Development of a semicircular roofing system in ferrocement

Ir. P. J. van Stekelenburg
M. S. Mathews MSCE
Ir. J. C. Walraven
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Ir. P.J. v. Stekelenburg 1)
M.S. Mathews 2)
Ir. J.C. Walraven 1)

1) University of Technology, Delft, Holland
2) University of Technology, Madras, India
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1. **Motivation and objectives**

The objective of the research program was to develop a ferrocement roofing system, which is cheap, durable and can be made on self-help basis with the minimum of technical assistance. In this section a number of aspects connected with the choice of the constructional system are discussed.

**Shape:** From several proposals the concept of a curved roof of ferrocement was chosen. A cylindrical shape was preferred over other forms, since, by virtue of its uniform curvature, it is possible to make all elements, to be assembled to a complete roof structure, on a single mould. Furtheron the depth of the supporting wall may be reduced as a result of the high depth of the roof itself.

**Size of the roof:** The span chosen was 3 m, since this was considered to be acceptable for a room in a house. The roof can be built up of single units, which can be extended in longitudinal direction by applying mortar to the overlapping meshes from the two shell units. Each unit is a composition of three elements. For laboratory testing the width adopted was 1 m (fig. 1).

![Fig. 1. Ferrocement roof unit.](image)

While in the actual construction the roof acts as a cylindrical shell, under laboratory conditions this shell is reduced to an arch; hence the load carrying capacity is correspondingly less.
The roof thickness chosen was 12 mm. The basis for this choice was that the thickness must be small enough to admit transport of the single elements by man-power and at the same time it must be thick enough to accommodate the meshes. However the thickness was increased at the edges to take care of support stresses.

Support conditions: A bolted connection was chosen for the support. The edge was stiffened with a beam, to be connected in a rotation free way to the supporting wall structure, which provides a support reaction against horizontal displacements (fig. 2).

Fig. 2. Support condition.

Assembly procedure: It would be fairly difficult to make the unit as a monolithic one, since this would require elaborate shuttering. Therefore it seemed appropriate to subdivide the unit into smaller elements. These elements must be simple to make and small in weight. To make a reasonable choice for the subdivision into elements a number of considerations have to be taken into account. An optimum solution must be obtained with regard to the following points:

- Ease of manufacturing
- Ease of handling (low weight)
- Ease of erection and fixing in place
- Ease of maintainance
- Suitability for self help.
It was felt that making a roof-unit out of 3 elements satisfies these requirements fairly well (fig. 1).

Joints: For the structural concept adopted, the quality of the joint is an important parameter, both for the strength of the structure and for the ease of its assembling. In the experimental program the joint detailing has been object of investigation.

Shuttering: The type of shuttering adopted will have an important influence on the economics of the system and on the extent to which self help will be feasible. Hence, different situations have to be considered.

The first situation corresponds to the laboratory condition and the second is related to the field conditions.

- The models made in the laboratory must be very accurate and reproducible. Therefore it is important to ensure close dimensional tolerance. So a wooden mould was designed for this purpose, details of which are presented later.

- The method of construction to be adopted in the field must be very simple. It must be such, that the people are able to manufacture the units on their own with a minimum of technical assistance. A possible solution is that earth (a mixture of sand and clay) is formed corresponding to the profile of the unit, which is circular in this case. The top of such a wall can be covered with a layer of cement mortar. The units can be cast on this surface.

2. Experimental program

a. Parameters

As reinforcement of the roof elements, layers of chicken mesh were used. For the tests 25 x 0.8 mm mesh of corrugated steel (see fig. 3) was chosen as a compromise between ease of penetration of mortar and reinforcement ratio.
One layer of mesh provides a total steel section of 40.2 mm²/m. Four types of connections were tested, details of which are represented in fig. 4.

Type 1. This joint is formed by making the meshes, extending from two adjacent elements, overlap and grouting them, after the elements have been brought in position.

Type 2. Here the elements overlap and are bolted together.

Type 3. In this case an articulated bolted joint was adopted. This ensures a better flow of stresses, compared to the type 2 joint.

For each type of joint two test specimens were made, one with 4 layers of mesh and one with 6 layers of mesh (the average thickness of the elements with 6 layers of mesh exceeded the intended thickness of 12 mm; the average value was 14 mm). After these experiments one additional test was carried out. This test differed from the previous ones in the detailing of both the reinforcement and the joint. A detailed representation of this joint is given in fig. 4.4 (type 4) and the figures 5a and b.

The reinforcement consisted in this case of 6 layers, which were however separated in two groups of three layers by means of split bamboo rods (fig. 6), resulting in a greater inner lever arm and consequently in a greater stiffness after cracking.
Reinforcement overlap (grouted connection)

Discontinuous bolted connection (dry connection)

Continuous bolted connection (dry connection)

Bajonet connection (grouted connection)

Fig. 4. Joint details.
Fig. 5a and b. Joint type 4 before grouting.

Fig. 6. Split bamboo rods as spacers to ensure a greater inner lever arm after cracking.
A survey of the total test program is presented in Table 2.1. Each model has been assigned an identifying number, indicating the type of the joint and the number of meshes.

**DETAILS OF TEST THE PROGRAM**

<table>
<thead>
<tr>
<th>Identifying model number</th>
<th>Type of joint</th>
<th>Number of meshes</th>
<th>Number of bolts or ties per m' of the joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4</td>
<td>1</td>
<td>4</td>
<td>-</td>
</tr>
<tr>
<td>1.6</td>
<td>1</td>
<td>6</td>
<td>-</td>
</tr>
<tr>
<td>2.4</td>
<td>2</td>
<td>4</td>
<td>2 × 9 bolts M12</td>
</tr>
<tr>
<td>2.6</td>
<td>2</td>
<td>6</td>
<td>2 × 9 bolts M12</td>
</tr>
<tr>
<td>3.4</td>
<td>3</td>
<td>4</td>
<td>2 × 9 bolts M12</td>
</tr>
<tr>
<td>3.6</td>
<td>3</td>
<td>6</td>
<td>2 × 9 bolts M12</td>
</tr>
<tr>
<td>3.6</td>
<td>4</td>
<td>6</td>
<td>3 × 9 ties Ø 1.25 mm</td>
</tr>
</tbody>
</table>

Table 2.1. *(*) see fig. 4.

b. Material properties

- The mixture, used to manufacture the specimens, had a maximum gravel size of 4 mm. The sand contained 17.5% fine material < 125 μ by means of the application of quartz powder. The cement weight (cement class B) was 415 kg per 2300 kg mix. The water cement ratio was 0.5. Details of the mix composition are given in Appendix I.

The values of the strength control tests are represented in Table 2.2.

<table>
<thead>
<tr>
<th>Model no.</th>
<th>Flexural tensile strength, obtained on 40 × 40 × 150 mm prisms (N/mm²)</th>
<th>Compression strength, obtained on cubes 40 × 40 × 40 mm (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4</td>
<td>9.57</td>
<td>43.4</td>
</tr>
<tr>
<td>1.6</td>
<td>10.0</td>
<td>63.6</td>
</tr>
<tr>
<td>2.4</td>
<td>10.5</td>
<td>63.6</td>
</tr>
<tr>
<td>2.6</td>
<td>11.6</td>
<td>69.1</td>
</tr>
<tr>
<td>3.4</td>
<td>10.2</td>
<td>66.1</td>
</tr>
<tr>
<td>3.6</td>
<td>10.4</td>
<td>63.2</td>
</tr>
<tr>
<td>4.6</td>
<td>8.6</td>
<td>41.2</td>
</tr>
</tbody>
</table>

Table 2.2. Strength control values.
The mortar in the joints had a maximum grain size of 2 mm, while the distribution was built up according to a Fuller curve. Quartz powder was added with a quantity of 2.66 kg/10 liter mixture, and cement class B with a quantity of 4.66 kg/10 liters mixture. This mortar was used for the joints of 1.4., 1.6. and 4.6. In the other models, the small gaps were filled with porion, a monomer with fine sandfilling on a water base. The chicken mesh (0.8 mm - 25 mm apart) had an elastic behaviour up to a stress of 300 N/mm². The edge-reinforcement was of the quality FeB 40-HW.

c. Method of manufacturing

The length of the mould was such that 6 elements could be cast at a time. These 6 elements consisted of 4 support elements and 2 centre elements. Thus 2 specimens, one with 4 layers and one with 6 layers, were made in a single operation (fig. 7-9).

At first a plastic sheet was spread over the mould to act as a separating medium between the mould and the specimen. Then half the number of the layers of chicken mesh were placed over the mould. The meshes were stretched over the mould and securely fixed in position. Subsequently the meshes were plastered with the concrete mix up to half the thickness of the shell, using a vibrating rubbing board. Then the other layers of meshes were fixed in position and the plastering operation was completed, by compacting also this second layer of mortar on the mould. The reinforcement of the edge beams was placed in position together with the second half of the meshes. The holes in the edge beams were made using short plastic pipes fixed with screws on the mould. After hardening these pipes were taken out. For the fabrication of the holes for the bolted joints rubber plugs were used, which were nailed on the mould. Gauges were used to ascertain the proper placement of plugs and pipes. In model 4.6 after applying the first layers of mortar, enough to cover the three lower meshes, split bamboo rods, soaked with water during 48 hours, were placed, following the curvature of the mould, 200 mm apart, and fixed at the lower meshes. Thus the three upper meshes were kept at a distance of 6 mm from the lower
ones. The elements remained on the mould for three days and were kept moist by means of covering with wet cloths during this period. The elements were removed on the fourth day, and mounted on scaffolding at the 7th day (to prepare the joints for grouting, to assemble the elements in units) (fig. 11).

d. Instrumentation and testing procedure

After assembling, the units were subjected to a line load on the top, according to the system represented in fig. 10 and 12. The vertical load on the top was applied by a hydraulic jack and controlled by a load cell. One of the edges was bolted on a moment resisting swaying structure, placed on one-direction pendel plates, kept in place only by a load cell (A). The other edge was bolted to a moment resisting support, fixed at the floor. Fifteen deflection readings in radial direction were measured on the unit (fig. 10 and 12). The load was applied in steps. Each step was followed by 90 seconds constant load, after which the measuring data were taken.
Fig. 7, 8, 9. Manufacturing of the test specimens.
Fig. 10. Arrangements for the test of a ferrocement roof unit.
Fig. 11. Scaffolding to assemble the elements to units and to prepare the joints for grouting.

Fig. 12. Specimen during testing.
3. **Behaviour under stepwise increased loading and failure**

In all specimens the first crack occurred in the top of the specimen. At further loading this was extended to a system of parallel cracks, forming a typical "ferrocement hinge" (fig. 13). The next cracks occurred in the joint or in the neighbourhood of the joint (Fig. 14).

![Fig. 13. Typical ferrocement hinge, located at the inner side of the roof under the top.](image13)

![Fig. 14. Hinge in the vicinity of the joint.](image14)
For the first series of specimens (joint type 1) only one crack developed at the joint, after which a severe loss of stiffness was observed. In the other series, similar systems of parallel cracks as in the top developed along the side of the joint. During the development of this second hinge the stiffness of the whole structure decreased gradually.

Instability occurred when, after a considerable deflection, a third crack was formed near the basis at the inside of the specimen. The individual specimens are compared using the load-top deflection relation. A comparison between the specimens of the first three series, revealing the influence of the number of meshes (and element thickness) is represented in fig. 15.

It is obvious that the increase with two layers of meshes in combination with a slight increase in shell-thickness has a favourable influence on the ultimate resistance of the structure. Furthermore it is obvious that the failure of the ferrocement roof is very gradual, which is in sharp contrast to the type of failure that occurs in the case of asbestos cement sheet.

A comparison of the influence of the different types of joints can be made by means of the figures 16a and b, in which the load-top deflection curves for specimens with the same number of mesh-layers have been collected. The results indicate that the unit utilizing the monolithic joint (type 1) was obviously less strong than those with bolted joints. This has to be blamed on the fact that due to shrinkage of the assembling mortar, the joint was already partially cracked before loading, in combination with a local non-uniform distribution of the mesh, which appeared to be concentrated at the most unfavourable (inner) side of the roof. However, the improved monolithic joint (type 4) behaves very well.

The combination of improved joint and split bamboo spacers, to increase the inner lever arm, showed even a higher stiffness and a higher ultimate resistance than the specimens with the bolted joints. It must be noted that after cracking at the top hardly any reduction of the stiffness can be observed. As it has to be realised that the bolted connections are relatively costly and more complicated, it may be considered as very favourable that this improved, feasible system gives the best results.
Fig. 15. Load-top deflection curves for the different series.
Other data, such as the load at which the first crack occurs at the top ($P_{cr}$), the load at which the first crack in the negative moment region occurs ($P_n$) and the ultimate load ($P_u$) are given in Table 3.1. Data, concerning support reactions and deflections are presented in Appendix II.

<table>
<thead>
<tr>
<th>Specimen nr.</th>
<th>$P_{cr}$ (kN)</th>
<th>$P_n$ (kN)</th>
<th>$P_u$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4</td>
<td>0.7</td>
<td>1.2</td>
<td>1.3</td>
</tr>
<tr>
<td>1.6</td>
<td>1.1</td>
<td>1.6</td>
<td>1.9</td>
</tr>
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<td>1.7</td>
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<tr>
<td>2.6</td>
<td>1.4</td>
<td>2.0</td>
<td>2.6</td>
</tr>
<tr>
<td>3.4</td>
<td>1.45</td>
<td>1.9</td>
<td>2.0</td>
</tr>
<tr>
<td>3.6</td>
<td>1.8</td>
<td>2.4</td>
<td>2.8</td>
</tr>
<tr>
<td>4.6</td>
<td>1.5</td>
<td>2.7</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Table 3.1.
4. Theoretical considerations and calculation of the load deflection behaviour of a semi-circular roof

Since the roof is symmetrical with regard to the vertical line through the top, only one half of the construction is considered (Fig. 17).

The moment in an arbitrary section, making an angle $\Theta$ with the vertical axis through the top is equal to:

$$M_\Theta = M_\circ + N.R(1-\cos \Theta) - \frac{1}{2} P.R \sin \Theta$$

The energy, absorbed in the considered system, due to bending is:

$$W = \frac{1}{2EI} \int_0^{\pi/2} M_\Theta^2 Rd\Theta$$

With the method of Castigliano, the displacements and rotation of the top can be expressed as:

$$\delta_\theta = \frac{\partial W}{\partial M_\Theta} = \frac{\partial W}{\partial M_\circ} = \frac{1}{2EI} \int_0^{\pi/2} 2M_\circ \frac{\partial M_\circ}{\partial N} Rd\Theta$$

$$= \frac{R^2}{EI} \int_0^{\pi/2} (M_\circ + NR(1-\cos \Theta) - \frac{1}{2} PR \sin \Theta)(1-\cos \Theta)d\Theta$$

$$= \left\{ M_\circ \left( \frac{\pi}{2} - 1 \right) + NR\left( \frac{3}{4} \pi - 2 \right) - \frac{PR}{4} \right\} \frac{R^2}{EI} \quad (1)$$
When the construction is uncracked, both the rotation $\phi$ and the horizontal displacement $\delta_h$, expressed in (1) and (3), must be equal to zero. These conditions lead to the following expressions for $M_o$ and $N$:

$$M_o = 0,154 \text{ PR}$$  \hspace{1cm} (4) \\
$$N = 0,45 \text{ PR}$$  \hspace{1cm} (5)

As a result, the distribution of moments in the uncracked stage is as represented in Fig.18.

Fig.18. Distribution of moments in the uncracked stage.
Substitution of (4) and (5) in (2) yields an expression for the displacement of the top:

$$\delta_v = 0.0112 \frac{PR^3}{EI}$$

(6)

Due to this displacement an additional second order moment $M_{\text{add.}} = \delta_v N$ is caused. Combining (5) and (6), this moment can be expressed as:

$$M_{\text{add.}} = (0.0112 \frac{PR^3}{EI}) 0.45 P = 0.0054 \frac{P^2 R^3}{(EI)^2}$$

(7)

A comparison of (4) and (7) leads to the conclusion that second order effects can be neglected.

2. Behaviour of the roof element in the cracked state.

As can be concluded from Fig. 18 the first crack must be expected in the top. The distribution of forces in the element after cracking depends on the rotational stiffness of the top hinge. Therefore at first the moment-rotation relation of cracked ferrocement with chicken mesh will be derived, the results of which are later used in the calculation.

2.1. Derivation of the moment-rotation relation of cracked ferrocement with chicken mesh reinforcement

A characteristic feature of chicken mesh is that the reinforcement ratio is not for each section the same (Fig. 19).

![Fig. 19. Variation of the reinforcement ratio in ferrocement with chicken mesh.](image-url)

Due to the hexagonal structure, two areas can be distinguished. It can be derived, that the reinforcement ratio over the length $l_1$ is only 50%
of that over \( l_2 \). However, in the uncracked stage this does not lead to significant differences in stiffness for the combined material for the low reinforcement ratios considered (\( \leq 2\% \)), so that it may be presumed that no preference exists for one of the areas with regard to the arising of cracks.

Cracking over the lengths \( l_1 \) or \( l_2 \) leads to different mechanisms. It will be investigated separately in what deformation the effect of cracking in both areas results.

I. Cracking within the length \( l_1 \):

Since the surface of the steel is rather smooth, the capacity to transfer bond stresses is neglectable, so that it is reasonable to presume, that the steel wire AB is subjected to a uniform stress over the full length \( l_1 \).

Furtheron the anchoring at both sides of AB is of high quality, due to the contact pressure between steel and concrete, increasing the bond (friction) between the two materials. A strain distribution as represented in Fig. 20 can be expected.

![Fig. 20. The effect of cracking within the length \( l_1 \).](image)

As a result the total displacement, due to a force \( F \) in the wire AB, is:

\[
\delta_{AB} = \frac{F \cdot l_{1}}{E_{a} \cdot (2A)}
\] (8)
II. Cracking within the length $\lambda_2$.

When a crack arises over the length $\lambda_2$ a mechanism will occur as represented in Fig. 21, due to the fact that the steel wires have a comparably low bending stiffness. When the external load is equal to that in the preceding case I, this leads to forces $1/2 \, F$ in both wires BC and BD. The total elongation of BC is:

$$\delta_{BC} = \frac{1/2 \, F \, \lambda_1}{E_a \, A}$$

(9)

This result is equal to that, found in case I, so that it seems reasonably to suppose that the place of the crack is not of influence on the deformation after cracking.

After the formation of cracks the steel is stressed. A section with a unit area $1 \, \text{mm}^2$ contains an amount of reinforcement of $\rho \, \text{mm}^2$. (Fig. 22).
When a stress $\sigma$ acts on the section, the strain after cracking can be calculated by means of (8) and (9):

$$
\varepsilon = \frac{\Delta l}{l_0} = \frac{\sigma}{E_a \rho}.
$$

So the stiffness of the cracked ferrocement is equal to:

$$
E_{fc} = \frac{\sigma}{\varepsilon} = E_a \rho = n\rho E_c
$$

(10)

This results in stress and strain distributions as presented in Fig. 23.

Fig. 23. Stress and strain distributions in cracked ferrocement.

The condition of compatibility of strains is fulfilled by the relation:

$$
\frac{\varepsilon_c}{\varepsilon_o} = \frac{x}{d-x}
$$

(11)

The condition of equilibrium results in:

$$
\frac{1}{2} \varepsilon_c E_c (d-x) = \frac{1}{2} (4d^2 \rho^2 + 4(1-np)(d^2 \rho))
$$

(12)

Combination of (11) and (12) yields an expression for the depth of the compression zone:

$$
x = \frac{2dn\rho + \sqrt{4d^2 n^2 \rho + 4(1-np)(d^2 \rho)}}{2(1-np)}
$$

(13)

The internal moment is equal to:

$$
M = \frac{2}{3} dB \int_0^x \frac{1}{2} x \varepsilon_b E_c = \frac{1}{3} dB \int_0^x \varepsilon_b E_c b
$$
So the concrete stress in the top of the section is:

\[ \varepsilon_c = \frac{3M}{bdxE_c} \]

The curvature \( K \) can be expressed as:

\[ K = \frac{\varepsilon_c}{x} = \frac{3M}{bdx^2E_c} \]

The total rotation \( \psi \) over the range \( l_1 \) (Fig. 19) is:

\[ \psi = K.l_1 = \left( \frac{3}{dx^2E_c} \right) M \]

or,

\[ \psi = C_0M \quad \text{with} \quad C_0 = \frac{3}{bdx^2E_c} \]  \hspace{1cm} (14)

The total rotation which is necessary in a cracked element to reach the cracking moment again depends on the number of cracks, and can be put equal to:

\[ \psi_{cr} = \alpha.C_0M_{cr} \]

in which \( \alpha \) is the number of cracks (Fig. 24)

---

Fig. 24. Moment-rotation relation, dependent on the number of cracks.
2.2. **Calculation of the load-deformation relation of the roof**

To be able to calculate the load-deformation relation of the roof, the rotational stiffness of the top hinge must be taken into account. In the case under consideration (Fig. 25), the maximum rotation $\phi$, that can occur without the arising of new cracks, is:

$$\phi_{\text{max}} = \frac{c_0}{2} M_{\text{cr}}$$  \hspace{1cm} (15)

and after $\alpha$ cracks (in which $\alpha$ is the total number of cracks at both sides of the axis of symmetry):

$$\phi_{\text{max, } \alpha} = \frac{\alpha c_0}{2} M_{\text{cr}}$$  \hspace{1cm} (16)

![Fig. 25. Rotation of the top hinge.](image)

So, if the number of cracks in the top is equal to $\alpha$, the relation between the moment in the top $M_o$ and the rotation $\phi$ is:

$$M_o = \frac{2\phi}{\alpha c_0}$$  \hspace{1cm} (17)

This relation is valid until $M_o$ reaches again the value $M_{\text{cr}}$. 
The decrease of the rotational stiffness of the top hinge as a function of the number of cracks is represented in Fig. 26.

Fig. 26. Rotation stiffness of the hinge as a function of the number of cracks at the top.

The distribution of forces in the roof can now be calculated (Fig. 27).

Fig. 27. Deformation of the roof as a function of the load.

Considerations of symmetry lead to the condition:

\[ \delta_h = 0 \]

Substitution of this value in (11) gives:

\[ N = 0.702 \, P - 1.602 \, \frac{M_o}{R} \quad \text{(18)} \]

The rotation \( \phi \) can be obtained by (18) and (3):

\[ \phi = \frac{R^2}{EI} \left\{ \frac{-M_o}{R} \cdot 0.657 + 0.100 \, P \right\} \quad \text{(19)} \]

After one crack the relation between \( M_o \) and \( \phi \) is given by (17). Combining this expression with (19) the relation between \( M_o \) and \( P \) is obtained:

\[ M_o = \frac{0.100 \, R^2 \, P}{0.657 \, R + \frac{C \, EI}{2}} \]
In the same way it can be derived that, when \( \alpha \) cracks are formed in the top, this relation is expressed by:

\[
M_{o,\alpha} = C_{Ra} \cdot P \quad C_{Ra} = \frac{0.100 \frac{R^2}{\alpha C_o EI}}{0.657R + \frac{1}{2}}
\]

which is the general relation between the moment in the top and the external load \( P \).

The relation between \( P \) and the vertical displacement \( \delta_v \) can be obtained from (2) and (20):

\[
\delta_{v,\alpha} = \frac{PR^3}{EI} \left(0.042 - 0.199 \frac{C_{Ra}}{R} \right)
\]

So, when the number of cracks is known, the distribution of moments in the roof can be calculated (Fig. 28).

![Fig. 28. Distribution of moments after cracking in the top.](image)

When the number of cracks in the top of the roof increases, the rotational stiffness decreases, so that the chance that in another part of the construction the cracking moment is reached is enhanced. As far as this is concerned, the moment in the roof for \( \Theta = 45^\circ \) seems to be a good criterion. The calculation of the load deflection curve can be carried out stepwise. At first the load at which the first crack is formed in the top is calculated by means of equation (4): the vertical displacement is obtained by (6).

Next the relation between \( M_o \) and \( P \) for the construction with one top-crack is calculated by means of (14) and (20). The maximum load which can be reached, without the formation of new top-cracks, is when \( M_o \) is about to reach the value \( M_{cr} \). For this value of \( P \) it is controlled, whether the moment \( M_{45^\circ} \) has exceeded the cracking moment. If not, the vertical displacement of the top can be calculated, combining (18), (20) and (2).
For reasons of symmetry it is assumed that two other cracks are formed at the end of this loading stage, so that the same procedure can be followed for $\alpha = 3$. When in a certain loading stage the moment $M_{45}$ is exceeded, the stiffness of the roof is appreciably reduced (Fig. 29),

![Fig. 29. The roof after the formation of two hinges.](image)

so that it is assumed that the ultimate capacity is reached.

In this way characteristic points of the load-vertical displacement curve can be constructed (secantial method). (Fig. 30).

![Fig. 30. Construction of characteristic points of the load-vertical displacement curve (secantial method).](image)

2.3. Calculation of the load-vertical displacement curve for the roofs 2.4 and 2.6

To compare the results of the theoretical calculation with the values, obtained in the test, the calculation method is applied to the specimen 2.4 and 2.6.

Specimen 2.4.

Data: $d = 12$ mm.

- 4 layers of mesh: reinforcement ratio $\rho = 0.0134$
- $f_{bu} = 10.6 \text{ N/mm}^2$ (determined by control specimens)
- $E_c = 48.3 \times 10^3$ (" " " " )
- $E_I = 7.2 \times 10^3$
- $M_{cr} = f_{bu}W = 254400 \text{ Nmm}$
The results of the calculations are summarized in Table 4.1.

<table>
<thead>
<tr>
<th>α</th>
<th>x(mm)</th>
<th>C₀</th>
<th>Cᵦ</th>
<th>Pmax (N)</th>
<th>δᵥ(mm)</th>
<th>M₄5₀° (Nmm)</th>
<th>Remark</th>
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<td>19.97</td>
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Table 4.1. Results of calculation for specimen 2.4.

Specimen 2.6.

Data: d = 14 mm

6 layers of mesh: reinforcement ratio ρ = 0.020
fᵦᵤ = 10.6 N/mm² (determined by control specimen)

\[ E_c = 48.3 \times 10^3 \text{ N/mm}^2 \]

\[ EI = 1.104 \times 10^{10} \text{ Nmm}^2 \]

\[ M_{cr} = 346267 \text{ Nmm} \]

The results of the calculations are summarized in Table 4.2.

<table>
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<th>α</th>
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<th>C₀</th>
<th>Cᵦ</th>
<th>Pmax (N)</th>
<th>δᵥ(mm)</th>
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Table 4.2. Results of calculation for specimen 2.6.
In fig. 31 the theoretical load-vertical displacement curve is compared with the experimental one. It may be concluded that a reasonable agreement with the experimental curves is obtained. High accuracy may not be expected, since the thickness of the roof, which is a highly influential parameter, cannot be established very accurately. Fig. 13 showed a picture of the fully developed top hinge of specimen 2.4. As can be seen the number of cracks is in reasonably good agreement with the predicted one (13). The calculation method, as suggested here, can be used to get an impression of the strength of roofs with other dimensions.
5. Conclusions

1. The test results indicated that the specimens with monolithic joints were not as strong as those with the bolted joints. A slight modification of the monolithic joint in combination with split bamboo rods, to assure a great inner lever arm after cracking, gave the best results while the costs were low compared with the specimens with bolted joints. The load carrying capacity of the elements tested, varied from 125 kg (4 layers of mesh, monolithic joint) to 340 kg (6 layers of mesh, improved monolithic joint). As such the ultimate resistance of the last type of specimen is more than 4 times the service load of 75 kg (a man’s weight).

2. The effect of the soaked split bamboo, separating the top and bottom layers of mesh, has an important influence of the stiffness after cracking.

3. The failure of the ferrocement roofing is very gradual, which is in sharp contrast to the type of failure that happens in the case of asbestos cement sheet.

4. The system of ferrocement roofing developed, is quite economic. The thickness of the element is just 12-14 mm for a span of 3 m. Besides, only chicken mesh, which is the cheapest kind of wire mesh, is used for reinforcing the shell.

5. The calculation method developed to predict the behaviour of the structure up to failure, fits the test results rather well and could be used to get an impression of the ultimate resistance when other dimensions are to be considered.

6. The principle of prefabrication of elements, which are manageable and transportable by hand, joined together on the site to a complete structure, is rather well practicable in ferrocement, provided that the joints are carefully handled.
6. **Suggestions**

1. The construction on a self-help basis is one of the most important aspects of this roofing form. However for the construction to be carried out on a self-help basis certain changes and modifications have to be incorporated. Considering the mould, in the self-help construction, it may not be feasible to make an elaborate shuttering as made in the laboratory. Instead, a sand mould could be made. This can be done by shaping a basis-mould of a sand-clay mixture which is topped with a layer of cement mortar. The elements can be casted on this mould.

2. When edge beams are used the supports could be restricted to columns in places where the edge beams meet (fig. 31). The size of the edge beams can be chosen independent on the thickness of the roof elements. In this way an extended field of application is born.

![Fig. 31. Ferrocement roof with edge beams, only supported by columns.](image)

3. To facilitate the manufacturing process a modified shape of the roof elements can be considered. This is shown in fig. 32. This system consists of a gutter unit and a shell unit. Here the shell unit is of greater length than the gutter unit which is made smaller and slightly thicker.
Fig. 32. Roof with gutter elements.
4. Edge beams could be made using ferrocement shuttering elements (fig. 33).

Fig. 33. Production of edge beams using ferrocement shuttering elements.
5. To make the joints, an adequate scaffolding should be used for the erection of the elements, to guarantee a good fitting of all elements involved (fig. 11).

To simplify the labour, inherent to the grouting of the joints, a retarder, applied on the mould, should be used in places where the fresh grout should have some bond on the existing ferrocement element. Furtheron the surface of the freshly casted concrete of the element can be roughened by a steel brush. The joints should be kept moist during at least 4 days at both (outer and inner) sides of the shell by using soaked rags or cell tissue, covered by a material that prevents it from drying.

Afterwards it is advisable to keep the rags in position for another four days to prevent too quick drying out.
APPENDIX I

Composition of the concrete mixture

Per m³: 1680 kg sand according to sieve analysis
415 kg cement
208 kg water (w.c.f. = 0.5)
25 l air

Total 2303 kg

Sieve analysis

<table>
<thead>
<tr>
<th>Opening diameter (mm)</th>
<th>percentage</th>
</tr>
</thead>
<tbody>
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</tr>
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8. APPENDIX II

Deformation of roof 4.6