

REINFORCED MASONRY SHEAR WALLS: CYCLIC LOAD TESTS IN CONTRAFLEXURE

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SUMMARY:

Nine 2.4 m high by 1.6 m long reinforced masonry walls (140 mm) were cycled under reverse curvature loading. The walls were designed to investigate the effect of vertical preload and the effectiveness of repair by epoxy grout injection together with certain matters raised by the Draft Code for Design of Reinforced Masonry¹, in particular effect of extent of grouting of cells, and size and distribution of reinforcement.

The novel loading system highlighted the undesirable effects of grout settlement and of relying on fewer but larger diameter reinforcing bars in masonry design. The difficulty of separating the shear resisting functions of horizontal and vertical reinforcement was also apparent.

1. INTRODUCTION:

The new Draft Standard Specification for Design of Reinforced Masonry¹ (the "Draft Code") represents a very significant development of its outdated predecessor, NZSS1900, chapter 9.2:1964. The last sixteen years has witnessed a trend to the ultimate strength method of design and the development of the concept of structural members which dissipate energy under earthquake loading, as described in the seismic Loadings Code for buildings² which was published in 1976, and these two major developments are accommodated in the Draft Code.

In spite of considerable testing in New Zealand^{3,4,5,6,7,8,9} since the mid-sixties, masonry remains a very complex material and its reluctance to be prescribed by theoretical considerations even under laboratory conditions is further enhanced by the sensitivity and supervision in the field. Indeed, ultimately "the problem is not one of theory but of practice".¹⁰

The Loadings Code², assumes that reinforced masonry has an ability to dissipate 80% as much seismic energy as reinforced concrete for structures of the same geometry. Figure 1 summarizes wall types and implied energy absorbing ability ('ductility demand'). The inference to be drawn from comment clause 3.2 of the Loadings Code² is that reinforced masonry 'end zones',¹ i.e. parts of elements where seismic energy absorption is concentrated, should be able to execute 8 load reversals (4 load cycles) to the design ductility demand without losing more than 20% of their strength.

The Draft Code¹ permits 'partially grouted' reinforced masonry although not in 'end zones'. Notwithstanding this prohibition, the present series of tests has permitted some comparison of the performance of partial and fully grouted construction during inelastic load excursions in the recognition that

ductility demand at unintended locations in a building is a real possibility during earthquake loading.

A minimum reinforcement bar diameter of 10 mm is required by the Draft Code,¹ and in the case of Grade 380 steel and for 140 mm block, a maximum bar diameter of 16 mm for bars which lap within the blockwork. (Note the code imposes no restrictions on the use of Grade 380 steel). Upper limits on spacing, together with minimum reinforcement ratios permit a range of bar layouts, and the test series has compared the performance of some extreme cases.

Masonry piers, particularly where coupled by deep spandrels, may exhibit a point of contraflexure within a storey height under seismic loading. Previous test series^{7,8,9} have loaded walls through a concrete beam at mid storey height, thus resulting in effectively zero moment at the point of loading. The walls are of aspect (height/length) ratio of not greater than one and thus 45° diagonal tension cracks always intersect either top or bottom beam. In designing the present test series, it was thought that the previous forms of loading may not truly simulate contraflexure loading in masonry piers because of this restriction on development of diagonal crack width, thereby possibly enhancing the wall performance. The loading system described in Figure 2 was therefore developed for the present series of testing to truly simulate contraflexure loading. This system obtains the same magnitude of moment at the top and bottom of the wall whatever the difference in cracking extent within the wall. Additional wall units in the series were designed to afford direct comparison with particular units tested in single curvature at the University of Canterbury.

2. TEST PROCEDURE & MATERIALS PROPERTIES:

Wherever possible the walls were subjected to the same loading history. For the first two cycles, the test unit was loaded to a maximum of approximately 75% of the theoretical yield load, based on measured material properties. The

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yield displacement was determined by linearly extrapolating the deflection at this load, providing the wall had cracked. For uncracked sections, an arbitrary yield deflection was chosen, because linear extrapolation would not be valid. Next the wall was subjected to two full cycles to twice yield displacement ($DF = 2$), then two cycles to $DF = 4$, and one cycle at every subsequent even Ductility Factor but finishing with two cycles at the final Ductility Factor.

Table 1 gives properties of reinforcing steel, respective sizes of which came from the same "heat-batch". Test results for three block high prisms at age 6 months are given in Table 2. The mortar compressive strength at age 6 months ranged from 14.6 to 26.4 MPa.

3. DESCRIPTION OF TEST UNITS:

Eight reinforced masonry wall units were tested, each of dimensions 2.400 high, 1.600 wide and 140 wide. In the case of fully-grouted walls, the interior blocks were open ended bond beams as is standard MWD practice, whereas a combination of standard and knock-in units was used to construct partially grouted test specimens in order to contain the extent of grouting. Starter bars were used to simulate normal site practice, and the ends of vertical reinforcing were welded to steel within top and bottom beams. Lap splice lengths were used as specified in the Draft Code¹, viz 80 and 60 bar diameters respectively for Grade 380 and 275 reinforcement. Except where minimum steel percentage was being investigated, Grade 380 steel was used as the Draft Code¹ does not restrict its use in any situation. Blocks were laid by a Registered Mason who was asked to work only to the standard of normal practice on site.

TABLE 1

REINFORCING STEEL PROPERTIES

Bar Size	Grade	Yield Stress (MPa)	Ultimate Stress (MPa)	Elastic Modulus (GPa)	Percent Elongation
D10	275	320	443	192	27
D12	275	353	482	197	25
D10	380	477*	702	198	19
D12	380	389	619	191	21
D16	380	454	701	193	19
D20	380	434*	674	194	20

*0.2% proof stress

Blockwork, reinforcing and grouting details for the eight wall units are given in Figure 3, and the basis for the choice of respective units is as follows:

Unit 2

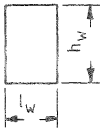
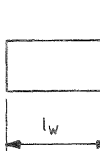
The Draft Code¹ requires a minimum area of reinforcement in both horizontal and vertical directions to be not less than 0.07% of the gross wall area at right angles to the reinforcement. In addition, the sum of vertical and horizontal reinforcement ratios is to be not less than 0.2%. Unit 2 was designed to test this requirement, and from Figure 3, the vertical reinforcement ratio is 0.136% and for horizontal bars, 0.081% giving a total of 0.217%. Further, only reinforced cells were grouted and so this wall is representative of those with a minimal grout and reinforcement content.

Unit 3

A minimum bar diameter of 10 mm is specified in the Draft Code¹, although no smaller than a 12 mm bar may be used in the 'end-zone' of a wall. Unit 3 was designed to test the effect of small bars at close centres by way of comparison with other units (Units 5 and 7) which have a similar percentage of vertical reinforcement comprising three bars instead of the eight of Unit 3.

Unit 4

The Draft Code¹ permits single storey Class III buildings in seismic Zone B and C to be reinforced horizontally with as little as one bar at 2.400 centres. Unit 4 was designed to test this provision and contained 0.28% of vertical reinforcement and no horizontal steel over its 2.400 height, as the reinforcing in top and bottom beams provide this. Thus,

CATEGORY	DESCRIPTION	ASSUMED DUCTILITY FACTOR (4/S _M)
1.	<u>'DUCTILE WALL'</u>  $\frac{h_w}{l_w} \geq 1.0$ $l_w \geq 0.8m$	2.1-3.3
2.	<u>'WALL OF LIMITED DUCTILITY'</u>  $\frac{h_w}{l_w} < 1.0$	2.1 (flexure)
3.	<u>'WALL OF LIMITED DUCTILITY WITH LARGE OPENINGS'</u> - ignore strength end stiffness of spandrel elements.	1.4 (flexure)
4.	<u>'LOW DUCTILE WALLS'</u> - elastic response	0.8

MASONRY WALL TYPES AND DUCTILITY DEMAND, ACCORDING TO DRAFT REINFORCED MASONRY CODE⁽¹⁾

Fig. 1.

OMITTED: BUCKLING FRAMES
 VERTICAL LOAD JACKS
 CHANNEL ON SIDES OF TOP BEAM

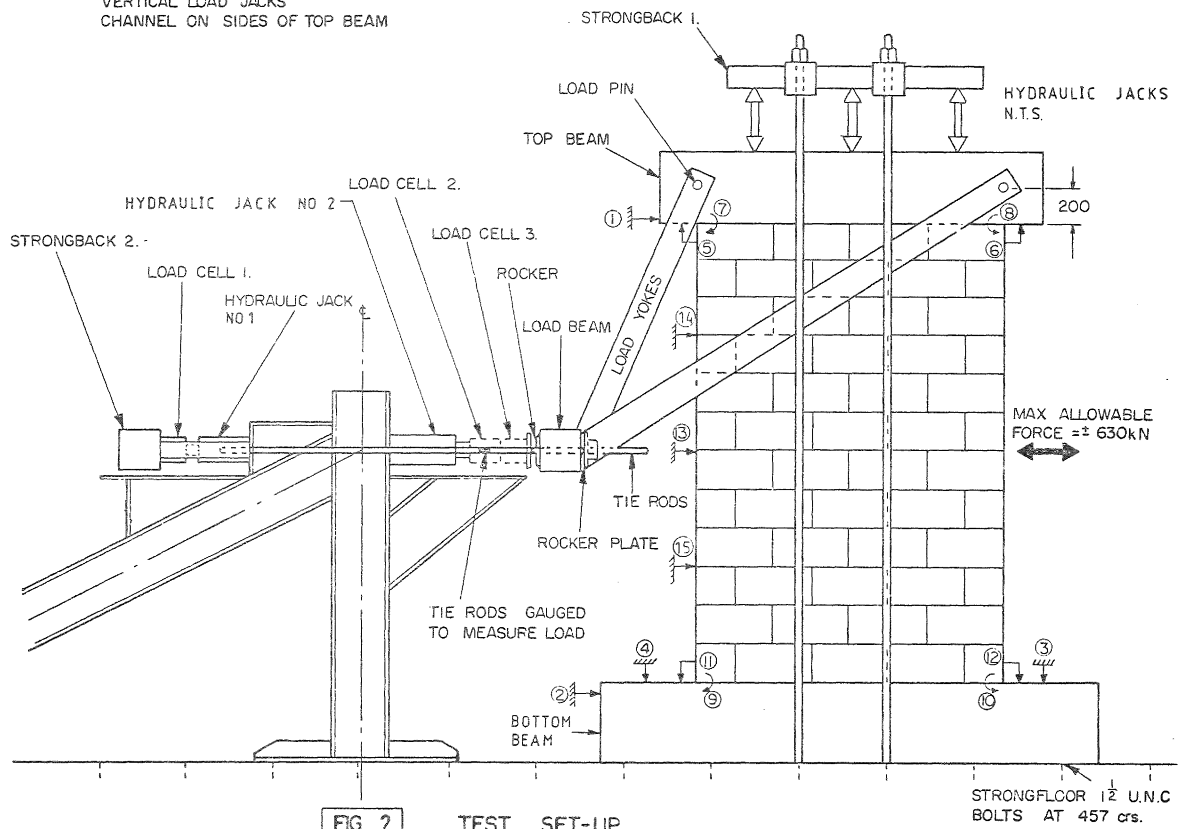
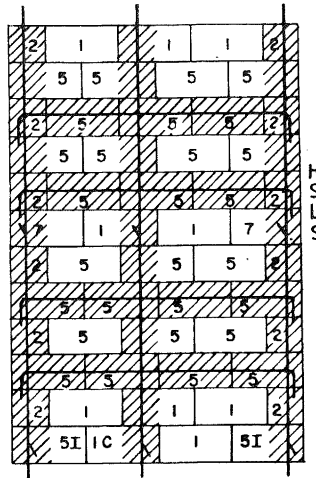
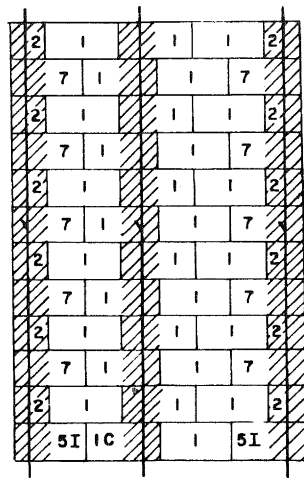


FIG. 2 TEST SET-UP



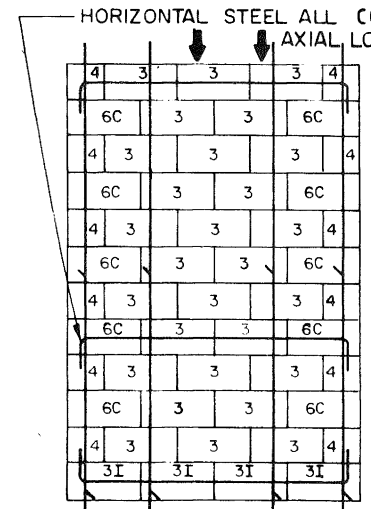
HORIZONTAL STEEL AT LAYERS SHOWN ONLY

UNIT NO. 5
ALL STEEL D16 GRADE 380
PARTLY FILLED WALL AS SHOWN



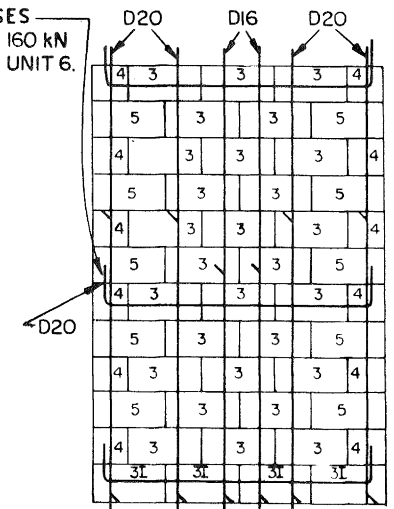
NO HORIZONTAL STEEL

UNIT NO. 4
ALL STEEL D16 GRADE 380
PARTLY FILLED WALL AS SHOWN

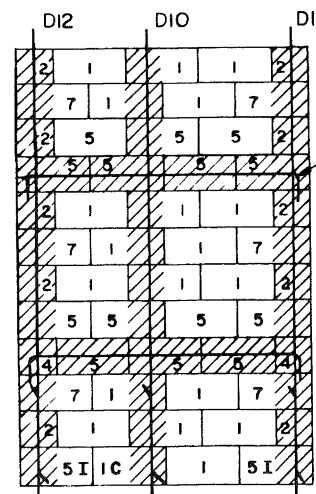


HORIZONTAL STEEL ALL COURSES
AXIAL LOAD = 160 kN
UNIT 6.

UNIT NO. 6 and 8
ALL STEEL D16 GRADE 380
ALL CELLS FILLED.

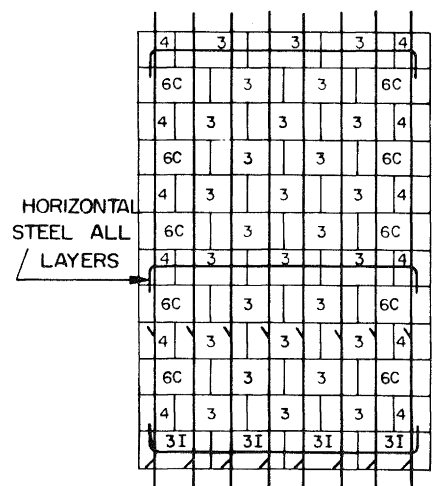


UNIT NO. 9
ALL STEEL GRADE 380
ALL CELLS FILLED



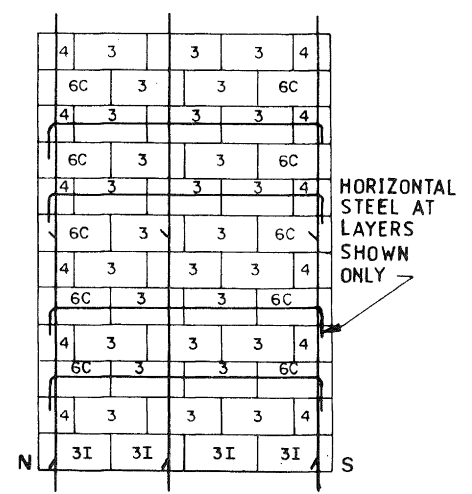
D 12 HORIZONTAL STEEL AT LAYERS SHOWN

UNIT NO. 2
ALL STEEL GRADE 275
PARTLY FILLED WALL AS SHOWN



HORIZONTAL STEEL ALL LAYERS

UNIT NO. 3
ALL STEEL D10 GRADE 380
ALL CELLS FILLED.



HORIZONTAL STEEL AT LAYERS SHOWN ONLY

UNIT NO. 7
ALL STEEL D16 GRADE 380
ALL CELLS FILLED

FIG. 3 DETAILS OF BLOCKWORK AND REINFORCING

TABLE 2

PRISM TEST RESULTS

Unit No.	Block Type	f'_m (MPa)
1	15.16	14.6
2	15.16	14.9
3	15.01	11.6
4	15.01	18.6
Average		14.9

the total steel percentage complies with the Draft Code¹. The wall was grouted only at reinforced flues.

Unit 5

This wall has reinforcement identical to Unit 7; however, Unit 5 is only partially grouted. It does not therefore comply with the Draft Code¹ in an 'end zone' but was designed primarily to obtain a measure of the effect of partial grouting by direct comparison with Unit 7. Reinforcement spacings of 600 horizontally and 400 vertically are more conservative than the Draft Code¹ requirements of 600 both ways for Class II and III buildings in Zone A, but comply with MWD requirements for public buildings¹².

Unit 6

This test wall unit was preloaded with a vertical load of 160 kN, resulting in an average compressive stress of 0.71 MPa, similar to that of Unit A5 on the series reported by Priestley⁸.

Reinforcement ratios in respective directions were similar to those of Priestley's unit A2, being 0.72% horizontally and 0.36% vertically. The unit was thus reasonably heavily reinforced in the horizontal direction, and was designed to test the effect of preload by comparison with Unit 8.

Unit 7

Reinforcement is identical with that of Unit 5, but Unit 7 is fully grouted.

Unit 8

This unit was designed to correspond with Unit A2 in the studies of Priestley⁸. Thus reinforcement ratios were similar in respective directions, as were bar spacings. The reinforcement is identical to Unit 6.

Unit 9

With horizontal reinforcement ratio of 1.12% and vertical ratio of 0.74%, this heavily reinforced wall was designed to correspond with Priestley's⁸ Unit A1.

Unit 10

Unit 8 was repaired by epoxy resin injection by Tedec (Masterton), renamed Unit 10 and tested 6 days after repair.

4. RESULTS:Unit 2

Figure 4 shows hysteresis loops. Cracking commenced during cycling to $DF = 1$ and generally followed mortar courses near top and bottom of the unit. Very little cracking occurred in the middle third of the wall.

After completion of testing, the wall was broken up in order to examine the condition of grout. All cells intended to be filled contained solid grout. However, sealing of other cells was seen to be inadequate as evidenced by the migration of grout to about two thirds of the cells in the lower half of the wall.

Much of the wall horizontal movement after stage N (See Fig. 4) was due to slip at the 11th mortar course which at that stage measured 2 mm and at stage Q (see Fig. 4) attained 8 mm. This observation is borne out by comparison of deflection plots at mid-height and top of wall unit (Fig. 4(i) and 4(ii)).

Some concrete spalling occurred near the test completion at the wall ends at about the 11th mortar course i.e. the bottom face of the top course of blocks. The pattern of cracks indicated that the wall was pivoting on a corner toe as it approached failure.

Unit 3

Cracking became visible during the $DF = 1$ cycles. With increasing applied loading, there developed a combination of diagonal cracks across the blocks and cracks along the mortar joints, although the middle third of the wall remained uncracked.

Figure 5 shows hysteresis loops and it is evident that cycles at greater deflection amplitudes dissipate energy through an increased tendency to pure sliding action rather than through the formation of a yield plateau.

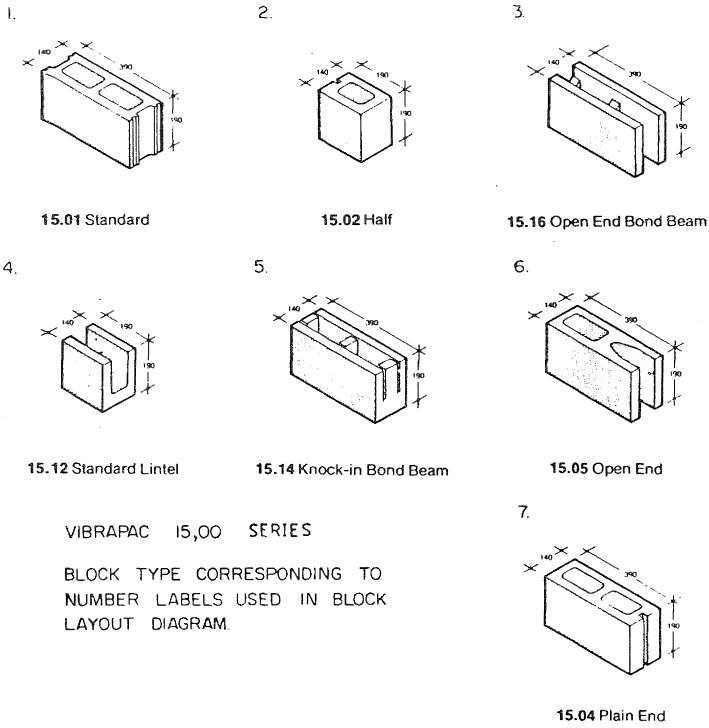
Examination of the wall after testing showed that the grout had indeed filled all cells.

Unit 4

This unit exhibited a striking pattern of diagonal cracking with progress of the loading sequence. As a result of the X-cracking, damage was concentrated in the middle third of the wall height (Figure 6).

Figure 7 shows hysteresis loops which evidence a steadily growing loss of elastic stiffness with increase of loading.

Inspection of the wall after completion of testing showed the grout to have migrated only into one cell other than those intended to be filled.

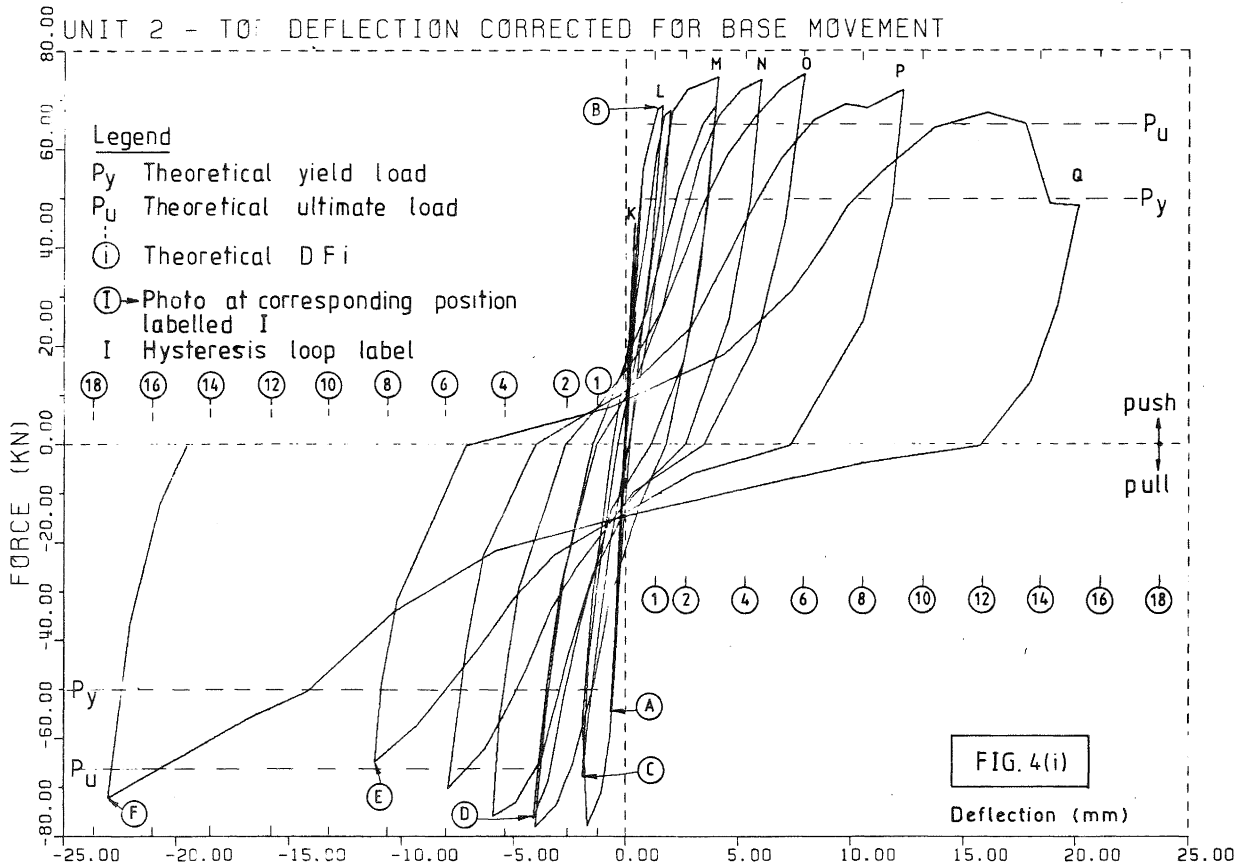


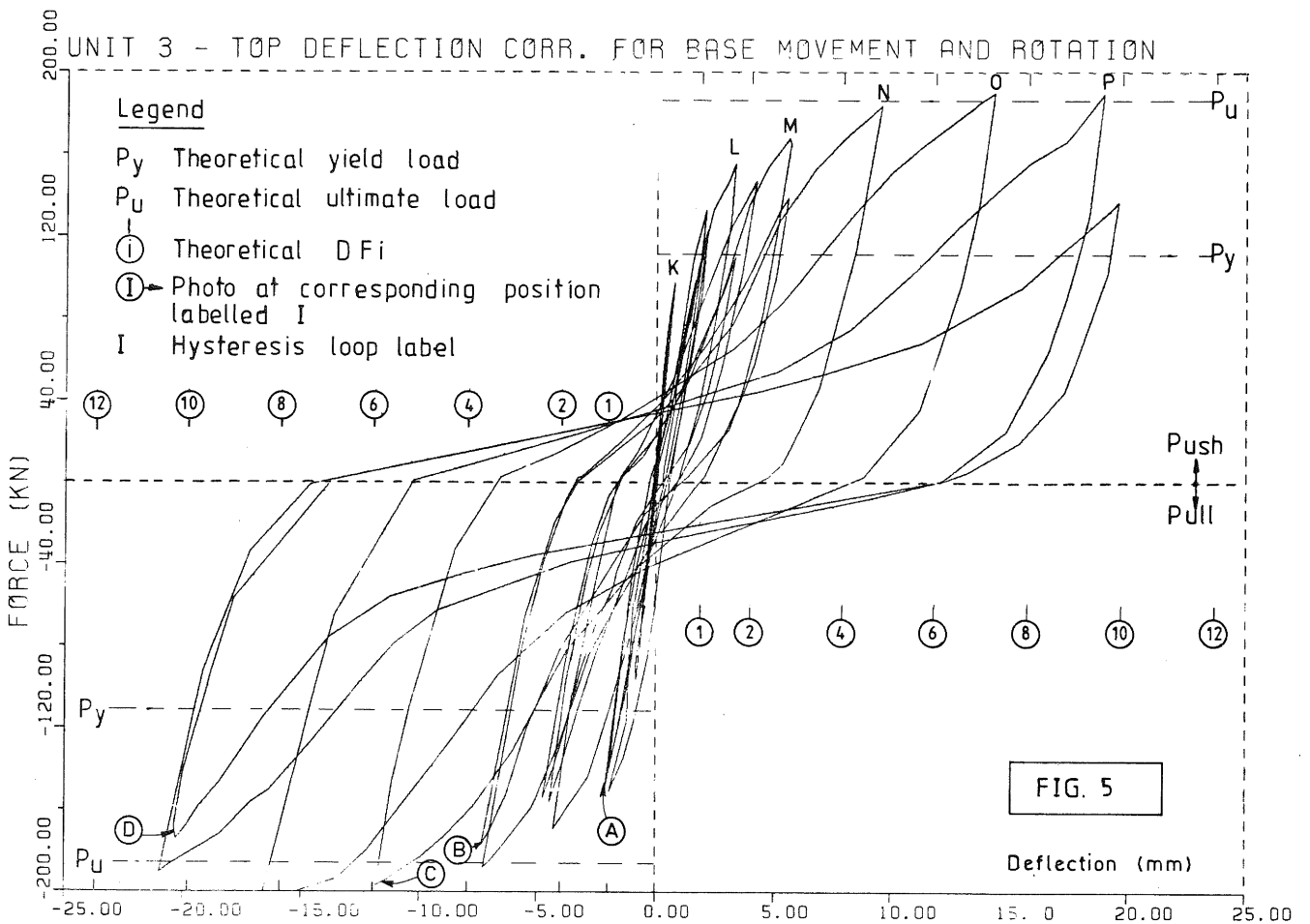
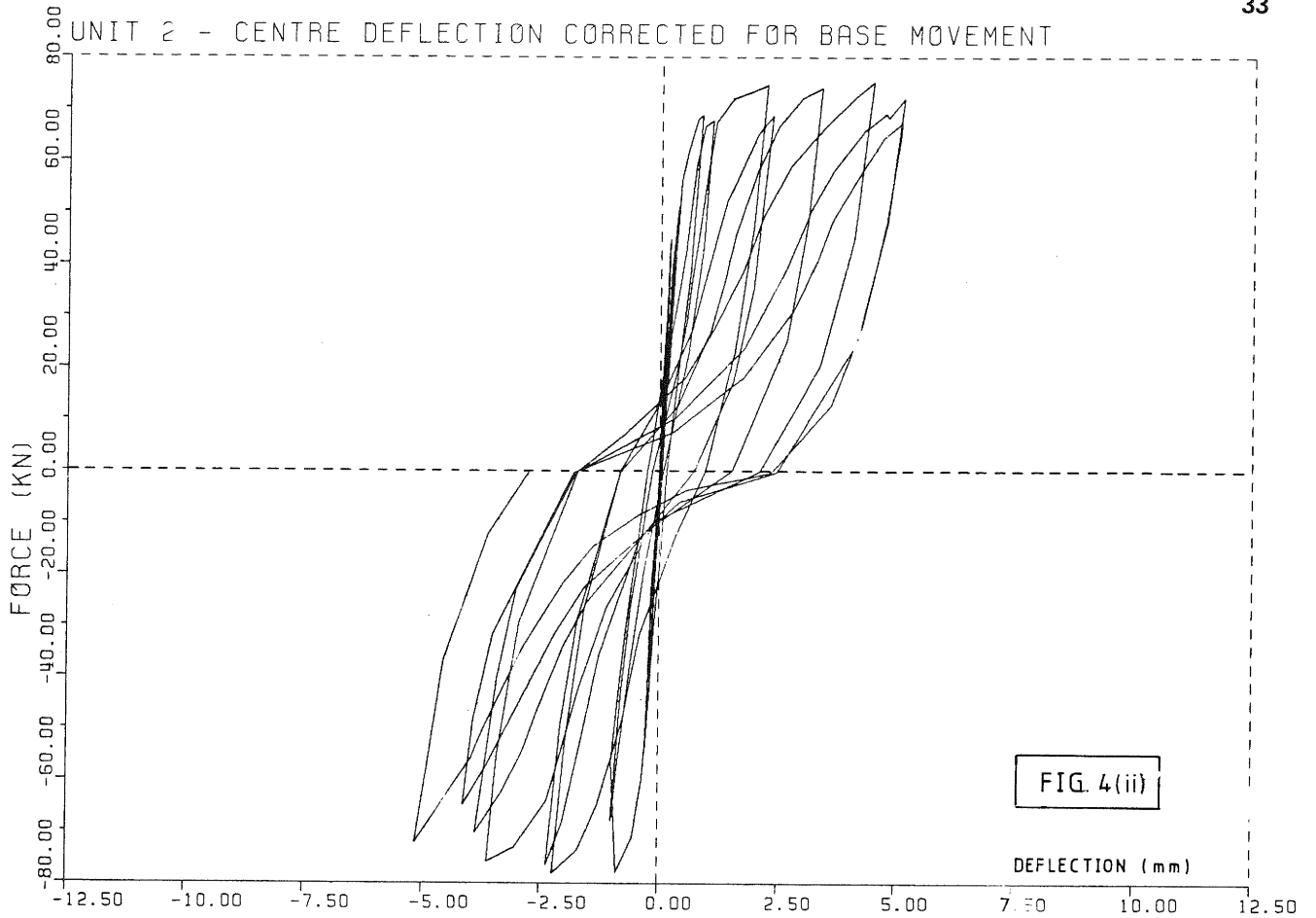
VIBRAPAC 15.00 SERIES
 BLOCK TYPE CORRESPONDING TO
 NUMBER LABELS USED IN BLOCK
 LAYOUT DIAGRAM.

ADDITIONAL NOTES

1. WITH THE EXCEPTION OF UNIT 9, THE LEG ON THE HORIZONTAL STEEL WENT DOWN ON ALL EXCEPT THE BOTTOM LAYER WHICH WENT UP. DUE TO SPACE CONSTRUCTION THE LEG WENT UP ON UNIT 9. AT ALL LAYERS.
2. THE SYMBOL I ON DIAGRAMS INDICATES THE BLOCKS WERE INVERTED TO FACILITATE CLEANING OUT.
3. THE SYMBOL C ON DIAGRAMS INDICATES THE BLOCKS WERE CUT. FOR BLOCK 6 THE UPPER HALF OF THE CENTRE WEB WAS REMOVED, AND FOR OTHER BLOCKS A CLEAN OUT PORT WAS CUT.

FIG. 3 CONTINUED





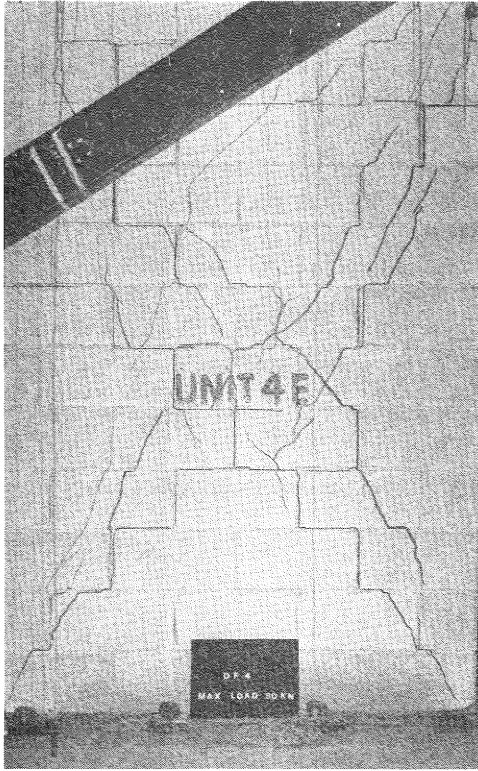


FIGURE 6 UNIT 4: CRACK PATTERN

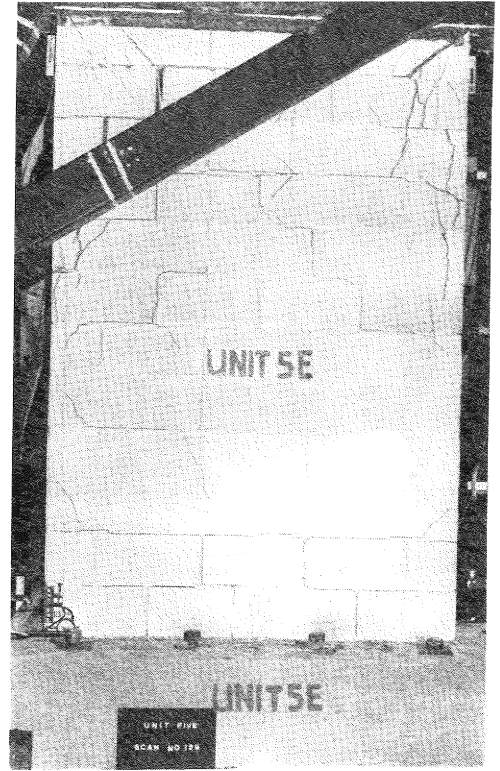
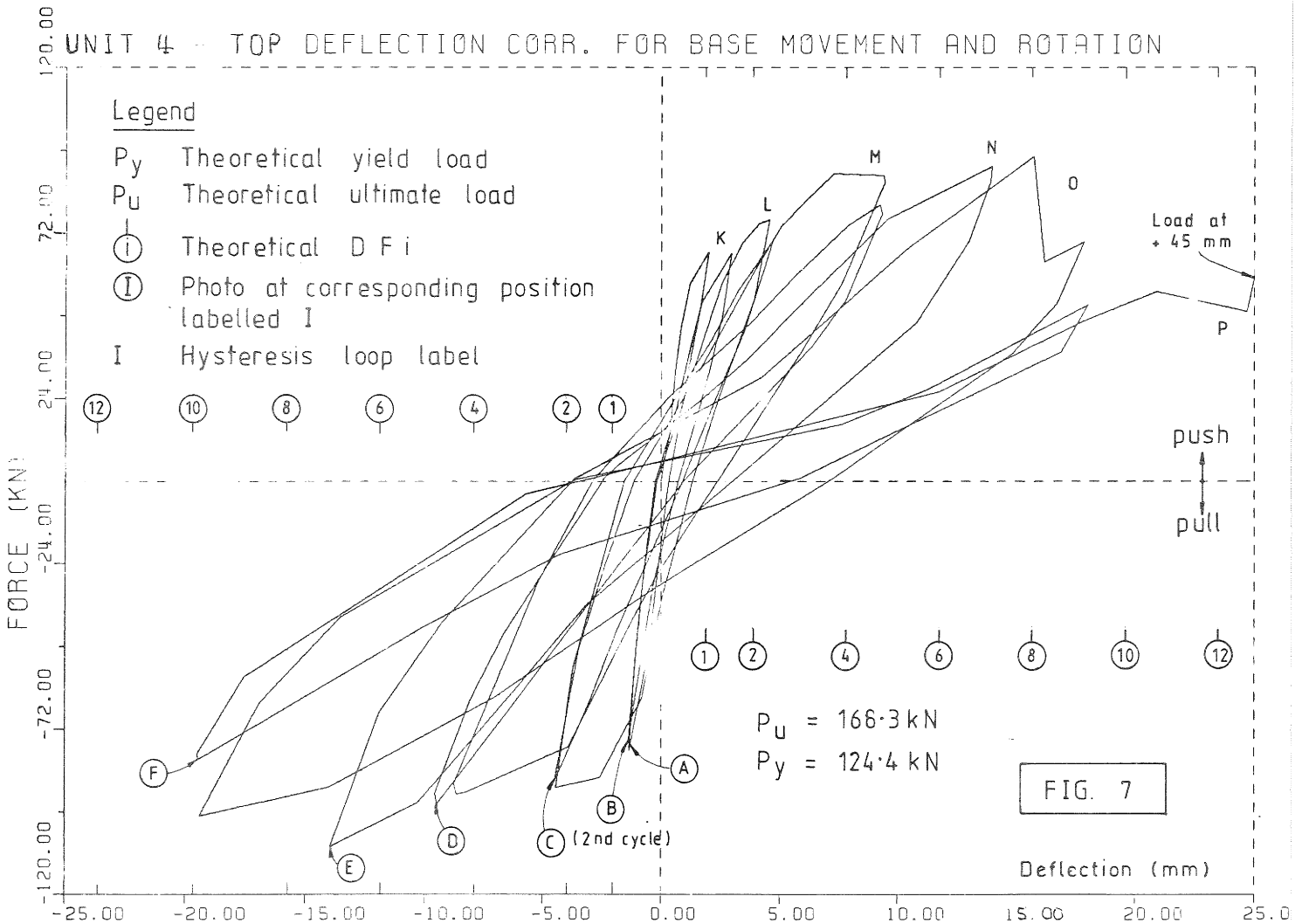


FIGURE 9 UNIT 5: CRACK PATTERN



A maximum width of 4 mm was recorded for a diagonal crack, the size being apparently facilitated by the absence of horizontal reinforcement.

Unit 5

Horizontal cracking within the two mortar courses adjacent to top and bottom beams was evident during the second cycle to $DF = 0.75$. The advent of a wide diagonal crack during the second cycle to $DF = 2$ resulted in a large drop in wall strength. With increase of applied deflection, the form of hysteresis loop (Fig 8) evidences the development of sliding along mortar courses. At the same time, further strength loss resulted from flexural compression-and-tension-induced damage to the upper edge blocks (Fig 9).

Comparison of deflections at mid-height with those at top of wall (Fig 8(i) and 8(ii)) confirms the observation that damage (wall distortion) was mainly located in the upper half of the wall unit.

Examination of the wall after test showed the grout to be of good quality and to have migrated to only two unintended cells (at the bottom next to the filled cell).

Unit 6

Five cracks in the upper mortar courses became apparent near the end of the 2 cycles at $DF = 0.75$. With subsequent loading, these cracks developed and a 1.5 mm diagonal crack developed during the $DF = 2$ cycles. Most cracking occurred in the top half of the wall, and clearly evidenced a bending-shear disposition. Failure was associated with spalling of compression concrete at the top of the wall and buckling of reinforcement.

Fig 10 shows hysteresis loops which while evidencing considerable loss of stiffness, are nevertheless dissipating significant energy at ductilities in excess of 4. However, rapid loss of strength also commences at this point, together with a marked increase in sliding which was observed to be taking place along the top mortar courses.

Unit 7

Although reinforcement is identical to that of Unit 5, this wall panel concentrated most of its damage in the top few courses of blocks. Noticeable out of plane movement of face shells was evident during load cycles to $DF = 6$. Loss of strength resulted from degradation of the wall top edge while extensive sliding occurred along the horizontal mortar joints, especially at the bottom of the top course of blocks.

Figure 11 shows hysteresis loops.

Unit 8

Figure 12 shows hysteresis loops and the lack of evidence of sliding here

is borne out by the observation that a pattern of horizontal cracking continuous over the wall length did not eventuate. Horizontal cracking emanating from edges pointed to a bending failure mechanism, although some diagonal cracking was also apparent. Only small deflections were applied to this unit as it was required to be subsequently retested after repair.

Unit 9

A network of fine diagonal cracking developed during testing of this wall. Loss of face shells occurred at the top course of blocks, and, during cycling to $DF = 6$, a sliding mechanism developed at the horizontal mortar course between the two upper courses of blocks. Fig 13 shows hysteresis loops.

Unit 10

This repaired Unit 8 still retained cracks less than 0.15 mm, and only cracks at the top corners and between bond beams and wall were wide enough to repair.

By the end of the first cycle to $DF = 2.5$, all the unrepaired cracks had reappeared, and major new cracks had appeared. With increase of load, crack movement tended to be confined to the top of the wall. This was apparently associated with tensile failure of two vertical reinforcing bars which had been heated to facilitate construction of Unit 8. (Study of hysteresis loops indicates that this unfortunate pre-heating of Grade 380 steel had little effect on Unit 8 which was not loaded to high deflections). Figure 14 shows hysteresis loops.

Examination of the wall after testing revealed that epoxy had penetrated along some bars thus aiding bond between bar and concrete. Further, bonding of cracks in grout through the entire grouting thickness was achieved, and subsequent cracks formed to one side rather than along the original cracks.

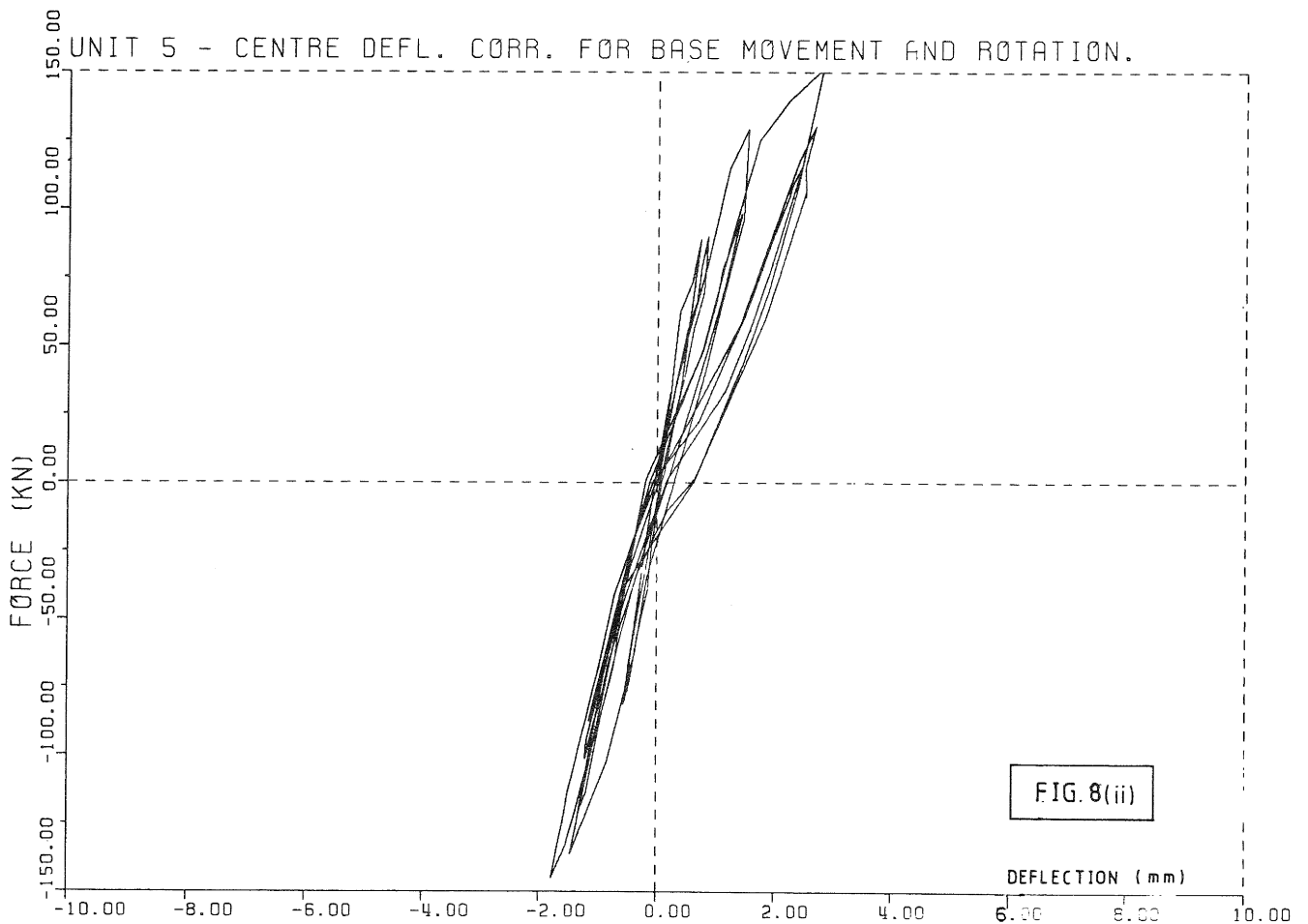
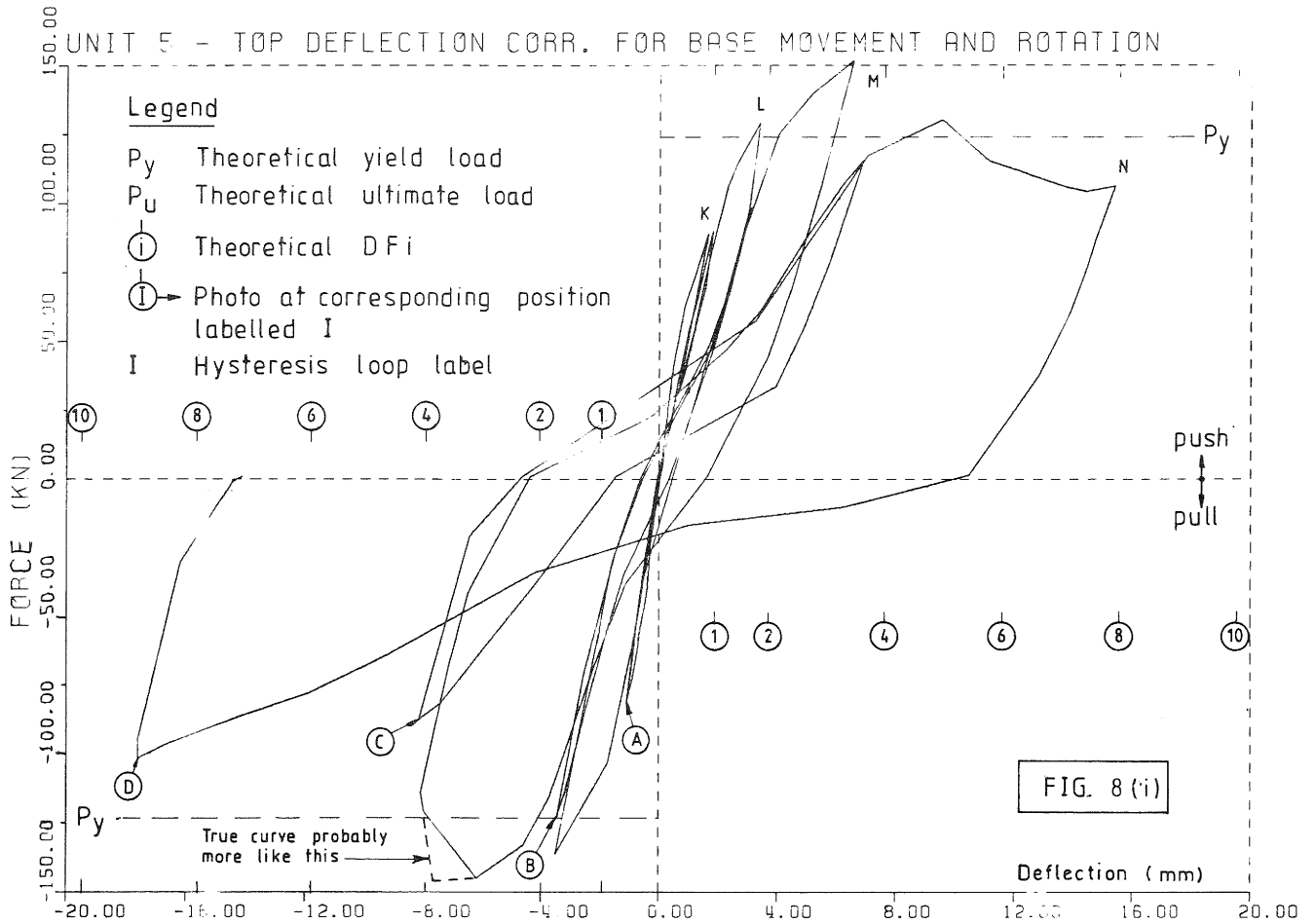
5. DISCUSSION:

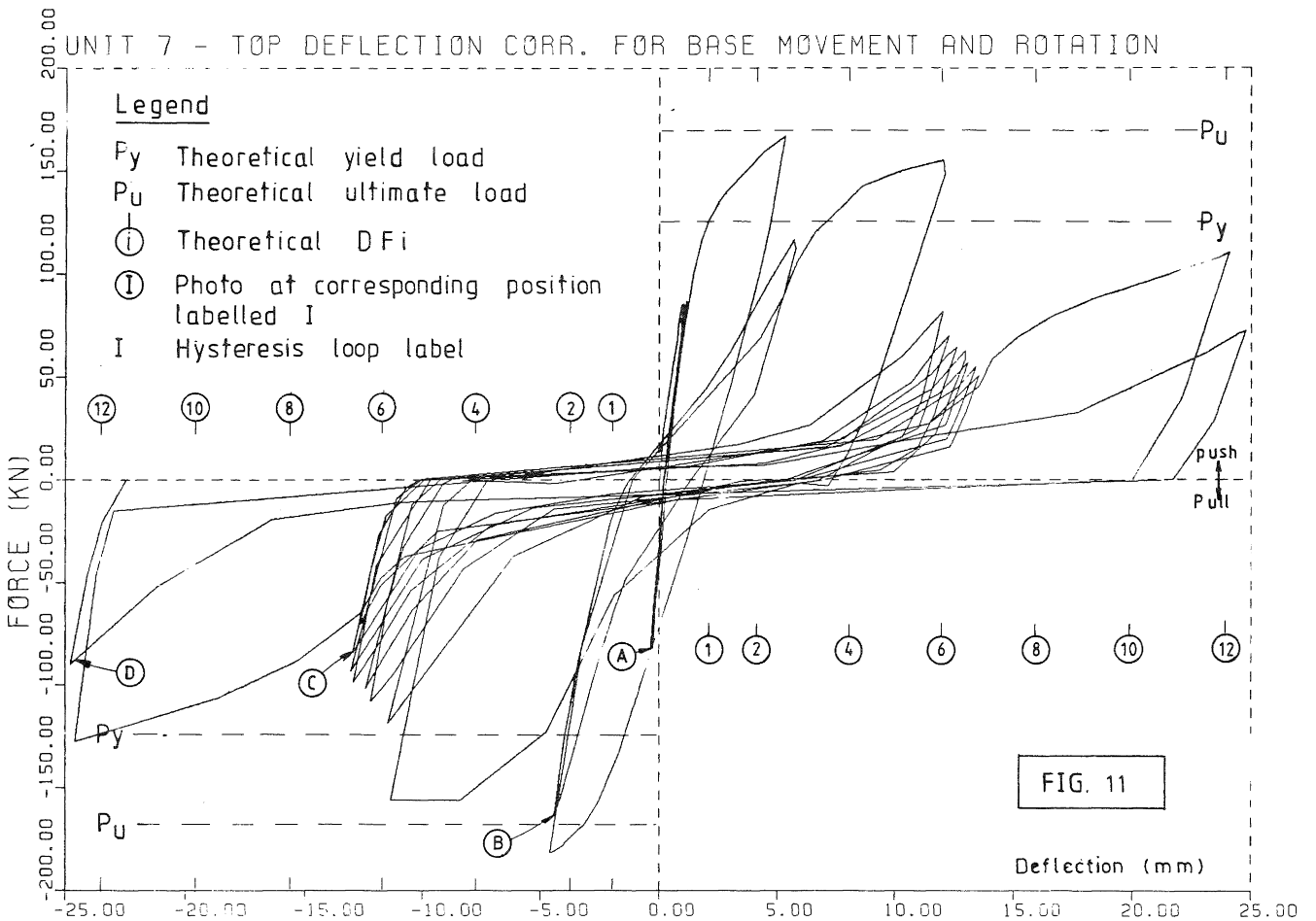
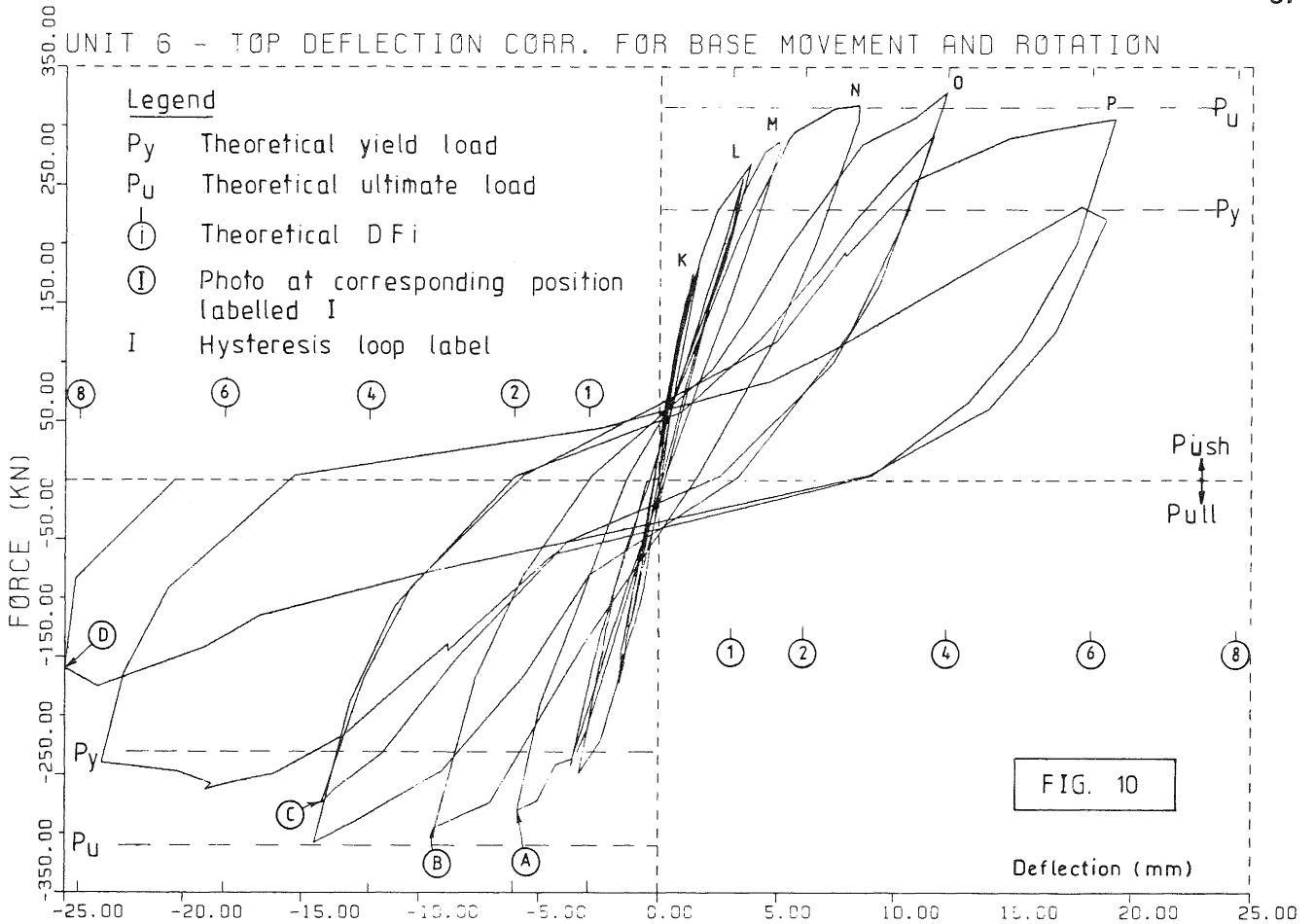
5.1 Method of Applied Loading

As is evident from Figure 2, the loading system applied moments of equal magnitude and opposite sign to wall ends, resulting in a point of contraflexure at mid-height. On the other hand, the loading system used at the University of Canterbury (e.g. reference (8)) applies the horizontal loading as pure shear at the top of the wall, which therefore becomes the point of contraflexure.

The aspect ratio of walls (height/length) in the present test series is greater than 1, unlike the Canterbury walls^{8,9}, so that 45° cracks are not restrained from growth by intersection with a top or bottom beam at both ends.

The ratios of height to point of contraflexure divided by wall length are 0.75 in both the series reported by Priestley⁸ and the present series.





UNIT 8 - TOP DEFLECTION CORR. FOR BASE MOVEMENT AND ROTATION

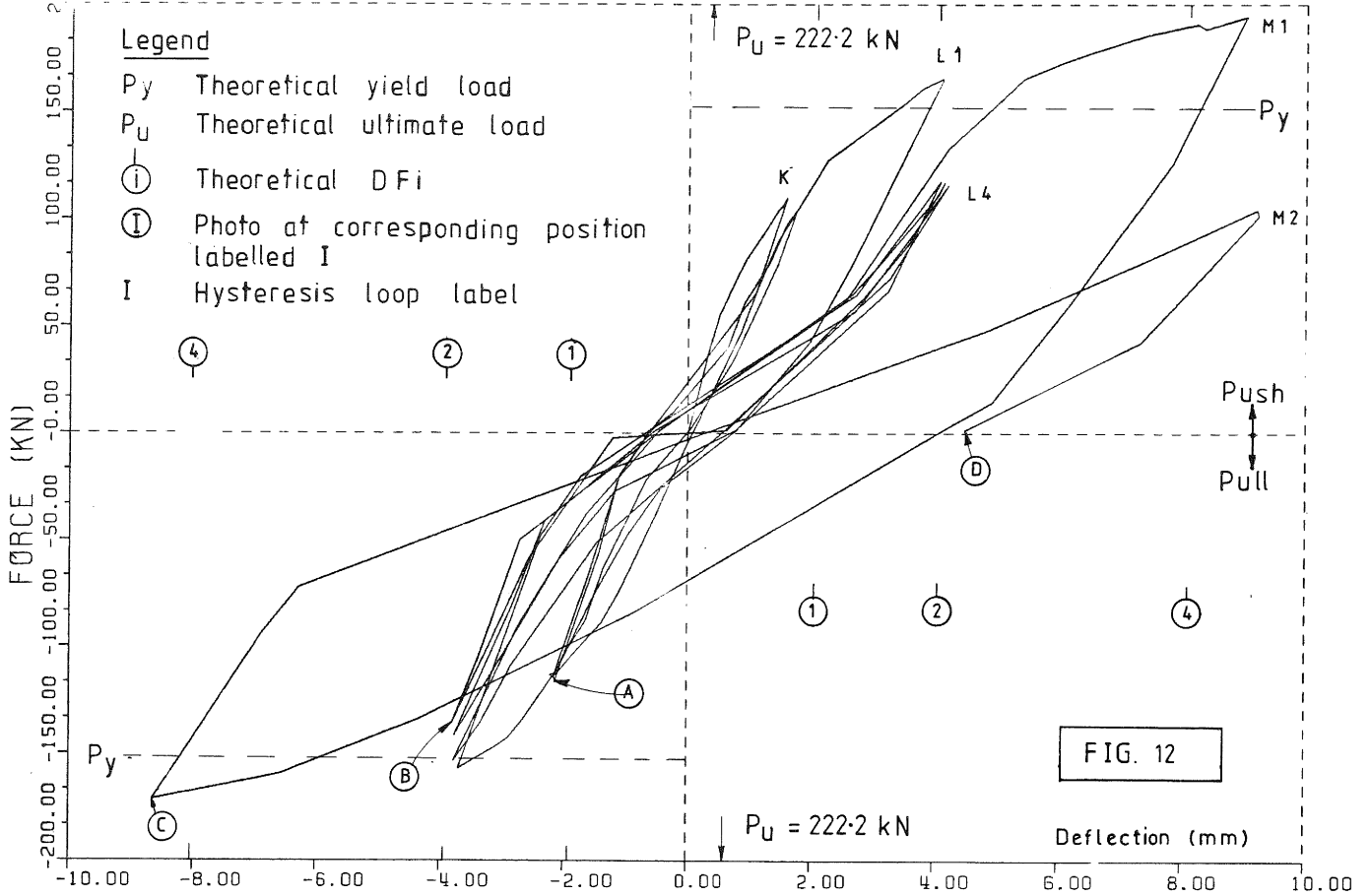


FIG. 12

UNIT 9 - TOP DEFLECTION CORR. FOR BASE MOVEMENT AND ROTATION

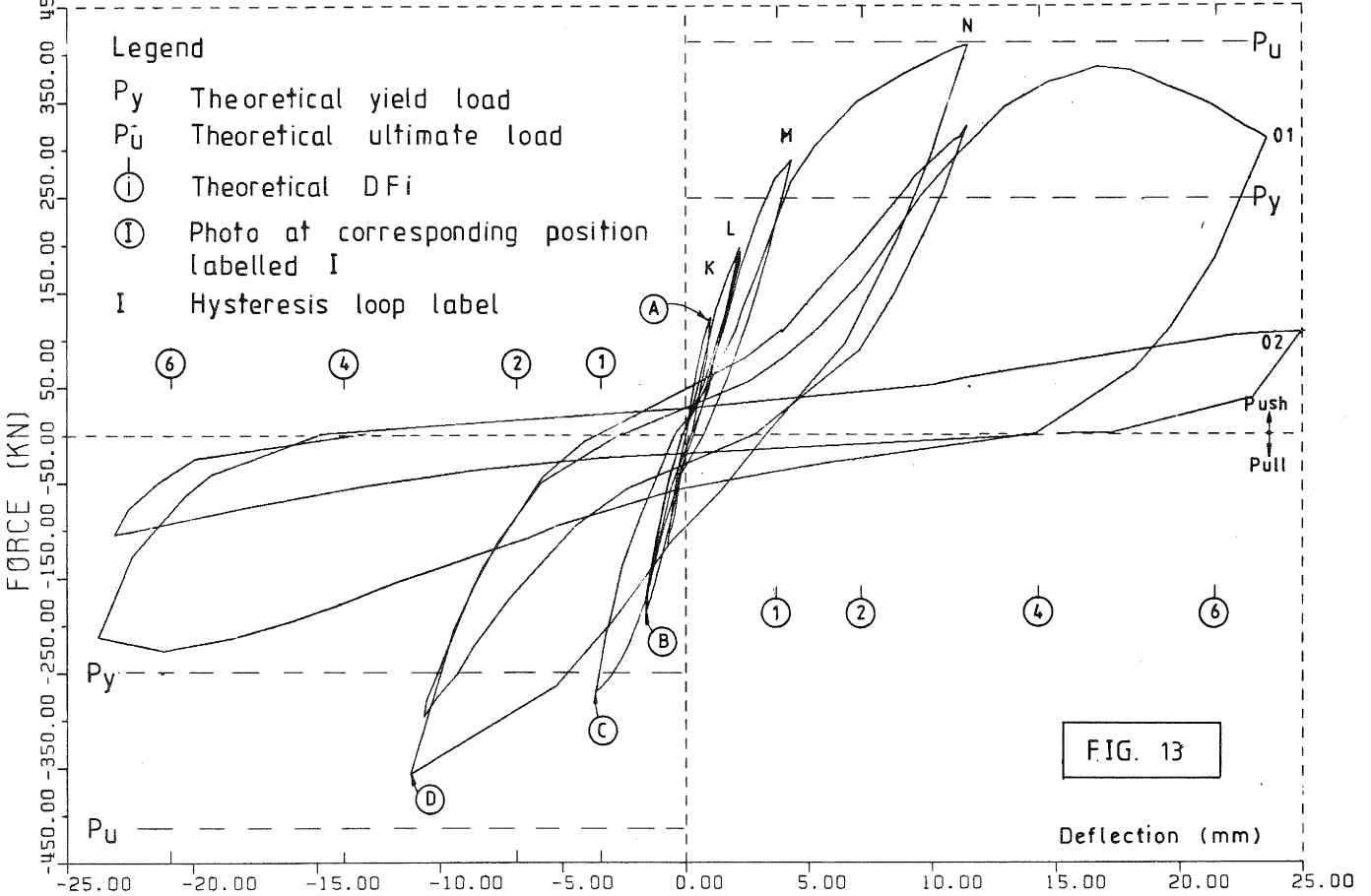


FIG. 13

Further, Units 8 and 9 in the present series have approximately the same bar sizes and vertical and horizontal steel ratios as Priestley's Units A2 and A1, respectively. Thus the two sets of units afford a means of comparing the two loadings systems and, in particular, of ascertaining whether or not the top beam on the Canterbury tests gives rise to an artificially high strength for those walls that would in reality be loaded in contra-flexure. Direct comparison is possible once loads and deflections for the respective wall series are normalised¹¹. Comparisons of hysteresis loops from respective test series are shown in Figures 15 and 16. The Canterbury curves have first been scaled with respect to the present series to give the same ultimate load and associated deflection values.

A comparison of Units 8 and A2 at high ductility values is not possible because the former unit was not tested to high ductilities. Nevertheless, the two units compare well at low ductilities. The decrease of strength of Unit 8 on its last cycle may have been enhanced by the effect of heating two vertical (high yield) bars during placing.

The super-position of (normalised) hysteresis loops for Units 9 and A1 shows a striking similarity in terms of strength, and the University of Canterbury Unit has only slightly greater ductility capability than Unit 9. Both Units 9 and A1 showed very similar patterns of damage with compression failure commencing at wall edges together with a well-developed pattern of diagonal cracking.

It is therefore concluded that differences in hysteresis loop of between the two similar pairs of wall units do not result from different methods of loading and that, for the units compared, the two series of tests gave very similar answers. For the case of the partially grouted Unit 4, however, the crack pattern (Fig 6) suggests that its performance would be enhanced if the unit was tested with a loading beam cast at mid-height as in the University of Canterbury rig. However direct comparison is not possible.

5.2 Extent of Filling of Cells

Comparison of hysteresis loops (Fig. 17) indicates that the fully grouted Unit 7 is about 20% stronger than partially grouted Unit 5. This concurs with the observation of Moss and Scrivener⁵.

While firm conclusions regarding relative ductility are not possible because of the different load histories, a substantial difference is not apparent.

A significant feature is that none of the partially grouted units exhibited sliding failure characteristics (Units 2 and 4 were also partially grouted). Conversely, of the fully grouted units (5 specimens), three developed a sliding mechanism in upper mortar courses, with associated substantial loss of strength.

It is significant that, in the present

study, the sliding failures have occurred along mortar courses near the top of the wall and usually in the course at the underside of the top row of blocks. In construction of the present series, a high lift grouting procedure¹³ was used, and although the construction of these walls is expected to be better than in the case of more extensive walls in the field, nevertheless settlement of the grout by as much as 25 mm was observed at wall top surfaces prior to pouring of the top beams (the grout was several weeks old at this stage). This suggests the occurrence of a lack of homogeneity due to separation between grout and block (see also reference¹⁴) in the upper region of the walls, which subsequently becomes a zone of concentrated failure. The existence of weaker grout at the top regions of the piers is not borne out by a series of prism tests reported in reference¹¹. Further investigation of this problem is under way. In such a situation, the mode of failure of the wall has little relationship with horizontal or vertical reinforcing steel percentages, as attested by the variation of steel in Units 6, 8 and 9, each of which lost considerable energy-dissipating capacity through failure in sliding along an upper horizontal mortar course. Neither is this form of failure mechanism predicted by theoretical consideration of wall aspect ratio.

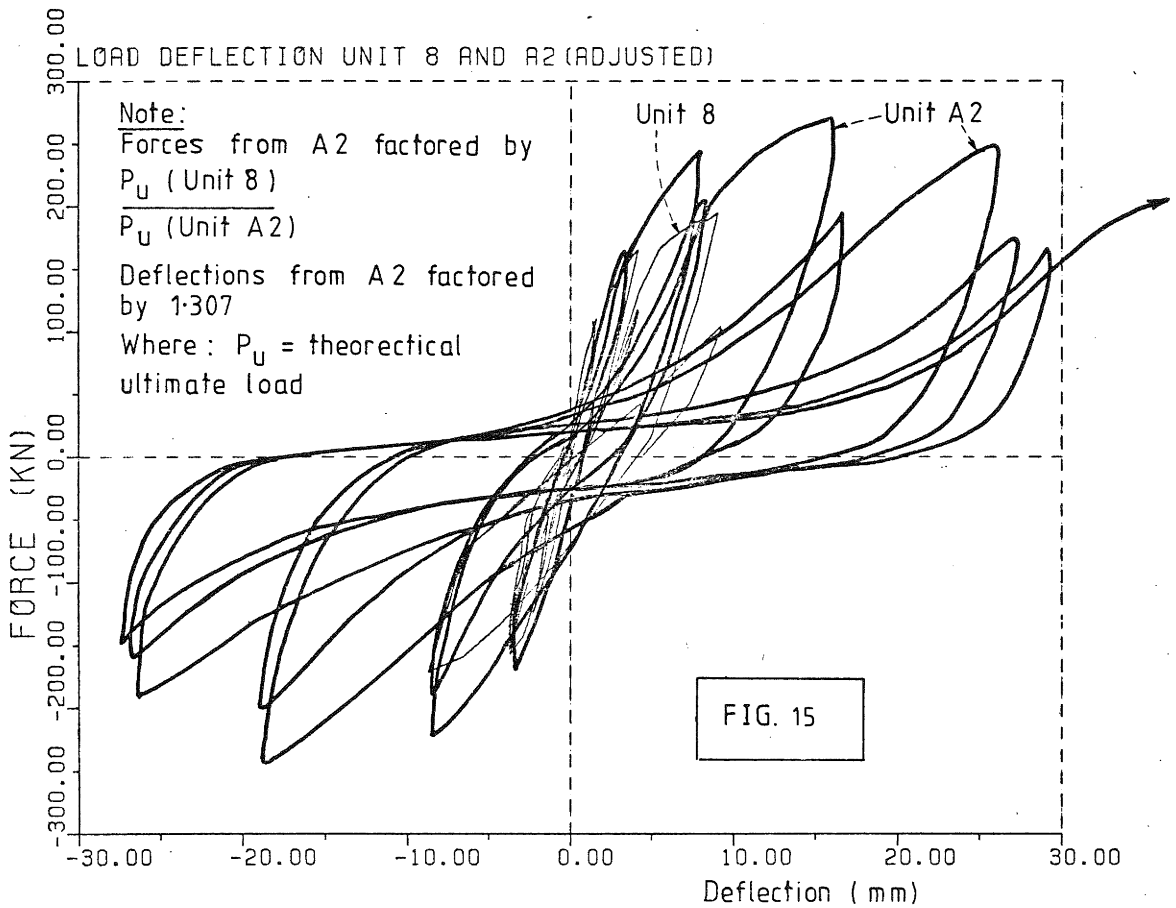
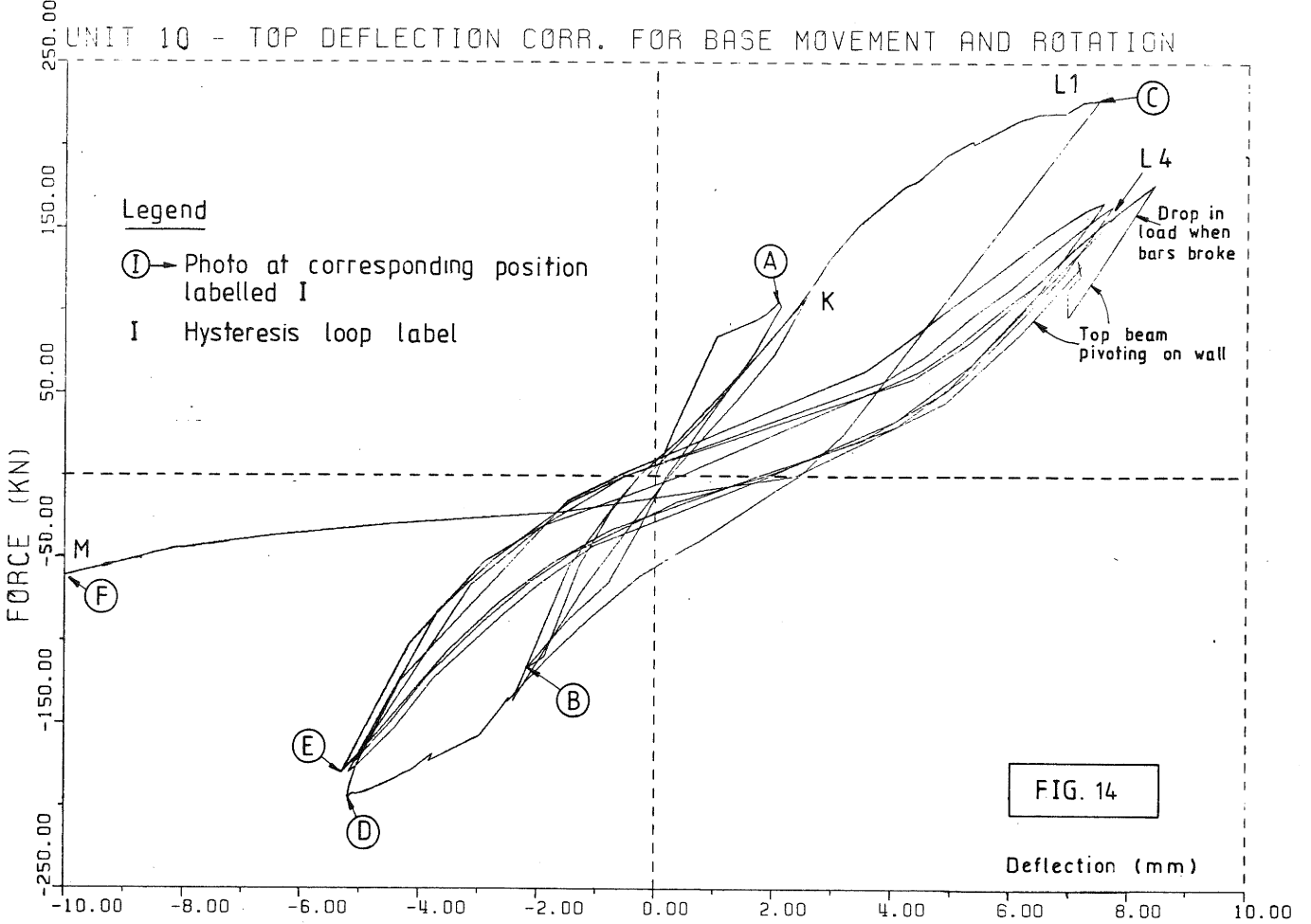
5.3 Performance of Walls with Minimum Grout and Reinforcement Quantities

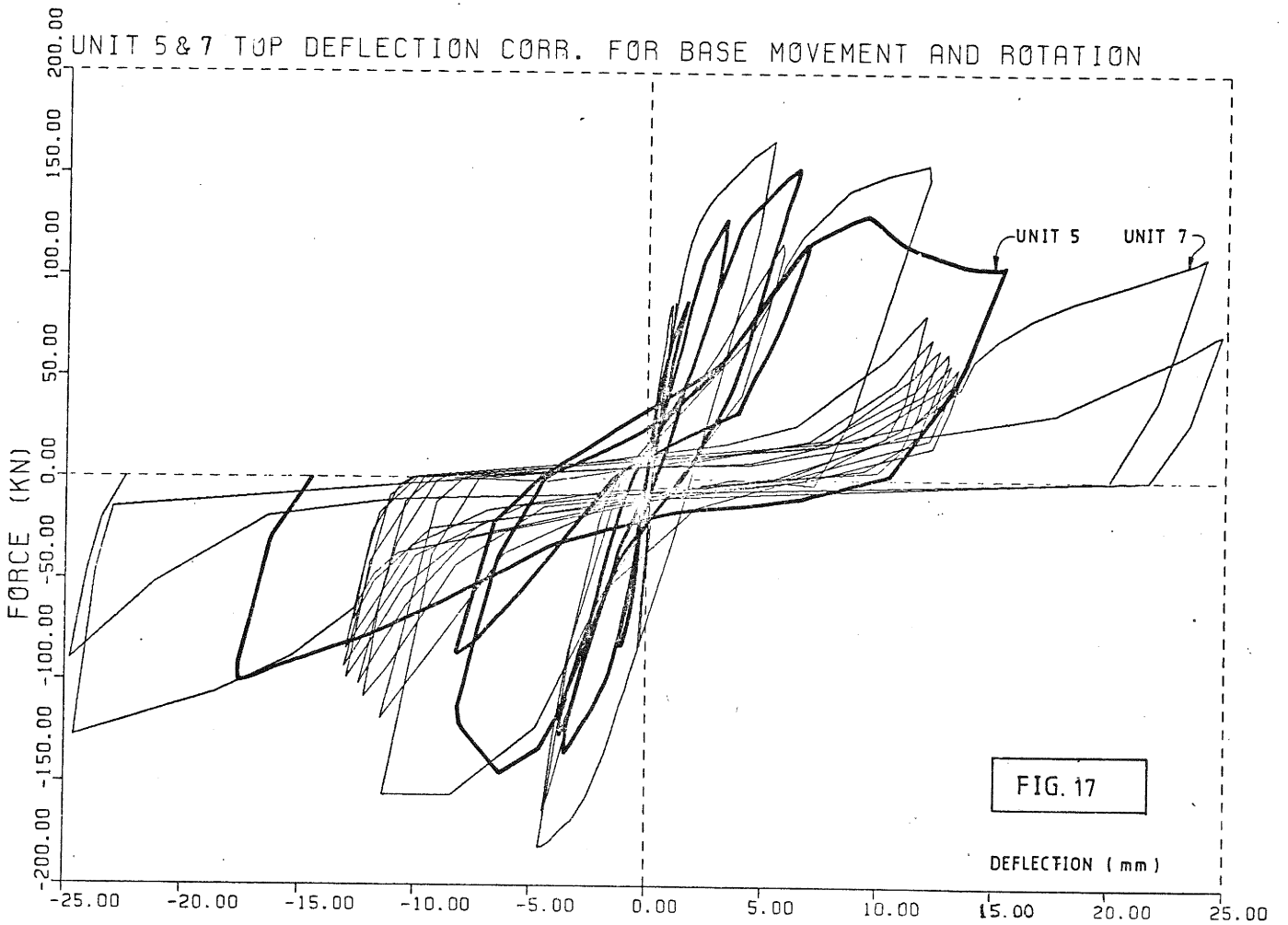
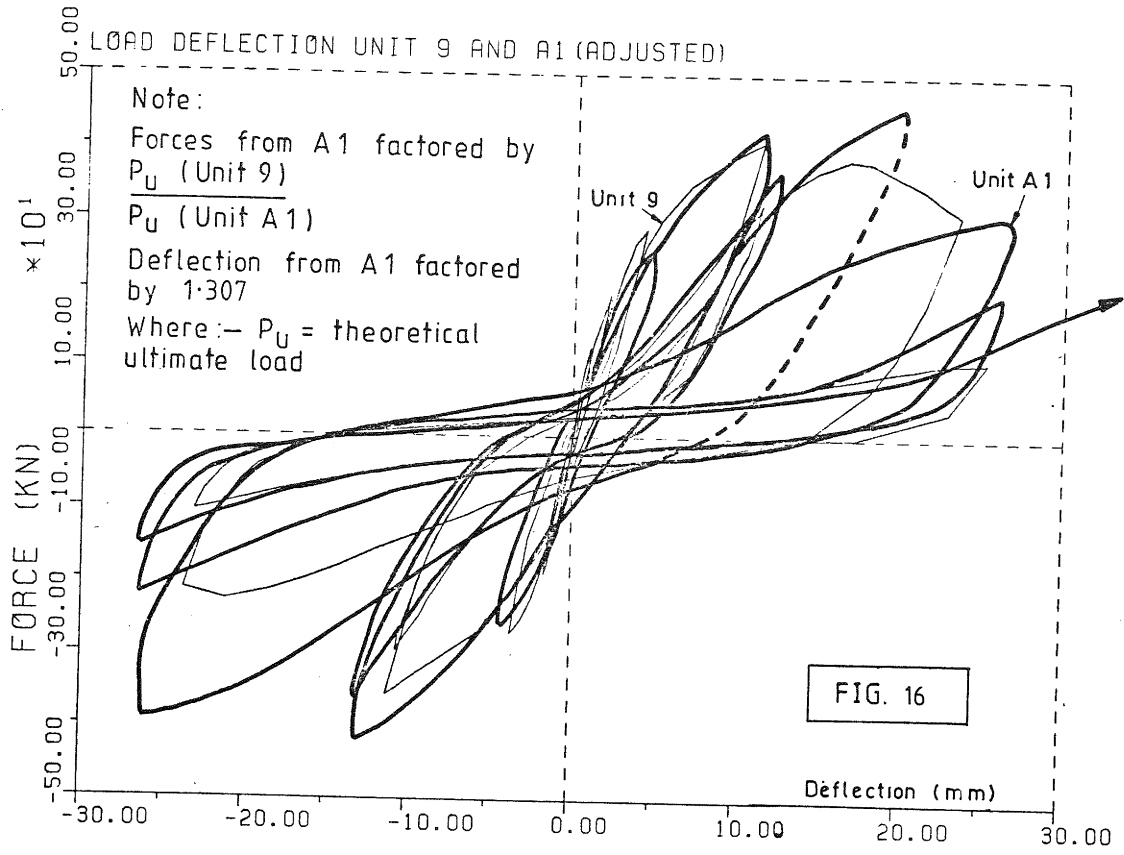
Units 2 and 4 were designed to test these conditions. By implication, partially grouted wall sections are not permitted in the Draft Code¹ in regions other than those which would experience unanticipated and modest ductility demand. Units 2 and 4 performed well and their inelastic behaviour was not characterised by sudden degradation. However, the strength of Unit 4 was not at all predictable from theory (see Section 5). Moreover, the test series did not consider serviceability criteria which, rather than seismic criteria, form the basis of the minima.

5.4 Size and Spacing of Reinforcement

Units 3 and 7 had similar reinforcement ratios (0.24 - 0.28%), both horizontally and vertically, the pattern in the former case consisting of small bars (D10) at close centres (200 mm) and the latter being D16 at 600 mm approximately. Both units evidenced sliding failures at high deflection ductilities (about DF = 6) and the loss of energy dissipating capability was more dramatic in the case of Unit 7.

The degradation of Unit 7 commenced at a DF in excess of 2 and was associated with flexural compression failure at the top edges. This developed into a sliding failure. Flexural compression failure was not at all apparent in the case of Unit 3, and sliding became evident only at DF in excess of 8. With essentially the same reinforcement ratios in respective directions, neither bending nor shear theory explains the difference in failure mode. It is therefore related rather to





bar size and spacing. Hence small bars, closely spaced "basketting" reinforcement¹² are to be aimed for in detailing of energy-absorbing areas of RHM construction, in order to reduce degradation of strength and stiffness.

Using the measured mean compressive grout strength (f_g) of 26 MPa, the formula of Priestley and Bridgeman⁷ describing shear transferred by dowel action of a Grade 380 bar of diameter, d , becomes

$$V \text{ (kN)} = 0.062 d^2 \quad (1)$$

Shear friction theory, on the other hand, with a coefficient of friction of 0.7¹, predicts a shear strength from a Grade 380 bar of diameter, d , of

$$V \text{ (kN)} = 0.201 d^2 \quad (2)$$

or three times the strength of a dowel. The latter formula predicts a resistance to sliding of 154 kN for Unit 7 and, when the end bars are discounted following compression failure of wall ends, one third of this strength or 51 kN is predicted. The observed values of about 150 kN and 60 kN respectively compare well with shear friction theory, and dowel action does not therefore provide significant resistance.

5.5 Reinforcement Direction and Failure Mode:

The problem of shear resistance of reinforced masonry is in reality very complex and it is likely that both horizontal and vertical reinforcement play a significant part in providing resistance before the onset of visible cracking, the former providing a horizontal tension component in conjunction with diagonal masonry compression struts, and the vertical reinforcement providing a shear friction resisting mechanism. With the development of a failure mode through horizontal cracking, the vertical reinforcement alone provides shear resistance; however, the widening of diagonal tension cracks is resisted primarily by horizontal reinforcement.

Further, the theoretical separation of bending and shear mechanisms is difficult. There is little difference between the predicted ultimate shear strength of the walls tested using bending theory from that assuming a shear friction resisting mechanism, for the present series of tests, as shown below: The theoretical ultimate wall moment is given by

$$M_u = A_s f_y (b/2 - a/2) \quad (3)$$

$$\dot{\bar{}} = 0.45 b A_s f_y \quad (4)$$

in the case of vertical steel being uniformly distributed across the wall length where

b = wall width,

a = depth of compression block at ultimate moment.

For the present test series, with point of contraflexure at mid-height,

$$V_u = 2 M_v / h \quad (5)$$

$$= \frac{2 \times 0.45 \times 1600 \times A_s f_y}{2400}$$

$$= 0.6 A_s f_y \quad (6)$$

On the other hand, shear friction theory¹ predicts an ultimate shear strength of

$$V_u = 0.7 A_s f_y \quad (7)$$

Thus the observation that most walls failed close to their theoretical ultimate flexural load does not prove that the failure mechanism was flexural and not shear, or vice-versa.

Figure 18 indicates the degree of correlation between the actual ultimate (maximum) load sustained by the respective test units and that predicted by the postulate of various governing resisting mechanisms. The theoretical ultimate load for diagonal tension was calculated from the sum of the yield strengths of bars cut by a 45° crack.

The assumption involved in capacity design procedures, in particular, is that a structural unit will choose the weakest internal force resisting system at the mode of failure; hence ductile slender elements are made stronger in shear, a "non-ductile" failure mode, than in bending. In Figure 18, the symbol in brackets after the unit number indicates the mode of failure observed from the pattern of cracking before any rapid strength degradation took place (around $DF = 2$ to 4).

In some cases, the mode of failure does not correspond with the predicted weakest resisting mechanism. Nevertheless, except in the case of Unit 4 which had no horizontal steel, the observed failure mode is associated with a strength that accords with theory, and is not greatly different from the predicted weakest mode. Units 8 (and 9) demonstrates convincingly that a flexural mode of failure can be forced by the provision of a large quantity of horizontal reinforcement.

5.6 Comparison of Theory and Experiment:

Prism tests¹¹ for the materials used gave an ultimate strength, f'_m , of 16 MPa, average modulus of elasticity, E_m of 6.66 GPa. From this information, the theoretical wall, displacement at first yield of a bar, $\Delta y'$, was then calculated. 'Yield displacement' was then defined as:

$$\Delta y = \Delta y' \times \frac{M_u}{M_y}$$

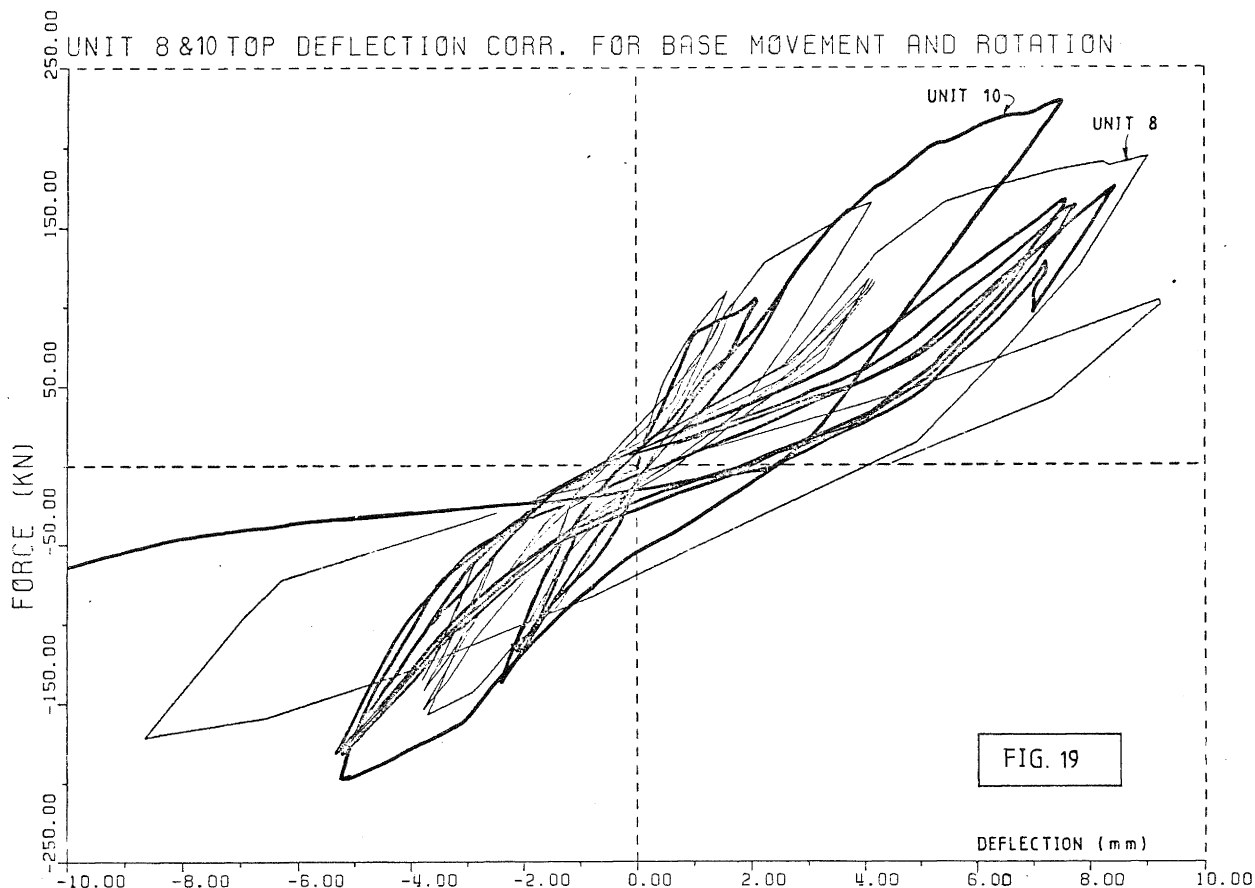
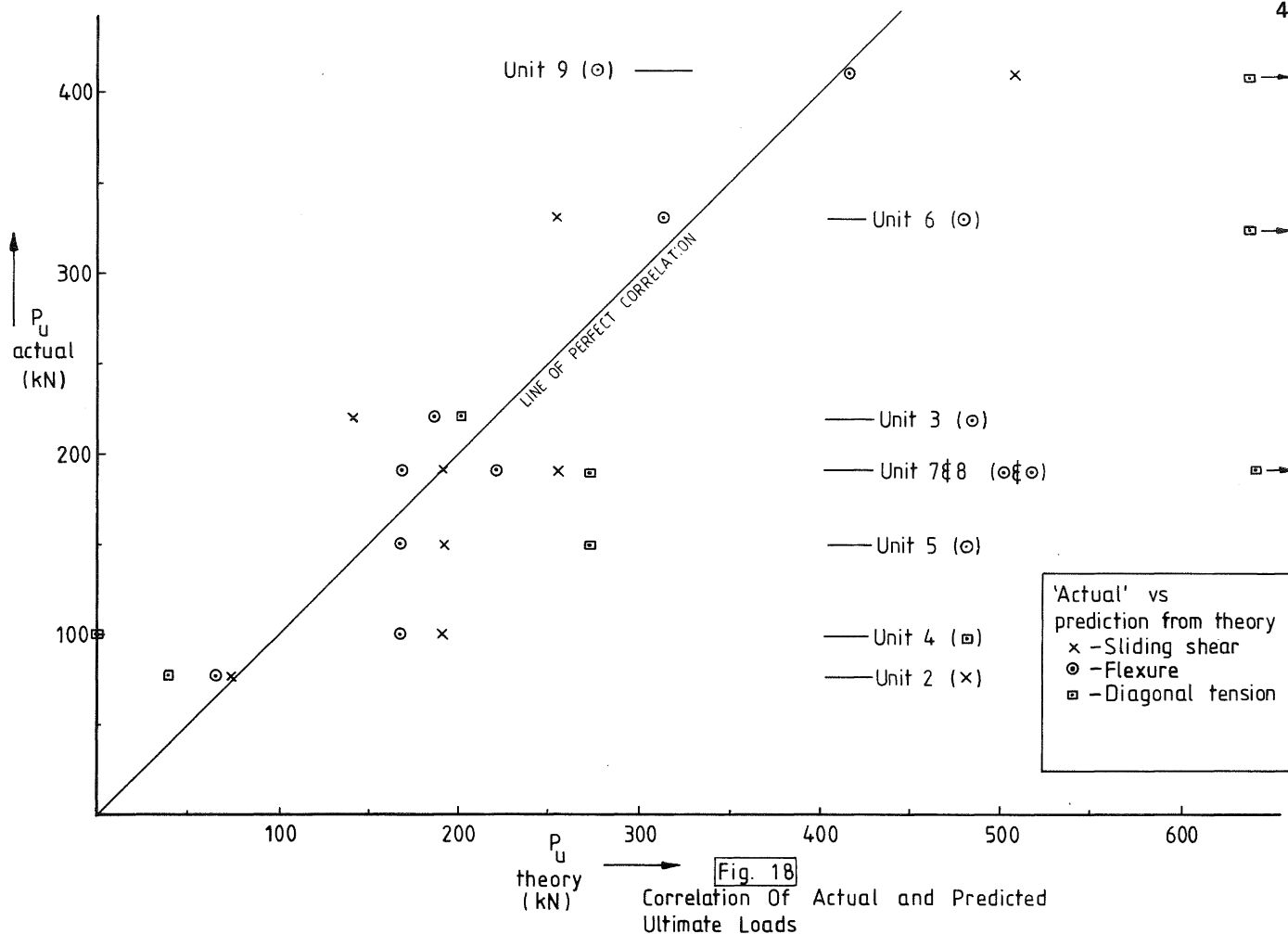


TABLE 3

THEORETICAL AND EXPERIMENTAL LOADS DEFLECTIONS
AND ELASTIC STIFFNESS VALUES -

UNIT	P _Y (kN)	THEORETICAL		EXPERIMENTAL		THEORETICAL	EXPERIMENTAL
		y (mm)	K _Y (kN/mm)	y	K _Y ^{***}	Δu (mm)	Δu (mm)
2	49.5	1.45	34.14	0.05*	92.0	41.24	20
3	111.6	2.34	47.69	2.69	65.7	18.82	> 20
4	124.4	2.31	53.86	* *	48.0	23.07	15
5	124.4	2.31	53.85	4.20	46.0	23.07	14
6	229.5	3.68	62.36	3.23	64.4	11.14	20
7	125.7	2.33	53.95	2.80	87.6	23.11	15
8	151.7	2.45	61.92	5.06	67.7	17.51	
9	250.1	4.31	58.03	5.49	0.0	9.39	16

* units effectively uncracked at this stage

** results indeterminable from plot

*** measured from load/deflection plots

Table 3 compares theoretical and experimental results. Generally actual yield deflections are greater than predicted by theory. This is probably mainly due to slippage along horizontal mortar joints.

The wall ultimate deflections can be calculated using theory presented by Priestley¹⁶ assuming a rectangular ultimate compressive block of strain 0.002. From Table 3 the ultimate deflections of the lightly reinforced walls are less than predicted by theory because brittle cracking of the blocks occurred before masonry crushing as evidenced on Unit 7, is the compression block did not reach the high assumed strain levels.

If we ignore partially filled Units 4 and 5, the average experimental stiffness of remaining cracked units is 77.9 kN/mm (c.o.v. of 17%) which implies an effective modulus of elasticity of 2.4 GPa for fully grouted units, rather less even, than the 5.0 GPa proposed by Priestley¹⁷. Note that the effective modulus is less than the measured prism modulus E_m described above because of the contribution of both flexural and shear cracking. The partially grouted units exhibit an average stiffness of only 60% that of fully grouted units.

5.7 Influence of Preloading Reinforced Masonry:

Comparison of hysteresis loops from Units 6 and 8 (Figures 10 and 12) shows a substantially better performance of the preloaded Unit 6, in terms of both strength and ductility.

The increase in observed strength of about 120 kN is approximately equal to the preload (160 kN axial compression) multiplied by a coefficient of friction of 0.7. Thus the lateral load capacity

of a wall with preload may be that without preload plus the preload $\times 0.7$. However, two sets of tests reported by Priestley⁸ show an increase in ultimate strength as a result of preloading of about 65% and 150% of the theoretical shear friction provided by the preload, and therefore at this stage, quantitative information for the designer as to the benefits of preload is not conclusive. Mayes et al¹⁵ report an increase in ultimate strength as a result of preload, but although a better ductility factor resulted, the maximum displacement achieved was reduced.

5.8 Effectiveness of Repair of Reinforced Masonry:

Figure 19 gives a comparison of hysteresis loops obtained for Unit 8 and for the same Unit after repair by epoxy injection and subsequent testing (Unit 10). At the time of failure of the previously heated reinforcing bars, the behaviour of the repaired wall was at least as good as Unit 8. Thus, repair by epoxy injection is a feasible proposition.

6. CONCLUSIONS:

6.1 The system of loading the wall in double curvature did not give rise to significantly different results compared with the single curvature loading reported from the University of Canterbury, at least for the walls compared. However, where single curvature loading would inhibit the propagation of diagonal cracking, this correspondence would probably not occur.

6.2 Partially grouted wall units exhibit ductility which compares well with that of fully grouted units. However, there is a significant reduction in strength and stiffness.

6.3 Wall units with minimum grout and reinforcement quantities resisted a load near to their theoretical ultimate and did not suddenly degrade.

6.4 The tests confirm that, at least for

the height to length ratio of walls tested, sufficient quantities of horizontal steel can ensure an essentially flexural failure within the range of ductility demand currently envisaged.²

6.5 The loading system highlighted the reduced strength near the top of units, both flexural compression and shear. This is probably a result of grout shrinkage from face shell interior surface. Further testing is under way to substantiate this.

6.6 Small diameter reinforcing bars at close centres lead to markedly better inelastic performance than similar quantities in the form of a few large bars. The additional limitations on bar spacing required by the Draft Code² in potential plastic hinge regions seem adequate.

7. ACKNOWLEDGEMENT:

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