Seventh Canadian Masonry Symposium
McMaster University
Hamilton, Ontario June 4-7, 1995

PREFABRICATED FACADE ELEMENTS IN PRESTRESSED MASONRY

A.J. Hogeslag[1] en D.R.W. Martens[2]

ABSTRACT

This paper presents the results of an experimental and theoretical research on the possibilities of prefabricated facade elements in prestressed masonry. Deformation behaviour and flexural strength have been studied by means of bending tests. The bond strength between reinforcement and masonry has been determined using pull-out tests. The suitability of non-metallic reinforcement has also been evaluated. In conclusion, the possibilities of applying prefabricated facade elements in prestressed masonry are indicated as well as their restrictions.

INTRODUCTION

In contemporary architecture masonry is still frequently used as a building material for facade elements, thanks to its generally recognized favourable properties regarding appearance, durability and cost price. Tough working conditions and the dependence on weather conditions in the traditional execution of brickwork constructions often lead to an inefficient way of building.

An improvement of working conditions may be realised by prefabrication and automation. Additional advantages of an industrialised process are: short construction time, high quality, practically independent of weather conditions, and employment of unskilled and semiskilled workers.

There is already a production technique for the prefabrication of facade elements in masonry, which involves attaching a load bearing internal wall leaf of reinforced concrete to the back of the masonry element. A major disadvantage of this type of prefab elements is the heavy weight. This problem may be solved by developing a facade element without a load bearing concrete leaf. In this case the masonry itself should be able to withstand the various loads to which it is subjected during its entire life span (manufacture, transport, erection, thermal movements, gravity forces and wind load). Because of the

[1]Professsor at Delft University of Technology, Faculty of Architecture
[2]Lecturer at Delft University of Technology, Faculty of Architecture

limited tensile strength of masonry this solution can only be found by reinforcing or prestressing the element. In order to limit crack formation as well as deformations it is recommended to prestress masonry. In an industrialised production process advantage can be taken of the technique of pretensioning.
In the research described in this paper the possibilities of applying prefabricated facade elements in prestressed masonry (pretensioning) have been analysed experimentally and theoretically. The evaluation of the test results has been translated into design rules. The possibilities and restrictions in architectural design and in construction have also been studied.

EXPERIMENTAL RESEARCH

Bending tests perpendicular to the bed joints
In order to test the flexural behaviour of prestressed masonry, four-point-bending tests were carried out on 3 masonry elements with a width of 1095 mm, a height of 2810 mm and a thickness of 100 mm (Fig. 1).

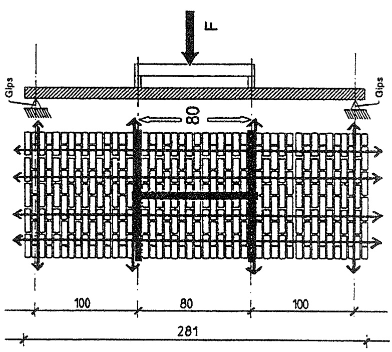


Fig. 1. Test arrangement at bending tests perpendicular to the bed joints

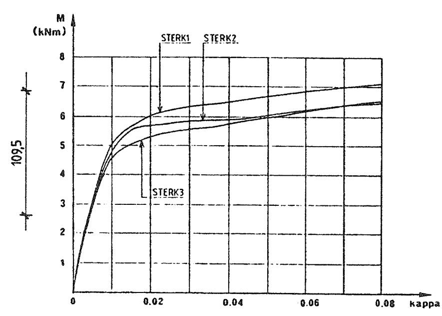


Fig. 2. Experimental M-κ-diagrams at bending tests perpendicular to the bed joints

The test elements were made by placing the bricks in stretcher bond on a horizontal mould. Four steel prestressing tendons with a diameter of 5.2 mm were then put lengthwise through the holes in the bricks (Fig. 1). The total prestressing force was 104 kN. Four prestressing tendons were placed in transverse direction. After pretensioning, the joints and holes were filled up with a high-quality grout (Cuglaton SK-B). When this grout had sufficiently hardened (after approximately 36 hours), the prestresssing force was transmitted to the masonry element. After 28 days deformation-controlled four-point- bending tests were carried out. The deflection was measured at 6 different points. Fig. 2 shows the moment-curvature diagrams (M-κ-diagrams) as determined experimentally.

Bending tests parallel to the bed joints
In an analogous way deformation-controlled four-point-bending tests were carried out on masonry elements which measured 730 mm (width) x 1640 mm (height) x 100 mm (thickness) (Fig. 3). Now, three prestressing tendons of 5.2 mm in diameter were put into the bed joints. The total prestressing force was 78 kN. The M-κ-diagrams determined experimentally are shown in Fig. 4.

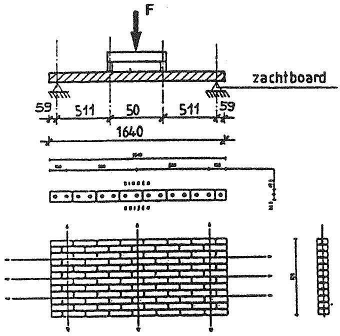


Fig. 3. Test arrangement at bending tests parallel to bed joints

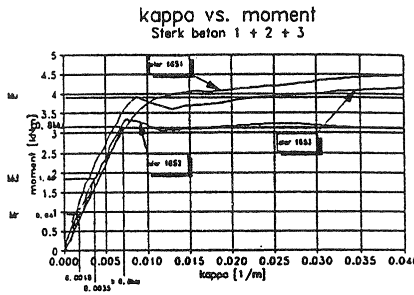


Fig. 4. Experimental M-κ-diagrams at bending parallel to the bed joints

Pull-out tests with steel tendons
As the bending tests parallel to the bed joints showed that there was debonding of the reinforcement, pull-out tests were carried out to evaluate the bond strength between masonry and reinforcement. These tests were done both on steel tendons put into holes in the masonry and on tendons embedded in the bed joints (Fig. 5). The test elements were cut from the masonry elements which had been tested on bending. In the pull-out tests only those tendons were tested that had not been loaded in the bending tests. The test arrangement is shown in Fig. 6, and the results of the pull-out tests are shown in Fig. 7.

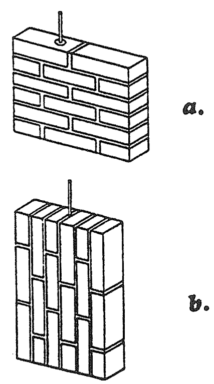


Fig. 5.a. Test elements with rods through holes in the bricks
b. Test elements with rods in the bed joints

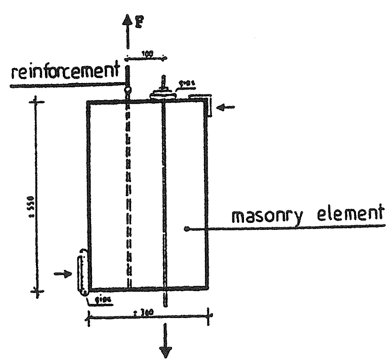
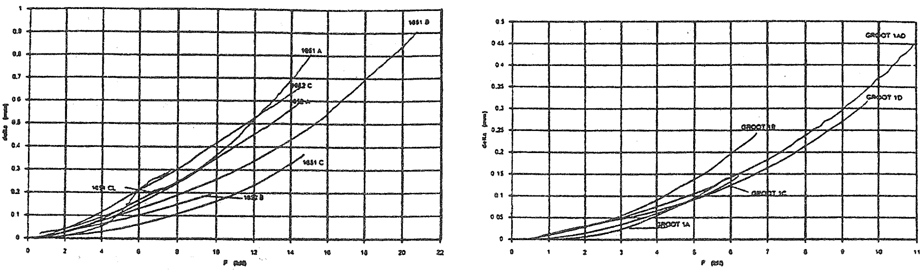


Fig. 6. Test arrangement of the pull-out tests



a. b.

Fig. 7.a. Force-deformation diagrams of the pull-out tests with tendons through the holes (F-δ-diagrams)
b. Force-deformation diagrams of the pull-out tests with tendons in the bed joints.

Pull-out tests with carbon fibre reinforced plastic rods
As masonry is a porous material the risk of corrosion has to be taken into account. A solution to this problem may either be found in the application of hot-dip galvanised steel prestressing tendons or in the use of non-metallic prestressing elements. In view of the favourable properties of carbon it was decided to conduct pull-out tests on carbon fibre reinforced plastic rods with a diameter of 5 mm (Dialed, Mitsubishi Kasei). In order to evaluate the effects of geometry, test elements were made by hand in different sizes. Fig. 8 shows the results of the pull-out tests on elements with a height of 65 mm.

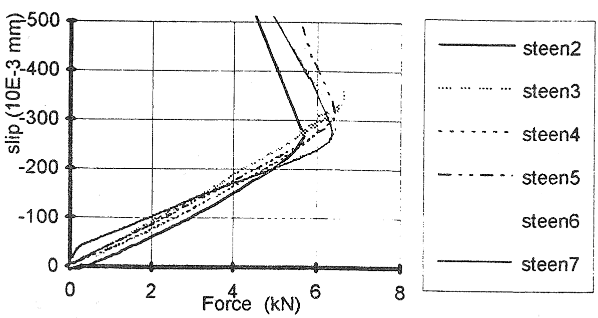


Fig. 8. Results of the pull-out tests with carbon fibre reinforced plastic rods.

EVALUATION OF THE TEST RESULTS

Bending tests perpendicular to the bed joints
From the results of the bending tests it can be deduced that the flexural strength equals 0.3 N/mm². This corresponds fairly well with the value of the bond strength between joint and brick as stated in the Dutch Standard NEN 6790. Based on the deformation

measurings in the elastic region, the average value of the modulus of elasticity was calculated to be 8000 N/mm².

As appears from the force-deformation diagram (Fig. 2), the prestressed masonry elements show a good ductile behaviour.

The theoretical calculation of the ultimate bending moment and the moment-curvature-relationship was based on the material properties derived from compressive tests on the bricks and from tensile tests on the prestressing tendons. The mechanical properties of the grout (Cuglaton SK-B) were provided by the manufacturer. For masonry a parabolic stress-strain relationship was introduced, whereas a bilinear law was applied for the σ-ε-diagram of the prestressing steel. The initial calculations have shown that the model of tension stiffening which is valid in concrete constructions gave no satisfactory results for masonry (Fig. 9).

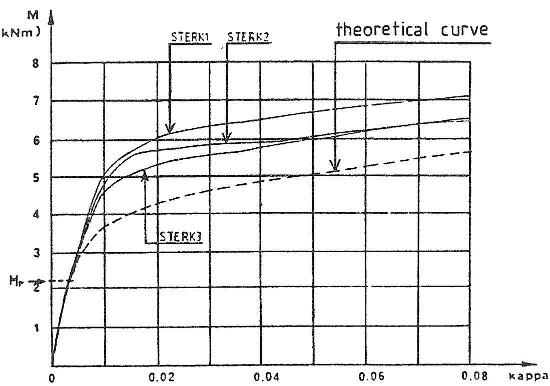


Fig. 9. Theoretical M-κ-diagram with the model of tension stiffening in concrete and the M-κ-curves determined experimentally.

As a consequence, a new model of tension stiffening was developed for masonry, taking into account that the tensile strength of bricks is greater than the bond strength between mortar and bricks. This means that crack spacing is always equal to the distance between two joints. The numerical implementation of this principle leads to the following formula for the modified bending moment M, taking into account tension stiffening (Fig. 10.):

$$M = \frac{M_0 \cdot M_g}{\mu \, . M_0 + (1-\mu) . M_g} \qquad [1]$$

M_0: bending moment calculated on the basis of uncracked section
M_g: bending moment calculated on the basis of completely cracked section

$$\mu = \beta \, . \left(1 - \frac{2}{3} \cdot \frac{Ld}{Lcr}\right) \qquad \text{if } Lcr > 2\, Ld \qquad [2]$$

$$\mu = \beta \cdot \frac{2}{3} \cdot \left[\frac{Lcr}{Ld} - \frac{1}{4} \cdot \left(\frac{Lcr}{Ld} \right) \right]^{2} \quad \text{if } Lcr < 2\,Ld \qquad [3]$$

with

Lcr: crack spacing (distance between joints)
Ld : anchorage length (function of steel stress)

$\beta = 1$ if steel stress is smaller than or equal to yield stress [4]

$$\beta = \left[\frac{\kappa u - \kappa}{\kappa u - \kappa y} \right]^{1/2} \quad \text{if steel stress exceeds yield stress} \qquad [5]$$

with κy: curvature at yielding of reinforcement
κu: curvature at ultimate bending moment

When applying this model for tension stiffening of masonry, a theoretical M-κ-diagram was obtained which fairly matches the curves determined experimentally (Fig. 11).

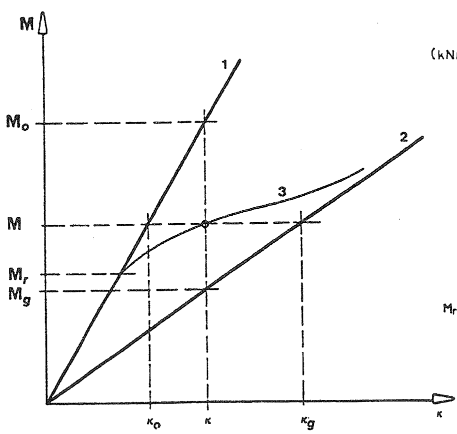


Fig. 10. Modified moment taking into account tension stiffening

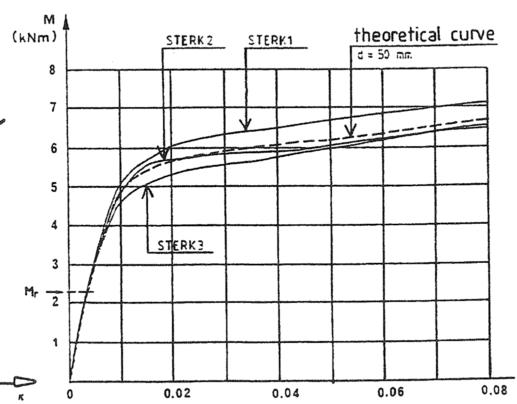


Fig. 11. Theoretically modified M-κ-diagram and experimental M-κ-curves

Bending tests parallel to the bed joints
In the bending tests parallel to the bed joints the same theoretical calculations were made as with the bending tests perpendicular to the bed joints. The ultimate bending moment determined experimentally, now appeared to be smaller than the theoretical value. As can be derived from the moment-curvature diagrams (fig. 4), this is due to the debonding of the reinforcement. This conclusion led to further research on the bond strength between reinforcement and masonry.

Pull-out tests with steel tendons
Analysis of the test results shows that when the reinforcement is perpendicular to the bed joints, the average shear stress at the ultimate bending moment is approximately twice as large (2.3 N/mm^2) as is the case with reinforcement in the bed joints (1.25 N/mm^2). This difference in bond strength can be explained by the fact that the mortar covering on the reinforcement is much thicker when reinforcement has been put into the bricks through holes (12.5 mm) than in the case of reinforcement in the bed joints (2.5 mm) (Fig. 12). Due to its thin covering the quality of this mortar is slightly inferior, resulting in a reduction of the bond strength. Furthermore, it appears that when the reinforcement is put in the bed joints, an eccentric position of the reinforcement has a greater negative effect on the bond strength than in the case with reinforcement through holes in the bricks.

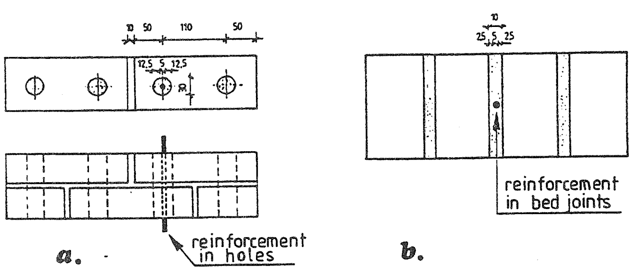


Fig. 12. Mortar covering with reinforcement perpendicular to the bed joints (a) and with reinforcement in the bed joints (b).

In a pull-out test the distribution of the shear stresses along the tendon is quite complex. So far, no general theoretical solution is known which satisfies all the boundary conditions. For reinforced concrete a number of solutions simplifying the actual stress distribution have been suggested, as shown in Fig. 13. All models are based on a τ-δ-relation, which links the relative slip δ of the reinforcement in relation to the concrete and to the corresponding shear stress τ. In the theoretical study the validity of the 5 different models shown in Fig. 14 was evaluated. To this end a comparison was made between the theoretical force-deformation diagrams and the diagram of a representative test 1652A (Fig. 15), which indicated that the Noakowski and Vandewalle models come closest to reality. The model with constant shear stress, forming the basis of most regulations, does not satisfactorily correspond with reality. In order to formulate design rules on a solid scientific basis, further research will be required to determine the maximum average shear stress based on the mechanical properties of reinforcement and brick respectively.

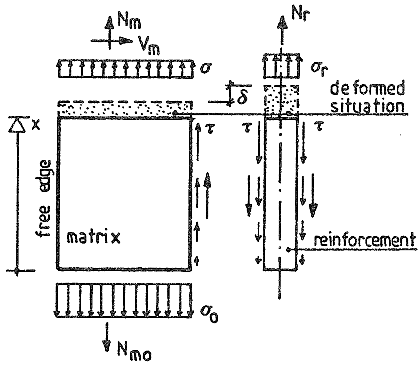


Fig. 13. Simplified deformation and stress distributions in pull-out tests

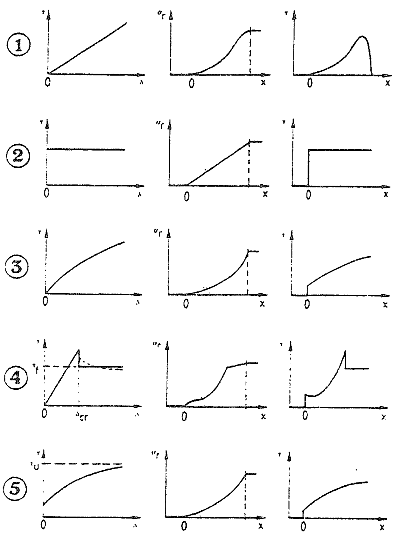


Fig. 14. Different τ-δ-models
1. linear model
2. constant shear stress
3. Noakowski model
4. Naaman model
5. Vandewalle model

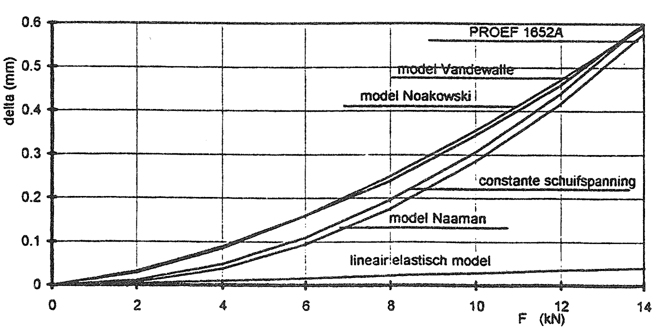


Fig. 15. Theoretical and experimental F-δ-curves

Pull-out tests with carbon fibre reinforced plastic rods
From an analysis of the results of the pull-out tests with carbon fibre reinforced plastic rods, it follows that the average shear stress at the ultimate bending moment is approximately 3 N/mm², which is considerably higher than in the case with steel prestressing tendons. The F-δ–diagrams have been calculated theoretically in accordance with the Noakowski and Vandewalle models. Here, too, the theoretical and experimental curves fairly agree.
In 1994 Granzier did a numerical research into the possibilities of prestressing masonry with carbon fibre reinforced plastic rods. In a follow-up project experiments will be carried out to investigate whether or not splitting of the masonry occurs in prestressing facade elements with carbon fibre reinforced plastic rods.

DESIGN RULES

Basic assumptions
With prefabricated facade elements in prestressed masonry limited crack formation under wind load is acceptable. The normative load seldom occurs and in view of its transient character, the cracks will open up for a very short time only, provided that this does not cause any plastic deformations. Therefore, it is recommended that all materials should behave elastically in the design state.

Design formulas
The design formulas are based on the bilinear stress-strain diagrams for masonry and prestressing steel according to Dutch Standards (NEN 6790 and NEN 6720) (Fig. 16).

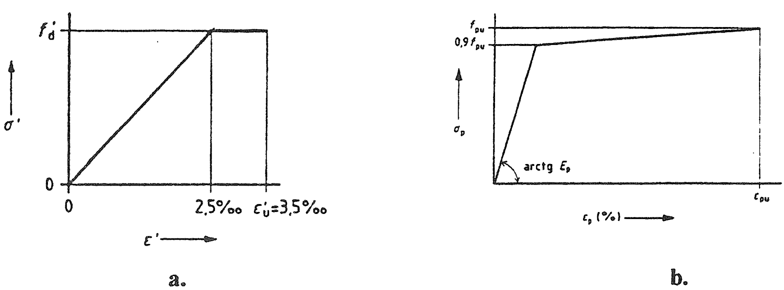


Fig. 16. a. σ-ε-diagram for masonry according to NEN 6790
b. σ-ε-diagram for prestressing steel according to NEN 6720

The minimum effective depth of the masonry element can be calculated as follows:

$$d = \beta \cdot \sqrt{\frac{Md}{b \cdot f'md}} \qquad [6]$$

$$\beta = \frac{3}{Ep} \cdot \frac{(360 - 320.\eta) . fpud + Ep}{\sqrt{(540 - 480.\eta) . fpud + Ep}} \quad [7]$$

with

Md : design bending moment
f'md: design compressive strength of masonry
η : loss of prestress
Ep : modulus of elasticity of prestressing steel

The depth of the compressive face x, the lever arm z, the recommended prestressing reinforcement Ap and the prestressing force are then determined by the following formulas:

$$x = \frac{Ep . d}{(360 - \eta . 320) . fpud + Ep} \quad [8]$$

$$z = d - x/3 \quad [9]$$

$$Ap = \frac{Md}{0.9 . fpud . z} \quad [10]$$

$$P0 = 0.8 . fpud . Ap \quad [11]$$

On the basis of these formulas it is possible to compose design-diagrams and design-tables (see table 1).

d = 45 mm			d = 50 mm			d = 55 mm		
Md / b . f'md (mm²)	100 . Ap / b . f'md (mm3/N)	P0 / b . f'md (mm)	Md / b . f'md (mm²)	100 . Ap / b . f'md (mm3/N)	P0 / b . f'md (mm)	Md / b . f'md (mm²)	100 . Ap / b . f'md (mm3/N)	P0 / b . f'md (mm)
50	.0934	1.20	50	.0841	1.08	50	.0764	.98
100	.1869	2.41	100	.1682	2.17	100	.1529	1.97
150	.2803	3.61	150	.2523	3.25	150	.2293	2.95
200	.3738	4.81	200	.3364	4.33	200	.3058	3.94
250	.4672	6.02	250	.4205	5.42	250	.3822	4.92
300	.5606	7.22	300	.5046	6.50	300	.4587	5.91
350	.6541	8.42	350	.5887	7.58	350	.5351	6.89
400	.7475	9.63	400	.6728	8.67	400	.6115	7.88
450	.8409	10.83	450	.7569	9.75	450	.6880	8.86
500	-	-	500	.8410	10.83	500	.7644	9.85
550	-	-	550	.9251	11.92	550	.8409	10.83
600	-	-	600	1.0092	13.00	600	.9173	11.82
650	-	-	650	-	-	650	.9938	12.80
700	-	-	700	-	-	700	1.0702	13.78

Table 1. Design table for facade elements prestressed with steel tendons FeP 1770 and with a loss of prestress η = 20%

Additional calculations

After designing the prestressed masonry element, the capacity of shear strength, the bond strength (transfer length and anchorage length) and the deformation should be evaluated. Further experimental research will be required in order to formulate more detailed calculation rules.

ASPECTS OF EXECUTION

Production

An essential condition for a profitable production of facade elements in prestressed masonry is that the application of prestressing tendons should be simple. This can be achieved in several ways: by putting the reinforcement through holes in the bricks, by putting the reinforcement in slits cut into the bricks, or by putting the reinforcement in the joints between the bricks. The latter method is most suitable for automation, but the disadvantage of placing the prestressing tendons in this way is in the restriction that not all brickwork bonds can be used.

In the production process the geometrical restrictions of the elements must be taken into account in view of the capacity of cranes and the requirements of transport.

Erection process

For erection purposes the width of the joints should be determined in accordance with the accepted geometric tolerances of the facade elements.

To ensure structural integrity, the anchors should be flexible enough to allow unrestrained thermal and moisture movements of the facade element. In addition, attention should be paid to the durability af the anchors (corrosion).

ARCHITECTURAL IMPACT

The design of facade elements

Prestressed masonry is to be considered a new material with its own design aspects. On the one hand there is the inevitable presence of joints, whereas on the other hand new types of brickwork bonds can be applied more economically than in the case of traditional non-prestressed masonry.

The most economic form of an element is the plane rectangle. In the case of elements with window openings, the sizes of the openings are restricted to certain maximum values. Subsequently, it is recommended to consider the architectural design in such a way that the facade can be composed of rectangular full elements.

Detailing

Good detailing of the joints and the edges of facade elements is a must to achieve an esthetic whole. Attention should be paid to these aspects right from the initial design stage.

CONCLUSIONS

Experimental and theoretical research has shown that prefabrication of facade elements in prestressed masonry is technically feasible. The elements show a satisfactory ductility in bending. Practical design rules have been drawn up to determine the dimensions of the facade elements. Further experimental research will be required to evaluate and refine the calculation methods. Particular attention should be paid to the bonding between reinforcement and masonry. The mortar properties will have to be geared to the properties of the reinforcement (either steel prestressing tendons or carbon fibre reinforced plastic rods) and of the bricks used.
Contacts are being made with the industry in order to develop a suitable automated production method.
The approach to architectural design should be based on the idea that prestressed masonry is a new material whose properties differ from those of traditional masonry. Now it is up to the architects to discover the possiblities of this new building material and to exploit it in an optimal and creative way.

ACKNOWLEDGMENTS

The authors wish to thank SUMITOMO EUROPE BV for providing the carbon fibre reinforced plastic rods needed for this research project.

REFERENCES

Granzier S.H.A., Het inleiden van voorspanning in metselwerk, Numeriek onderzoek naar de mogelijkheden van het voorspannen van metselwerk met, danwel zonder aanhechting (VMA en VZA), TU Eindhoven, oktober 1994.

Hogeslag A.J., Martens D.R.W., Geprefabriceerde gevelelementen in voorgespannen metselwerk, Delft, herwerkte versie, oktober 1994.

Hogeslag A.J., Martens D.R.W., Hechting tussen wapening en metselwerk, Delft, augustus 1994.

NEN 6790, Steenconstructies (Nederlandse Norm), 1991

NEN 6720, Betonconstructies (Nederlandse Norm), 1991

Walraeven J.C., Galjaard J.C., Voorgespannen Beton, dictaat, Technische Universiteit Delft, januari 1993.