volume fractions of 1.26% and 1.60% in the longitudinal direction whereas the fiber reinforced ferrocement sections had total volume fractions of 3% and 5% (Table 1a).

Two different sizes of galvanised mild steel welded wire mesh were used, with mesh openings of 25 x 25mm and 50 x 50mm, having wire diameters of 1mm and 1.6mm respectively (Table 1). Although these mesh openings are larger than those generally used, they facilitate closer spacing of mesh openings by staggering the location of the meshes. Further, these particular mesh sizes were used to enable steel fibers to be incorporated in the specimens. The meshes had elastic modulus of 95-120 GPa and yield and ultimate strengths of 218-236 MPa and 288-314 MPa respectively.

Series C -- Four different types of polypropylene mesh with different sizes of mesh opening were used in this series. To be compatible with the low modulus fiber, a low modulus lightweight mortar matrix was used. This series had a low fiber volume percentage in the longitudinal direction varying from 0.24% to 3.78% (Table 1b).

Series D -- Cement paste was used as a matrix in this series, with polypropylene mesh with smaller mesh opening as reinforcement. The variables here were the type and number of layers of mesh, the fiber volume varying from 1.88 to 22.60%.

The reinforcement used in Series C and D was woven polypropylene fabrics of the type used in the carpet industry and in civil engineering in road and drainage systems. Five different grades (Table 1b) were used: the numbers defining the mesh size (24/31, for example) refer to the number of fibers in the weft and warp directions respectively per 100mm. The warp runs in the direction of main reinforcement (i.e. longitudinal), and provides the strength, whereas the weft is interwoven between these fibers running in the transverse direction. The weft fibers transfer the strength of the warp fibers to the composite by creating a strong mechanical bond with the matrix. The elastic modulus of the polypropylene fibers ranged from 2.8 to 3.8 GPa while their ultimate strength ranged from 150 to 440 MPa. The polymer grid used in these tests had a mesh size of 8 x 9mm with mean elastic modulus of 3.0 GPa and ultimate tensile strength of 245 MPa.
Series E -- For comparison with the behavior of established thin sheet materials and the polypropylene reinforced sheets, tests in this series were carried out on factory produced glass reinforced cement (GRC) sheets, having a thickness of 6mm and a density of 1700 kg/m³. The GRC boards contained Cemfil II Alkali Resistant Fibers 37mm long and a small amount of cellulose fibers; the glass content was in the range of 2.5 to 5%.

Matrix Properties

From practical, economic and technical considerations, the matrix used for the thin sheets was designed to have high workability, low water to binder ratio and adequate strength. Excellent flow characteristics were required to allow the matrix to penetrate through several layers of mesh with fibers added, particularly with the thin fabrics used in Series C and D. All mixes therefore contained a superplasticizer, and in addition, 30% to 70% by weight of cement was replaced by fly ash. The fine aggregate was also restricted to a maximum size of 2.36mm. Natural dry sand was used for the normal weight matrices and an expanded clay aggregate for the lightweight matrices. In order to reduce the bleeding characteristics of the matrix, a special mixing procedure was adopted (5), and this proved very effective.

For Series A and B, the final matrix chosen contained 70% cement replacement with fly ash, had a water to binder ratio of 0.3 and averaged 28 day and 1 year compressive strengths of 43.5 MPa and 91.0 MPa respectively. The lightweight mortar matrix used for Series C tests had an average dry density of 1600 kg/m³ and contained 50% by weight cement replacement with fly ash. This matrix had a compressive strength of 17 MPa at 28 days and about 31 MPa at 6 months. The cement paste matrix for Series D had a water to binder ratio of 0.4, 50% by weight cement replacement with fly ash and compressive strengths of 36 MPa at 28 days and 54 MPa at 6 months respectively. The 28 day flexural strengths of the three mortar matrices averaged 7.5, 4.1 and 3.6 MPa respectively. The matrix for the glass and cellulose fiber sheets in Series E also contained fly ash. These sheets were factory produced and precise details of the matrix are not known.

Testing Details

All the specimens were cured in a fog room and tested at 28 days by loading at the third points. The rate of loading was kept constant at 0.50mm/min. The mortar matrix strains at the tension and compression faces were measured through electrical resistance strain gages and the central deflection through dial gages. All the strain measurements were continuously recorded. The cracking behavior in the constant moment region
was also monitored throughout the test. Three plate specimens were tested for each variable. At least twelve measurements of the section depth were made for each sheet and the average flexural stress values calculated using simple elastic bending theory.

TEST RESULTS AND DISCUSSION

The test results showing the limit of proportionality (LOP) stress and the nominal modulus of rupture (MOR) together with the average section depth of the specimens and crack spacing are shown in Tables 2 to 6. Typical load-deflection and stress-strain relationships measured on the 500 x 100 x 10mm specimens are also shown for each type of composite. All the load-deflection and stress-strain curves showed an initial linear portion followed by a non-linear post-cracking stage. Typical crack patterns are also presented.

Steel Fiber Concrete Sheets (Series A)

Table 2 and Fig.1 show the performance characteristics of sheets reinforced with steel fibers. Fiber volumes of 3% to 5% are relatively high for steel fibers, and the data show that there is a small decrease in LOP values as the fiber volume is increased. On the other hand, an increase in fiber volume increases the MOR; the maximum increase of about 50% in the flexural strength occurred when the fiber volume was increased from 3% to 4%. There was only a modest increase in MOR when the fiber volume was further increased to 5%. About 4% thus seems to be the optimum fiber volume from strength considerations, the MOR for this fiber content being about 24-25 MPa with a crack spacing of about 7mm. The average crack spacing also decreased dramatically, from about 35mm to about 7mm, when the fiber volume was increased from 3% to 4%, emphasizing that 3% fiber volume was inadequate for optimum control of cracking and flexural strength. Thus from both flexural strength and crack control considerations, about 4% fiber volume appears to be the optimum fiber content for steel fibers. There was only a marginal increase in crack control when the fiber volume was increased to 5%, confirming that any increase in fiber content beyond 4% was not being effective in enhancing the engineering properties of the composite.

Table 2 shows two more important performance characteristics. There is a distinct and consistent size effect, the larger specimens showing a decrease in LOP and MOR values of about 5%. In practice, factory produced sheets may be many times larger than the largest size of 1000 x 300 x 10mm sheets tested in this study. Further decrease in flexural strength can
therefore be expected in large sheets. In addition, sheets tested with the cast face in tension again showed a distinct, although small, loss in strength and increase in crack spacing. The flow and cohesive characteristics of the matrix are thus important factors to be considered in large scale production. A matrix that holds the fibers in place and yet prevents fiber bundling will obviously optimize both the fiber and sheet performance characteristics.

Ferrocement Sheets (Series B)

Table 3 and Fig. 2 show the behavior characteristics of ferrocement sheets without steel fibers. An increase in fiber volume (in the longitudinal direction) again produced a modest increase in LOP and MOR stresses. The 25mm square mesh reinforced sheets developed cracks at about the same spacing as the transverse wires; however, with the 50mm mesh, additional cracks developed between the initial cracks formed along transverse wires, and the final crack spacing was reduced from 50mm to about 36mm. This increased crack control is partly due to the welded mesh and partly to the improved properties of the matrix which was able to develop adequate bond to allow multiple cracking to occur (Fig. 2).

When short steel fibers were added to the ferrocement sections, substantial improvements in both flexural strength and crack control were achieved (Table 3). The effects on LOP of varying the mesh volume and the fiber volume combinations are marginal, but both MOR and crack spacing were greatly influenced to advantage by varying the proportions of the mesh and fiber content. With the same number of layers of mesh, increasing the fiber volume increased both flexural strength and crack control, and the best improvements were achieved with the smaller 25mm square mesh. At 5% total steel volume, flexural strength of about 25 MPa and crack spacings of 11 to 12mm were achieved (Fig. 3) with this mesh and fiber combination. At 3% total steel volume, increasing the number of layers from 2 to 4 was not beneficial in strength, although the crack control improved. At 5% total steel volume, increasing the mesh layers from 2 to 4 increased the flexural strength by 30% to 50%, without affecting the crack control. On the other hand, with 4 layers of 25mm square mesh, adding about 1.7% fiber volume increased flexural strength by 20% to 50% and reduced crack spacing by about half. Increasing the fiber volume further to about 3.7%, more than doubled the flexural strength.

The important conclusion derived from these data is that for a given type and size of mesh and fiber, it is possible to find an optimum mesh content and fiber volume from which the maximum benefit to flexural strength and crack control can be achieved. For the materials used in this study, 4 layers of 25mm square welded mesh with 3.74% fiber (0.5 x 40mm) volume
in random orientation produced flexural strength of 32 to 37 MPa and crack spacings of 12 to 13mm. These are very remarkable performance characteristics for a 10-12mm thick sheet, and show that such sheets can be utilised in a wide range of practical applications.

Sheets with Woven Polypropylene Fabrics (Series C and D)

Two series of tests were carried out with these fabrics. Series C consisted of sheets with low modulus lightweight mortar matrix and low fiber volumes (in the longitudinal direction) up to about 3.8%. Table 4 presents strength and cracking data for these sheets; Fig.4 gives typical load deflection and stress-strain relationships and Fig.5 the crack pattern. Obviously, low volumes of such mesh are not always functionally adequate; however, it is interesting to note that even as low a mesh volume as about 0.5%, gave LOP values comparable to the plain unreinforced sheet and distinct post-cracking behavior prior to failure (Table 4). With the lightweight matrix, a mesh volume of about 3.8% (24/31 mesh) gave flexural strengths of 7 to 9 MPa, and crack spacings of 6 to 8mm (Fig.5). The mesh geometry with the woven fabrics has thus an important effect both on strength and the post-cracking behavior.

Specimens in Series D were made with a cement paste matrix and two of the most promising woven fabrics, namely, the 24/31 and 31/47 fabrics. The strength and cracking data for these sheets are shown in Table 5. Figure 6 shows the load-deflection and stress-strain behavior, and the crack patterns in Fig.7.

A comparison of data in Tables 4 and 5 of sheets with the 24/31 fabric shows that a lightweight matrix is not necessarily the best matrix to be used with a low modulus mesh, if strength were the main requirement. With a mesh volume of 3.78%, the use of a stiffer matrix increased flexural strength by 25% to 30% to about 9.5-11.5 MPa and retained similar crack spacing of 6 to 8mm.

With the woven fabric 31/47, doubling the mesh volume from 4.52% to 9.04% nearly doubled the flexural strength (from 8-9 MPa to 14-21 MPa) and reduced crack spacing by nearly half from about 9-9.5mm to about 5mm (Fig.7). With 8 to 10 layers of such fabric, flexural strengths of 23-24 MPa and 35-40 MPa were achieved for the larger and smaller test specimens respectively, and a crack spacing of about 3.0 to 3.5mm. With polypropylene fabrics, as with other reinforcing elements, there was again a distinct size effect – a fact that should not be overlooked when performance of large size sheets is evaluated on the basis of laboratory scale tests. This is particularly important when considering LOP values, although this may not be the sole factor for service load stresses.
The data in Table 5 seem to indicate that with 31/47 fabrics, about 8 layers of fabric appear to be optimum from both strength and crack control considerations. In practice corrugated sheets perform much better, and it should be possible to achieve excellent strength properties and crack control with such sheets reinforced with woven fabrics.

Glass Reinforced Cement Sheets (Series E)

These sheets were factory produced with an average sheet thickness of 6mm, and a total fiber volume of about 5% with a small amount of cellulose fibers included. The glass fibers were the improved Cemfil II fibers. The matrix again contained fly ash. The flexural behavior characteristics of these sheets are given in Table 6 and Figs.8 and 9.

These data show three significant factors which are not always recognised in practical usage. Firstly the production process has an effect on the performance of the sheets. Those cut transversely to the production line showed reduced flexural strength by up to about 20%. Secondly, unlike steel fiber reinforced sheets, sheets tested with the cast face in tension gave higher strengths than those tested with the mold face in tension. Finally, there is again a distinct size effect on flexural strength. Large sheets with the cast face in tension had strength losses of about 20%; in practice, with very large sheets, these losses could be higher. The differences in behavior between the longitudinally and transversely cut sheets and the size effect can be further enhanced when the sheets are exposed to cyclic environmental conditions when both strength and ductility can be adversely affected (5).

A comparison of the data in Tables 5 and 6 shows that to obtain flexural strength comparable to that of a 5% GRC sheet, about 6 layers of the 31/47 polypropylene woven fabric will be required and will give about 20 MPa for the large 1m long sheets and about 25 MPa for the 0.5m long sheets. However, exposure to cyclic wetting/drying or heating/cooling conditions will drastically reduce the strength and ductility of GRC sheets whereas the polypropylene reinforced sheets will be practically unaffected by such climatic effects (6).

To illustrate the differences in flexural behavior between sheets reinforced with glass fibers, polypropylene open networks (NETCEM) and polypropylene woven fabrics, Fig.10 is presented to compare the results of this study with those of others (7-9). For clarity, the curves up to the LOP have been omitted in the figure. It can be seen that on the basis of flexural behavior of specimens of comparable size, the polypropylene woven fabrics compare very favorably with both NETCEM and GRC.
CONCLUSIONS

This paper presents comparative data on the flexural strength and cracking behavior of thin sheets reinforced with a wide range of reinforcing elements. Sheets of 10-13mm thickness were reinforced with short steel fibers, welded steel mesh without and with steel fibers and polypropylene woven fabrics having a wide range of mesh geometry. These data are compared with test results of factory produced GRC sheets having a thickness of 6mm and containing a small amount of cellulose fibers. From the data presented in this paper, the following conclusions can be drawn, some of which are specific to this study.

1. With steel fibers, about 4% by volume appears to be the optimum fiber content giving a flexural strength of 24-25 MPa and a crack spacing of about 7mm. Sheets tested with the cast face in tension showed a distinct, although small, reduction in strength and increase in crack spacing.

2. Sheets reinforced with 25mm square welded mesh developed cracks at about the same spacing as the transverse wires. With a 50mm square welded mesh and a suitable matrix it was possible to reduce the crack spacing to about 36mm, lower than the transverse wire spacing.

3. With a combination of welded steel mesh and steel fibers, it was possible to optimize both flexural strength and crack control. With 4 layers of 25mm square welded mesh and a steel fiber volume of 3.74% in random orientation, it was possible to obtain flexural strengths of 32 to 37 MPa and crack spacings of 12 to 13mm.

4. With woven polypropylene fabrics, even as low a mesh volume as 1%, gave LOP values comparable to that of the plain unreinforced sheet and distinct post-cracking behavior prior to failure. With a mesh volume of about 3.8% (24/31 mesh) and a lightweight matrix, flexural strengths of 7 to 9 MPa and crack spacings of 6 to 8mm were obtained. However, a lightweight low modulus matrix does not necessarily give the best results with a low modulus fabric.

5. With the 31/47 fabrics, 8 to 10 layers of reinforcement produced flexural strengths of 23-24 MPa and 35-40 MPa for large and small sheets respectively with a crack spacing of about 3.0 to 3.5mm.

6. With factory produced GRC sheets, specimens cut transversely to the production line gave about 20% lower flexural strength than those cut in the longitudinal direction. Further, sheets tested with cast face in tension showed higher strengths than those tested with the mold face.
in tension.

7. All the fiber reinforced cement sheets showed a distinct size effect on flexural strength. This factor should not be overlooked when factory produced large sheets are assessed on the basis of laboratory scale tests.

8. To obtain a flexural strength comparable to that of a GRC sheet with 5% fiber volume, about 6 to 7 layers of the 31/47 polypropylene woven fabric will be required. However, cyclic exposure conditions have a substantial adverse effect on the strength and ductility of GRC sheets whilst polypropylene reinforced sheets remain practically unaffected by such climatic effects.

9. Apart from strength improvement, crack control is the major, and probably the most important, property imparted by fiber and/or mesh reinforcement. Large ductility and energy absorption properties consequently ensue.

10. The data presented here show that a wide range of reinforcing elements are available for thin sheets for applications in roofing, cladding, partition walls and other structures made up with thin wall elements.

REFERENCES


6. Swamy, R. N. and Hussin, M. W., Effect of curing conditions


Table 1a Details of specimens in Series A and B.

<table>
<thead>
<tr>
<th>Series</th>
<th>Specimen designation</th>
<th>Wire mesh</th>
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<th>Total specific surface (S_{RL} cm²/cm³)</th>
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<td>Mesh type</td>
<td>Wire yield strength (N/mm²)</td>
<td>Volume fraction (ρ)</td>
<td>Specific surface (V_{RL} cm²/cm³)</td>
<td>Volume fraction (ρ)</td>
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Note: * (a) means mild face in tension. ** (b) means cast face in tension. † volume of reinforcement in longitudinal direction. ‡ specific surface of reinforcement in longitudinal direction.
Table 1b Details of specimens in Series C, D and E.

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<th>Specimen designation</th>
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<th>No. of mesh layers in specimen</th>
<th>Fiber percentage by volume (VR</th>
<th>Specific surface (SRL cm²/cm³)</th>
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<td>0.38</td>
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<td>WGT(b) GRC</td>
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</table>

Note

× unless otherwise stated, the reinforcement is polypropylene mesh: see text for definition of mesh size.
* (a) means mold face in tension.
** (b) means cast face in tension.
† volume of reinforcement in longitudinal direction.
‡ specific surface of reinforcement in longitudinal direction.
§ total volume fraction as specified by the manufacturer.
Table 2  Limit of proportionality and modulus of rupture (series A)

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<th>Specimen identification</th>
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<th>Limit of proportionality</th>
<th>Nominal modulus of rupture</th>
<th>Average crack spacing at failure (mm)</th>
<th>Section depth (mm)</th>
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<td>( N/mm^2 ) C.V. %</td>
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<td>10.00</td>
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<td>-</td>
<td>7.50 5.7</td>
<td>10.00</td>
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<td>16.27 17.15 7.1</td>
<td>34.6 36.3</td>
<td>10.07 10.02</td>
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<td>9.07 10.31 3.8</td>
<td>24.10 25.10 8.5</td>
<td>7.1 7.1</td>
<td>10.29 10.30</td>
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<td>16.17 17.87 6.9</td>
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<td>10.6 12.9</td>
<td>10.21 10.20</td>
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Note: * The letter W is meant for sheet 1000x300x10 mm size, the letter F is meant for flexural specimen, 500x100x10 mm plate. †(a) means mould face in tension. †(b) means cast face in tension.
<table>
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<th>Specimen identification</th>
<th>No. of specimen</th>
<th>$V_{RL}$ (%)</th>
<th>Limit of proportionality</th>
<th>Nominal modulus of rupture</th>
<th>Average crack spacing at failure (mm)</th>
<th>Section depth (mm)</th>
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<td>N/mm²</td>
<td>C.V. %</td>
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Table 4  Limit of proportionality and modulus of rupture (series C)

| Specimen identification | No. of specimen | \( V_{RL} \) (%) | Limit of proportionality | Nominal modulus of rupture | Average crack spacing at failure (mm) | Section depth (mm) *
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<td>N/mm(^2) C.V. %</td>
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* average of 12 measurements
Table 4 (cont'd)

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<td>Section depth (mm)</td>
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Note + W - 1000 x 300 x t mm sheets
      F - 500 x 100 x t mm plates
      f - 150 x 50 x t mm plates
* total volume fraction as specified by the manufacturer
The measured thickness of GRC specimen was 5.96 mm at 30 different measurements with C.V. = 2.1%
Fig. 1--Load-deformation behaviour of sheets reinforced with steel fibers (series A)
Fig. 2--Cracking behaviour of ferrocement sheets without steel fibers (series B)
Fig. 3--Load-deformation curves of ferrocement sheets with steel fibers (series B)
Fig. 4--Load-deformation behaviour of sheets reinforced with woven polypropylene fabrics (series C)
Fig. 5--Cracking behaviour of sheets reinforced with woven polypropylene fabrics (series C)
Fig. 5 (cont'd) -- Cracking behaviour of sheets reinforced with woven polypropylene fabrics (series C)
Fig. 6(a)—Load-deformation characteristics of sheets reinforced with woven polypropylene fabrics (series D)
Fig. 6(b)—Load-deformation characteristics of sheets reinforced with woven polypropylene fabrics (series D)
Fig. 7--Crack patterns of sheets reinforced with woven polypropylene fabrics (series D)
Fig. 7 (cont'd) -- Crack patterns of sheets reinforced with woven polypropylene fabrics (series D)
Fig. 8(a)—Load-deformation behaviour of GRC sheets (series E) Mould face in tension; L: tested longitudinally
Fig. 8(b) -- Load-deformation behaviour of GRC sheets (series E)
Cast face in tension; T: tested transversely
Fig. 9 -- Cracking behaviour of GRC sheets
Fig. 10—Comparative load-deformation behaviour of thin sheets made from GRC, NETCEM, and polypropylene fabrics.
Tensile Behavior of Thin Ferrocement Plates

by R.N. Swamy and Y.B.I. Shaheen

Synopsis: This paper presents comprehensive test data on the tensile behavior of ferrocement plates, 12.5mm thick. The main variables investigated were mesh geometry, specific surface, volume fraction, mesh yield strength and skeletal bars. The specimens were specially designed to ensure failure in the gage length. The matrix was proportioned for high strength, high workability and high durability with low water to binder ratio, and contained 50% cement replacement with fly ash. Cracking and deformation were monitored throughout the loading range. The results showed that the composite properties of elastic modulus and ultimate tensile strength could be very satisfactorily predicted. However, the cracking behavior for a wide range of mesh geometry could not be satisfactorily predicted by a single unique relationship. There was, however, a good correlation between the composite properties of ultimate tensile strength and ultimate flexural strength. The results show that by suitable design of the matrix and the reinforcement, high strength ferrocement sheets with high crack resistance can be developed for a variety of structural applications.

Keywords: composite materials; cracking (fracturing); deformation; durability; ferrocement; fly ash; modulus of elasticity; mortars (material); plates (structural members); tensile strength; tests; welded wire fabric
ACI member R. Narayan Swamy is on the faculty of the University of Sheffield. His main research interests are in concrete materials and concrete structures. He is a member of ACI Committees 544, Fiber Reinforced Concrete, and 549, Ferrocement. He is the recipient of the George Stephenson Gold Medal from the Institution of Civil Engineers and the Henry Adams Diploma from the Institution of Structural Engineers, England.

Yousry Bayoumy Ibrahim Shaheen has carried out extensive studies on ferrocement concrete plates. His research interests are in concrete material behavior, ferrocement and structural behavior.

INTRODUCTION

Ferrocement sections are noted for their high ductility and toughness, and are capable of carrying loads for long periods of time (1-3). Thin ferrocement plates of high strength and crack resistance could therefore be used not only for roofing and cladding, but also for other unusual applications such as shell structures, storage containers and ships. However, since the behavior of the section depends on the close interaction between the cement matrix and the mesh reinforcement, the correct choice of these two components is an essential first step for the successful long term performance of the material.

The matrix has to be able to perform several roles. Since it has to penetrate several layers of mesh, sometimes of small mesh openings, excellent flow characteristics of the matrix are essential if a dense composite is to be fabricated, and the matrix is to fill the composite fully, particularly behind or underneath the different layers of wires. Further, the cover to the mesh reinforcement is very small, often only of the order of 5mm, or sometimes even less; it then becomes very important to ensure that the matrix is capable of developing a compact structure that will resist penetration of carbonation and chloride ions, the two factors that eventually lead to steel corrosion.

The other main difficulty is in the choice of the mesh geometry. Meshes are produced in a wide range, with a wide choice of mesh opening, wire diameter and mesh fabrication. Much more confusing, and technically frustrating, is the high variability of the mechanical properties of the mesh, particularly its elastic modulus, yield strength and ultimate strength. The mesh geometry and the mesh properties can thus
impart quite different reinforcing properties to the composite, and the overall performance of the material can be very variable, particularly when the results obtained in different countries are compared.

The aim of this paper is to establish some definitive patterns of cracking and deformation of thin ferrocement sheets of 12.5mm thickness in direct tension. Emphasis has been given to three significant aspects of material fabrication. The test specimen geometry is chosen to ensure uniform stress distribution uninterrupted by the gripping stresses at the ends of the specimen. The matrix was designed for high strength, high flowability and high durability. The reinforcement consisted of mild steel welded mesh or high tensile steel woven mesh with a wide range of mesh geometry and mesh strength. The effect of the presence of skeletal bars was also studied. The results are compared to sections reinforced with small size rebars alone.

EXPERIMENTAL PROGRAM

Test Specimen Geometry

Bearing in mind the inherent difficulties in testing thin sheet specimens in direct tension, the test specimens were especially designed to ensure failure away from the grips and the ends of the specimen. The test geometry used is shown in Fig.1, the overall size of the specimen being 560 x 100 x 12.5mm, with a test gage length of 300mm. The ends of the specimens were thickened to 37.5mm, and additionally reinforced with further layers of mesh to improve the resistance to gripping stresses, and impede pull-out of meshes and premature failure. Prior to testing, two steel plates were bonded to each end of the specimen by a special adhesive, ensuring adequate bonding between the steel and cement matrix. All the precautions taken produced the desired cracking and fracture characteristics as shown in Fig.1.

Test Program

The experimental program consisted of seven series of tests as shown in Table 1. The test specimens were reinforced with various types and amounts of mesh and/or steel bar (skeletal) reinforcement. The mesh used was mild steel galvanised welded mesh, mild steel ungalvanised welded mesh, and high tensile woven mesh, all of different geometry and wire diameter. The skeletal steel consisted of 4mm and 5mm dia. bars of mild steel and 5mm dia. high tensile steel. The details and mechanical properties of the reinforcement used
in the tests are shown in Table 2. The main variables studied are: number of mesh layers, geometry of the mesh opening and wire diameter, type of steel, combination of mesh and skeletal bars, specific volume and volume fraction of the reinforcement. Comparative tests were also carried out on specimens reinforced with steel bars alone. The reinforcing details of each series are shown in Table 1.

Matrix Properties

The matrix was designed to have high flow characteristics, high strength, low water to binder ratio and high durability. To achieve these properties, 50% of the cement was replaced with a good quality Type F fly ash and a sulphonated melamine formaldehyde superplasticizer was also used. Two mixes, both having the same water to binder ratio of 0.3, were designed for 28 day compressive strengths of 55-60 MPa and 65 to 70 MPa for use with mild steel (mix M4) and high tensile steel reinforcement (mix M3) respectively. Both mixes had one day strength of 17 to 18 MPa (on 50mm cubes). Mix M4 had a 28 day strength of 67 MPa and mix M3, 74 MPa. At 6 months, these strengths had increased to an average value of 99 MPa and 102 MPa respectively. The 28 day flexural strength measured on 500 x 100 x 25mm plates averaged 8 MPa for both mixes while the elastic modulus at the same age averaged 31 to 32 GPa (4).

Fabrication and Testing

The test specimens were cast in specially prepared molds, cured in a fog room (20°C, RH 100%) and tested at 28 days. A uniform cover of 3mm was maintained for all the test specimens, except for Series E where the cover was 2.5mm. A special technique was adopted in the mold which ensured a constant cover during reinforcement fabrication and casting. This consisted of threading fine steel wires through holes in the mold which were then tensioned by means of lock nuts. The reinforcing cages were supported on these wires. Further, the entire reinforcement was tied together into a rigid case using mortar spacers made from the same mortar matrix. Matrix strains were measured on both faces using a mechanical extensometer and electrical resistance gages; the latter were also used to monitor the occurrence of the first crack, and further cracking was monitored throughout the tests. The tests were carried out at a cross-head speed of 1mm/min. Three specimens were tested for each variable.
TEST RESULTS AND DISCUSSION

From the large amount of data obtained from the tests, only the more important aspects of tensile behavior are discussed below.

Load-strain behavior

Typical load-composite strain curves are shown in Figs. 2 to 4 for Series A (mild steel mesh + skeletal bar), Series D (mild steel mesh only) and Series F (high tensile steel mesh) respectively. The load-strain curves depicted similar characteristics to load-deflection curves in flexure (5), and showed varying degrees of the three typical stages of elastic, elasto-plastic and plastic behavior. The extent to which these three stages are each predominant depends on the geometry, volume fraction, surface area, and type of mesh. In general, the addition of skeletal bars to mild steel mesh did not make much difference to the first crack load, whereas high tensile woven meshes tended to give lower first crack loads to comparable areas of mild steel mesh (Figs. 3 and 4). Further, the elasto-plastic and plastic stages could not always be completely defined, particularly for high tensile steel meshes as spalling of the matrix cover and debonding from the reinforcement invariably occurred with such meshes.

As expected, test specimens reinforced with skeletal bars alone showed low first crack loads compared to specimens reinforced with mesh or mesh and skeletal bars (Fig. 5). On the other hand, for the same volume fraction, specimens reinforced with skeletal bars generally gave much higher ultimate strengths compared to specimens with similar meshes because of their higher tensile strengths, and were therefore able to sustain much higher tensile strains prior to failure.

It must, however, be borne in mind that strain measurements on the matrix include crack widths, once the cracks are formed. The matrix strains cannot therefore be strictly interpreted as the matrix strain; rather, the strains represent the composite tensile strain capacity which enables it to preserve its structural integrity and composite behavior throughout the loading range. The very large composite ultimate strains shown in Table 3 are thus a measure of the ductility of the composite and its ability to remain as a single unit.

In ferrocement sheets, the behavior of the specimen and the composite action is very much a function of the interaction between the cement matrix and the steel wires. The matrix contribution depends on its tensile strength (and strain capacity) whilst the mesh contribution depends on the effective reinforcement ratio and the steel stress. These values have
been calculated for a typical specimen S1D and the results are illustrated in Fig.6.

**Composite stress at first crack**

The stress at first crack of ferrocement sheets in tension is known to be influenced by the specific surface of the mesh wires and the mesh volume fraction, both in the loading direction. The composite first crack strength is shown in Table 3, and the results relating composite stress at first crack to specific surface and volume fraction are shown in Figs. 7 and 8. These figures also show the best fit regression lines for each variable. It is clear that the results are too scattered to give any reliable theoretical best fit relationship. Considering the large number of variables included in this study, these data confirm that such relationships can only be at best very, very approximate.

**Composite modulus**

The elastic modulus of the composite in the elastic stage can be predicted by the law of mixtures (particularly for welded and woven meshes),

\[ E_{ct} = E_m V_m + E_s V_R \]  \hspace{1cm} (1)

where \( E \) and \( V \) represent the elastic modulus and volume fraction and \( m, s \) and \( R \) represent the matrix, steel and longitudinal steel. The predicted values gave satisfactory correlation with the experimental results as shown in Table 4.

**Ultimate tensile strength**

At failure, the matrix in a ferrocement section is heavily cracked and carries no load. The ultimate load that a test specimen can then take in tension is approximately equal to the ultimate load taken by all the reinforcement in the loading direction. Table 5 shows the comparison between the experimental composite ultimate strength and the reinforcement ultimate strength. The results show very good correlation as indicated in Fig.9. The following regression equation represents a good best fit line with a coefficient of correlation of 0.978.

\[ \sigma_{ult} \cdot V_{RL} = 0.96 \times \text{ult. comp.stress} - 0.0088 \]  \hspace{1cm} (2a)
Thin-Section FRC and Ferrocement

or, in effect,

\[ \sigma_{\text{ult}} \cdot V_{RL} = 0.96 \times \text{ult. comp. stress} \]  \hspace{1cm} (2)

where \( V_{RL} \) is the volume fraction in the longitudinal direction.

**Relation between ultimate tensile and flexural strength**

Test specimens reinforced similarly were also tested in flexure, but not reported here. Both these studies indicated a direct and consistent relationship between composite direct tensile strength and composite flexural strength as shown in Fig.10. The best fit line has the following equation:

\[ \sigma_{\text{ut}} = 0.31 \sigma_{\text{flex}} + 0.66 \]  \hspace{1cm} (3)

where \( \sigma_{\text{ut}} = \) ultimate composite tensile strength and \( \sigma_{\text{flex}} = \) ultimate composite flexural strength. The correlation coefficient for this equation is 0.857, and the relationship between the two properties can be seen to be good.

**Cracking behavior**

Cracking is a random phenomenon subject to a large degree of scatter. Many research studies have attempted to correlate the mean crack spacing to the volume fraction of the matrix \( V_m \) and/or the specific surface of the mesh steel in the longitudinal direction \( S_{RL} \) (6,8). The results of this study plotted in Fig.11 show considerable discrepancy between the theoretical values and the experimental crack spacings. These results only confirm similar discrepancies reported earlier by the senior author (5).

The main reason for this discrepancy is the fact that there are other important variables such as the mesh tensile strength which has a considerable influence on crack spacing and crack width. Further, the geometry of the mesh, its volume and the presence of skeletal bars also affect cracking behavior as shown by the senior author elsewhere (5, 9). From an extensive analysis of the data obtained from this study (10), it was found that, because of the large variations in mesh geometry, its elastic modulus and tensile strength, and the large number of parameters involved, it is unlikely that a single unique equation can be developed, that will be not only simple but will also give meaningful engineering correlation that can be used in practice. However, a parameter, represented
by the product of the yield strength of the mesh and the volume fraction of the mesh in the steel direction, was found to be satisfactorily correlated to cracking behavior. Such equations take the following form:

\[
\text{crack spacing} = A - B \cdot f_y \cdot V_{RL}
\]  
\[
\text{crack width} = \{C - D \cdot f_y \cdot V_{RL}\} \varepsilon_m
\]

where A, B, C and D are numerical constants, \( f_y \) is the yield strength of the mesh reinforcement, \( V_{RL} \) is the volume fraction of steel in the longitudinal direction and \( \varepsilon_m \) is the average tensile strain on the face of the specimen (10). Because of the large number of such equations involved, they are not presented here. However, Table 6 shows the correlation obtained between the predicted values and the experimental final crack spacing. The data show very good correlation, bearing in mind the randomness of the cracking phenomenon. The important conclusion from these studies is that it is unlikely to be able to find a single, unique equation that will satisfactorily predict the cracking behavior for all types of meshes, but that for a given geometry and type of mesh, it is possible to develop equations that will have engineering significance.

The cracking behavior and the failure pattern of all but one test specimens is shown in Fig.12. It is clear that excellent crack control can be obtained by choosing the appropriate geometry and volume fraction of mesh reinforcement. Equally, bars alone (Series E and G) cannot give the type of crack control obtained with meshes or meshes and skeletal bars.

Energy absorption and toughness

Figures 2 to 4, and Table 3, show the large ductility and energy absorption capability of ferrocement sections. These are quantified here, energy absorption by the area under the load-elongation curve, and the toughness by the ratio of the total area under the load-elongation curve to that up to the first crack load. These results are shown in Table 7. This data can only be looked upon as qualitative values since they cannot be translated directly to design. The very high toughness values for Series E (mild steel bars only) and Series G (high tensile bars only) are slightly misleading because of their low first cracking load and consequent small areas of load-extension up to first crack. Nevertheless, these results give an overall picture of the capability of ferrocement section to resist dynamic forces.
CONCLUSIONS

The main conclusions derived from these tests are given below.

1. All the test specimens showed to varying degrees the three distinct stages of load-composite strain behavior, namely, elastic, elastoplastic and plastic. With high tensile steel woven meshes, the last two stages could not be clearly defined because of the spalling of the cover steel.

2. The addition of skeletal bars to mild steel mesh did not make much difference to the first crack loads. High tensile woven meshes tended to give lower first crack loads compared to mild steel welded mesh, because of their higher elastic modulus and nature of their weave. Test specimens reinforced with skeletal bars alone also gave lower first crack loads compared to those with mild steel mesh.

3. All the specimens showed very large tensile strain capacity prior to failure which is a measure of the close interaction between the matrix and the mesh to preserve composite action and structural integrity.

4. Although both the specific surface of the mesh wires and the mesh volume fraction in the loading direction influence the composite stress at first crack, the relationships were too scattered to give a reliable numerical best fit relationship.

5. The composite elastic modulus on the other hand, could be satisfactorily predicted by the law of mixtures.

6. The ultimate composite tensile strength was approximately the same as the reinforcement ultimate strength.

7. There was also a good correlation between the ultimate composite tensile strength and the composite flexural strength.

8. Because of the large number of parameters influencing cracking behavior including mesh geometry, elastic modulus and tensile strength, the tests showed no single unique relationship to define final crack spacing in a meaningful engineering way. Existing equations showed poor correlation to the data developed in these tests.
9. For a given type of mesh, it was possible to establish a good correlation between a regression equation involving mesh volume fraction and strength to crack spacing. However, no simple unique relationship was found to be able to predict cracking behavior satisfactorily for all types of mesh geometry.

10. All the test specimens showed excellent ductility, energy absorption capability and toughness.

11. Ferrocement sections with a wide range and combinations of mesh geometry and/or skeletal bars have very satisfactory engineering properties that make them highly suitable for a variety of structural applications beyond those of roofing and cladding.

CONVERSION FACTORS

1 lb. = 0.4536 kg
1 in. = 25.4mm
1 lbf/in'. = 6895 Pa
1 ft.lb. = 1360.5 Nmm

REFERENCES


9. Swamy, R. N. and Hussin, M. W., Flexural behavior of thin fiber reinforced and ferrocement sheets, (This Symposium).

Table 1 Test details.

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<th>Ultimate strength (N/mm²)</th>
<th>Ultimate strain (millistrain)</th>
<th>Modulus of Resilience (N/mm²)</th>
<th>% Reduction of area (q)</th>
<th>Fracture Toughness (N/mm²)</th>
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† Stress corresponding to strain of 5 millistrain

* Galvanized square mild steel welded mesh

** Square high tensile steel woven mesh

x Square mild steel welded mesh
Table 3 Composite first crack strengths, composite ultimate strengths and strains

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Table 4 Experimental and theoretical values of composite modulus of elasticity

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$\text{Av} = 0.956$
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Table 5  Composite ultimate strength and reinforcement strength

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* Average of three specimens.

\[ Av = 1.003 \]
\[ S.D. = 0.0734 \]
Table 7  Energy absorption and toughness

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<td>42164</td>
<td>664.51</td>
</tr>
</tbody>
</table>

* L-E  Load elongation  
F.C.L.  First crack load  
* U.L.  Ultimate load
Fig. 1--Test geometry and specimen under test
Fig. 1 (cont'd)—Test geometry and specimen under test
Fig. 2—Load-composite strain behaviour of ferrocement sheets reinforced with mild steel mesh and skeletal bars (series A)
Fig. 3--Load-strain behaviour of ferrocement sheets reinforced with mild steel mesh only (series D)
Fig. 4—Load-composite strain characteristics of ferrocement plates reinforced with high tensile steel mesh (series F)
Fig. 5--Load-composite strain behaviour of plates reinforced with mild steel skeletal bars alone (series E)

<table>
<thead>
<tr>
<th>PLATE</th>
<th>$V_R$ %</th>
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<tbody>
<tr>
<td>S1E</td>
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</tr>
<tr>
<td>S2E</td>
<td>0.78</td>
</tr>
<tr>
<td>S3E</td>
<td>1.02</td>
</tr>
<tr>
<td>S4E</td>
<td>1.57</td>
</tr>
<tr>
<td>S5E</td>
<td>1.88</td>
</tr>
<tr>
<td>S6E</td>
<td>2.67</td>
</tr>
</tbody>
</table>
Fig. 6—Matrix and reinforcement contributions to composite behaviour
Fig. 7--Relation between composite first crack stress and specific surface of reinforcement.
Fig. 8--Relation between composite first crack stress and volume fraction of reinforcement
Fig. 9--Relation between composite ultimate strengths and tensile strength of reinforcement
Fig. 10--Relation between composite tensile strength and its flexural strength
Fig. 11--Influence of specific surface on final crack spacing
Fig. 12--Cracking behaviour of ferrocement sheets in tension
Investigation of Precast Ferrocement Planks Connected by Steel Bolts

by T.S. Krishnamoorthy, V.S. Parameswaran, M. Neelamegam, and K. Balasubramanian

Synopsis: Precast thin ferrocement planks have replaced wood for a variety of applications. Present knowledge about joining them using steel bolts or similar means is very limited. While bolted connections are commonly employed in steel construction, their suitability for connecting precast reinforced concrete or ferrocement elements is yet to be fully investigated, particularly when subjected to both bending and direct tension. A series of tests were carried out at the Structural Engineering Research Centre, Madras, India, on precast ferrocement planks connected together using steel bolts for transferring tension and flexural moment across the joints. In the first series of experiments the efficiency of lap and butt types of joints using steel bolts was investigated. The variables included diameter, location and spacing of the bolts, thickness of the planks, and the cross linking of steel bolts by additional steel plates across the joints.

Keywords: bolted connections; bolts; connections; failure; ferrocement; joints (junctions); lap connections; panels; pressure; strains; structural design; tensile strength; tests; thickness
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INTRODUCTION

Precast ferrocement planks are commonly employed in the construction of water tanks, boundary walls, kiosks, and floors and roofs for low-cost houses. It is easy to fabricate them and the process of manufacturing them is also quite simple and less expensive. They can be produced in the form of long strips and cut to convenient lengths depending on the actual usage. The strips or planks can then be joined together by suitable means to form the final structure. At present there is not much information available on connections using steel bolts. The simplest type of connection is the lap joint in which one strip is placed over the other with an overlap. In the other type the two strips butt against each other and the connection is made by means of a pair of steel plates on either side of the elements. Behaviour of both was studied.
Failure of a joint may be due to any one or combination of the following causes:

-the tensile stress in the reinforcing steel may exceed its ultimate failure stress resulting in tension failure of the strips;

-the shear stress in the steel bolt may exceed its ultimate shear strength resulting in shear failure;

-the pressure exerted by the steel bolt on the contact area surrounding the bolt hole may cause local crushing and tearing of the strip around the hole;

While failure due to the first cause will mean achieving the full strength of the joint, the other two types of failures do not reflect this. Efforts were taken during the investigation to avoid totally failure of the joints by the shearing of the bolts. The third type of failure (crushing of concrete) could not be avoided totally in the experiments, but the parameters causing such a failure were, however, studied.

EXPERIMENTAL INVESTIGATIONS

Ferrocement test specimens of the shape shown in Fig 1(a) & (b) and having a thickness of 15 mm or 20 mm were cast using wooden moulds. While the specimen shown in Fig 1(a) is a full length, unjoined piece, the one shown in Fig 1(b) consists of two identical pieces which were used for evaluating the performance of lap and butt joints. Figs 2(a) & (b) give the details of the lap and butt joints which were studied during the investigations. The bolt holes in the test specimens were made in the conventional manner using a hand-operated electric drill.

The details of reinforcement provided in the specimens are given in Figs 3(a) & (b). The mix proportions used for casting the test specimens and specifications of the mortar and welded and chicken meshes (which were kept the same in all the specimens) are given below:
Cement : Fine sand - 1:2 by weight;
Water cement ratio - 0.42
Average compressive strength - 40 N/square mm
of cement mortar
(measured using 70.7 mm x 70.7 mm x 70.7 mm cubes)
Welded mesh: Grid size - 50 mm x 50 mm
Diameter - 4.0 mm
Ultimate strength - 800 N/square mm
Chicken mesh - Hexagonal
Thickness - 24 gauge;
ultimate strength - 160 N/square mm

Details of the specially fabricated testing frame used for applying the load on the test specimens are given in Fig 4. Two hydraulic jacks of 50 kN capacity symmetrically placed on the top platen of the test frame helped in applying the tensile load on the test specimen. A third jack was used for calibrating and measuring purposes. A proving ring of 20 kN capacity was used in conjunction with this jack.

Strain gauge locations were marked on the specimens and steel pellets were pasted on either side of specimens at these places to measure the strains during the application of load. The load was applied on the specimen gradually and the tensile strains were measured using a 'Pfender' mechanical strain gauge having a least-count of 0.001 mm.

DISCUSSION OF TEST RESULTS

Twenty four specimens were tested. Five were full-length, straight specimens cast without any joint. Two thicknesses were used 15 mm and 20 mm.

Twelve specimens were tested using a lap joint between the two precast strips. The thickness of the planks was varied. The joints were made using either a single bolt or two bolts arranged in a vertical row. Steel side plates of 5 mm thickness were used in some of the specimens to connect the two bolts on either side of the joint. The diameter of the steel bolt as well as the lap length were also varied to study their influence on the behaviour of the joint.
Seven specimens were tested using the butt joint. The joint was achieved by providing two 5 mm thick end plates on either side of the plank across the joint. The thickness of the planks was 15 mm in the case of three specimens and 20 mm in the rest. The diameter of the bolt used was either 10 mm or 12 mm. The number of bolts used was two or four.

All the twenty-four specimens were tested to find out their ultimate load-carrying capacity under direct tension. The results of the tests carried out on unjoined, full-length straight specimens and on specimens connected by steel bolts using either a lap or a butt joint are given in Tables 1, 2 and 3, respectively. Since the same reinforcement as given in Fig 1 (b) was provided in both 15 mm and 20 mm thick test specimens, the ultimate load-carrying capacity of these specimens, when they do not incorporate any joint, will be more or less the same, if it is assumed that the specimen fail by fracture of the mesh reinforcing steel. This was exactly the case in the case of full length straight specimens in which the failure was noticed primarily by the fracture of steel, simultaneously followed by crushing of concrete near the bolt holes at the two gripping ends. The average ultimate failure load of both 15 mm thick and 20 mm thick specimens was found to be 19 kN. The results of the tests are given in Table 1.

The results of tests carried out on the twelve specimens using the lap joint with and without the side connecting plates between the bolts are shown in Table 2. In many specimens, the edge distance was kept at 25 mm and the lap length only was varied. It may be seen from this table that the specimens with a lap length of only 60 mm failed at a load of 4 kN when only one bolt was used. When the lap length was increased to 100 mm and two bolts were used, there was noticeable improvement in the performance of the joint. The performance of the specimen improved very much further for the same lap length, when two bolts with 5 mm thick steel connecting plates were used. The provision of the side connecting plates helped to achieve an ultimate tensile load varying from 16 kN to 18.5 kN which is almost equal to the load-carrying capacity of the unjoined, full length specimen. Further increase of the lap length to 125 mm did not result in any further improvement in the load-carrying capacity.
It may be concluded that a lap length of 100 mm with two bolts and side steel plates on either side of the joint can be expected to result in a very efficient transfer of load and a small amount of moment (caused due to eccentricity of load at the joint) through the joint. A lap length equal to six times the thickness of the ferrocement element is recommended for the design purposes. It is also seen from Table 2 that increasing the diameter of the steel bolt also results in an increase in the load-carrying capacity of the joint. This is because the contact pressure exerted by the bolts on the surrounding ferrocement mortar at the time of loading gets distributed over a large area when large diameter bolts are used. Further tests are contemplated to arrive at a definite recommendation on the size of the bolts to be used for such lap joints. The edge distance of 25 mm provided for the end bolts is also found to be adequate for effective transfer of load across the joint when steel side plates are used. An edge distance equal to two and a half times the diameter of the bolts used may, therefore, be used for general design purposes in such cases.

The results of tests carried out on specimens which were connected by butt joints are given in Table 3. It may be seen that the load-carrying capacity is very much less when the edge distance to the bolts is kept small. Significant improvement is achieved when this distance is increased. Here again, the efficiency further improves when the diameter and number of bolts are increased along with the increase in the edge distance. An edge distance equal to six times the diameter of the bolt used is recommended for design purposes.

The tensile strains measured on the surface of the lap jointed and butt jointed test assemblies using the Pfender mechanical strain-measuring gauge are plotted against applied tensile stress and shown in Figs 5 & 6, respectively. Since the strain measurements were taken at regions close to load application or transfer, they do not truly represent the average, uniform strains one would have normally expected to get during a standard tension test. They are, however, useful in comparing the relative behaviour of the two types of joints taken for the present study.
The strain values indicate that the butt-jointed specimens show a uniform and linear variation of strain with the applied load as compared to specimens provided with lap joints. In the case of the latter, the small bending moment caused due to the eccentric pull across the section at the joint not only contributed to variation of the tensile strains on the two opposite faces of the precast planks, but also to the increase in the tensile strains in the specimens under a specified applied load, as compared to strains suffered by butt jointed specimens for the same load. The difference found in the slopes of the stress-strain diagrams may also be attributed to this fact. Fig 7. shows some of the tested specimens after their failure.

CONCLUSIONS

The paper presents the results of a limited number of tests carried out on thin precast ferrocement elements connected together by means of steel bolts and subjected to direct tension. Two types of joints were used, namely, lap and butt-types. From the tests reported in this paper, the following conclusions may be drawn:

-It is possible to use steel bolts for connecting thin precast ferrocement elements for the effective transfer of load and moment across the joint.

-Both lap and butt-types of joints can be used for achieving the connection.

-Use of a single bolt in a lap joint is not very effective in the transfer of load across the joint. When the edge distance is as small as 25 mm, a minimum of two bolts is recommended. Provision of a steel end plate connecting the bolts is found to increase the load-carrying capacity of the joint to a value equal to one that is obtainable in the case of an unjointed full-length element.

-In the case of a lap joint, a minimum lap length equal to six times the thickness of the precast element is recommended when the edge of the first bolt from the edge of the precast element is 25 mm.
- In the case of a butt joint, an edge distance equal to at least six times the diameter of the bolt used is recommended.

- While large diameter steel bolts are found to increase the contact bearing area and thereby minimise the possibility of cracking and tearing of the mortar around the bolt holes, it has not been possible to make any specific recommendations regarding their size vis-a-vis the thickness of the precast elements from the limited tests carried out so far.

Further investigations on the behaviour of precast ferrocement elements connected by steel bolts and subjected to combined loads are in progress with a view to formulating specific criteria for the design of joints.

ACKNOWLEDGEMENT

The authors wish to express their appreciation for the assistance and cooperation rendered by Mr. A.B. Narambu, Graduate Trainee, and the technical staff of the Concrete Composites Laboratory, SERC, Madras. The paper is being published with the kind permission of the Director, Structural Engineering Research Centre, Madras, India.
### TABLE 1: Results of Tests on Full Length, Straight Ferrocement Planks

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Thickness (mm)</th>
<th>Failure load (kN)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>15</td>
<td>20.0</td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>15</td>
<td>18.0</td>
<td>Failure by yielding of steel followed by cracking near bolt hole</td>
</tr>
<tr>
<td>S3</td>
<td>20</td>
<td>17.5</td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td>20</td>
<td>20.0</td>
<td></td>
</tr>
<tr>
<td>S5</td>
<td>20</td>
<td>20.0</td>
<td></td>
</tr>
</tbody>
</table>

### TABLE 2: Results of Tests on Specimens Connected by Lap Joints

*Identified by the alphabet, thickness (mm), diameter of steel bolt (mm), and the total number of bolts across the joint, respectively*

<table>
<thead>
<tr>
<th>Description of specimen length</th>
<th>Lap length (mm)</th>
<th>Edge distance (mm)</th>
<th>Size of 5mm thick MS side plate (mm x mm)</th>
<th>Ultimate load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1/15/10/1</td>
<td>100</td>
<td>50</td>
<td>...</td>
<td>8.0</td>
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<tr>
<td>L2/15/10/1</td>
<td>60</td>
<td>30</td>
<td>...</td>
<td>5.0</td>
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<td>L3/15/10/2</td>
<td>100</td>
<td>25</td>
<td>100 x 50</td>
<td>16.0</td>
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<tr>
<td>L4/15/10/2</td>
<td>100</td>
<td>25</td>
<td>100 x 50</td>
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<td>30</td>
<td>60 x 50</td>
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<td>60 x 50</td>
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<td>100</td>
<td>25</td>
<td>...</td>
<td>10.0</td>
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<tr>
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<td>25</td>
<td>100 x 50</td>
<td>18.0</td>
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<tr>
<td>L10/20/12/2</td>
<td>100</td>
<td>25</td>
<td>100 x 50</td>
<td>18.5</td>
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<td>125</td>
<td>25</td>
<td>125 x 50</td>
<td>17.8</td>
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<tr>
<td>L12/20/12/2</td>
<td>125</td>
<td>25</td>
<td>125 x 50</td>
<td>17.0</td>
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<tr>
<td>Description of specimen</td>
<td>Edge distance of specimen (D)</td>
<td>c/c of bolt (D1)</td>
<td>Size of MS plate (mm x mm)</td>
<td>Ultimate load (kN)</td>
</tr>
<tr>
<td>-------------------------</td>
<td>-----------------------------</td>
<td>-----------------</td>
<td>---------------------------</td>
<td>------------------</td>
</tr>
<tr>
<td>B1/15/10/2</td>
<td>65</td>
<td>130</td>
<td>180 x 50</td>
<td>12.0</td>
</tr>
<tr>
<td>B2/15/10/4</td>
<td>45</td>
<td>90</td>
<td>140 x 50</td>
<td>10.0</td>
</tr>
<tr>
<td>B3/15/12/2</td>
<td>65</td>
<td>130</td>
<td>180 x 50</td>
<td>18.0</td>
</tr>
<tr>
<td>B4/20/10/2</td>
<td>25</td>
<td>50</td>
<td>100 x 50</td>
<td>6.0</td>
</tr>
<tr>
<td>B5/20/10/2</td>
<td>45</td>
<td>90</td>
<td>140 x 50</td>
<td>12.0</td>
</tr>
<tr>
<td>B6/20/10/2</td>
<td>65</td>
<td>130</td>
<td>180 x 50</td>
<td>14.0</td>
</tr>
<tr>
<td>B7/20/12/2</td>
<td>45</td>
<td>90</td>
<td>140 x 50</td>
<td>13.0</td>
</tr>
</tbody>
</table>

* Identified by the alphabet, thickness (mm), diameter of steel bolt (mm), and the total number of bolts across the joint, respectively.
NOTE:
ALL DIMENSIONS
ARE IN mm

FIG. 1 DETAILS OF TEST SPECIMEN
FIG. 2 DETAILS OF CONNECTION

LEGEND

- $t_f$: Thickness of the Plank
- $l_p$: Lap Length
- $d_1$: Edge Distance
- $d_2$: C/C Distance of Bolts

(a) LAP JOINT

(b) BUTT JOINT
FIG. 3 DETAILS OF REINFORCEMENT

(a) WELD MESH 50x50x4
CHICKEN MESH 24 gauge

(b) WELD MESH 50x50x4
CHICKEN MESH 24 gauge

NOTE: ALL DIMENSIONS ARE IN mm

FIG. 4 DIRECT TENSION TEST FRAME

LEGEND
1. FERROCEMENT SPECIMEN
2. HOLDER
3. TIE ROD
4. ST. JACK
5. PRESSURE HOSE
6. PUMPING UNIT
7. PROVING RING
8. TEST FRAME
9. PRESSURE DISTRIBUTOR
FIG. 5 STRESS VS AVERAGE TENSILE STRAIN (LAP-JOINTS)

FIG. 6 STRESS VS AVERAGE TENSILE STRAIN (BUTT-JOINTS)
FIG.7 FAILURE PATTERN OF THE TESTED SPECIMEN
Behavior of Thin Sheet FRC Under Impact Loading

by A. Bentur, S. Mindess, and C. Yan

Synopsis: Thin section FRC panels may be subjected to localized impact. In this study, thin sheet FRC materials, made with asbestos fibres in different matrices, were tested under impact loading, using a drop-weight instrumented impact machine. The impact properties were characterized in terms of the peak bending load, and the fracture energy (computed as the area under the load-deflection curve). Companion specimens were tested under static loading. The specimen dimensions were about 200 mm wide, 600 mm long, and 6-12.7 mm thick.

In all cases, the peak bending loads were considerably higher under impact loading than under static loading; however, the fracture energies were always higher under static loading. These effects can be explained in terms of the porosity of the interfacial matrix, and the degree of bundle separation of the asbestos fibres.

Keywords: asbestos; asbestos cement products; cements; failure mechanisms; fiberboard; fibers; impact strength; impact tests; load-deflection curve; reinforcing materials
A. Bentur, Associate Professor in the Department of Civil Engineering, Building Research Station, Technion, Haifa, Israel, is a member of ACI 544 - Fiber Reinforced Concrete. He is also active in RILEM. His research interests include the mechanical properties and durability of fibre reinforced cements and concretes.

S. Mindess, Professor of Civil Engineering at the University of British Columbia, Vancouver, Canada, is a member of ACI 446 - Fracture Mechanics of Concrete, and is active in RILEM and ASTM. His research interests include fibre reinforced concrete, impact testing, and fracture mechanics.

C. Yan, Graduate student, Department of Civil Engineering at the University of British Columbia, Vancouver, Canada, is a doctoral candidate, studying the effect of dynamic loading on the bond between concrete and reinforcing bars.

INTRODUCTION

Thin sheet fibre reinforced cement composites are widely used as panels and roofing materials. They thus may be subjected not only to static loading, but also to impact loading. However, while the behaviour under static loading of such sheet materials is fairly well known (1), their behaviour under impact has not been studied in any systematic way, not least because no standard methods exist for impact testing of such materials.

In the present study, asbestos cement thin sheet materials were tested, using an instrumented drop-weight impact machine. The impact behaviour was characterized in terms of the maximum impact loads and the fracture energies. Companion specimens were tested under static loading for comparison purposes.

It is true that asbestos-cement building materials are being phased out because of the health hazards associated with asbestos. However, it is important that proper base-line data be acquired for asbestos-cement materials, so that substitutes for asbestos in thin sheet fibre reinforced cements can be properly assessed.

EXPERIMENTAL PROCEDURES

Specimens

Four different types of thin sheet specimens were tested. The specimen designations and characteristics are given in Table 1. They were all commercially prepared. The specimens were of varying thicknesses; the other specimen dimensions were a length of 600 mm and a width of 200 mm.
Test Method

Impact tests were carried out using an instrumented drop-weight impact machine, designed and constructed at the University of British Columbia. This impact machine, and the method of analysis, have been described in detail elsewhere (2-4). Briefly, the machine is capable of dropping a 345 kg mass impact hammer from heights of up to 2.4 m. For the tests reported here, a drop height of 0.2 m was used, giving a measured impact velocity of about 1.86 m/s. Strain gauges mounted in the striking end of the impact hammer (the "tup") are used to record the contact load between the drop hammer and the beam. An accelerometer mounted near the centre of the specimen was used to monitor the response of the flat sheets themselves to the impact load. A data acquisition system was used to record the tup load and accelerometer readings simultaneously at 200 microsecond intervals. At this rate, the secondary oscillations on the load and acceleration traces will not be recorded.

In order to analyze impact tests of this type, it is necessary to separate the inertial load (i.e. the load required to accelerate the specimen from rest) from the total load measured by the instrumented tup. Assuming that the displacements and accelerations along the length of the specimen are linearly distributed (3), the distributed inertial forces can be replaced by a generalized inertial load acting at the centre of the sheet,

\[ P_1(t) = \rho A \ddot{u}_c(t) \left[ \frac{l}{3} + \frac{8}{3}(h^3/l^2) \right] \]  

where

- \( P_1(t) \) = generalized inertial load
- \( \rho \) = mass density of the specimen
- \( A \) = cross-sectional area of the specimen
- \( \ddot{u}_c(t) \) = acceleration at centre
- \( l \) = span of the total specimen
- \( h \) = length of specimen overhanging the supports

Then, the bending load acting on the sheet can be evaluated from

\[ P_b(t) = P_t(t) - P_1(t) \]
where
\[ P_b(t) = \text{generalized bending load} \]
\[ P_t(t) = \text{observed tup load} \]
\[ P_i(t) = \text{generalized inertial load from Eq. (1)} \]

Two successive integrations of the midspan acceleration with respect to time yield the midspan deflection
\[ u_0(t) = \int_0^t \ddot{u}_0(t) \, dt \tag{3} \]

where \( u_0(t) \) is the deflection, and \( \dot{u}_0(t) \) is the velocity.

It must be noted here that Equations 1 and 2 are strictly valid only for the elastic analysis of beams. Clearly, by the time the peak load has been reached, and certainly in the post-peak region of the curve, the FRC composites are significantly cracked. In addition, with this particular geometry, the specimens act more as a plate than as a beam. Thus, the analysis presented here can only given an approximation of the true fracture energy. Nonetheless, it is felt that it can provide useful comparative data amongst different FRC composites of the same geometry.

It would probably be preferable to measure \( P_b(t) \) directly, though this is difficult in practice. However, it has been shown (2) that obtaining \( P_b(t) \) through an instrumented support anvil does not give significantly different values from those computed from Eq. (2).

From these data, the bending load vs. deflection relationships can be determined. The fracture energy of the specimen was taken to be equal to the area under the \( P_b(t) \) vs \( u_0(t) \) curves, carried out to the point at which the load fell back to 1/3 of the peak load. The value of 1/3 was chosen arbitrarily, simply as a convenient way of comparing different materials. Typical time plots of \( P_t, P_i \) and \( P_b \) are given in Fig. 1, for specimen Ca-2.

Companion specimens were also tested in conventional static 3-point bending, using an Instron testing machine. The rate of loading for these specimens was such that failure occurred in about 3 minutes. For the impact specimens, the peak load occurred within about 5 ms, with failure occurring in less than about 20 ms.
EXPERIMENTAL RESULTS

The peak bending loads, fracture energies, and deflections at "failure" (i.e. at the point at which the load had fallen back to 1/3 of the peak load) are given in Table 2. Though only a limited number of specimens were available for testing, the results were quite consistent. From Table 2, it may be seen that, in all cases, the peak loads under impact loading were higher than those under static loading. However, both the fracture energies and the deflections at failure were lower under impact loading than under static loading.

To supplement the mechanical measurements, the fracture surfaces of the specimens broken in impact were examined in an SEM, and the following observations were made:

Specimen Ca-1: The fracture surface revealed pulled-out asbestos bundles (Fig. 2a). The matrix around the root of the pulled-out bundles was granular and showed some cracking (Fig. 2b). The bundle showed a considerable degree of separation into very fine filaments. This could be seen at the root of the pulled-out bundle (Fig. 2b) and became much more marked at the end of the bundle (Fig. 2c).

Specimen Ca-2: The mode of failure was pull-out of a fibre bundle (Fig. 3a). The matrix around the root of the bundle was very dense (Figs. 3a; 3b), and this is also evident in the zone around the "hole" left after the bundle had been pulled out (Fig. 3c). The matrix is considerably denser than that of specimen Ca-1. There is some filamentization of the bundle, and separation into smaller units (Figs. 3b; 3d). However, this separation does not appear to be as fine as in specimen Ca-1 (e.g., compare Fig. 2c with Figs. 3b; 3d). In specimen Ca-2, the edge of the bundle is much better defined (Figs. 3a; 3d).

Specimen Ca-3: Here too, the main mode of fracture is pull-out of a fibre bundle (Fig. 4a). There is very little contact between the fibre bundle and the surrounding matrix, and the matrix around the fibres is very porous (Fig. 4b). Within the bundle, there is separation into smaller filament units (Fig. 4b), similar to that in specimen Ca-1.

Specimen Ca-4: The mode of failure in this specimen was also dominated by pull-out of the fibre bundles (Fig. 5a). The matrix at the root of the bundle was very porous (Fig. 5b). There was only a small degree of separation of the bundle into filaments throughout its length (Figs. 5a; 5b). The bundles appeared to be much more closely held together than in specimen Ca-1. Of particular interest is the end of the pulled-out bundle, which seemed to be very clearly defined, and cut in a plane almost perpendicular to the longitudinal axis (Fig. 5a), with almost no separation into individual filaments (Fig. 5c). This is quite different from the frayed ends of the bundles in specimen Ca-1 (compare with Fig. 2c).
DISCUSSION OF RESULTS

The increase in strength under impact loading compared to static loading is consistent with the well-known strain rate sensitivity of cementitious materials, whose apparent strengths increase with increases in loading rate (5). This does not, however, explain why the fracture energy decreased with an increase in loading rate. For concretes reinforced with steel or polypropylene fibres, it is generally found (2) that fracture energies increase with increasing strain rates. The results obtained here can only be explained by a change in the mode of failure in impact as compared to static loading. From the SEM observations, it is clear that all of the specimens failed by pull-out of the fibre bundles from the matrix. The differences in the densities of the matrices were also evident in the micrographs.

Strength

High strengths appeared to be associated with higher densities of the matrix around the fibre bundles. The SEM micrographs indicate a more compact interfacial matrix in Ca-2, which has the highest strength. The interface in Ca-1 is less compact, and the strength is also somewhat lower. The weakest material, Ca-4, also has the most porous matrix. Specimens Ca-3 and Ca-4 have similar porosities and similar impact strengths. However, there was hardly any filamentization of the fibre bundles in Ca-4, which may lead to less efficient reinforcement.

Fracture Energy

A higher fracture energy was associated with a greater degree of separation and filamentization of the fibre bundles. Ca-4, with very little filamentization, had a very low fracture energy, while Ca-1 and Ca-2, with the greatest degree of filamentization, had the highest fracture energies. While Ca-3 is similar in bundle separation to Ca-1, it is much lower in toughness; this is due to the more porous interfacial matrix.

CONCLUSIONS

1) All of the materials failed primarily by fibre pull-out under impact loading.

2) The differences in mechanical properties of the different materials appear to be a function of both the degree of bundle separation and the porosity of the interfacial matrix.

3) The strength appears to be more sensitive to the structure of the interfacial matrix, while the fracture energy is more sensitive to the degree of filament separation. The
degree of separation is probably a function of the production process.

ACKNOWLEDGEMENTS

This work was supported in part by a grant from the Natural Sciences and Engineering Research Council of Canada.

REFERENCES


### TABLE 1 - SPECIMEN DESIGNATIONS

<table>
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<tr>
<th>Thickness (mm)</th>
<th>Density (kg/m³)</th>
<th>Type of Material</th>
<th>Asbestos Content (% by weight)</th>
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<tr>
<td>Ca-1*</td>
<td>5.9</td>
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<td>Cement</td>
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<td>Ca-2*</td>
<td>6.0</td>
<td>1682</td>
<td>Cement</td>
</tr>
<tr>
<td>Ca-3*</td>
<td>5.6</td>
<td>801</td>
<td>Cement + perlite</td>
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<td>Ca-4**</td>
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<td>~800</td>
<td>Gypsum</td>
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Note: All materials were conventionally cured in air.

*Produced by Turners Building Products Ltd., Mission, B.C., Canada

**Source unknown.

### TABLE 2 - PROPERTIES OF THE THIN SHEET MATERIALS UNDER BOTH STATIC AND DYNAMIC LOADING

<table>
<thead>
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</tr>
<tr>
<td>Ca-4***</td>
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<td>0.55</td>
</tr>
</tbody>
</table>

*average of two specimens.

**average of 5 specimens, except that for Ca-4 only 3 specimens were tested.

***equivalent value, 1/4 of the measured values given in the line above, since the thickness of Ca-4 was about twice that of the other three specimens.
Fig. 1--Typical curves of $P_t$ vs $t$, $P_I$ vs $t$, and $P_b$ vs $t$, for specimen Ca-2
Fig. 2(a) -- General view, showing pulled-out bundles

Fig. 2(b) -- The root of a pulled-out bundle

Fig. 2 -- Fractured surface of Ca-1
Fig. 2(c)—Higher magnification within a bundle showing separation and filamentization

Fig. 2(cont'd)—Fractured surface of Ca-1
Fig. 3(a)--Pulled-out bundle and the dense matrix around it

Fig. 3(b)--High magnification of the root of a pulled-out bundle

Fig. 3--Fractured surface of Ca-2
Fig. 3(c)--The dense matrix around the "hole" left by a pulled-out bundle

Fig. 3(d)--The end of a bundle

Fig. 3(cont'd)--Fractured surface of Ca-2
Fig. 4(a)--Pulled-out bundle

Fig. 4(b)--The end of a bundle

Fig. 4--Fractured surface of Ca-3
Fig. 5(a)--Pulled-out bundle

Fig. 5(b)--The root of a pulled-out bundle surrounded by a porous matrix

Fig. 5--Fractured surface of Ca-4
Fig. 5(c)--The end of a bundle

Fig. 5(cont'd)--Fractured surface of Ca-4
Thin Sheet Glass and Synthetic Fabric Reinforced Concrete
60-120 Pound pcf Density

by M. Schupack

Synopsis: The background to the development of two types of thin, fabric reinforced, portland cement concrete sheets is described and range of properties given. Both normal weight and lightweight mortars (including cellular mortars) were used as a matrix. Glass or synthetic fiber continuous reinforcement in the form of fabric scrims and/or non-woven three dimensional fabric were used. The materials developed are potential substitutes for plywood, cement asbestos and other type sheet material which requires the properties of weather resistance, incombustibility, non-biodegradability and economy. The test results also suggest that the matrix and reinforcement concepts developed will lead to applications in other reinforced concrete uses. The thin sheet materials lend themselves to easy manufacture in a comparatively simple plant.

Keywords: cellular concretes; concretes; density (mass/volume); fabric; fibers; glass fibers; mortars (material); reinforced concrete; reinforcing materials; synthetic fibers
M. Schupack, of Schupack Suarez Engineers, a U. S. pioneer in prestressed concrete, has also developed fibrous concrete products and composites. He serves on numerous committees dealing with the promulgation of codes and practices including ACI Fibrous Concrete and is a past director of ACI and PCI.

INTRODUCTION

The development of two types of hydraulic cement mortar, fabric reinforced, thin sheet described herein originated from a proprietary development project. The task was to develop an inexpensive durable sheet material that could be manufactured in a flat position and then be "cold" bent into a two foot diameter cylindrical drum (Fig. 1). After the writer developed this sheet material successfully it was realized that this technology had other possible uses as a substitute for plywood, cement asbestos, and other types of waterproof sheets. Several versions of lightweight, flexible, durable and relatively inexpensive sheet material evolved from the original development.

The first generation sheets were reinforced with surface scrim (a scrim is a two-dimensional woven or non-woven fabric) of coated fiberglass or alkali resistant synthetic yarn encapsulated in a low water-cement ratio, cellular, portland cement mortar matrix. The mortar matrix density could range from 50 to 120 lbs. per cubic ft., depending on the required end use.

The second generation development was a more ductile and tougher material using a reinforcing means in the form of three-dimensional non-woven fabric, similar to a geotextile filter fabric. It was found that a non-woven three dimensional fabric could be impregnated with a special cellular mix to produce an almost ductile material. This portland cement based material can have densities in the range of 50 to 120 pcf. The material can be cut with hand tools, can be nailed close to the edge without predrilling (3/8" edge distance for a 1/4" thick sheet) and not show significant cracks, and is particularly tough under impact.
FIRST GENERATION FABRIC CONCRETE - STRESSED SKIN

Design Requirements

In order to provide ductility, durability, relatively low cost and easy workability, the following points had to be considered:

1. The most efficient use of the most expensive component of the composite - the fabric scrim. Obviously this is achieved by placing the scrim at the extreme fiber of the sheet thus achieving a stressed skin construction.

2. Obtaining a scrim that would be durable in the mortar matrix and tolerate the alkalinity of portland cement as well as in the environment in which it is to be used.

3. The combination of reinforcing means and encapsulating matrix that would provide required strength, ductility and adequate bond to the reinforcing fabric.

4. Provide easy workability in the field, preferably with usual hand and power tools.

5. Be reasonably dimensionally stable.


7. Be non-biodegradable.

8. Be manufactured with minimum of labor and relatively low plant cost.

9. Capable of being surface textured.

Concept Development

It was found that, by placing the scrims with practically zero mortar cover, in effect achieving a stressed skin construction, the most structurally efficient use of the relatively expensive reinforcing means could be achieved. This was achieved even with lower modulus of elasticity yarns in the 1 to 2 million psi range. Utilizing a stressed skin construction with practically zero cover to the reinforcing means requires deterioration-resistant fabric. The fabric material had to be resistant to water, high alkalinity,
ultraviolet, and other exposures in the environment in which the sheet is to be used. Since corrosion does not have to be a concern as with steel reinforcement, the matrix to be used could be pervious to an acceptable extent. Penetration of chlorides and presence of water and oxygen is not a concern in regards to the durability of the reinforcing means. Since reinforcing steel corrosion is not a problem, a more pervious concrete matrix as provided by low water-cement ratio, water-retentive, portland cement cellular mortar matrix could be used in the density range of 50 to 120 pcf. This matrix encapsulates the tension providing reinforcing means and provides the flexural compression block of the composite.

To increase toughness and edge strength as needed, the introduction of select fine synthetic discrete fibers at relatively small volume percentage - up to about 1% - can be used. This also enhances the nailability at closer edge distance.

Unlike plywood, asbestos cement, and other common sheet material, it is possible to design this type of sheet material to suit the end need. The practical problem of manufacturing standard sheets, which can be so easily varied, creates a general planning problem. Therefore, in the utilization of the concept, several standard sheets would be developed to suit the more common needs.

The choice of reinforcement in the form of textile type fabric is extensive when compared with the limited choice of steel. Besides difference in strength, moduli of different yarn in the fabric can range from about 0.5 to 20 million psi. Also, the configuration and spacing of the yarn in the fabric have a significant effect on performance. All these factors, including bond of the fabric, have to be considered in the choice of fabric. For instance, if energy absorption is the major consideration, a lower modulus material is preferred. If, say, a 1/4" sheet of about 90 pcf density is to be bent into a 12" radius and still be able to control cracking, a modulus of about 2 to 5 million psi and relatively close spacing of yarn is desirable. For a stiff high strength sheet using a matrix of 120 pcf density, a high modulus of 8 million psi or over is preferable.

In our development work we found that it was essential to assure that the matrix bonded the top reinforcing scrim as cast. If any bleed water developed at the level of the top scrim, the bond was compromised to some degree. This was controlled by chemical means as described in Reference 1.

Obviously, because of the different types of fabric reinforcement available in the market today (and potentially, the more ideal fabrics which may become available from the
textile industry in the future), the selection becomes a major research effort. This is even more complicated by the wide range of choice of hydraulic cement matrixes which, usually for thin sheets, makes up about 15% of the material cost.

Physical Properties

Some of the typical properties of the stressed skin sheet are shown in Fig. 2. It can be seen that a wide range of properties are achievable within a wide range of matrix densities. The densities which exceed about 100 lbs. per cubic ft., are not as flexible and should be predrilled for attachments. To give a basic understanding of the behavior of the first generation sheet material, Fig. 3 shows a typical deflection curve of an 83 pcf density material utilizing a coated glass scrim. The crack pattern for a polyester scrim and a 70 pcf matrix density is shown in Fig. 4.

SECOND GENERATION FABRIC CONCRETE - THREE DIMENSIONAL CONTINUOUS REINFORCEMENT

Concept Development

To achieve a tougher material than the composites described above, another composite was developed using a similar first generation matrix technology with a three dimensional (3-D) fabric for reinforcement (2). In the development work, a polyester non-woven three dimensional fabric was used successfully. It had to be "filled", without separating out the water from the mortar, because of the basic filtering action of the non-woven filter fabric. This was achieved by chemical means, similar to those developed by the author in obtaining water retentive grout for post-tensioning tendons (Reference 3).

Three-Dimensional Reinforcing

The 3-D reinforcing is made up of continuous fibers distributed in a random pattern in the x, y and z directions. Various geotextile filter clothes have this construction. The 3-D reinforcing should preferably have a more open structure to permit easier penetration of the matrix. Naturally the size of the sand particles that can be used depends on the openness of the non-woven fabric. It has been found that a non-woven fabric that supplies about 1 to 3% of fibers by
volume provides a very tough system. The non-woven fabric itself, in these volumes, will provide over 1000 psi flexural strength with a cellular mortar matrix density of 80 pcf. This is substantially superior to discrete polypropylene fibers used at this general volume ratio in an 80 pcf cellular concrete. The writer's unreported work using chopped polypropylene and P.V.A. fibers up to 1.5% by volume in cellular 80 pcf density mortar matrix achieved flexural strengths of under 500 psi. Besides the higher tensile strengths achieved, even with lightweight matrix, the flexural behavior is ductile and the crack distribution excellent. In order to achieve greater flexural strength, scrim can be attached to the non-woven fabric. Strengths up to 5000-6000 psi in flexure with density in the order of 90 pcf can be achieved with this combination. The writer is not aware of any hydraulic cement cellular concrete composite using chopped fibers that can achieve these flexural strengths.

**Typical Physical Properties**

The physical properties of an 80 pcf cellular mortar matrix, reinforced with polyester non-woven fabric (11.1 oz. per sq. yd.) can be seen in Fig. 5. The actual ultimate strength extends beyond the curve plotted but was not determined because of the large deflections. A 95 pcf density matrix reinforced only with a similar non-woven fabric achieved a maximum stress of 1479 ppsi when the test was discontinued because of large deflections. The curve had similar shape and similar ductility and an approximate visual cracking stress of about 600 psi. In Fig. 6 note the well distributed crack pattern obtained in this 0.195" thick specimen. Properties as described above can not be achieved with discrete fibers in lightweight or normal weight mortars in the writer's experience, particularly when lower modulus fibers are used.

**Durability**

The general durability of the two generations of sheet material seems to be excellent. Exposure of 60 to 90 lb. density sheet material in a tidal estuary as shown in Fig. 7, subjecting these specimens to 40 to 50 freeze-thaw cycles a year, has indicated no significant deterioration of the panel after 2 years. The tidal estuary that was used as an exposure station has a salinity slightly less that of the ocean. Some specimens were actually placed into the highly organic marsh, others were nailed to a rack and all were inundated twice a day by the tides. The hydrogen sulfide derived from the
organics of the marsh are often noticeable by smell. In Fig.
8, the first generation specimens that were exposed on the
side of a building in Norwalk, Connecticut, subjected to snow,
rain, and frost, have indicated no deterioration after 5
years.

Some of these test specimens were tested and cracked
previous to exposure. Also the fibers used, were supposedly
attacked by the alkali in portland cement. Considering the
above two points, the exposure test we performed should
establish a level of confidence in the use of low water-cement
cellular portland cement mortar in combination with
"non-corroding" synthetic fibers. A possible explanation of
why there is no apparent alkali attack of the polyester fiber
is that the thin cellular mortar sheets may rapidly
carbonate, thus minimizing the time the polyester fibers are
exposed to the alkaline environment.

GENERAL POTENTIAL FOR THE USE OF THIN SHEET MATERIAL AND FOR
THE OVERALL CONCEPT OF STRESS SKIN CONSTRUCTION AND
THREE-DIMENSIONAL REINFORCEMENT

The materials described above have potential for use as
sheet material which can be a substitute for plywood, cement
asbestos and other water resisting sheet materials. It has
the following properties:
- Low cost.
- Will not burn.
- Density depending on end use - from 50 to 140 pcf.
- Flexural strength - up to 7,000 psi.
- Crack resistant.
- Can be fabricated to have high impact strength.
- Weatherproof, i.e., can be used in the tidal zone.
- Vermin and insect proof.
- Modulus of elasticity can be controlled.
- Can be bent to relatively small radii after being
  completely cured. (1/4" thickness to 12" radius)
- Can be designed to be nailed and worked with regular
  hand tools.
- Can have sustained flexural load capacity even at 50%
  of ultimate.
- Can be readily manufactured without complicated and
  expensive equipment.

The materials can be designed to be used as siding, siding
underlayment, shingles, sub-flooring, duct work, left-in-place
forms, applications where bendable sheets can be a
convenience, and other applications. Taking this concept
further, it is envisioned that fabric reinforcement can be
used to reinforce, in a stress skin manner, bridge decks and
pavements.
The second generation material which produces exceptional ductility requires only about 1 to 3% of fibers by volume compared to 4 to 6% of chopped glass fiber reinforced panels of similar toughness. Besides, in the second generation 3-D material, there is the additional advantage that the three dimensional reinforcement also permits the use of a cellular lightweight matrix which achieves excellent durability. It is the contention of the author that the use of continuous synthetic reinforcement has not been given adequate attention considering its vast potential. Since the problems of reinforcement corrosion seems to be plaguing numerous concrete structures, particularly where chlorides are available, it behooves us to consider the potential of this type of reinforcement and where it can be effectively used. There are certainly limitations in its use because of fire ratings although, basically, the composite is non-combustible.

CONCLUSIONS

The work we have done over a decade indicates that synthetic as well as glass continuous reinforcement are viable reinforcing means of regular and cellular concrete. Our work was limited to up to 2" thick specimens mostly using cellular concrete.

Fabric technology that will achieve the most efficient and durable fabric to produce fabric concrete is, theoretically available. Unfortunately, the yarn and textile industry has not had enough interest in these developments to see the vast potential. The cooperation between the chemical yarn industry and the textile weavers in conjunction with the concrete industry is needed. Our frustration in developing these methods and composite materials has been that we have not been able to obtain the fabric type and configuration of our choice. All our work has been done on fabric that was commercially available and thus our choice was always a compromise. With the great growth in the geotextile industry and further knowledge made available to architects and engineers on the potential of using alternate means of reinforcing concrete, both regular and lightweight, it is our hope that the textile industry will be motivated to supply these needs in the future.
References


Fig. 1--"Cold" bending 1/4" thick, 90 pcf density fabric concrete sheet after completely cured
<table>
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<tr>
<th>FABRIC</th>
<th>SHEET THICKNESS</th>
<th>MODULUS OF SHEET ELASTICITY 1,000 psi</th>
<th>MATRIX DENSITY pcf</th>
<th>STRESS AT VISUAL 1ST CRACK psi</th>
<th>FLEXURAL STRESS (BASED ON HOMOGENOUS SECTION AT) ULTIMATE psi</th>
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Fig. 2--Typical properties of fabric concrete (Neat cement cellular mortar)
STRESS - DEFLECTION CURVE

Flexure Test Panel No. 6C-37D - Load Control Machine
Cellular Mortar Matrix 83 pcf Thickness 0.84"
( Coated Glass Scrim Reinforced - Tensile Strength 400 lbs/inch )

FIGURE 3
Fig. 4--Crack pattern for polyester scrim in a 70 pcf matrix
STRESS - DEFLECTION CURVE

Flexure Test Panel No. PFF3-7 - Cellular Mortar Matrix 80 pcf
(Non-woven Polyester Fabric Reinforced)

FIGURE 5
Fig. 6—Crack pattern for a polyester non-woven three dimensional fabric in a 95 pcf matrix
Fig. 7--Exposure tests of fabric concrete specimens in Village Creek Tidal Estuary, Long Island Sound, Norwalk, CT. Specimens are shown at low tide. At high tide, specimens are covered by 2-3' of water.
Fig. 8--Coated glass scrim reinforced 1/4" thick panels - 90 pcf density—after five years exposure on the outside wall of a building in Norwalk, CT