Research into the Behaviour of Pocket-Type Retaining Walls

by

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Abstract

The type of construction, the development and details of the design guidance currently available in the U.K. are described. Previous experimental work, cost studies and practical experience are reviewed and areas that require further research are indicated. The experimental work currently being carried out is described in detail. The behaviour of the first walls in the test series, which failed either by the steel yielding or by simultaneous steel yielding and brickwork crushing is described. A preliminary assessment of the results against the draft U.K. Code of Practice recommendations is made.

1. INTRODUCTION

Brickwork has been used extensively in the United Kingdom for the construction of retaining walls but much of this was before 1900. In particular, there are many examples of mass brickwork retaining walls, some of considerable height, forming railway cuttings. The use of reinforced brickwork for retaining walls is relatively rare, however, despite the fact that the concept was introduced as long ago as 1825 by Sir Marc Isambard Brunel[1] when he used reinforced brickwork caissons for the construction of the first tunnel under the River Thames. In fact the use of reinforced brickwork generally in the United Kingdom is rare, but there is a growing interest and the amount of research is increasing[2].

There are three common types of reinforced brickwork cantilever retaining walls, and these are usually known as grouted cavity, quetta bond and pocket-type walls (Figure 1.). Of these the pocket-type wall is the most efficient in resisting lateral earth pressure. As it contains the reinforcement, concentrated in pockets formed in the brickwork, at regular intervals along the tension face of the wall. These pockets are subsequently filled with high slump concrete to form a composite construction. The design was developed by the brick industry in the U.S.A. and was described by Abel and Cochran[3]. Figure 2 shows three variations of wall recommended by the Brick Institute of America[4]. Figure 2a shows the pocket-type wall in its simplest form, with the pockets wholly within the uniform width of the wall and normally spaced at 1.0 - 1.3 m centres. The type shown in Figure 2b may be designed as a 'T' section which results in more efficient use of materials and also has the advantage of requiring no formwork during concreting.

The retaining wall shown in Figure 2c is a combination of the others and has the practical advantage that the same flange thickness may be used for the full wall height to produce an economical wall. It would be usual for the other wall types to have a reduction in thickness at intervals up their height, as shown in Figure 3. In the present study the structural behaviour of only the simplest form of pocket-type retaining wall is being investigated.

2. DESIGN GUIDANCE

Until recently there was only a limited amount of guidance available in the U.K. for the designer intending to use pocket-type construction. A new British Standard for reinforced masonry is in draft form but the current recommendations are included in a standard that covers all types of load-bearing walls, CP 111 : 1970[5], and contains a few paragraphs only on reinforced brickwork. For the design principles the user is referred to a reinforced concrete Code, CP 114 : 1969[6], which recommends the use of either elastic analysis with the maximum stresses limited at working loads or load factor analysis. As a result publications by the British Ceramic Research Association[7], the Brick Development Association[8] and Structural Clay Products Ltd.[9,10], were produced to give the necessary design guidance. In each of these publications the wall is considered to act as a homogeneous cantilever for pocket spacings of up to 1 m centres and as a series of 'T' sections with reduced flanges for greater pocket spacings. For these greater pocket spacings it is also necessary to check the capacity of the brickwork to span between the pockets.

In preparing the draft British Standard on reinforced and prestressed masonry note was taken of the previous design guidance, in particular that in SP 91[7], as this uses the limit state philosophy which was that adopted for the Code. Pocket-type walls have been treated as flanged members, irrespective of pocket spacing, as have those reinforced hollow concrete blockwork walls with regularly spaced protruding ribs. The design formulae have been made consistent with those for flanged members in the concrete Code, CP 110 : 1972[11].

The draft assumes that for reinforced masonry structures the ultimate limit state will be critical. The design, therefore, is carried out using the partial safety factors appropriate to the ultimate limit state. Recommendations are given to ensure that the serviceability limit states of deflection and cracking are not reached.

The design moment of resistance is taken as the lesser of the values given by equations 1 and 2.

$$M_d = \frac{f_f}{\gamma_{mm}} b\, t_f \quad (d-0.5\, t_f) \qquad \dots\dots\dots\dots \quad 1.$$

$$M_d = \frac{f_y}{\gamma_{ms}} A_s \quad (d-0.5\, t_f) \qquad \dots\dots\dots\dots \quad 2.$$

where:- M_d = design moment of resistance

A_s = area of reinforcement

b = effective width of the flange

d = effective depth of the reinforcement

f_f = characteristic compressive strength of reinforcement masonry in bending

f_y = characteristic tensile strength of reinforcement

t_f = flange thickness

γ_{mm} = partial safety factor for compressive strength of masonry (2.5 or 2.8)

γ_{ms} = partial safety factor for strength of steel (1.15)

t_f is taken as the lesser of t_u and 0.5d, where t_u is the thickness of the unit on the compressive side of the pocket.

b is taken as the lesser of the pocket spacing and the breadth of the pocket plus 12 t_u.

The shear resistance of the section is considered to be provided by the whole of the effective depth of the section, except in cases where the brickwork thickness between the pockets is lower, then this actual thickness is used. At the design load the average shear stress should not exceed f_v/γ_{mv} where f_v is the characteristic shear strength and γ_{mv} is the partial safety factor for the shear strength of the brickwork. The characteristic shear strength increases with the percentage of reinforcement in the section, however the data from which the recommended values have been derived is from tests on grouted cavity construction and was used because of the lack of more relevant information. The value of γ_{mv} recommended is 2.5.

3. PREVIOUS RESEARCH AND COST STUDIES

In 1976 Maurenbrecher et. al.[12] reported the results of tests on five reinforced brickwork cantilever walls tested at the Building Research Station. One of these was a pocket-type wall 3 m high, 1 m wide and 327 mm thick at the base, stepping down to 215 mm at mid height. Failure was due to shear at the change in section caused by the combination of a number of adverse factors. It became clear from the other wall tests in the series that even with a large amount of tensile reinforcement used in conjunction with the medium strength brickwork failures were generally due to the steel yielding.

In 1974 a pocket-type wall was constructed at one of the brickworks of George Armitage and Sons Ltd. This is shown in Figure 3. The design of this wall is described by Maurenbrecher[13] who also reports that after 500 days the wall had deflected 24 mm at the top, 8 mm of this was due to sliding, it is not known how much of the remainder was due to rotation and how much to the stem bending.

The economic feasibility of pocket-type walls has been studied by Maurenbrecher et. al.[10] and by Haseltine and Tutt[8]. In both cases comparisons were made between reinforced concrete walls and reinforced brickwork pocket-type walls.

The results of the first of these cost studies[10], which was done in 1972-3 was based on an actual job and indicated that for walls up to 6 m high a pocket-type wall was more economical than a comparable reinforced concrete wall with a ribbed finish. For walls greater than 6 m reinforced concrete walls become more economical.

The cost study by Haseltine and Tutt in 1975[8] compared grouted cavity, mass brickwork, quetta bond, stepped and plain pocket-type reinforced brickwork walls with two types of reinforced concrete walls - a ribbed finish and a brick-faced finish. Walls up to 4 m high only were considered. The results indicated that reinforced brickwork pocket-type retaining walls were the most economical wall for wall heights above 1 m. The authors suggested that for walls greater than 4 m high it was reasonable to assume that similar results would be obtained. The cheapest of the reinforced concrete walls was the one with the ribbed finish which was between 30% and 49% more expensive than pocket -type construction, depending on the height.

Recent examinations of relative costs have indicated that the conclusions of the above studies are still broadly relevant. For walls between 1 m and 4 m in height, although rough shuttered concrete walls are cheaper than pocket-type walls, when any effort is made to improve the finish to the concrete wall the reverse is true.

4. TEST PROGRAMME

The preliminary programme, which has just been completed, consisted of four tests on full size walls. Table 1. gives details of these walls, each of which was 1½ brick thick (327 mm), 3 m high and 2 m long. The aim of the preliminary tests was to investigate whether the present design formulae for flexure and shear contained in the draft B.S. code on reinforced masonry were adequate.

All the walls were built off a reuseable steel base to which the reinforcement was anchored. The brickwork was built around the reinforcement which rose vertically in the pockets formed, at 1 m spacing, by the bonding pattern. When the brickwork was completed shuttering was clamped to the rear face of the wall and a nominal strength 25 N/mm^2 concrete poured into the pockets and subsequently compacted by a poker vibrator to prevent any large air pockets forming. The concrete mix was proportioned 1:2:3, cement:sand:20 mm aggregate (20 mm max. size) by weight, had a water:cement ratio of 0.6 and a slump of 75-120 mm. It was placed on consecutive days in two lifts, each of 1.5 m. The wall was covered with a polythene sheet and allowed to cure for 28 days before testing.

Each wall was laterally loaded at three horizontal positions on the tension face using a series of hydraulic rams connected to spreader beams, the rams reacting against an A frame. The rams applied a load which gave a similar effect at the base of the wall to that of a triangular pressure distribution. The load was applied in increments and after each one deflections were measured on the rear face (tension face) by electrical transducers whilst steel strains and vertical strains in the brickwork were recorded at the base of the wall. A demec gauge, 150 mm long, was used to measure brickwork strains and electrical resistance strain gauges were used to measure the steel strains. In all the tests except RW1 rotations were measured at four positions up the height using a system of optical levers.

5. RESULTS

The results are given in Table 2. Initially, in each wall test there was a low strain in the steel and a deflection at the top of the wall which increased with applied bending moment either linearly or at a slowly increasing rate. When the tensile strain in the brickwork reached between 60 and 180 microstrain a crack occurred at the brick-mortar interface beneath the lowest course of bricks. This crack generally occurred when the moment was less than 20% of the failure moment, and at this instant the rate of increase of the steel strain increased. There then followed a period when the deflection, steel and brickwork strains increased approximately linearly with load until the wall was close to failure. Each test was terminated when the deflection began to increase at a rapidly increasing rate with increased load. Consequently at what was considered to be failure the steel had passed its yield point but had not broken.

Wall RW1 failed by the steel yielding with no indication of any shear or compression failures. The reinforcing bars in the centre of each pocket in wall RW2 strained less than the outer bars. When these outer bars began to yield additional stress was transferred to the central bars and these began to yield. This wall failed as a balanced section and there were considerable areas of brickwork on the front face which had spalled away. Walls RW3 and RW4, both of which contained a relatively small amount of steel, failed when it yielded.

For all except wall RW1 the vertical compressive strain in the brickwork was lowest in front of the pockets, for wall RW1 the reverse was true. The possibility of the brickwork arching between the pockets and thus causing a complex stress situation has been considered but there was not other evidence of this occurring and it seems unlikely with a 1 m pocket spacing. There was no evidence of the walls twisting out of their plane.

In walls RW1 and RW2 the strain distribution through the brickwork, which was measured at the end of the wall, was slightly curved in the compression block, and in the case of wall RW2 in the tension block also. However, cracks either in or near the gauge length can affect the accuracy of the measurement of tensile strain. In walls RW3 and RW4 the strain distributions were linear which was also found by Maurenbrecher[12]. Examples of the variation of deflection and steel strain with applied bending moment are shown in Figure 4.

6. DISCUSSION

It is clear from the tests carried out so far that walls with this pocket spacing behave as homogeneous cantilevers and that the draft Code of Practice will need to acknowledge this. In the case of walls RW3 and RW4 it makes a significant difference. Table 2. shows that the characteristic bending moment (setting partial safety factors to unity) derived using the flanged member approach (equation 2) is 176 kNm, which is well below the measured failure moments of 222 kNm and 221 kNm. When the measured yield strength of the steel is used and the lever arm is not restricted to the Code value the predicted ultimate bending moments are quite accurate at 208 kNm and 216 kNm respectively. Although some 4% of this increase in bending moment is accounted for by the increase in the yield value, the remainder is due to the conservative assumption for the value of the lever arm. When the measured ultimate steel strength is used in equation 2 the predicted moment is higher than the measured one which is reasonable, as although the bars had yielded they had not broken. For the walls with the larger amount of reinforcement the difference is not so important.

When the moment of resistance based on the brickwork is calculated based on the measured brickwork strength and the neutral axis is not limited to the flange thickness the failure moments are altered by a maximum of 23%. This is an increase except in the case of wall RW1, where the brickwork pier strength was considerably lower than the Code characteristic strength. The removal of the restriction on the lever arm in this calculation is not critical as designs are generally based on a prediction of steel failure.

From Table 2. it is also clear that the Code estimate of the ultimate shear strength of the wall is too low for the walls with the greater amount of reinforcement where the load at flexural failure was approximately twice this value. The draft Code values for the characteristic shear strength of masonry are given in Table 3. For walls with the lower percentage of reinforcement the estimate is roughly equal to the failure loads, although the failures were flexural. Although it is to be expected that the Code values would be lower than the measured ones as they are intended to be a lower bound, the difference for walls RW1 and RW2 is much too large to be accounted for in this way.

It is important to consider whether there are any unnecessary restrictions placed on the design of pocket-type walls because of the underestimation of their shear resistance. To show, in a fairly simple way, the importance of the design for shear three pocket-type retaining walls of height 3 m, 4 m and 5 m have been considered. Each wall has the following common design parameters:-

effective depth 280 mm
characteristic strength of steel $425/mm^2$
width 2 m
pocket spacing 1 m mortar mix $1:\frac{1}{4}:3$ proportioned by volume
partial safety factor for strength of steel 1.15
partial safety factor for compressive strength of brickwork 2.5
partial safety factor to be applied to earth pressure load 1.6
partial safety factor for shear strength of brickwork 2.5
specific weight of retained earth $18\ kN/m^3$
coefficient of active earth pressure 0.33

For each of the required design moments of resistance equation 1. was used to determine the required minimum compressive strength of the brickwork and hence the minimum brick strengths that would ensure that the sections were under-reinforced. The required areas of steel were determined by using equation 2. and for these amounts of reinforcement the design shear resistances of the walls were found using the information in Table 3.

The results are given in Table 4. and these indicate that shear will always be the critical design parameter. This may be due to the size of the partial safety factor for shear which is 2.5, and if all of the design shear resistance values are multiplied by 2.5 they are at least 20% larger than the design shear load. The size of the partial safety factor is in the absence of firm data a matter for engineering judgement, however even if the current value were reduced it is clear that for the taller walls and consequently larger amounts of reinforcement shear would remain the critical design parameter. The results for the more heavily reinforced walls given in Table 2 show that the Code estimate of the characteristic shear resistance is approximately half the shear face acting when flexural failure occurred. Table 4 shows that for the 5 m wall the design shear load (120 kN) is approximately twice the design shear resistance (58 kN). Consequently even with the present partial safety factor the preliminary experimental evidence indicates that the critical design parameter for the 5 m wall should not be its shear resistance. It is clear that, in particular for the larger walls, revised shear strength figures are urgently required.

7. CONCLUSIONS

The use of reinforced brickwork pocket-type retaining walls in the U.K. has been limited despite their economic advantages. Design guidance has developed to support the current inadequate Code of Practice recommendations and this is now being incorporated in a new Code. Evidence from a preliminary test programme indicates that with a 1 m pocket spacing the walls fail in flexure. Comparisons with the draft Code design formulae indicate that there is an advantage if, for this pocket spacing the wall is designed as a homogeneous cantilever rather than a flanged member. Designs to the draft Code will also be dominated by the requirements for shear resistance and for taller walls in particular a revision of the draft Code estimate is necessary.

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REFERENCES

1. BEAMISH, R. Memoirs of the Life of Sir Marc Isambard Brunel. Longman, Green, Longman and Roberts, London, 1862.

2. SUTHERLAND R.J.M. Brick and Block Masonry in Engineering Proc. I.C.E. Part 1. Design and Construction, 70, 31-63, 1981.

3. ABEL, C.R. and COCHRAN, M.R. A Reinforced Brick Masonry Retaining Wall with Reinforcement in Pockets. Proceedings of the Second International Brick Masonry Conference. Edited by H.W.H. West and K.H. Speed, Stoke-on-Trent, B. Ceram. R.A. 1971. pp. 23-28

4. BRICK INSTITUTE OF AMERICA. Reinforced Brick Masonry Retaining Walls, Parts 1, 2 and 3, Tech. Notes 17E, and 17F, 1964 and 179, 1965.

5. BRITISH STANDARDS INSTITUTION Structural Recommendations for Load Bearing Walls, C.P. 111 : 1970.

6. BRITISH STANDARDS INSTITUTION The Structural Use of Reinforced Concrete in Buildings C.P. 114 : 1969.

7. BRITISH CERAMIC RESEARCH ASSOCIATION Design Guide for Reinforced and Prestressed Clay Brickwork, Spec. Publ. 91, 1977.

8. HASELTINE, B.A. and TUTT, J.N. Brickwork Retaining Walls, Brick Development Association, Windsor, England, August 1977.

9. JOHNSON, G.D. Reinforced Brickwork Retaining Walls Design Guide Structural Clay Products Ltd., Tech. Note SCP 15, Hertford, England, 1979.

10. MAURENBRECHER, A.H.P., BIRD, A.B., SUTHERLAND, R.J.M., and FOSTER, R.D. Reinforced Brickwork, Vertical Cantilevers Vol. 1, Structural Clay Products Ltd., SCP10, Hertford, England, 1976.

11. BRITISH STANDARDS INSTITUTION Code of Practice for the Structural Use of Concrete. C.P. 110 : 1972

12. MAURENBRECHER, A.H.P., BIRD, A.B., SUTHERLAND, R.J.M. and FOSTER, D. Vertical Cantilevers in Reinforced Brickwork of Clay Bricks Vol. 2 Structural Clay Products Ltd. SCP 11, Hertford, England, 1976.

13. MAURENBRECHER, A.H.P. A Pocket-Type Reinforced Brickwork Retaining Wall. Edited by D. Foster. Structural Clay Products Ltd. SCP 13, Hertford, England, 1977.

TABLE 1.

Wall Details

Wall No.	RW 1	RW 2	RW 3	RW 4
Breadth (mm)	1900	1995	1990	2000
Depth (mm)	330	322	330	325
Effective Depth (mm)	275	263	289	289
Percentage Reinforcement	0.92	0.92	0.28	0.28
Brick Type	A	B	B	A
Mean Brick Strength (N/mm^2)	84.8	21.6	25.4	64.7
Mortar Type	$1-\frac{1}{4}-3$	$1-\frac{1}{4}-3$	$1-\frac{1}{4}-3$	$1-\frac{1}{4}-3$
Mean Mortar Cube Strength (N/mm^2)	11.7	13.5	14.9	13.3
Mean Concrete Cube Strength (N/mm^2)	34.1	36.8	32.9	32.0
Mean Brickwork Pier Strength (N/mm^2)	16.2	6.9	8.4	16.8
Mean Measured Yield Strength (N/mm^2)	442.8	442.8	479.9	479.9
Mean Measured Ultimate Strength (N/mm^2)	620.7	620.7	642.8	642.8
Code Characteristic Brickwork Strength (N/mm^2)	21.6	7.8	8.7	18.1
Code Characteristic Steel Strength (N/mm^2)	425	425	460	460

TABLE 2.

Test Results and Predicted Values

Wall No.	RW 1	RW 2	RW 3	RW 4
Failure Moment (kNm)	367	450	222	221
Code M_d (brickwork) (eq.1) kNm	1130	405	507	1060
Code M_d (steel) (eq.2) kNm	459	434	176	176
M_d (steel) using measured parameters (kNm)	477	392	208	216
M_d (steel) using ultimate strength (kNm)	677	755	272	286
M_d (brickwork) using measured parameters (kNm)	1044	428	628	1263
Shear Force at Failure (kN)	550	519	229	228
Code Characteristic Shear Stress (N/mm^2)	0.53	0.53	0.39	0.39
Code Characteristic Shear Force (kN)	227	278	224	225

TABLE 3.

Characteristic Shear Strength of Reinforced Masonry

Percentage Reinforcement %	Up to 0.15	0.5	1.03	1.5	2.0
Characteristic Shear Strengths N/mm^2	0.35	0.45	0.55	0.60	0.65

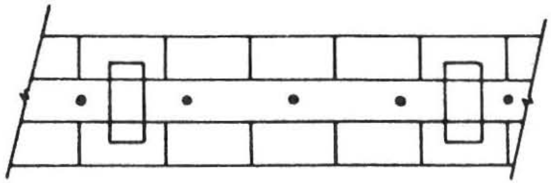
TABLE 4.

Design Comparisons

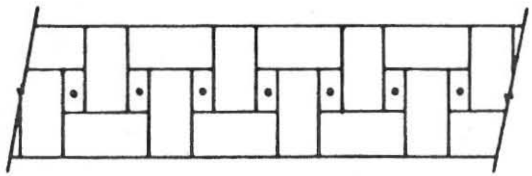
Wall Height m	Required Steel Area (mm^2)	Proportion of Reinforcement (%)	Required Characteristic Strength of Brickwork (N/mm^2)	Required Mean Brick Strength (N/mm^2)	Design Shear Load (kN)	Design Shear Resistance of Wall (kN)
3	1020	0.18	3.8	8.2	43.2	40
4	2418	0.43	9.0	27.0	76.8	48
5	4722	0.84	17.7	63.0	120.0	58

FIGURE 1. Three Forms of Reinforced Brickwork Retaining Wall shown in Plan.

(a) Grouted Cavity



(b) Quetta Bond



(c) Pocket-type

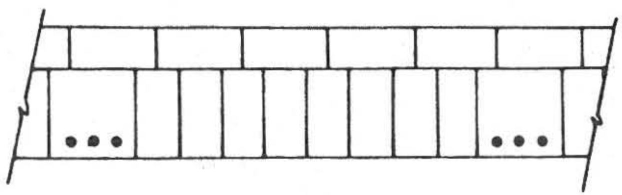
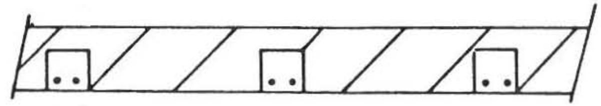
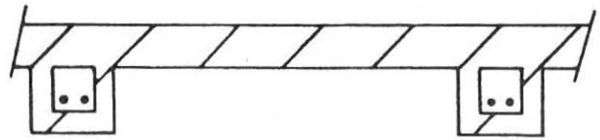


FIGURE 2. Three Variations of Pocket-type Walls.

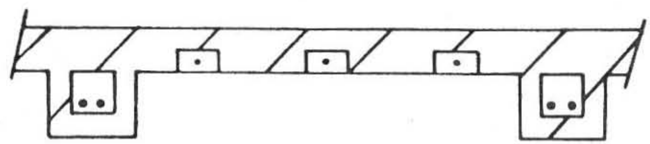
(a) Simplest type



(b) 'T'-section with stiff piers



(c) Combination of (a) & (b)



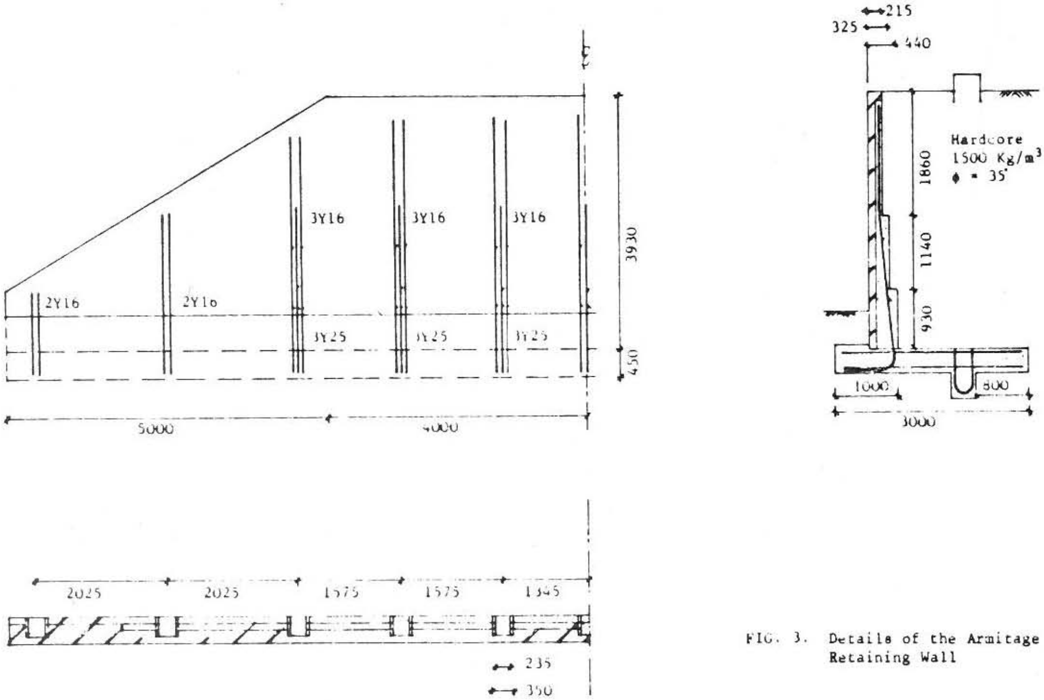


FIG. 3. Details of the Armitage Retaining Wall

FIGURE 4. VARIATION OF STRAIN IN THE REINFORCEMENT AND DEFLECTION OF THE TOP OF THE WALL WITH BENDING MOMENT