PARTIALLY REINFORCED CONCRETE MASONRY WALLS SUBJECTED TO LATERAL LOADS

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ABSTRACT The paper deals with lateral load tests on partially reinforced and unreinforced hollow concrete block masonry walls. The effective width of the reinforced cores is investigated. Theoretical estimates or cracking loads, deflections (pre- and post-cracking) and ultimate loads are compared with the observed values.

1. INTRODUCTION

Masonry walls constructed of hollow concrete blocks with reinforced cores widely spaced are referred to as partially reinforced. Such walls span between top and bottom supports and resist wind load by vertical flexural action. Portion of the masonry each side of the grouted reinforced core may be considered to act with it and the Draft Australian Masonry Code (1) limits the effective width of this beam to 800mm, 4 times the thickness of the wall (t_w), or to the spacing of the reinforced cores whichever is least. There is little experimental justification for these requirements and hence a preliminary experimental program was carried out at the Masonry Research Centre, Deakin University on partially reinforced and plain concrete block masonry walls (2). All walls were tested in simple vertical flexure under uniform lateral pressure and load deflection behaviour was observed to ultimate load. The effective width of masonry associated with each reinforced core was assessed and theoretical deflections after cracking were obtained using a finite element program that allowed for variation of cross-sectional properties from course to course due to varying position of the reinforcement and propagation of cracking.

Theoretical estimates of deflections and ultimate load are compared with observed values.

2. DESCRIPTION OF WALLS

Table 1 summarises the partially reinforced and unreinforced walls in the test program. The reinforced walls had vertical reinforcement placed in cores at 1800 mm centres, a spacing much greater than normally permitted in the Australian code and hence the term partially reinforced. Reinforced cores were filled with 20 MPa grout and contained either a central C16 bar or 2-C16 bars placed centrally at the bottom but splayed outwards to be close to the wall faces at the top. Bond beams were located at top and bottom courses. Masonry was laid in face shell bedding with 1 cement : 1 lime : 6 sand parts by volume for the mortar. Unreinforced walls were laid in face shell bedding using mortar consisting of 1 cement : 5 sand with methol cellulose additive in the water.

3. WALL TESTS

All reinforced walls were tested by bolting the top and bottom bond beam back to a strong reaction frame and applying pressure incrementally in an air-bag inserted between the back of the wall and the reaction frame. Deflections were measured by dial gauges attached to an independent suspended frame.

The deflections of reinforced walls were fairly uniform across the length of the wall. In general an initial crack occurred along one bed joint, near mid-height and the width of the crack was not noticeably larger in the centre than at the ends.

As loading continued, subsequent cracks occurred at other bed joints until all bed joints from about 1/4 wall height to 3/4 wall height had cracked.

The load deflection curves were generally linear up to initial cracking with a short non-linear transition after cracking followed by a fairly linear section, at reduced stiffness, until ultimate load was approached. This behaviour is analysed theoretically in the following section. Results of wall tests are summarised in Table 2.

WALL NO.	SIZE OF MASONRY UNITS	HEIGHT	LENGTH	THICKNESS	NO. OF REINFORCED CORES	NO. OF C16 BARS PER CORE
A1	400x200x200	3200	2000	200	2	1 central
A2	400x200x200	3200	2000	200	2	2 splayed
A3	400x150x200	3200	2000	150	2	1 central
A4	400x150x200	3200	2000	150	2	2 splayed
A5	400x200x200	3200	3800	200	3	2 splayed
B	400x200x200	5400	7400	200	5	2 splayed
C1-C10	400x200x200	2600	2000	200		NIL

TABLE 1 SUMMARY OF WALL DETAILS

WALL NO.	FAILURE LOAD kPa FIRST CRACK	ULTIMATE	LOAD-DEFLN SCOPE S Pa/mm	REMARKS
A1	1.37	5.73	1168	
A2	0.68	6.90	1761	
A3	0.50	2.52	527	Grout did not penetrate properly
A4	0.54	5.05	388	Perpend joints cracked at 4.9 MPa
A5	1.04	5.39	1128	
B1	0.10	2.35	162	Loading discontinued for safety

TABLE 2 RESULTS FOR REINFORCED WALLS

Unreinforced walls were tested in a similar fashion to reinforced walls except that steel support beams were used instead of bond beams. Failure in the unreinforced walls occurred without any warning when a bed joint near mid-height cracked for the full length of the course.

On completion of each wall test the moduli of rupture of individual joints were measured by cutting the perpend joints at each end of the block and using a bond wrench. These walls had a practically linear load deflection behaviour up to failure. An elastic modulus of masonry was obtained by fitting a straight line to the experimental curve for some walls and using conventional bending theory. Results are summarised in Table 3.

WALL NO.	FAILURE LOAD OF WALL KPa	AV MODULUS OF RUPTURE OF JOINTS MPa	ELASTIC MODULUS GPa
C1	2.24	0.48	-
C2	2.31	0.49	-
C3	2.92	0.43	-
C4	1.74	0.43	3.99
C5	2.10	0.34	4.83
C6	1.96	0.44	-
C7	1.33	0.39	5.98
C8	1.16	0.39	4.70
C9	2.20	0.46	-
C10	2.66	0.48	8.08
MEAN	2.06	0.43	5.52

TABLE 3 RESULTS FOR PLAIN WALLS

4. THEORETICAL LOAD-DEFLECTION RESPONSE

Because of the wide spacing of vertical reinforcing it is unlikely that the full width of the wall will be effective in resisting lateral load. The Draft Australian Masonry Code specifies that partially reinforced walls shall be considered as separate reinforced beams with unreinforced masonry between them, the maximum width of beam generally being limited to 4 times the thickness of the wall. It is assumed here that load is resisted in vertical bending by cores acting as isolated beams having an effective width of k t_e, for interior beams and e + .5 kt for edge beams, where k is a constant to be determined. See Figure 1.

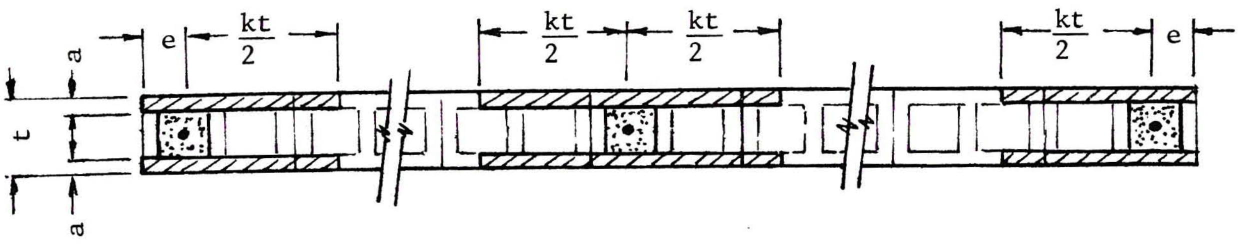

FIGURE 1 EFFECTIVE WIDTH OF BEAMS

The wall stiffness is dependent upon the concrete masonry units, the mortar in the joints, concrete grout in the cores and reinforcement steel in the cores. To simplify calculations the units and mortar were considered the one material-masonry. Properties of the composite section were obtained by transforming the steel and grout into masonry.

For the composite section:

$$E_m I_c = E_m \left(I_m + \frac{E_g}{E_m} I_g + \frac{E_s}{E_m} I_s\right)$$

$$\text{Or } E_m I_c = E_m I_m + E_g I_g + E_s I_s \qquad \text{........(1)}$$

Where, E_m = Elastic modulus of masonry = 5.5 GPa (from Table 3)
E_g = Elastic modulus of 20 MPa grout = 20 GPa
E_s = Elastic modulus of steel = 200 GPa
I_g = Second moment of area of grout (mm^4)
I_s = Second moment of area of steel (mm^4)
I_m = Second moment of area of face shell bedded masonry of width k t (mm)

$E_m I_c$ = Flexural rigidity of composite section N mm^2

I_m can be evaluated with reference to Fig. 1.

For two edge beams

$$I = 2 \cdot 1/12 \cdot (e + 0.5kt)\,[t^3 - (t-2a)^3]$$

$$I = (2e + kt)\frac{t^3}{12} \left[6\left(\frac{a}{t}\right) - 12\left(\frac{a}{t}\right)^2 + 8\left(\frac{a}{t}\right)^3\right]$$

For N interior beams

$$I = N\frac{kt^4}{12} \left[6\left(\frac{a}{t}\right) - 12\left(\frac{a}{t}\right)^2 + 8\left(\frac{a}{t}\right)^3\right]$$

Combining these for the composite section, with N interior beams

$$I_m = \frac{t^4}{12}\left[\frac{2e}{t} + k(N+1)\right]\left[6\left(\frac{a}{t}\right) - 12\left(\frac{a}{t}\right)^2 + 8\left(\frac{a}{t}\right)^3\right]$$

$$I_m = \frac{Ct^4}{12}\left[\frac{2e}{t} + k(N+1)\right] \qquad \ldots\ldots(2)$$

where $C = \left[6\left(\frac{a}{t}\right) - 12\left(\frac{a}{t}\right)^2 + 8\left(\frac{a}{t}\right)^3\right]$

The flexural rigidity of the composite section can be obtained from the initial slope (S) of the experimental load-deflection curve for the reinforced walls. For a wall of length L and height H

$$E_m I_c = \frac{5}{384} LH^4 S$$

$$E_m I_c = .013\ LH^4 S \qquad \ldots\ldots(3)$$

Experimental values of S(Pa/mm) are given in Table 2.

Combining equations 1, 2 and 3 gives:

$$k = \frac{1}{N+1}\left[12\,\frac{(.013\ LH^4 S - E_g I_g - E_s I_s)}{Ct^4\ E_m} - \frac{2e}{t}\right]$$

By substituting values of the various geometric properties and the experimental slope, S, the value of k has been determined for each test wall and is shown in Table 4.

WALL	k
A1	3.28
A2	6.55
A3	5.76
A4	2.55
A5	3.65
B1	4.78
AVERAGE	4.43

TABLE 4 EFFECTIVE WIDTH COEFFICIENT BASED ON OBSERVED LOAD-DEFLECTION SLOPE

The average value of 4.43 compares favourably with the value of 4 given in the Draft Australian Masonry Code. On the other hand if it is assumed the full length of the wall is effective an average calculated value of E_m = 2.614 GPa is obtained that does not compare favourably with the measured value of 5.2 given in Table 3.

4.1 Stress At First Cracking

For each wall, stress at midspan at first observed cracking load was computed on the following basis:

(1) Effective width of an interior beam equals 4t.

(2) Effective width of an edge beam equals e + 2t.

(3) The interior and edge beams receive loads proportional to their stiffnesses (so that deflections and also stresses in the edge and interior beams are same).

The computed values are shown in Table 5. The average value of 0.42 MPa for walls A1 .. A5 compares favourably with the average Modulus of Rupture of 0.43 MPa observed from unreinforced walls (Table 3).

WALL	STRESS AT MIDSPAN (MPa)	REMARKS
A1	0.55	
A2	0.26	Average of 0.42 MPa for walls A1 to A5
A3	0.426	
A4	0.406	
A5	0.45	
B1	0.126	Low cracking load
AVERAGE	0.37 MPa	

TABLE 5 MODULUS OF RUPTURE OF REINFORCED WALLS

4.2 Deflections After Cracking

In computing deflections both before and after cracking an effective width of beam equal to 4t or e + 2t as applicable was assumed. Prior to cracking gross section properties were used that in some cases vried from course to course because of the use of splayed bars. The first crack was assumed to develop at the joint with maximum bending moment applied, that is, at mid-height if self weight is neglected. Properties of the cracked section were calculated by using reinforced concrete methods to compute the position of the neutral axis and neglecting all transformed areas in tension except that of the steel.

From the first crack at mid-height other cracks were assumed to develop progressively towards the supports whenever the applied bending moment at a bed joint equalled or exceeded the observed moment at mid span at first cracking, M_{cr}. Deflections were computed on the basis of cracked section properties for that portion of the wall where the moment equalled or exceeded M_{cr}, on uncracked section properties in the remainder of the wall. Since such calculations are extremely tedious a finite element program using beam elements was used. The load-deflection relation obtained in this way consisted of a series of linear portions of progressively decreasing stiffnesses as indicated in Figures 2 .. 7.

Included in these figures are the experimental deflection curves. The computed results reflect the trend observed in test results but there are large discrepancies. Reasons for this include:

(1) Sensitivity of this method to the assumed level of first cracking load. This effect illustrated in Fig. 7 which shows the computed deflections for wall B1 for two values of cracking load:

(a) 0.1 kPa as observed in the test, and
(b) 0.3 kPa based on a modulus of rupture of 0.43 MPa.(Table 3)

(2) Variation of joint strength. The theory assumes that all joints have the same strength whereas bond strength varies considerably. The first crack is not always near mid-height and cracking does not progress symmetrically and uniformly from the first crack towards the supports.

4.3 Ultimate Loads

Ultimate loads were computed on the following basis:

(1) Section of mid-height of the wall is critical.

(2) All steel is at yield stress of 410 MPa at failure.

(3) Non-linear distribution of stress occurs in the compression zone represented by an equivalent rectangular distribution of 0.85 F_m' stress level (F_m' = 6.1 MPa).

(4) Load is distributed to cores (beams) in proportion to the tributary areas.

(5) The strength of the wall is determined by the weakest core (beam).

Computed and observed values are indicated in Table 6.

WALL	ULTIMATE LOAD (kPa) OBSERVED	COMPUTED	REMARKS
A1	5.71	5.28	
A2	6.90	8.59	
A3	2.52	3.28	Incomplete penetration of grout.
A4	5.05	5.07	
A5	5.39	5.49	
B1	2.35	1.98	

TABLE 6 ULTIMATE LOADS

4.4 Estimate of Effective Width Based on Observed Ultimate Loads

The maximum tensile force at ultimate load is known to be

$$Tult = A_s f_{sy}$$

The lever arm, a, at ultimate load is thus known to be

$$a = \frac{Mult}{Tult} \qquad \ldots (4)$$

The depth of the compression zone, x, is given by equating the tensile and compressive forces

$$x = \frac{Tult}{0.85\ F'm\ .\ kt} \qquad \ldots (5)$$

Since the c.g. of steel reinforcement is at the centre of the wall a is also given by

$$a = \frac{t}{2} - \frac{x}{2}$$

or

$$x = t - 2a \qquad \ldots (6)$$

from 5 and 6 we have

$$t - 2a = \frac{Tult}{0.85\ F'm\ kt}$$

$$\text{or} \quad k = \frac{T_{ult}}{0.85\ F'_m t\ (t-2a)} = \frac{A_s f_{sy}}{0.85F'_m (t - 2\ Mult/A_s f_{sy})}. \quad (7)$$

Substituting known values of A_s, f_{sy}, F'_m, t, Mult, the effective width coefficient, k, can be computed from 7. The computed value has an upper limit equal to the spacing between the beams. Table 7 shows the k values computed from observed ultimate loads. The average value of 4.265 compares favourably with 4 recommended by the code.

It may, however, be noted that at ultimate load wide changes in the width coefficient would result in only slight changes in the lever arm and ultimate load. The ultimate load is insensitive to the choice of effective width coefficient.

WALL	k	REMARKS
A1	4.143	
A2	1.906	
A3	1.74	Average 4.06
A4	3.483	
A5	3.643	
B1	9.474	Max. value limited by spacing between beams

TABLE 7 EFFECTIVE WIDTH COEFFICIENT BASED ON OBSERVED ULTIMATE LOADS

5. CONCLUSIONS

(1) Tests indicate that the effective width of reinforced core beams is close to the value of 4t as recommended by the draft Australian Masonry Code.

(2) The tests also confirm that cracking loads, and ultimate loads can be estimated using standard reinforced concrete type computations.

(3) A method of computing load deflection 'characteristics' allowing for progressive cracking gave reasonable correlation with experimental results. The method is capable of further development by allowing for variation of flexural joint strengths and axial loads.

6. REFERENCES

(1) "The Draft SAA Masonry Code", Standards Association of Australia, 1984.

(2) Baker, L.R. "Partially reinforced concrete masonry wall panels subjected to lateral load", Masonry Research Centre, Deakin University, Geelong. Report No. 84/1.

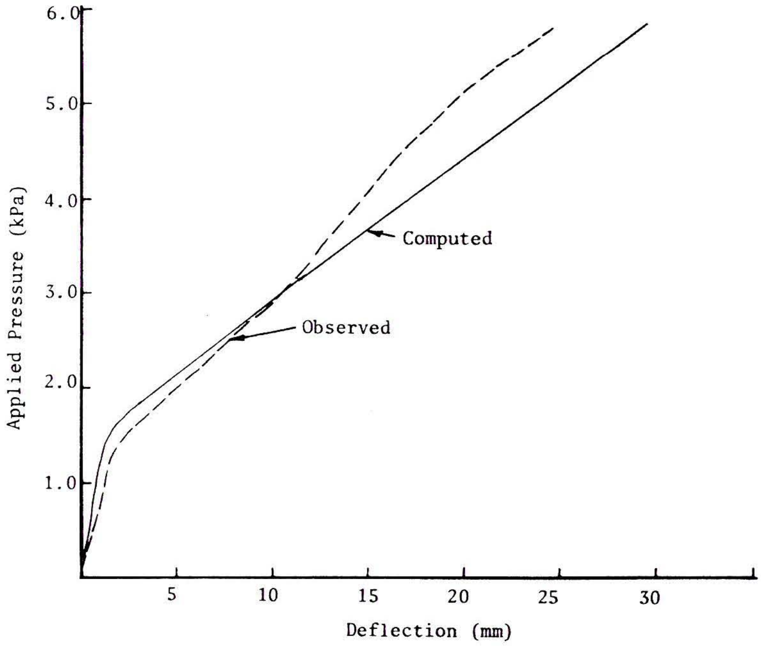

Figure 2 Wall A1

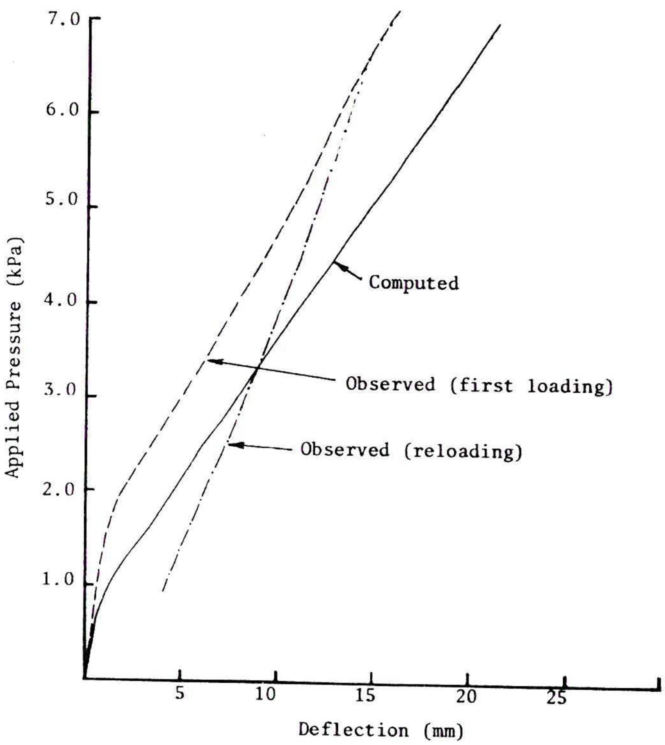

Figure 3 Wall A2

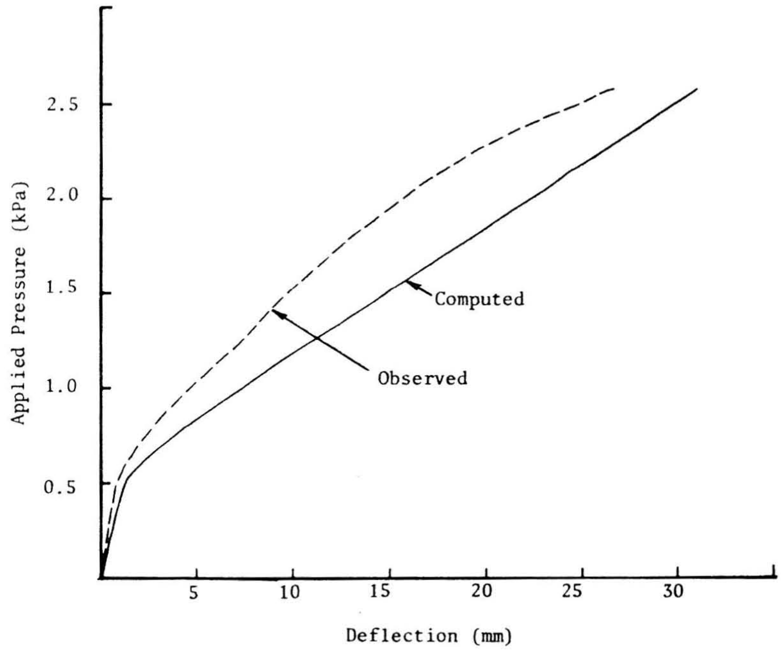

Figure 4 Wall A3

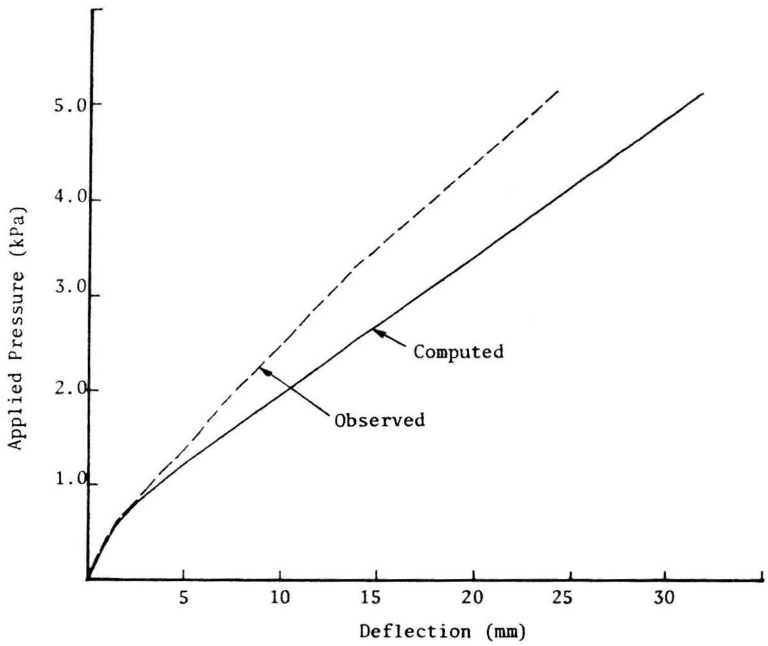

Figure 5 Wall A4

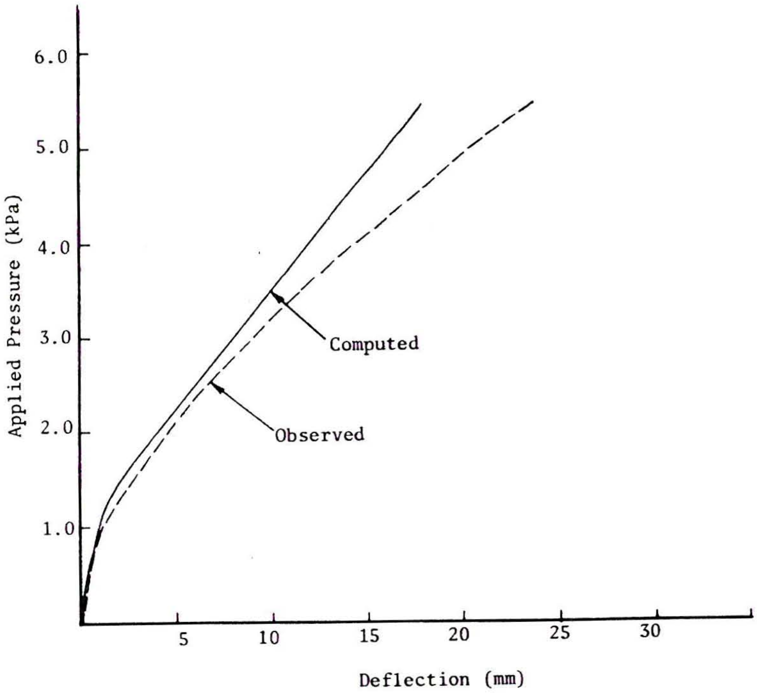

Figure 6 Wall A5

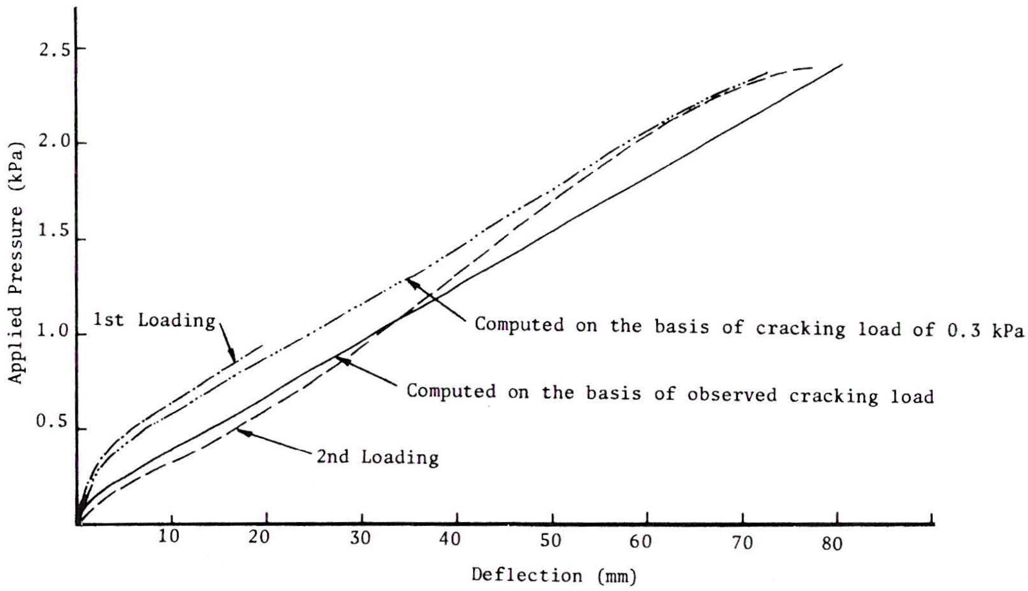

Figure 7 Wall B1