Performance Criteria for Ferrocement

Antoine E. Naaman*

The increasing application of ferrocement as a structural material has created the need for specifications related to its usage similarly to those developed for reinforced concrete and steel structures. To satisfy this need, some guidelines and requirements related to specific applications of ferrocement, such as boat building, have been already published. The guidelines and criteria presented in this paper are more general in nature and should be applicable to most ferrocement structures; they summarize and update previously published work and present a new expanded section on design requirements and satisfactory performance. Design examples are developed.

INTRODUCTION

The increasing application of ferrocement as a structural material has created the need for specifications related to its usage similarly to those developed for reinforced concrete and steel structures. Such need has been already identified by the ferrocement boat industry as witnessed by some recently published requirements on the construction of ferrocement boats [1-3]. In a recent paper Shah [4] has provided tentative recommendations for the construction of ferrocement water tanks. Similar publications will simplify the construction of ferrocement structures and guarantee their safe performance. Also, they are of great value to professional users and practitioners as they provide a more rational basis for design.

Legally speaking, specifications must come from and be officially approved by various technical organizations and governmental agencies. As to the author's knowledge, a document containing such specifications will not be available in the near future, it seems that some more general guidelines, not necessarily related to a specific application of ferrocement, will represent a valuable first step toward such a goal.

Based on (1) the above mentioned requirements [1-4], (2) an extensive series of tests to investigate the mechanical properties of ferrocement [5-10], (3) a number of tests on ferrocement wall panels [11] (which included corrugated panels and sandwich panels) and ferrocement water tanks [12] where reinforcing parameters, load deformation response, cracking, leakage and cost characteristics were studied, (4) an extensive survey of ferrocement and reinforced concrete related literature where some forms of specifications or recommendations were found and analyzed [13-24], and (5) a survey of the applications of ferrocement in some typical structures [25] where reinforcing parameters and other variables were observed (Table 1), the author is presenting these guidelines and the corresponding design criteria for the construction of ferrocement structural members.

They are divided into four major sections, namely: Materials, Testing, Design, and Construction.

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Table 1. Ferrocement: Current Ranges of Composition and Properties

<table>
<thead>
<tr>
<th>Wire-Mesh Reinforcement</th>
<th>Wire Diameter:</th>
<th>0.020 ≤ ϕ ≤ 0.062 inches (0.5 ≤ ϕ ≤ 1.5 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of Mesh:</td>
<td></td>
<td>Chicken wire or square woven or welded wire galvanized mesh; expanded metal</td>
</tr>
<tr>
<td>Size of Mesh Openings:</td>
<td>1/4 ≤ m ≤ 1 inch (6 ≤ m ≤ 25 mm)</td>
<td></td>
</tr>
<tr>
<td>Number of Mesh Layers:</td>
<td>Upto 12 layers per inch of thickness (Upto 5 layers per cm of thickness)</td>
<td></td>
</tr>
<tr>
<td>Fraction Volume of Reinforcement:</td>
<td>Upto 8% in both directions corresponding to up to 40 pounds of steel per cubic foot of concrete (630 Kg/m³)</td>
<td></td>
</tr>
<tr>
<td>Specific Surface of Reinforcement:</td>
<td>Upto 10 in²/in³ in both directions (Upto 4 cm²/cm³ in both directions)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Intermediate Skeletal Reinforcement if used</th>
<th>Type: Wires; wire fabric, rods; strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter:</td>
<td>1/8 ≤ d ≤ 3/8 inches (3 ≤ d ≤ 10 mm)</td>
</tr>
<tr>
<td>Grid Size:</td>
<td>2 ≤ G ≤ 4 inches (5 ≤ G ≤ 10 cm)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Typical Mortar Composition</th>
<th>Portland Cement: Any type depending on application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand-to-Cement Ratio:</td>
<td>1 ≤ S/C ≤ 2.5 by weight</td>
</tr>
<tr>
<td>Water-to-Cement Ratio:</td>
<td>0.4 ≤ W/C ≤ 0.6 by weight</td>
</tr>
<tr>
<td>Recommendations:</td>
<td>Fine sand all passing U.S. sieve No. 8 and having 5% by weight passing No. 100, with a continuous grading curve in-between</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Composite Properties</th>
<th>Thickness: 1/4 ≤ t ≤ 2 inches (6 ≤ t ≤ 50 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Cover:</td>
<td>1/16 ≤ c ≤ 3/16 inch (1.5 ≤ c ≤ 5 mm)</td>
</tr>
<tr>
<td>Ultimate Tensile Strength: Upto 5,000 psi (3,445 N/cm²)</td>
<td></td>
</tr>
<tr>
<td>Allowable Tensile Stress: Upto 1,500 psi (1,033 N/cm²)</td>
<td></td>
</tr>
<tr>
<td>Modulus of Rupture:</td>
<td>Upto 8,000 psi (5,512 N/cm²)</td>
</tr>
<tr>
<td>Compressive Strength:</td>
<td>4,000 to 10,000 psi (2,756 to 6,890 N/cm²)</td>
</tr>
</tbody>
</table>

GUIDELINES

Materials

Cement

The cement is to be ordinary portland cement and shall conform to the ASTM standard C-150. ASTM Type I is recommended; other types of portland cements may be used if special
properties such as high early strength and sulfate resistance are needed. The cement should be fresh and of uniform consistency. Where there is evidence of lumps or any foreign matter in the material, it should not be used. The cement is to be stored under dry conditions and for as short a duration as possible.

Fine Aggregates (Sand)

Sand shall be obtained from a reliable supplier and shall comply with ASTM standard C-33 for fine aggregates. It should be clean, hard, strong, free of organic impurities (ASTM C-40) and deleterious substances. It should be inert with respect to other materials used and of suitable type with regard to strength, density, shrinkage and durability of the mortar made with it.

Grading of the sand is to be such that a mortar of specified proportions is produced with a uniform distribution of the aggregate, which will have a high density and good workability and which will work into position without segregation and without use of a high water content. The fineness of the sand should be such that 100% of it passes standard sieve No. 8. Table 2 gives some guideline on desirable gradings.

Table 2. Guidelines on Desirable Sand Gradings

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing by Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>U.S. Standard Square Mesh</td>
<td></td>
</tr>
<tr>
<td>No. 8</td>
<td>100</td>
</tr>
<tr>
<td>No. 16</td>
<td>50-85</td>
</tr>
<tr>
<td>No. 30</td>
<td>25-60</td>
</tr>
<tr>
<td>No. 50</td>
<td>40-30</td>
</tr>
<tr>
<td>No. 100</td>
<td>2-10</td>
</tr>
</tbody>
</table>

Water

Water used in the mixing is to be fresh and free from any organic and harmful solutions which will lead to a deterioration in the properties of the mortar. Salt water is not to be used.

Admixtures

Special considerations shall be given to the addition of materials to the concrete for special purposes. Approval may be given by the consulting engineer, when the material is to be added to directly or indirectly reduce the water to cement ratio or according to approved standards, if any.

If admixtures are used they should conform to federal specifications SS-P-570B and ASTM C-260, C-494 or C-618. The use of additive not covered by the above specifications (such as polymer latex modified mortar) shall be based on test data to comply with the specified mortar properties.

Mortar Mix Proportions

The recommended mix proportions for a typical application are: sand-to-cement ratio by weight = 2, and water-to-cement ratio by weight = 0.45 to 0.5. The sand-to-cement ratio
may be increased to 2.5 when shotcreting is used to account for the loss of particles caused by the rebound. Other mix proportions than those recommended here (Table 1) and admixtures such as water-reducing agents may be used provided the specified properties such as compressive strength are attained. Attention should be paid to estimating the moisture content of the aggregate in order to provide a reliable control of the water-cement ratio.

Quantities of materials are normally to be determined by weight although the aggregates may be determined by volume, due allowance being made for bulking due to moisture.

The slump of the fresh mortar shall be the lowest possible that will permit thorough compaction and should not exceed 2.5 inches (6 cm); the 28-day compression strength of 3 in. x 6 in. (7.5 x 15 cm) moist cured cylinders should not be less than about 4000 psi (2756 N/cm²).

Control of the sand fineness modulus (ASTM C-33 or C-330), water-to-cement and aggregate-to-cement ratios should be carefully maintained to provide uniform properties throughout the structure. The general requirements for constituent materials and mortar quality shall also be guided by Chapters 3 to 6 of the ACI 318-77 building code requirements for reinforced concrete.

Paint Coating

For ferrocement water retaining structures paint is recommended on both surfaces. Any good quality epoxy paint such as, for example, Sikaguard HI-BILD 664/9 or equivalent is acceptable. The paint should be stable and durable for up to a specified temperature and pressure as well as chemically inert for the type of local water used.

Steel Reinforcement

The reinforcement should be clean and free from all loose mill scale, dust and loose rust, and coatings such as paint oil or anything that might reduce bond.

a) Wire mesh. Several types of mesh reinforcement can be used: galvanized chicken wire mesh, woven or welded square galvanized meshes, or expanded metal sheets. Square type meshes are recommended if isotropic properties of the ferrocement material and/or if accurate apriori prediction of these properties are desired. Woven or welded square galvanized wire mesh shall conform to ASTM standard A-185 with a wire diameter of not more than 1/16 in. (1.5 mm) and a yield strength at 0.2% offset strain, in excess of 65,000 psi (448 MN/m²).

b) Welded wire fabric. The welded fabric which can be used in combination with wire mesh shall comply with ASTM standards A-496 and A-497. The minimum yield strength of the wire shall be (according to ASTM A-82) 65,000 psi (448 MN/m²) for the smooth wire and 70,000 psi (483 MN/m²) for the deformed wire. The wire diameter shall preferably be less than 3/8 in. (10 mm) when used in ferrocement.

c) Rods, wires, strands. If used, reinforcing rods shall be as specified by ASTM A-615 and ASTM A-616 for grade 60. If prestressing wires or strands are used, they shall comply with ASTM A-421 and ASTM A-416. Preferably all shall have diameters smaller than 3/8 in. (10 mm) when used in the ferrocement wall.
Testing

This section applies essentially to large or unusual ferrocement structures where little previous experience exists. Such typical structures include water retaining or carrying structures and large water tanks of more than 10,000 gallons (40000 liters) capacity.

**Preliminary Tests**

The preliminary tests should include:

a) The compressive strength of the mortar (based on cubes or cylinders) in order to determine suitable mix proportions, and in accordance, for example, with ASTM standard C-496-66.

b) The stress-strain curves in tension of the different types of steel reinforcement used, especially the wire mesh in order to determine strength and modulus of the reinforcing system and predict crack widths.

c) The bending properties of ferrocement plates which are representative of the proposed design (ASTM C-78-64). These tests should reflect predicted strength and cracking behavior; they should show: (1) that failure is by rupture of the steel reinforcement (for better ductility), (2) that the modulus of rupture assuming a homogeneous elastic section should be in excess of a certain specified value, and (3) that the maximum crack width should be less than or equal to a specified value.

**Quality Control and Delivery Tests**

During and after the construction, the following tests should be performed in order to assure quality and uniformity of the ferrocement material and serviceability of the structure:

a) Specific weight of fresh mortar (ASTM C-138).

b) Amount of entrained air (ASTM C-260).

c) Compressive strength of mortar specimens cured in place of 1, 7 and 28 days, and moist cured for 28 days.

d) Split cylinder tests (ASTM C-496-62T) and flexural strength (ASTM C-293) on 28-day moist cured mortar specimens.

e) Tensile tests on samples of wire mesh reinforcement in order to determine their stress-strain characteristics.

f) Flexural tests at 28 days on ferrocement plates fabricated at the same time as the structure and cured on site, using a method identical to that for the structure.

g) For water retaining structures pulse velocity tests at several locations on the surface of the structure (after the curing period but before filling with water) to ascertain that no voids are present inside the wall and that the finished mortar is compact and homogeneous. Where the consulting engineer suspects any voids in the shell, he may request suitable cores to be drilled out for examination.

h) After completion, the structure is to be closely inspected for surface defects and care should be taken to patch these defects with epoxy filler cement or similar approved filler. Water retaining structures shall be filled with water, before being painted, in order to detect any leaking pinholes in the surface; these must be closed using an epoxy based adhesive or equivalent. The surface will then be presumed watertight and subsequently will be painted under dry conditions.
Design

Ferrocement is a thin highly reinforced shell of mortar in which the steel reinforcement is distributed throughout the section so that the material under stress acts approximately as a "homogeneous" material. Any design related recommendation for ferrocement structures should depend on the type of application and should be based on a rational analysis supported by test results. However, in view of the present state of knowledge of ferrocement and except for special applications where an accurate analysis may be necessary to guarantee safe performance, the following guidelines for ferrocement made with plain mortar and square steel meshes should lead to a satisfactory performance. Engineering judgement must be exercised if other types of meshes are used.

![Stress-Strain Curve](Image)

**Fig. 1. Typical idealization of the mesh stress strain curve**

1. The total volume fraction of reinforcement $V_r$, in both directions, shall not be less than 1.8%. The total specific surface of reinforcement $S_p$, in both directions, shall not be less than 2 in$^2$/in$^3$ (0.8 cm$^2$/cm$^3$). About twice these values are generally recommended (and more for water retaining structures). In computing the specific surface of reinforcement the skeletal steel may be disregarded while it may be considered in computing $V_r$.

<table>
<thead>
<tr>
<th><strong>Table 3. Reinforcing Parameters for Ferrocement</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Minimum $V_r$ (approx.)</strong></td>
</tr>
<tr>
<td><strong>Minimum $S_p$ (approx.)</strong></td>
</tr>
<tr>
<td><strong>Mesh opening</strong></td>
</tr>
<tr>
<td><strong>Number of mesh layers</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td><strong>Cover $C_{ovg.}$ (approx.)</strong></td>
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</table>

*It is recommended to double the above values for water retaining structures.*
b) The recommended average net cover of the reinforcement is about 1/12 in. (2 mm). However, a lesser value can be used provided the reinforcement is galvanized, the surface protected by painting and the crack width limited to a low value. It is also recommended that for thicknesses larger than 1/2 in. (12 mm) the net cover shall not exceed 1/5 the thickness \( t \) nor 3/16 in. (5 mm), in order to insure proper distribution of the mesh throughout the thickness.

c) For a given ferrocement material (without skeletal reinforcement) of thickness \( t \) the recommended mesh opening \( s \) should not be larger than \( t \). Furthermore, the number of layers of mesh \( N \) should be such that:

\[ N \geq 4t. \]

If skeletal reinforcement is used it is recommended that the skeletal reinforcement does not occupy more than 50% of the thickness of the ferrocement material. For this case use \( N \geq 4t' \) where \( t' \) is the thickness in which meshes are distributed, \( s, t, \) and \( t' \) are in inch units.

In order to predict the behavior of ferrocement under service load conditions, an elastic analysis similar to that of reinforced concrete "Working Stress Design" method is acceptable provided the modulus of the steel mesh system (which may be different from the modulus of the steel wire) is considered. Ultimate load can also be predicted for flexural members by analyzing ferrocement as a reinforced concrete member (or column) using the ACI ultimate strength design method. For tensile members the load at ultimate can be approximated by the load carrying capacity of the mesh reinforcement alone in the direction of loading.

If the working stress design method is used, allowable stresses for the constituent materials and for the composite have to be specified. Suggested values are shown in Table 4 and described below:

<table>
<thead>
<tr>
<th>Table 4 Allowable Tensile Stresses in the Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-water retaining structures</td>
</tr>
<tr>
<td>( \dot{f}_s = 0.60 f_p )</td>
</tr>
<tr>
<td>Ex. for ( f_p = 65 \text{ ksi} )</td>
</tr>
<tr>
<td>( \dot{f}_s = 39 \text{ ksi} )</td>
</tr>
<tr>
<td>Water retaining structures</td>
</tr>
<tr>
<td>( f_s = 30 \text{ ksi} )</td>
</tr>
<tr>
<td>Fatigue loading (maximum allowable stress range)</td>
</tr>
<tr>
<td>(fatigue life of about ( 2 \times 10^6 ) cycles)</td>
</tr>
</tbody>
</table>

d) The allowable tensile stress in the steel reinforcement may be generally taken as \( 0.60 f_p \), where \( f_p \) is the yield strength measured at 0.0035 strain; however, for water retaining and sanitary structures it is preferable to limit the tensile stress to 30 ksi (207 MN/m²) unless crack width measurements on a test model indicate that a higher stress would not impair performance. The above values hold provided the pitch weaving of the mesh system is moderate in order to insure an adequate modulus.
For pure tensile elements in ferrocement, an allowable tensile stress in the composite of up to \( \bar{\sigma}_t \) 1000 psi (7 MN/m²) may be used as a first approximation to determine the required thickness \( t \) of the element; then the required area of steel in the loading direction \( A_{sl} \) can be determined using cracked section analysis.

e) The allowable compressive stress in the composite may be taken as 0.45 \( f_c \) where \( f_c \) is the specified compressive strength of the mortar measured from tests on 3 x 6 in. (7.5 x 15 cm) cylinders.

f) For ferrocement structures to sustain a minimum life of two million cycles the stress-range in the steel must be limited to \( f_{ps} = 30 \) ksi (207 MN/m²). A value of \( f_{ps} = 36 \) ksi (248 MN/m²) can be used for one million cycles and 55 ksi (380 MN/m²) for 100,000 cycles.

Although the design of ferrocement structures is often based on stress limitations or strength criteria, the concept of serviceability, i.e., acceptable overall behavior under service load and during service life is becoming prevalent. Serviceability in concrete structures is often related to cracking behavior and crack widths. Little quantitative information, however, exists on cracking in ferrocement. The following limits and prediction equations can be used as a first approximation and until more substantial experimental data become available.

g) It is recommended that the maximum value of crack width be less than 0.004 in. (0.010 mm) for non-corrosive environment and 0.002 in. (0.005 mm) for corrosive environment and/or water retaining structures.

h) The maximum crack width for flexural members can be predicted as a first approximation using the following equation [8]:

\[
W_{max} \simeq \varepsilon_s \times S \times \beta = \frac{f_s}{E_R} \times S \times \beta
\]

(1)

where

\( \varepsilon_s \) = strain in the outermost layer of steel  
\( f_s \) = stress in the outermost layer of steel  
\( S \) = mesh opening size  
\( \beta \) = ratio of distances to the neutral axis from the extreme tensile fiber and from the outermost layer of steel  
\( E_R \) = modulus of the reinforcing system

[Note: \( E_R = E_S \simeq 29 \times 10^6 \) psi (203 \times 10^3 MN/m²) for welded mesh; \( E_R \) may vary from 15 to 25 \times 10^6 psi (105 to 175 MN/m²) \( 10^3 \) for woven meshes depending on the pitch of the weaving as observed in various tests]

Note that equation (1) is valid for both the U.S. and metric systems. It is an equation based on theoretical considerations and is recommended here mostly because of its simplicity. Regression equations based on experimental data [23] were also developed and can be used instead, to predict average and maximum crack widths in flexure. They are given by:

\[
W_{av} = \frac{23000}{E_R} (0.271 f_s - 3.73)10^{-4}
\]

(2)

and

\[
W_{max} = \frac{23000}{E_R} (0.324 f_s - 4.36)10^{-4}
\]

(3)
where

\[ W_{av} = \frac{15.9}{E_R} (f_s - 95) \]  \hspace{1cm} \text{......... (4)}

and

\[ W_{max} = \frac{15.9}{E_R} (1.2f_s - 111) \]  \hspace{1cm} \text{......... (5)}

where crack width is in mm and steel stress and elastic modulus are in MN/m².

i) To predict crack widths in tensile ferrocement members, one can refer to the theoretical equation developed by Huq and Pama [26]. It has the advantage of accounting for many of the parameters found important in the cracking response of ferrocement and can be adapted to estimate flexural cracking [27] as well. A less accurate empirical design procedure has been also developed [23] to predict cracking under tensile loading. It applies when the steel stress \( f_s \) is less than the yield strength and in any case less than 60 ksi (414 MN/m²). It is summarized as follows:

For any \( f_s \leq 20 \text{ S}_{RL} = f_{sta} \) (stress at crack stabilization)

\[ W_{max} = \frac{20}{E_R} \]  \hspace{1cm} \text{......... (6)}

where \( W_{max} \) is given in inches, \( S_{RL} \) in \text{in}^{-1}, \( f_s \) in ksi and \( E_R \) in ksi.

For \( f_s > 20 \text{ S}_{RL} \)

\[ W_{max} = 10^{-4} \left[ 6.9 + (f_s - 20 \text{ S}_{RL}) \right] \frac{29000}{E_R} \]  \hspace{1cm} \text{......... (7)}

If the metric system is used equations (6) and (7) would lead to the following:

For any \( f_s \leq 345 \text{ S}_{RL} \)

\[ W_{max} = \frac{3500}{E_R} \]  \hspace{1cm} \text{......... (8)}

where \( f_s \) is in MN/m², \( S_{RL} \) in cm⁻¹, \( W_{max} \) in mm and \( E_R \) in MN/m²:

for \( f_s > 345 \text{ S}_{RL} \)

\[ W_{max} = \frac{20}{E_R} \left[ 175 + 3.69 (f_s - 345 \text{ S}_{RL}) \right] \]  \hspace{1cm} \text{......... (9)}

j) Test results on cyclic loading of ferrocement beams have indicated that the average and maximum crack widths at a given load keep increasing with the number of loading cycles. This may be due to the effects of cyclic creep of mortar and bond deterioration at the mesh mortar interface due to fatigue.
The following relationship was found acceptable to represent crack width data, average or maximum, under fatigue loading up to about 90% of fatigue life:

\[ W = Ae^{Br} \] ......... (10)

where

- \( W \) = average or maximum crack width
- \( A \) = value of crack width at maximum load under static test.
  - It could be predicted from equations (1) to (5)
- \( e \) = base of Naperian logarithms
- \( r \) = cycle ratio; i.e., number of cycles at which crack width is calculated to the number of cycles to failure \( N_f \)
- \( B \) = a parameter determined from the experimental data and described in (9). A constant value of \( B = 1.67 \) is proposed for \( N_f \geq 450,000 \) cycles which is generally the case for most design situations.

In order to use equation [10] the number of cycles to failure \( N_f \) is needed. A regression analysis of the data [9] between stress range in the steel and the number of cycles to failure has led to the following relation:

\[ f_{sr} = 152.3 - 19.5 \log_{10} N_f \] ......... (11)

where

- \( f_{sr} \) = stress range in the outermost layer of steel in ksi
- \( N_f \) = number of cycles to failure

Thus if \( f_{sr} \) is known, \( N_f \) can be predicted, or if a certain value of \( N_f \) is required by the design the corresponding limit on stress range can be determined.

Using the metric system, equation [11] gives

\[ f_{sr} \approx 1050 - 134.5 \log_{10} N_f \] ......... (12)

where \( f_{sr} \) is in MN/m².

Illustrative examples showing the use of the above design criteria are given in Appendix A

**Construction**

*Fabrication of Reinfocing Cage*

The wire mesh and/or mesh fabric or rods are to be placed and shaped to the structure form. Adequate reinforcement are to be placed locally where higher stresses are expected.

The different layers of wire mesh should be carefully and securely tied together and to the central layer of coarser skeletal reinforcement (rods or welded wire fabric, if any) in order to provide an as even thickness as possible and to avoid movement during the placing of the mortar.

Any multiple discontinuities in reinforcement are to be avoided and an adequate overlap is to be provided according to the following recommendations: and overlap of at least 3 in
(7.5 cm) or six times the mesh size whichever is higher is recommended to assure continuity between the ends of a layer of mesh; where reinforcing rods are joined, the overlap must be in accordance with the recommendations of the ACI 318-77 Building Code, Section 7.

If holes are needed in the shell, they should preferably be cut before plastering. Adequate reinforcement must be placed locally around holes to account for stress concentrations. The locations of holes should preferably be where stresses and pressures are minimal. Additional protective coating such as epoxy sealers must be provided around connections.

**Mortar-Plastering**

Mixing, handling and compaction of the mortar should be consistent and closely supervised to ensure high quality material. The builder should be guided by established codes of practice such as the ACI 318-77 Building Code, Sections 4 and 5.

The mortar must be thoroughly compacted during placing to ensure the absence of voids around reinforcement and in the corners of any framework.

Under no circumstances should the mortar be compacted simultaneously from both sides of the reinforcement in one operation. Vibrators and hand rodding can be used if necessary to achieve better penetration and distribution.

The mortar should be placed within a reasonably short period of adding the mixing water and with continual agitation during the waiting period. During handling and placing of the mortar care shall be taken to avoid segregation of the mix.

When plastering of the structure is done in more than one operation, care is to be taken to ensure a sound joint; same shall hold for the main joint between the foundation and the tank wall. The use of bonding agents where mortar joints are made will be given special consideration and has to be approved by the consulting engineer. Under no circumstances is any bonding agent which is unstable in water to be used.

Apertures for various fittings should where possible be formed prior to placing the mortar (or may be cut after completion and curing of the hull if it is shown that this last method is equivalent). All apertures should have the bare concrete surface adequately sealed before fixing the skin fittings.

**Shotcreting**

The application of the shotcrete technique to ferrocement reduces the difficulty of adequate placement of low slump mortar with trowel or hand. The two known methods of shotcreting the "dry" and the "wet" method are both acceptable for application in thin shell structures. In either method the material and process must comply with the ACI Standard 506-66 "Recommended Practice for Shotcreting."

**Curing**

Ferrocement structures are to be properly cured once the mortar has taken its first set (which occurs 3 to 4 hours after mortar application). The set mortar or concrete is to be kept wet for a period dependent on the type of cement used and the ambient conditions. The
method of curing should normally be by water spray but other proven methods are acceptable such as steam curing and membrane curing. The temperature of curing must be above 50°F. The contractor should be guided by the ACI 318-77 Building Code, Section 5.

**Painting**

The paint if needed or desired should be applied after completion of curing and after the surfaces have been dried. Manufacturers instructions should be followed.

**ACKNOWLEDGEMENT**

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**APPENDIX A**

**Illustrative Design Examples**

The following variables will be used throughout: compressive strength of mortar \( f'_c = 5000 \text{ psi (35 MN/m}^2) \); elastic modulus of the woven reinforcing mesh \( E_R = 23000 \text{ ksi (159,000 MN/m}^2) \); steel stress-strain curve assumed elasto-plastic (Fig. 1) with a yield strength \( f_y = 70000 \text{ psi (483 MN/m}^2) \); allowable compressive stress in ferrocement \( f_{ck} = 0.45 f'_c = 2250 \text{ psi (15.5 MN/m}^2) \); modular ratio \( n = 5.4 \).

**Tensile Element**

Design the wall of a ferrocement cylindrical tank subjected, under service load, to a maximum hoop (ring) tension \( T = 500 \text{ lbs/linear in.} \)

Assuming an allowable stress \( f_{rk} = 1000 \text{ psi for the composite leads to the required thickness:} \)

\[
t = \frac{T}{f_{rk}} = \frac{500}{1000} = 0.5 \text{ in. (12.5 mm)}
\]

The amount of steel \( A_{sl} \) per inch wide strip in the loading direction is obtained assuming a cracked section and an allowable tensile stress in the steel \( f_y = 30 \text{ ksi (207 MN/m}^2) \). Note that this stress satisfies the recommended limits for water retaining structures and for fatigue loading; thus:

\[
A_{sl} = \frac{T}{f_y} = \frac{500}{30000} = 0.0167 \text{ in}^2 (10.6 \text{ mm}^2)
\]

Assume the reinforcement consists of woven wire mesh with 0.5 in. (12.5 mm) wire spacing and a wire diameter of 0.042 in. (1.07 mm). The cross-sectional area of one wire equals 0.00138 \text{ in}^2 (0.89 \text{ mm}^2); the number of wires needed per inch equals \((0.0167/0.00138 \approx 12)\) wires. As two wires exist per inch and per layer of mesh, six layers are needed to provide the required value of \( A_{sl} \). It can be checked that the thickness \( t \) can just accommodate them.
The total volume fraction of reinforcement in both directions, assuming square mesh, is given numerically by:

\[ V_f = \frac{2 \times 0.0167}{t} = 0.0668 \]

or 6.68%. The corresponding specific surface of reinforcement is given by:

\[ S_R = \frac{4V_f}{\phi} = \frac{4 \times 0.0668}{0.042} = 6.36 \text{ in.}^2/\text{in.}^3 (2.5 \text{ cm}^2/\text{cm}^3) \]

Note that the above values satisfy all the recommendations to qualify the composite as ferrocement.

As the predicted value of the steel stress at crack stabilization \(20 S_{RL}\) is high compared to the service stress, the maximum crack width is given by equation (6):

\[ W_{max} = \frac{20}{E_R} \approx 0.00087 \text{ in.} (0.022 \text{ mm}) \]

which is less than the limit of 0.002 in. (0.05 mm) recommended for water retaining structures.

The above design has largely satisfied most limitations. Note, however, that an allowable tensile stress of 1000 psi (7 MN/m²) in the composite generally leads to a high volume fraction of reinforcement. The reader may want to check that if a stress of 800 psi (5.5 MN/m²) was used, the following alternative design would have been acceptable: \(t = 0.625\) in. (16 mm) and same area \(A_f\) of longitudinal reinforcement; the reinforcement consists of a square skeletal steel grid \(2\) in. (5 cm) wire spacing with wire diameter of 0.125 in. (3.2 mm) and four layers of the woven wire mesh used previously; the corresponding value of \(V_f\) is 5.5% and \(S_R\) is 3.99 in.\(^2/\text{in.}^3\) (1.57 cm\(^2/cm^3\)); assuming the same modulus of elasticity as above leads to a stress at crack stabilization \(20 S_{RL}\) of 40 ksi (276 MN/m²) larger than the service stress in the steel; thus the same value of maximum crack width is also obtained from equation (6) and is satisfactory.

**Flexural Element**

Check the feasibility of using the ferrocement sandwich panel shown in Fig. 2a as a typical element of a one-way simply supported slab. Assume a dead load including weight of 20 lbs/linear ft and a live load of 60 lbs/linear ft; the live load corresponds to 40 lbs/sq. ft or approximately 195 kg/m². The span length is 12 ft (3.7 m).

1. Let us first determine the values of midspan moments; the subscript \(D\) is used to describe the effect of dead load, \(L\) that of live load, \(S\) that of service load and \(U\) that of ultimate load.

Thus:

\[ M_D = \frac{w_D A^2}{8} = 4320 \text{ lb in.} \]

\[ M_L = \frac{w_L A^2}{8} = 12960 \text{ lb in.} \]

\[ M_S = M_D + M_L = 17280 \text{ lb in.} \]
2. **Ultimate Strength Design.** The factorized ultimate moment to be resisted by the ferrocement section is given by:

\[ M_U = 1.4 M_D + 1.7 M_E = 28080 \text{ lb in.} \]

Using the ultimate strength design approach of the ACI code leads to two equations of equilibrium at ultimate behavior of the section: a force equilibrium equation and a moment equilibrium equation. These equations contain two unknowns namely "a," the depth of the compressive stress block and \( A_t \), the area of tensile reinforcement. In writing these equations it will be assumed at first that the section acts as a rectangular section at ultimate and that the influence of the reinforcement in the compressive zone is negligible. (These assumptions are confirmed by the results obtained for this example and there will be no need here to revise the design.) The two equations (Fig. 2b) are:

\[ A_z f_y = 0.85 f'_c \cdot ba \]  \hspace{1cm} (13)

\[ M_U = \phi A_z f_y \left( d - \frac{a}{2} \right) = \phi 0.85 f'_c \cdot ba \left( d - \frac{a}{2} \right) \]  \hspace{1cm} (14)

Replacing \( \phi \) by 0.9 and \( M_U \) by its value in equation (14) leads to a quadratic equation in "a" for which one positive root exists, namely \( a \approx 0.07 \text{ in.} \ (1.78 \text{ mm}) \). The corresponding location of the neutral axis is given by \( c = a/\beta_f \approx 0.07/0.80 \approx 0.09 \text{ in.} \ (2.23 \text{ mm}) \); as "c" is less than the flange thickness the section is acting as a rectangular section as previously assumed; thus equation (13) gives:
\[ A_e = \frac{0.85 f'_e ba}{f_y} = 0.0765 \text{ in.}^2 \ (49.35 \text{ mm}^2) \]

Using a square wire mesh having a wire spacing of 0.25 in. (6.35 mm) and a wire diameter of 0.025 in. (0.63 mm) leads to the reinforcement shown in Fig. 2c. Note that the tensile steel area provided amounts to 0.0825 in.\(^2\) (53.22 mm\(^2\)) slightly higher than the above required value.

For the above section it can be shown that the applied moment is substantially less than the maximum resisting moment by the concrete and that the shear resistance is largely adequate.

3 **Working Stress Design.** The allowable stress in the steel under full service load must be specified. One may consider first the value \( f_y = 0.60 f_y = 42 \text{ ksi} \ (290 \text{ MN/m}^2) \). However, the slab is subjected to fatigue under the repetitive applications of live loads; the stress range in the steel is limited to \( f'_{y} = 30 \text{ ksi} \); as the live load is 75% of the service load, the maximum allowable steel stress should be \( f'_{y} = 30/0.75 = 40 \text{ ksi} \ (276 \text{ MN/m}^2) \).

The required area of tensile steel is given by:

\[ A_y = \frac{M_y}{f_y J d} \tag{15} \]

where \( J \) is the lever arm between the tensile and compressive forces in the section. For the sandwich-type section shown \( J \) can be estimated as 0.95. Thus:

\[ A_y \approx \frac{17280}{40000 \times 0.95 \times 5.875} = 0.0774 \text{ in.}^2 \ (49.9 \text{ mm}^2) \]

The above value is not very different from that obtained using the U.S.D. approach and leads for all practical purposes to the same reinforcement shown in Fig. 2c.

4. To estimate the value of maximum crack width under service load, two approaches can be considered for this particular section: one, that the section is acting in flexure and the other that the lower flange is acting as a pure tensile member.

If flexure is considered, the maximum crack width can be estimated from equation (3) assuming \( f_y = f_{ya} = 40 \text{ ksi} \):

\[ W_{\text{max}} = \frac{23000}{E_R} [0.324 f_y - 4.36] \times 10^{-4} = 0.00086 \text{ in.} \ (0.022 \text{ mm}) \]

If pure tension in the lower flange is considered, the value of the specific surface of reinforcement in the flange in the loading direction must be determined. It can be shown that the volume fraction of steel in the loading direction is 1.83% and the corresponding specific surface is 2.93 in.\(^2\)/in.\(^3\) (1.15 cm\(^2\)/cm\(^3\)). Thus the stabilization stress is given by:

\[ f_{sta} = 20 S_{RL} = 20 \times 2.93 = 58.6 \text{ ksi} \ (404 \text{ MN/m}^2) \]

and as \( f_y < f_{sta} \) the maximum crack width can be predicted by:

\[ W_{\text{max}} = \frac{20}{E_R} = 0.00087 \text{ in.} \ (0.022 \text{ mm}) \]
which is surprisingly very close to the value obtained assuming flexure behavior.

Let us estimate the effect of fatigue on crack width. As the stress range in the steel was assumed equal to 30 ksi (207 MN/m²), the fatigue life of the Ferrocement element can be estimated at about 2 x 10⁶ cycles from equation (11). Using equation (10) with \( A = 0.00087 \) and \( B = 1.67 \) leads to the following predicted values of maximum crack widths at different cycles:

For \[ r = 0.25 \text{ (0.5 million cycles)} \] gives \( W_{\text{max}} = 0.00132 \text{ in. (0.034 mm)} \)

\[ r = 0.50 \text{ (1 million cycles)} \] gives \( W_{\text{max}} = 0.00201 \text{ in. (0.051 mm)} \)

\[ r = 0.75 \text{ (1.5 million cycles)} \] gives \( W_{\text{max}} = 0.00304 \text{ in. (0.077 mm)} \)

\[ r = 0.90 \text{ (1.9 million cycles)} \] gives \( W_{\text{max}} = 0.00391 \text{ in. (0.10 mm)} \)

It can be seen that although the maximum crack width under static load was substantially smaller than the recommended limit of 0.004 in. (0.1 mm), the effect of cyclic fatigue loading can significantly reduce the gap between them.

5. The above example dealt with a particular section of Ferrocement where the reinforcement was essentially concentrated in the flanges. If a rectangular section was used with several layers of reinforcement across the depth, the reader may want to refer to the work of Huq and Pana [26-27] for a more general and complete analysis. If the ACI code approach is used with U.S.D. or W.S.D., the section (with several layers of reinforcement) may be also analyzed as a column under pure bending and should lead to very realistic results.

REFERENCES


16. "Building Code Requirements for Reinforced Concrete," American Concrete Institute, Standard 318-77, Detroit, Michigan, USA.

17. American Society for Testing and Materials, Standards No. A116-73; A390-66; A475-72; A584-68; A586-68; A497-72; A641-71a; A185-73; A615-74a; and A82-72.


