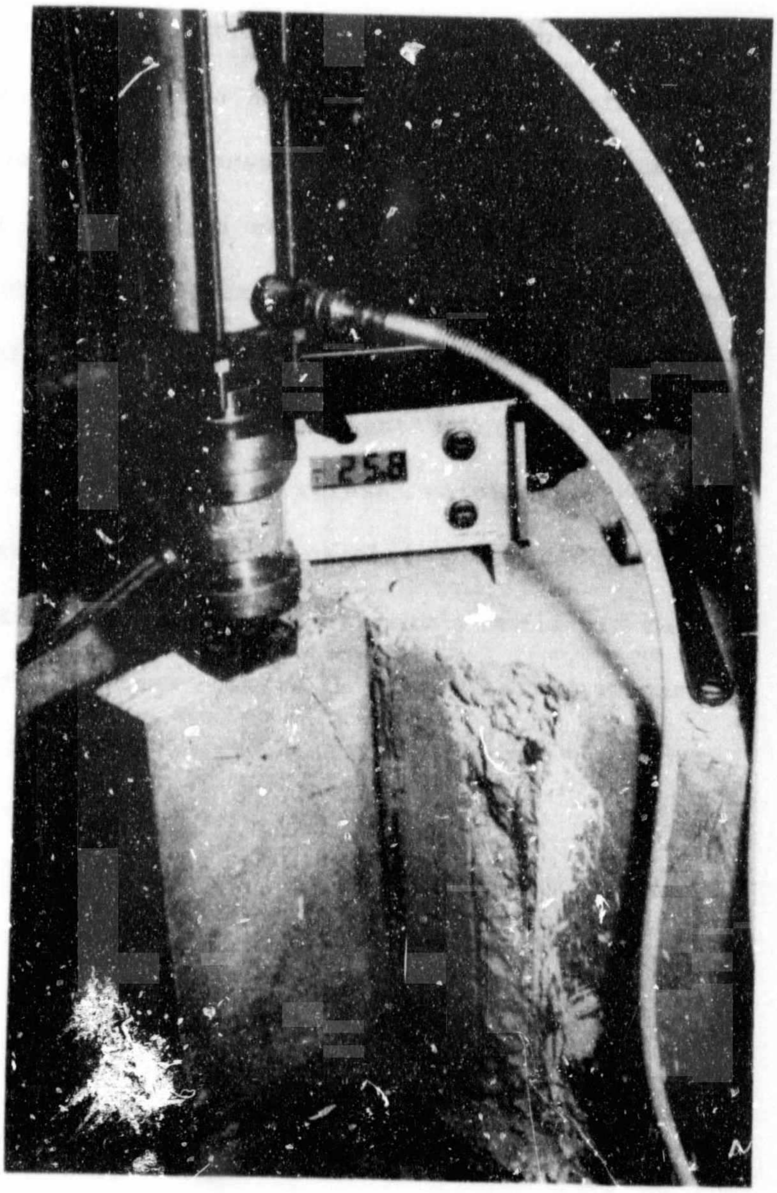

FIGURE 3.12 PHOTOGRAPH SHOWING DEEP PRISMATIC CORBELS WITH SIGNIFICANTLY REDUCED a/d RATIO.

clined diagonal crack as indicated in the figure. The measured average maximum shear stress for these specimen types at failure was of the order of 5MPa. An extrapolation of the curves derived in Chapter 4, shown in Figure 4.8, indicates that a resistant shear stress considerably in excess of 5MPa would be anticipated at an a/d ratio of 0,2 in terms of this extrapolation. A value of approximately 10MPa is in fact consistent with these curves for an a/d ratio of 0,2. The test results on these specimens of relatively extreme geometry are consistent with the order of magnitude of the ultimate shear stress values recorded by Walraven and Reinhardt and thus confirm that a cut-off or limit to the curves depicted in Figure 4.8 at extremely small a/d ratios appears to be likely. The test results for these corbel specimens are also tabulated in Appendix A.

3.5 GENERAL CONCLUSIONS FOR CORBELS

1. In general, for all other parameters being equal, the haunched in-situ corbel unreinforced for shear carries 3 to 4 times the vertical load at ultimate than the equivalent bolt-on corbel unreinforced for shear. As the bolt-on corbel relies on dowel action and friction for shear resistance, it can be concluded that an assessment of dowel action contributing of the order of 10% in the monolithic corbels, with a/d ratios somewhat smaller than unity, is reasonable. Since previous research[40] shows the contribution of dowel action to be about 20% for beams with a/d ratios in excess of 2, it can be concluded that the relative dowel action contribution for monolithic sections decreases with a/d ratio. The implication of this is that aggregate interlock shows a comparable increase with reducing a/d ratio in monolithic sections. This relative load sharing assessment is further justified by the observation that consistent displacements for both types of corbel were attained at the ultimate limit state of shear.

2. Both types of corbel, if unreinforced for shear, exhibit insufficient ductility to meet the requirements of a design which necessitates a reasonable redistribution of force.

3. For bolt-on units the initial shear crack is a dowel crack; for the in-situ units the initial shear crack is a steeply inclined diagonal crack.

4. If suitably reinforced for shear, both types of corbel can have their shear capacities raised to a value greater than their flexural ultimate limit state, or, for precast units, such that a ductile shear displacement mode is attained. Both types of corbel can thus be designed for the required load based on a ductile ultimate limit state. The shear reinforcement details would, however, differ between the in-situ and bolt-on units. The most important shear reinforcement in the bolt-on corbel is the vertical link encompassing the bolt. The shear reinforcement for the in-situ corbels is in the form of horizontal links as recommended by various codes of practice. It appears that neither current models for shear nor current codes quantify the performance of this type of shear reinforcement and a proportion of the flexural reinforcement is generally prescribed. While this appears to be an acceptable approach since shear capacity is thus linked to flexural capacity, it is considered appropriate that a universal model for shear should be able to qualify and quantify the behaviour of such horizontal links. This is considered particularly important because of the potential use of horizontal links in a wide variety of structural elements, such as pile-caps, deep beams, pad foundations and deep slabs.

5. Special attention must be paid to moving the load back from the face of the corbel in order to prevent front splitting failure. This is a general precaution and applies to any structural situation where large load bears near a free edge, including instances such as supports to simply-supported beams. The danger of this mode of premature failure is increased as the cover to the flexural reinforcement increases. It is thus recommended that a general requirement, of moving the edge of the load horizontally away from the face of the element by a cover depth beyond the last straight portion of the flexural reinforcement, is adhered to.

6. The strut-and-tie analogy for designing corbels tacitly assumes a flexural ultimate limit state. According to the tests of this programme, this was not generally achieved if shear reinforcement was not included in the corbel. If appropriate shear reinforcement is included, the strut-and-tie analogy is an excellent and simple predictor of the ultimate limit state[22].

7. For in-situ corbels unreinforced for shear, the resistant shear stress increased with increasing flexural steel ratio, although it was only moderately sensitive to this parameter. It is, however, significant that a universal model for shear should reflect this trend even at this relative extreme of the slope of the diagonal shear crack. An important observation in respect of the influence of the flexural reinforcement for the elements un-

reinforced for shear is that the shear resistance was impaired if the steel was not adequately anchored beyond the point of intersection of the diagonal shear crack. Adequate anchorage implies either mechanical anchorage or a full tension lap length in general. For compact corbels this can necessitate the welding of a transverse bar to the end of the flexural reinforcement. In contrast to this, local bond did not appear to influence ultimate shear resistance measurably, and a series of tests is undertaken in Chapter 5 in order to confirm this initial observation.

8. The shear resistance of corbels, particularly those unreinforced for shear, is dependent on concrete strength, and a universal model for shear evaluation should reflect the influence of this parameter.

9. Where cover to the specimens was varied , it was found that the shear resistance reduced as the effective depth reduced. This observation was consistent with the code approaches in general which assess shear stress in terms of the effective depth and not overall depth.

10. The evaluation of the prismatic corbels unreinforced for shear indicates that the presence of moment and shear in close proximity does not influence the shear resistance of the diagonal shear crack. It is postulated, although not observed in this test series, that the presence of large shear forces could inhibit

ductility of the moment-rotation relationship for a hinge formed at the face of a continuous support, although the attainment of the ultimate moment capacity is not impaired.

11. Although the haunched corbels unreinforced for shear appeared to exhibit a slightly lower shear resistance than the prismatic corbels unreinforced for shear, this discrepancy was not significant and also becomes irrelevant immediately shear reinforcement in the form of horizontal links is introduced. It is likely that the model to be proposed within the scope of this work could be used to investigate the geometric limits at which the haunched corbel unreinforced for shear might begin to exhibit significantly impaired shear resistance, although this will not have meaningful practical significance owing to the generally mandatory inclusion of horizontal link reinforcement.

4 PUNCHING SHEAR IN FLAT PLATES

A sense of uncertainty and disquiet is occasionally experienced by practicing engineers when evaluating the performance of reinforced concrete slabs in punching shear. This is certainly not without some justification. Current codes vary by substantial margins and these in turn can vary radically from older codes in the conceptual approach adopted and indeed in the final trends and predictions. In addition, tes[illegible] are not plentiful, especially in the evaluation of the [illegible] performance of deep slabs.

In this chapter the fundamentals of punching shear design are examined and current codes compared with test results on various slab and beam specimens. The feasibility of a universally applicable model for shear which also includes an evaluation of plate and slab elements is also given consideration. Certain parameters have received an in-depth examination in terms of this test programme, owing to the nature of the queries most often encountered, and also to determine their relevance and influence in a general model for shear.

4.1 SOME ASPECTS OF CURRENT CODES

The philosophy of evaluating the punching shear at a perimeter of 1,5h from the face of the column or column head was first introduced in CP110 and is now probably in fairly general use in countries using British codes. This approach has also been adopted in the South African code SABS 0100[6]. There is some justification for this concept. Laboratory tests on slabs indicate that, if unconstrained, the slab will fail in punching shear in a conical format which intersects the upper surface of the slab at approximately 1,5 to 2 times the depth, h, of the slab, from the column face. This corresponds to an a/d ratio of approximately 2 which is consistent with the trough (or minimum value) of Kani's valley of diagonal failure. If the failure stress of the specimen unreinforced for shear is evaluated on the resulting punching perimeter at the surface of the slab, it is immediately evident that the shear stress at failure for this situation coincides approximately with that for beam specimens unreinforced for shear. It is primarily for this second reason that the perimeter of 1,5h from the column face was chosen as the "affine" surface for the evaluation of punching shear in CP110. The physically observed dimensions of the unconstrained punching cone do, however, appear to have far more fundamental significance in terms of a qualitative and quantitative assessment of the mechanics of punching shear failure than merely providing a convenient affine surface.

If the punching shear capacity of the slab is evaluated by code and accepted design procedures, it is apparent that any load within the surface perimeter of the punching cone zone cannot logically be considered to be contributing to shear failure[36]. This phenomenon will have negligible effect on results for thin to medium thickness slabs with normal span to depth ratios, but will have a marked influence on deep slabs and slabs with small span to depth ratios. This phenomenon also indicates that the slope of the punching cone could become steeper than the unconstrained slope depending on the interaction between increased shear resistance as the slope increases and increased applied shear force owing to the reduction in the portion carried within the limits of the cone. There is thus considerable justification in developing a shear model which depends on the function of slope of the diagonal shear crack of the punching cone, as this function can then be minimised to establish the lowest shear collapse load for a particular structural geometry. These calculations will be relatively easily handled for slabs subjected to uniformly distributed loads, which is consistent with the loading normally used to establish the flexural ultimate limit state for slabs.

CP114 was condemned by many as being unsafe with respect to shear evaluation in particular and this is indeed why the approach in

CP110 varies significantly in this respect. CP110, however, is certainly not without flaws and has possibly received more criticism of its shear provisions than its predecessor. A radical philosophical change in code approach based primarily on research results has the danger of being untried over the full range of structures likely to be encountered by the code user. The inability to minimise the collapse energy function because the punching perimeter is fixed is one of the reasons why CP110 gives anomalous results in some instances, although the philosophy of starting at approximately an unconstrained perimeter is sound. This limitation appears to have been redressed in a simplistic manner in the draft BS0000, by checking an increased resistant shear stress at the support face.

However, CP110 does predict the punching shear capacity of thin to medium thickness slabs with "normal" span/depth ratios very well. This is evident in undertaking tests on a number of slab specimens. The code has also proved itself over the past years in application to structures of this category. There is thus considerable justification for maintaining the perimeter at approximately the position of unconstrained minimum collapse shear load, as opposed to other codes where a reduced perimeter is used with an enhanced resistant shear stress, but attention should be given to instances where anomalous results could be obtained. This implies the need for an evaluation of specimens having geometric properties that induce anomalies of this kind, in particular a study of the shear performance of deep slabs subjected to punching is warranted.

4.2 THE TEST PROGRAMME

In allusion to the observed areas of discrepancy mentioned above, the major parameters intended to be investigated were depth of slab and variations in shear-arm to effective depth ratio, (a/d ratio). In addition, it was felt necessary to consider the effects of change in percentage of flexural steel reinforcement and grade of concrete in conjunction with the effects of the major parameters in order to assess the universal influence of these parameters on resistant shear stress.

Other influences which required evaluation within the scope of such an assessment are the effect of moment transfer applied simultaneously with punching shear, methods of reinforcing for punching shear and the effects of prestressing the slab specimens. The prestressed slab specimens are evaluated in 4.7.

Some fifty reinforced concrete slab specimens were tested in this programme. The depths varied from 50mm to 300mm, the a/d ratios from 0,3 to 2,5 and the reinforcement ratios from 0,5% to 1,0%, while the grade of concrete varied from 20MPa to 40MPa. The typical plan dimensions of the specimens were 600mm by 600mm. Load was applied to the specimens through a "punching platen", intended to simulate a column or patch load on the slab. These punching specimens were generally square and varied in dimension from 80mm by 80mm to 120mm by 120mm. The slabs were supported symmetrically, on all four sides, by a simple, square line support, giving varying span dimensions in plan. This simulated the simplest arrangement of load on a slab analogous to the equivalent case of two point or single point loading of beams, in order to study the influence of variations in a/d ratio (in a two-dimensional sense) for slabs. Although this "line load" is hypothetical and not encountered in flat plates in practice, the objectives of the tests were specifically to determine shear resistance effect, with the load effect being considered independently. Adjustments in the load effect which can contribute to shear failure need to be made because the prototype loading closely approximates a uniformly distributed load in many instances. This approach is thus entirely consistent with those of two-point or single point loading for beams, which have been used by many researchers in the field of shear performance of reinforced concrete beams[24,29,40]. Standard high yield steel reinforcement of 450MPa average yield stress was used in the slab specimens. This was placed in the form of an isotropic mesh, welded at the perimeter only, to provided adequate anchorage. In certain of the specimens electric resistance strain gauges were attached to the reinforcement in order to monitor its behaviour under the influence of punching shear forces. Typical views of the reinforcement for these specimens are indicated in Figure 4.1 and typical 300mm deep slab specimens are shown in Figure 4.2.

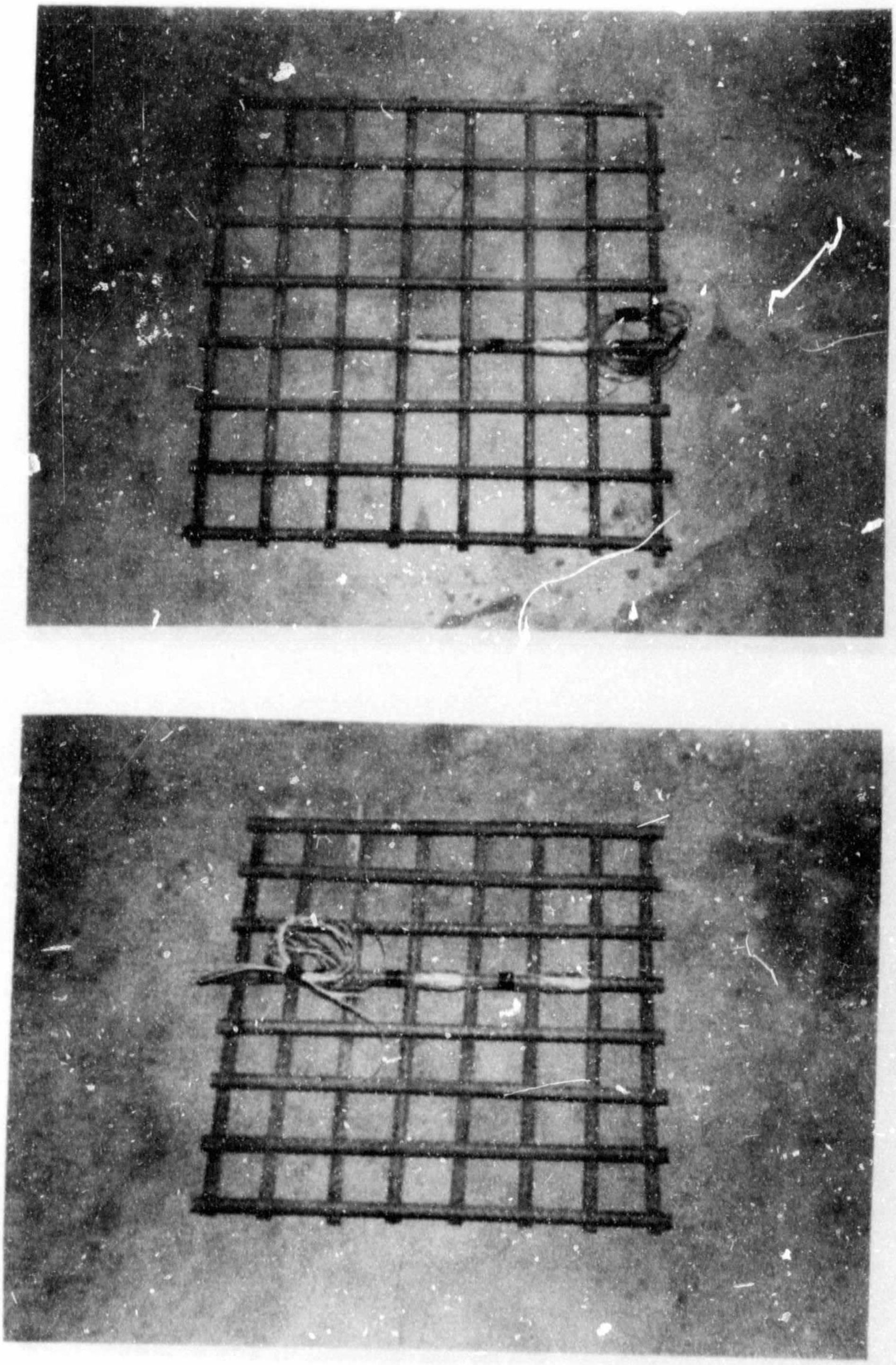

FIGURE 4.1 FLEXURAL REINFORCEMENT MESH USED IN SLAB SPECIMENS, SHOWING PLACING OF ELECTRIC RESISTANCE STRAIN GAUGES.

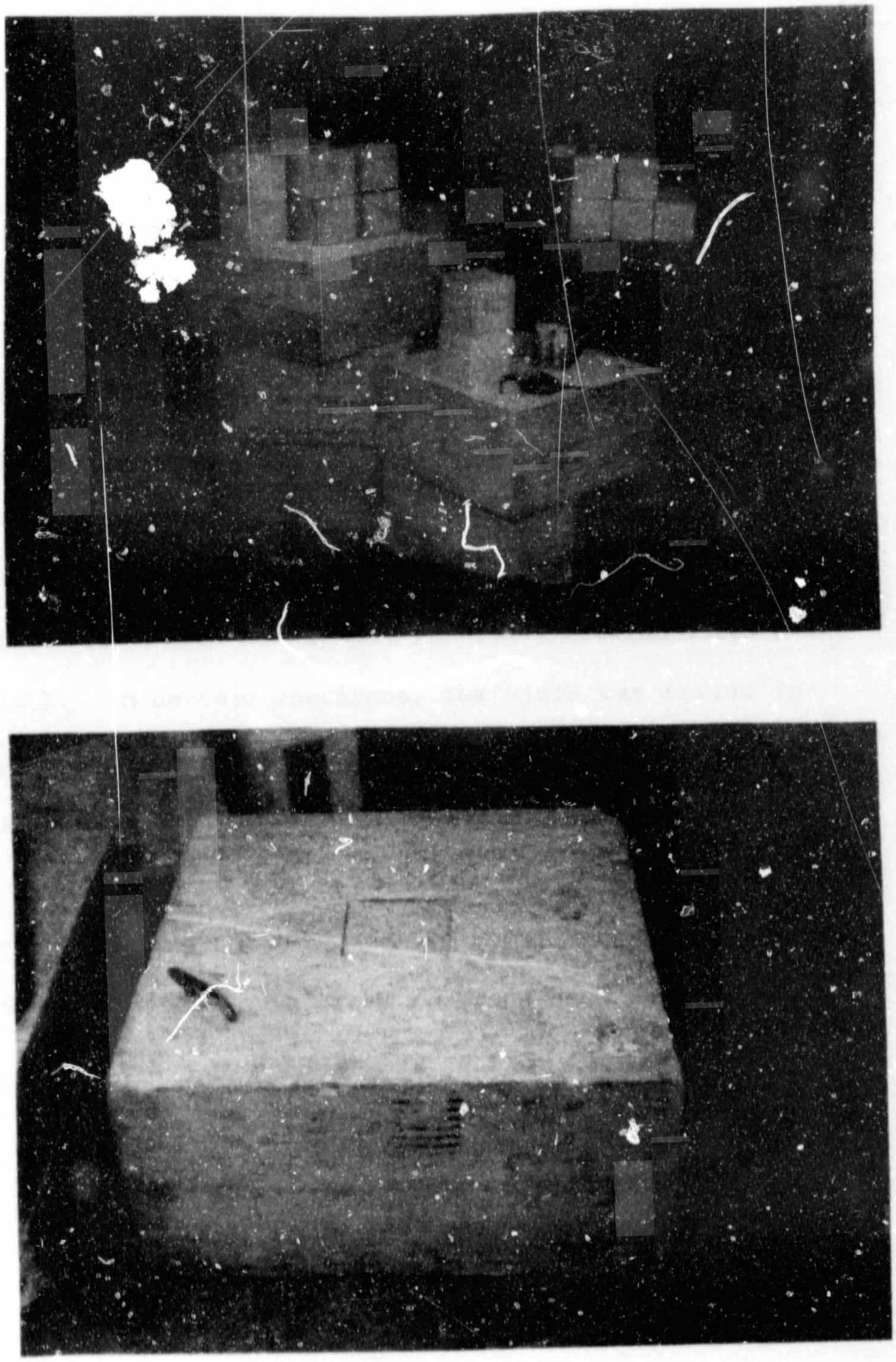

FIGURE 4.2 TYPICAL 300mm DEEP SLAB SPECIMENS TESTED IN PUNCHING SHEAR.

A depth of slab specimen of 300mm was the deepest that could reasonably be accommodated by the size and load capacity of the hydraulic testing machine. Because of this limitation some thirty beam specimens unreinforced for shear were tested in parallel with the slab specimens. The beam specimens varied from 150mm to 1200mm in depth, with other parameters remaining identical to the slab specimens, the intention being that the results could be correlated over the range of depths that coincided and could thus be extrapolated to 1200mm depth of slab with a high level of confidence. The beams tested were generally between 130mm and 150mm in width. In certain specimens, the width was varied intentionally to assess its possible influence on resistant shear stress. There was, however, no measurable effect of width of section on resitant shear stress and this was thus discarded as a parameter influencing the proposed universal model for shear. Typical 600mm and 1200mm reinforced concrete beam specimen, unreinforced for shear, are shown in Figure 4.3.

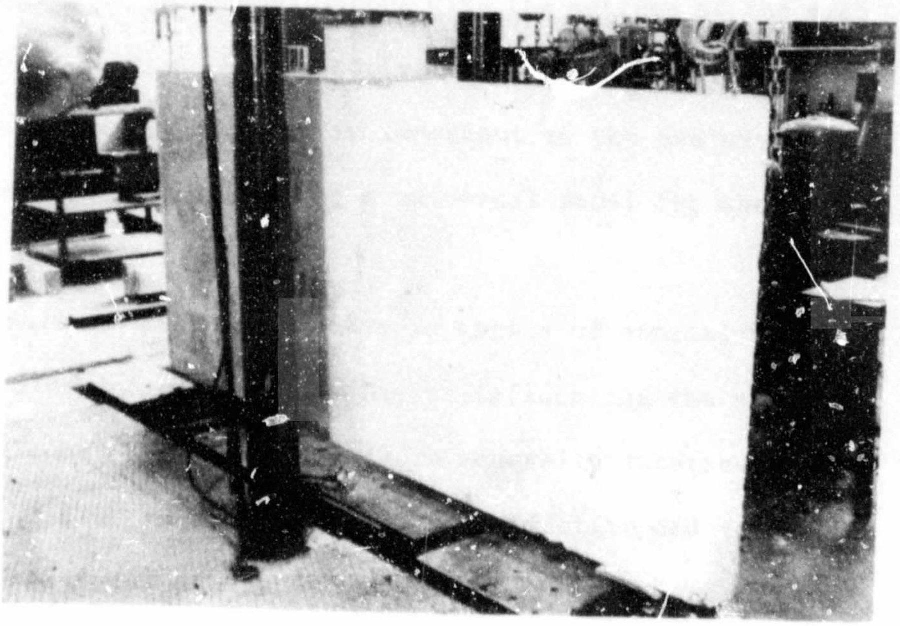

FIGURE 4.3 EXAMPLES OF 600mm AND 1200mm DEEP BEAM SPECIMENS TESTED IN THIS SERIES.

4.3 TEST RESULTS AND THE INFLUENCE OF THE MAJOR PARAMETERS

All the slab specimens tested failed in punching shear, in that they developed the typical cone of failure and the load-deflection curves all indicated brittle failure with no ductility. In the lightly reinforced specimens there was usually some evidence of the onset of yield line crack patterns observed at failure, although yield line collapse never occurred. A typical intersection of the punching cone with the surface of the slab is shown in Figure 4.4. A qualitative assessment of this form of unconstrained punching surface is important in the evaluation of punching shear performance using a universal model for shear.

The slab specimens with a/d ratios in excess of approximately 2 showed no indication of constraining or influencing the shape of the punching cone. This cone of failure generally intersected the surface of the slab so as to result in an effective a/d ratio (in two-dimensional form) of approximately 2. The shape of the cone intersection in these cases approximated the shape used in CP110, in that some arbitrary rounding of the corners appeared to occur generally. This is evident in Figure 4.4.

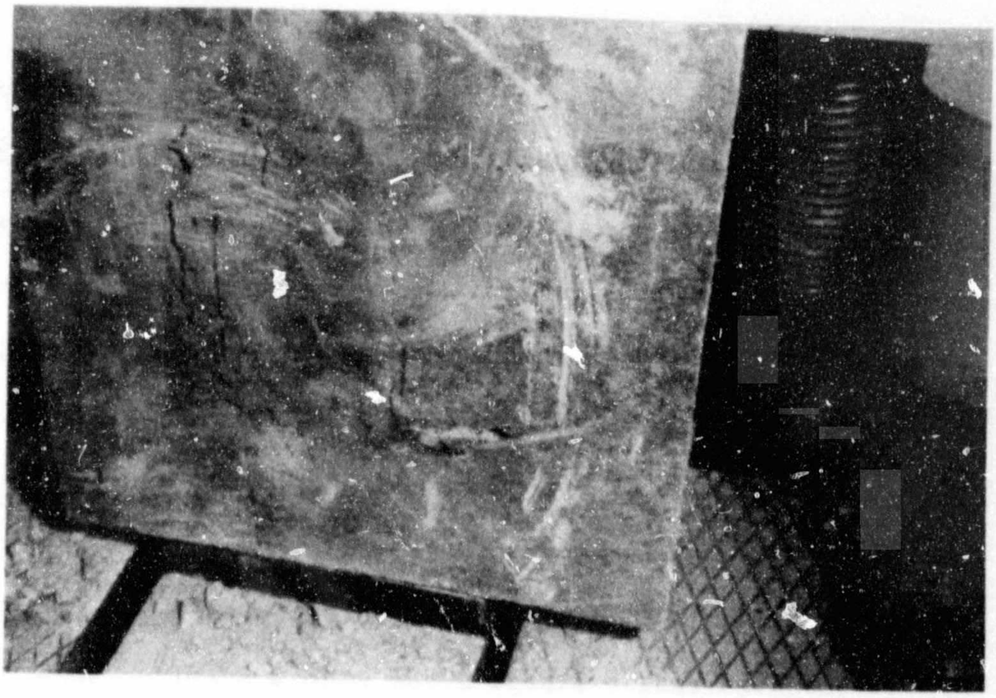

FIGURE 4.4 TYPICAL PUNCHING CONE INTERSECTION WITH SLAB SURFACE.

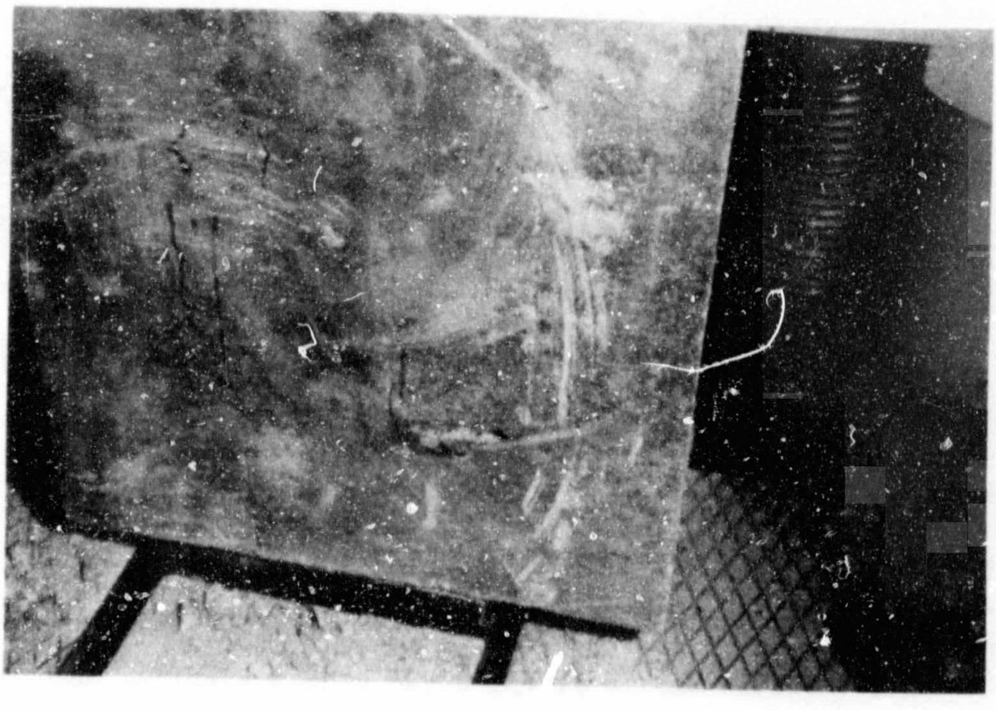

FIGURE 4.4 TYPICAL PUNCHING CONE INTERSECTION WITH SLAB SURFACE.

Taking cognizance of all the adjustments that have to be made to the mean ultimate shear resistance recorded in the tests, i.e. adjustments for depth, concrete grade, flexural steel ratio, from mean resistance to characteristic resistance. and further adjustment by a partial material factor of 1,25, fairly good correlation between the test results and the predicted CP110 and BS0000 code values were obtained for the slab specimens with "unconstrained" a/d ratios.

In tests where the punching cone was forced to form more steeply than it would if unconstrained, an enhanced value of resistant shear stress was observed, which was similar to that recommended by these codes. The plan shape of the affine surface at 1,5h from the column or support face has been changed from a a rounded format in CP110 to a square format in the draft BS0000. The implication is that the draft code will result in slightly less conservative assessments than CP110 for punching shear in general. For a comparative evaluation of the test results at the specific code perimeters of 1,5h, a square affine surface, consistent with BS0000, was used in order to make comparisons of code design stresses and test stress results.

In the following sections the general parametric behaviour of the specimens tested has been summarised and presented graphically for clarity. The detailed test results relating to these summaries are tabulated in Appendix B. A new, universal model for shear which is intended to reflect the observed universal parametric behaviour of shear resistance is developed in Chapter 8. This model can be applied to punching shear evaluation directly if cognizance is taken of a slightly reduced perimeter as is evident in the following sections. The model stress values are included in Appendix B for comparative purposes.

Of the parameters affecting the test results the following could be clearly identified:

4.3.1 FLEXURAL REINFORCEMENT RATIO

The percentage of flexural reinforcement, if adequately anchored beyond the point of intersection of the shear crack with the flexural reinforcement, did influence the ultimate resistant shear stress of the specimens tested. It is of note that strain measurements on the flexural reinforcement indicate that it is unlikely to be at yield in the vicinity of the point of intersection of this reinforcement with the diagonal shear crack. This observation is evident in the load-strain curves of Figure 4.5 and cognizance is taken of this in the formulation of a universal model for shear. While the limited plan dimension of the slab test specimens means that the shape of the bending moment diagram in

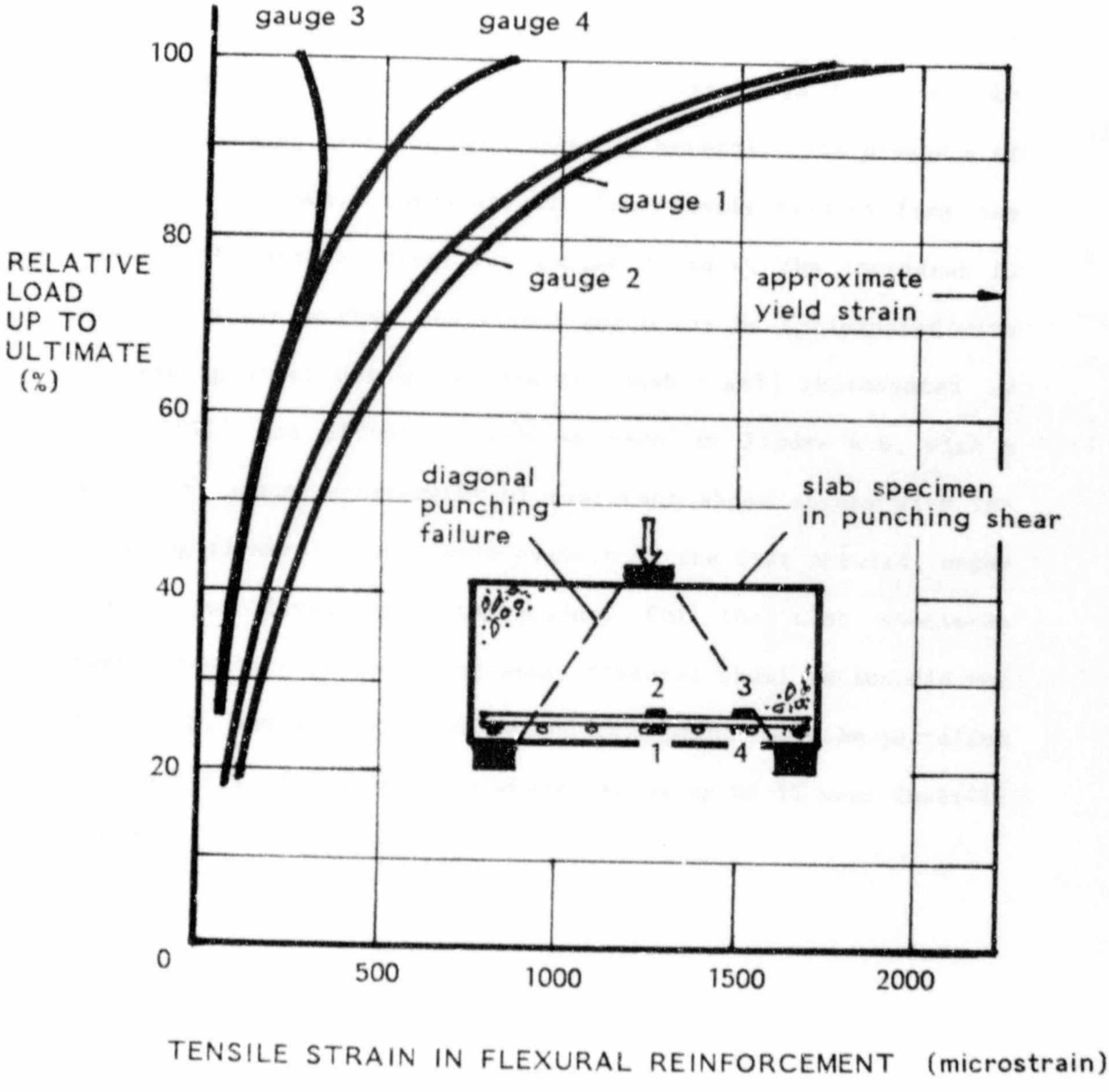

FIGURE 4.5 STRAIN MEASUREMENTS ON FLEXURAL REINFORCEMENT UP TO SHEAR FAILURE IN PUNCHING SLAB SPECIMENS.

the proximity of the punching platen is not identical to that of a prototype continuous flat plate structure, the rapid reduction of bending moment in the prototype relative to the position of the unconstrained punching surface will result in the above observation being true for both cases in general. The presence of dowel action double curvature is also clearly evident from the form of the strain curves for gauges 3 and 4. The increases in average shear stress resistance which can be anticipated with increasing steel percentage are reasonably well represented by both CP110 and BS0000 as can be seen in Figure 4.6, with a slightly reduced enhancement of resistant shear stress with increasing flexural steel ratio evident in the test results, especially for lower concrete grades. For the slab specimens considered in this test programme, flexural steel ratios did not exceed 1%, and extrapolation beyond this point could be justified on the basis of beam results where ratios up to 3% were investigated.

4.3.2 GRADE OF CONCRETE

The grade of concrete affected the ultimate shear resistance of the slab specimens tested. Reasonable correlation was obtained between the test results and the code values of CP110 and BS0000

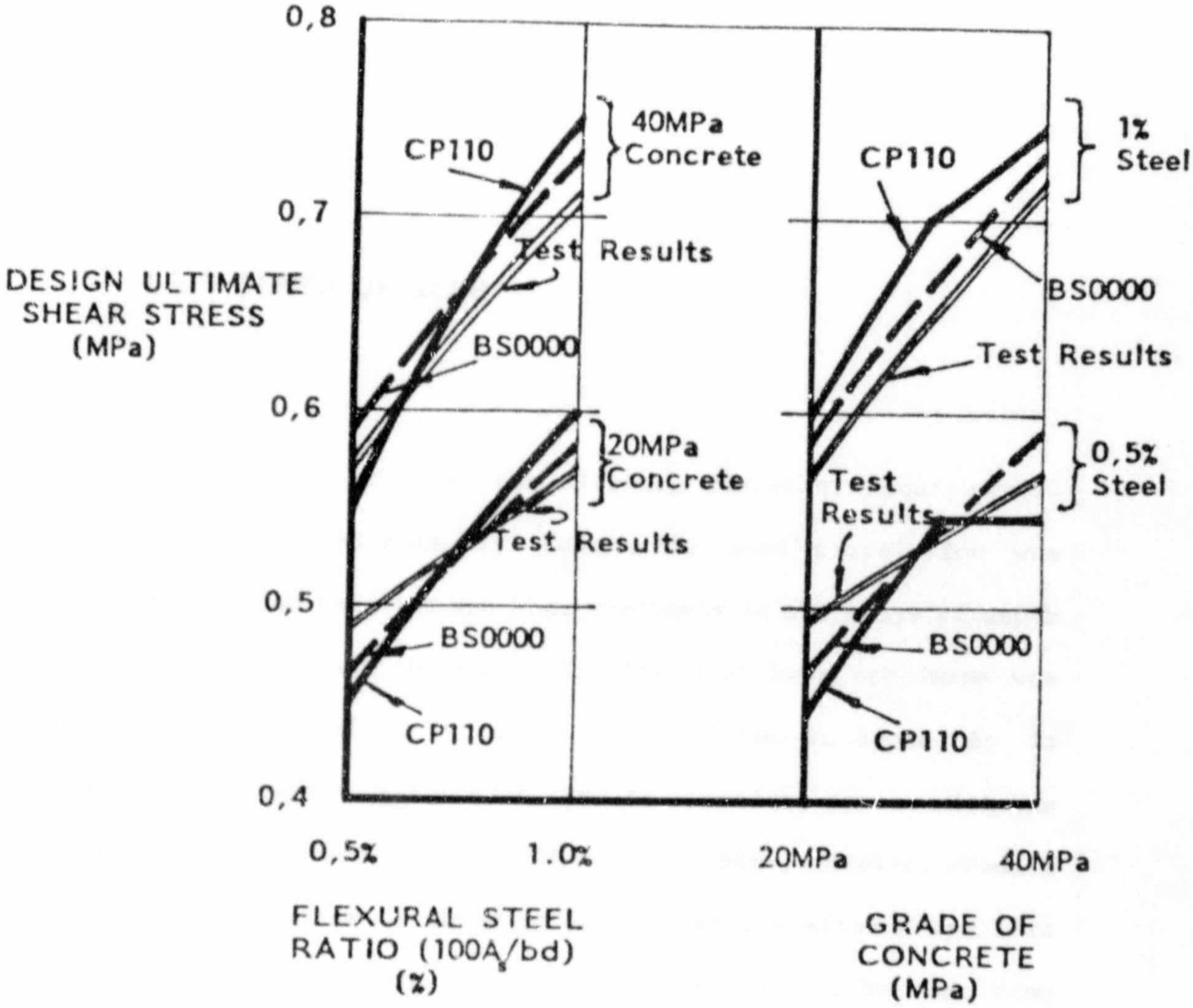

FIGURE 4.6 TEST RESULTS RELATED TO CODE VALUES FOR VARIATION IN DESIGN ULTIMATE RESISTANT SHEAR STRESS WITH STEEL RATIO AND GRADE OF CONCRETE.

as indicated in Figure 4.6. A universal trend in the influence of these parameters is thus evident in these test results and this should be reflected in an appropriate model for shear.

4.3.3 DEPTH OF SLAB

The depth of slab affected the ultimate resistant shear stress of the slabs tested markedly. Reasonably good correlation was obtained between the slab and beam specimens in the range of depth considered, and thus the curve for the deep beam specimens was used to predict values for slabs up to 1200mm in depth. As is evident in Figure 4.7, the test results correlate better with the curve for BS0000 than that for CP110. Both codes, however, predict no further reduction of shear stress at failure after a specific limiting depth of slab. The test results, as can be seen from Figure 4.7, indicate a continued reduction of shear stress at failure with increasing depth of slab.

BS0000 recommends a relationship of:

$$\xi = \sqrt[4]{(500/d)} \qquad (1 \le \xi \le 1{,}5)$$

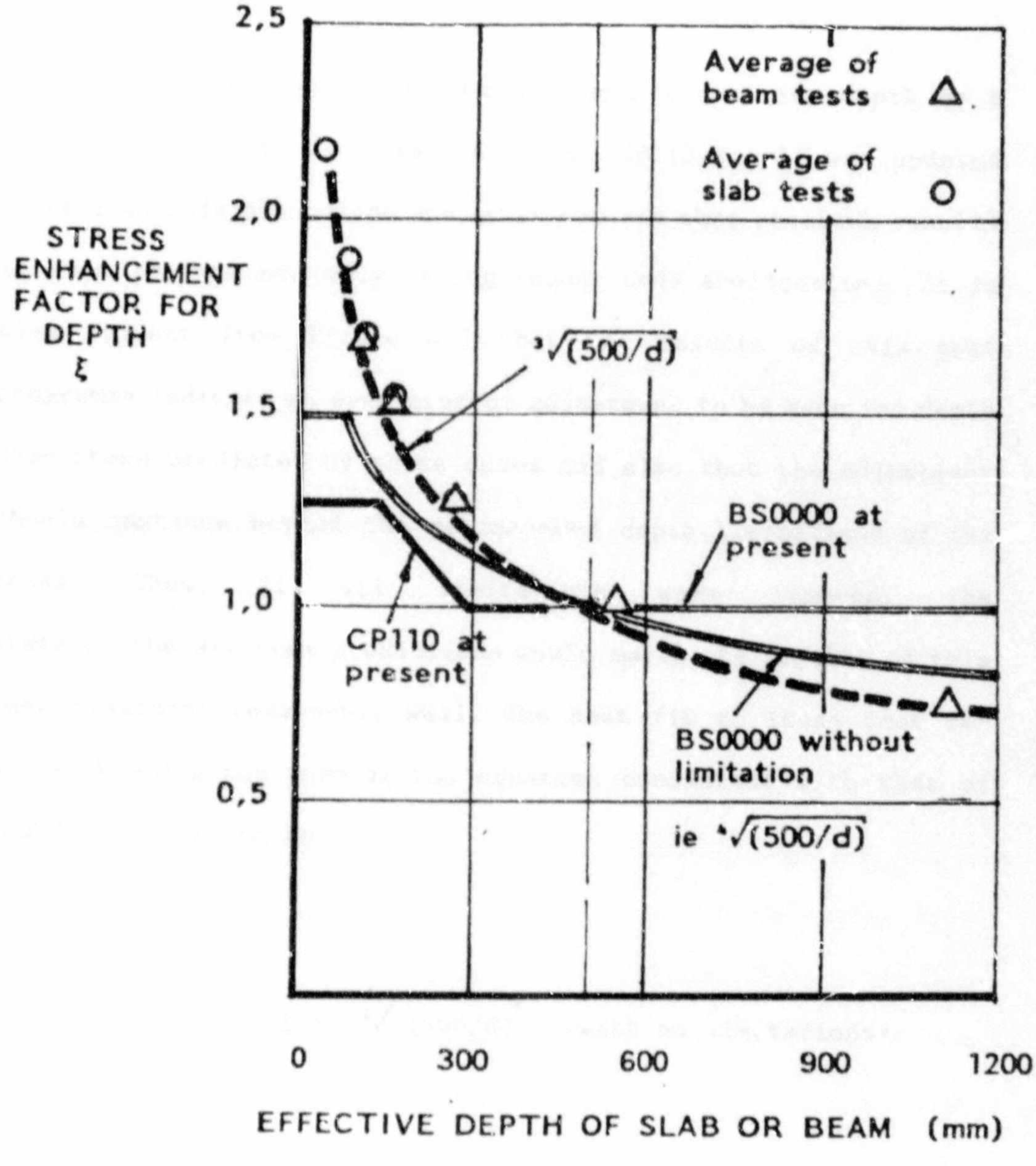

FIGURE 4.7 VARIATION OF ULTIMATE DESIGN RESISTANT SHEAR STRESS ENHANCEMENT FACTOR WITH DEPTH.

where ξ is the stress enhancement factor for depth.

d is the effective depth of the slab.

This code thus adjusts the design shear stress for depth by a larger factor than that used in CP110. CP110 itself was updated to reflect this phenomenon and it is evident that research results in this respect are only having recent code application. It is also evident from Figure 4.7 that the results of this test programme indicate an even greater adjustment to be made for depth than those predicted by these codes and also that the adjustment should continue beyond the recommended depth limitations of the codes. Thus, if all limitations were removed, the state of the art code predictions would match the results of this test programme reasonably well. The best fit to these test results, keeping the form of the equation consistent with that of BS0000, appears to be:

$$\xi = \sqrt[3]{(500/d)} \quad \text{(with no limitations)}$$

The trend of the test results of this programme are consistent with the results of other researchers in this field [4,24,25,29,43], in particular this equation correlates well with the curve given by Chana[4] in Figure 2.11. Explanations as to the reason for the reduction in resistant shear stress with depth are varied, including such effects as strain gradient across the section. The explanation that is proposed here is related to the proposed model for shear. The proposed model, developed in Chapter 8 of this thesis, is based on existing models which recognise the influence of three components of resistance to shear after the formation of the diagonal shear crack, for structural elements which are unreinforced for shear. These components are:

(a) Dowel action of the flexural reinforcement.

(b) Aggregate interlock along the diagonal shear crack.

(c) Shear transfer across the compression zone.

The major contributing factor here is considered to be that of aggregate interlock, with abrupt strain weakening once the ultimate, peak shear resistance is exceeded. It would appear from the results of this test programme that there is a relationship between the crack width of the diagonal shear crack and the abrupt loss of resistance of the aggregate interlock component in that the peak shear resistance appears to be attained at an absolute approximate average width of the diagonal shear crack which is relatively independent of the depth of the section being tested. It is thus proposed that once the diagonal shear crack reaches a certain absolute average value, abrupt loss of aggregate interlock is imminent. If the section is reinforced in such a way that the "ductile" contribution of the steel reinforcement, flexural or shear, is substantially smaller than that of aggregate interlock, then abrupt shear failure of the section follows. For structural elements unreinforced for shear, the flexural reinforcement is relatively inefficient in resisting shear force, in relation to the diagonal shear crack, and abrupt shear failure always follows loss of aggregate interlock in these cases. The only possible remaining ductile contribution to shear resistance after loss of aggregate interlock is that of dowel action, which is generally a small proportion of total shear resistance for monolithic sections. This assessment is based on the assumption that shear failure precedes flexural failure. In the event that the flexural reinforcement reaches uniaxial yield at the point where the diagonal shear/flexural crack intersects this reinforcement, then the flexural ultimate limit state has been attained. A hinge will form at this point in the structural element and even if this hinge is subject to shear force, this will be carried by dowel action and a considerably enhanced compression zone component. The investigation of the mechanics of the manner in which such a hinge carries the shear force is not within the specific scope of this thesis.

For deep sections, the remoteness of the flexural steel reinforcement and compression zone from the midpoint of the section result in this absolute value of crack width of the diagonal shear crack being achieved earlier than that for a shallow section if average shear stress is the parameter being used for the shear evaluation. The deep section will thus lose aggregate interlock earlier than a shallow section in terms of shear stress and will thus fail in shear at a lower stress. This phenomenon should be be reflected in the formulation of a universal model for shear.

4.3.4 SHEAR ARM TO DEPTH RATIO

In this test programme, the cone of punching was forced to form steeper in certain of the specimens than it would if unconstrained. The shear arm to depth ratio (or the a/d ratio) for these specimens was thus effectively reduced. This was achieved by reducing the size of the four-sided square support to the slabs being tested, while maintaining a symmetrical arrangement of support around the punching platen. The beam and adjusted slab results for these tests both exhibit clear parametric behaviour with varying a/d ratio, as indicated in Figure 4.8. Upon examination of this figure, it would appear that the beam test results are well represented by the adjustment used in CP110 and BS0000 of:

$$2d/a$$

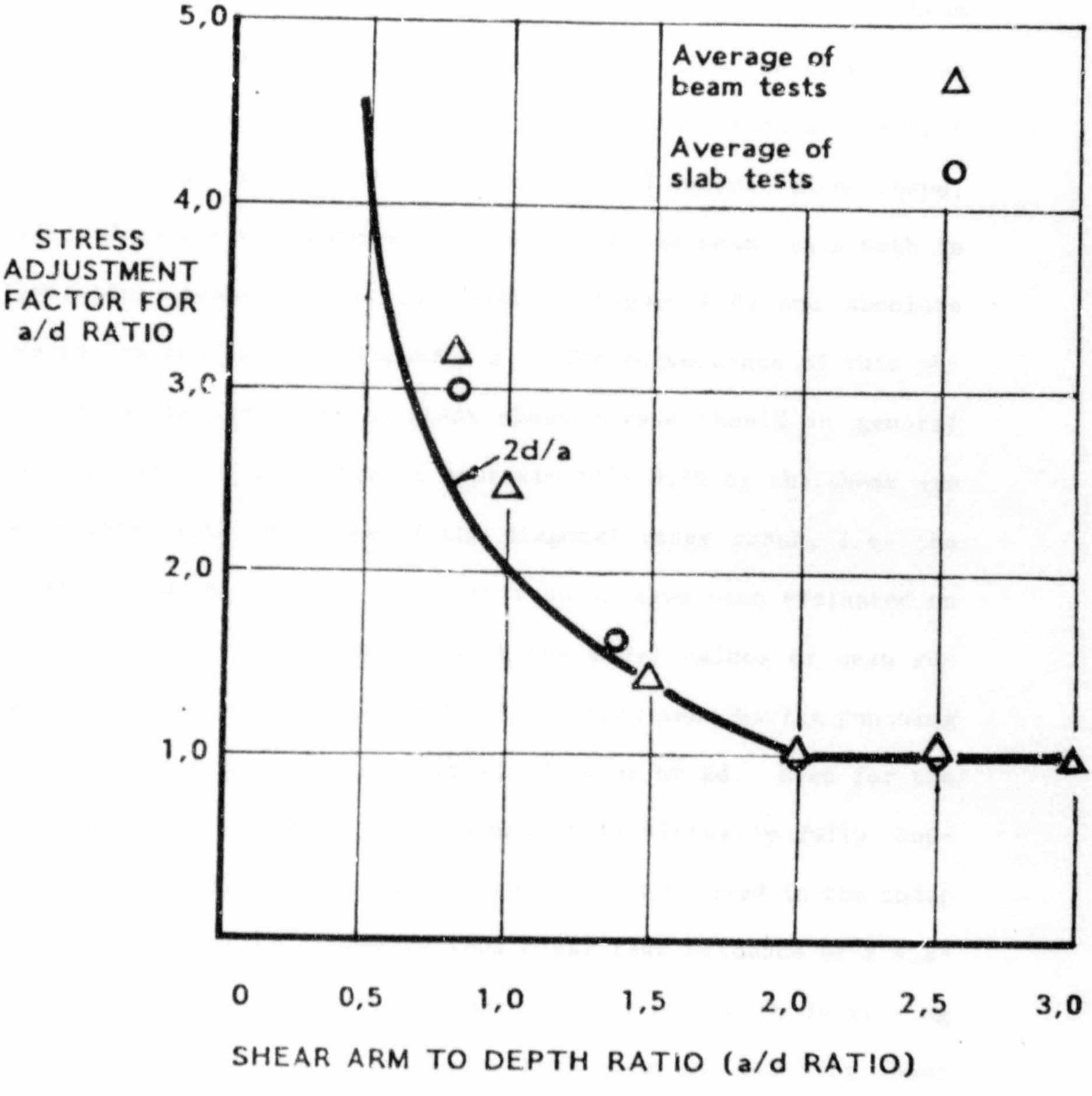

FIGURE 4.8 VARIATION OF ULTIMATE DESIGN RESISTANT SHEAR STRESS ADJUSTMENT FACTOR WITH a/d RATIO.

This is consistent with the results of other researchers in this field. The slab test results, however, if evaluated on the perimeter, appeared to be consistently slightly lower than the beam resistant stress results. Evaluating the test results at an affine perimeter of 0,85 of the true measured perimeter, using a rounded format consistent with the observed unconstrained shape, yielded values which correlated well with the beam tests both in parametric behaviour (as indicated in Figure 4.8) and absolute value (as indicated in Appendix B). The consequence of this observation is that the resistant shear stress should in general be applied at a perimeter of approximately 0,85 of the shear arm associated with the slope of the diagonal shear crack, i.e. the diagonal punching cone. The test results have been evaluated on this basis for the comparison with the model values of mean resistant shear stress in Appendix B for specimens having punching cones steeper than the unconstrained value of 2d. Even for the unconstrained specimens, this approach is virtually fully consistent, as 0,85 of 2d is very close to the 1,5h used in the codes under consideration. There is thus clear test evidence of a significant enhancement in resistant shear stress with increasing slope of diagonal shear crack for both beam and punching shear specimens, and a universal model for shear should reflect this trend for both these structural element types.

4.4 CASE STUDIES

In order to place the test results of the slabs unreinforced for shear in perspective relative to the existing and proposed codes of practice, three flat plate case studies have been considered. Certain parametric trends, such as depth of slab and span/depth ratios, are identified in these examples. Other parameters, however, are not intended to be evaluated and thus all examples are considered to be of Grade 25 concrete, to have 0,5% isotropic reinforcement top and bottom of the slab and to have typical cover to the reinforcement. In addition, only typical interior columns of a flat plate structure have been considered with only nominal moment transfer being required. Column sizes are considered to be typical and are not varied as a significant parameter in this exercise. The results of the case studies are in terms of the design ultimate total uniformly distributed load that the flat plate structure is capable of supporting. The case studies considered are:

(a) A 150mm thick slab supported on 300mm square columns at 3000mm centres.
(light industrial or office)

(b) A 350mm thick slab supported on 400mm square columns at 6000mm centres.
(heavy/medium industrial)

(c) A 1200mm thick slab supported on 1000mm square columns at 5000mm centres.
(floor to large silo)

The results for these case studies for both punching shear capacity and yield line capacity for a number of different codes and for the results of this test programme are presented in Figure 4.9. It is not the intention of this figure to plot a specific parameter on the horizontal axis, although the cases plotted on this axis do represent trends of both increasing depth and reducing span/depth ratios. The capacities of the case studies have been linked, simply to indicate greater clarity in the figure. The gap in the figure is to accomodate a change in vertical scale for the extreme case of the silo floor.

The evaluation depicted in Figure 4.9 does not take moment transfer into account, other than in the nominal manner recommended by CP110 and BS0000. It is of note that the arbitrary increase in punching shear resistance required for a typical interior column has been reduced from 25% in CP110 to 15% in BS0000. Test results in this test programme have thus been adjusted by 15% in this comparative evaluation. It is evident from this figure that prior to the introduction of a check at the column face (instituted in BS0000), CP110 predicted ultimate design loads which were radically higher than general for deep slabs with small span/depth ratios, because a check was only required at the

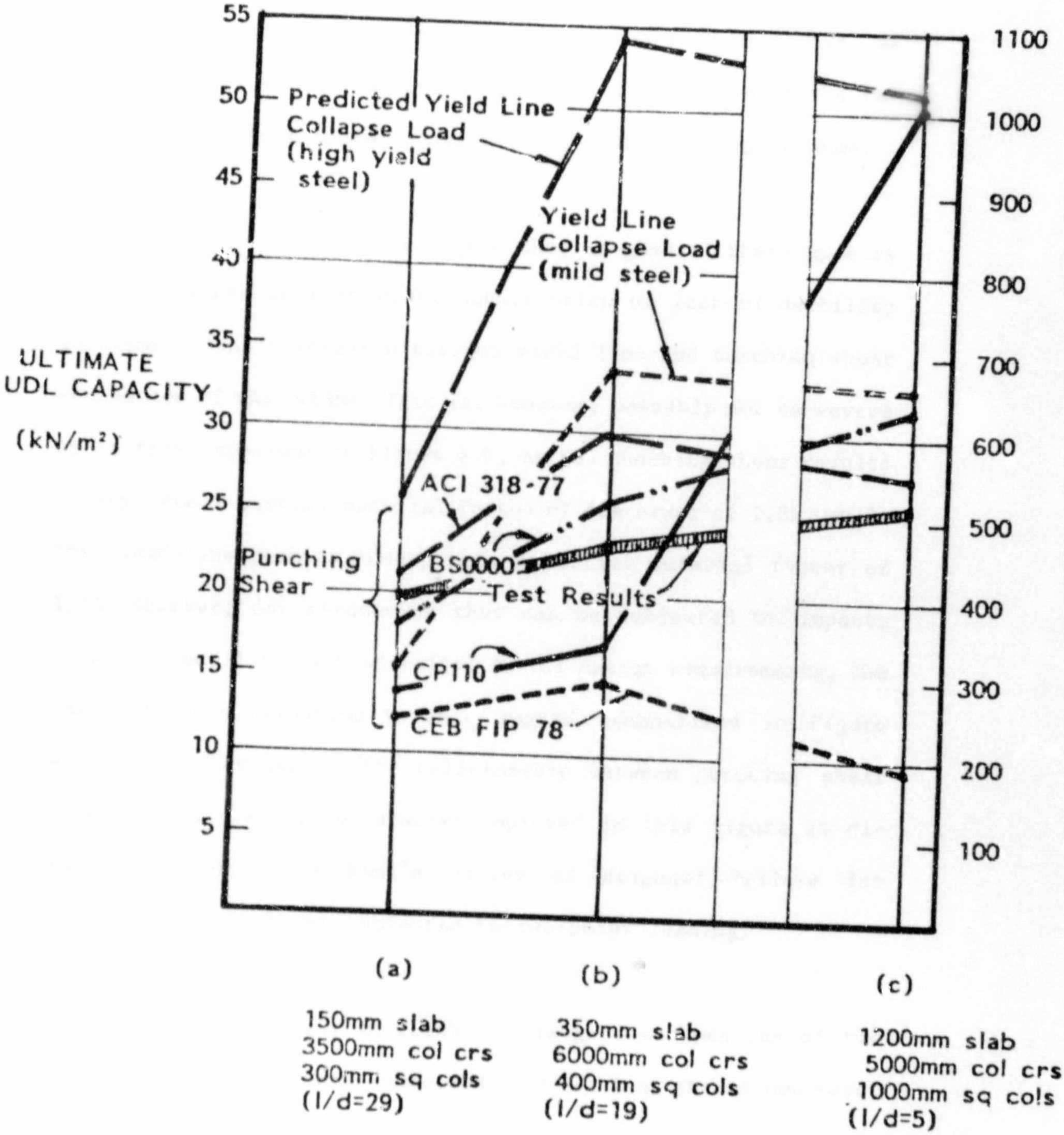

FIGURE 4.9 VARIATION OF ULTIMATE DESIGN UNIFORMLY DISTRIBUTED LOAD CAPACITY WITH CASE STUDY.

unconstrained punching perimeter. BS0000 has redressed this, as can be seen, but does not quite cover the most severe of the results which appear to be anticipated from this test programme.

It is also evident from the figure that in general there appears to be a danger in flat plate construction of lack of ductility in terms of the comparison between yield line and punching shear evaluation of the slabs. This is, however, possibly not as severe as is first apparent in Figure 4.9, as the punching shear results incorporate a partial material factor of the order of 1,25, while the yield-line results incorporate a partial material factor of 1,15. However, for structures that can be subjected to impact, or where ductility is fundamental to the design requirements, the unsatisfactory situation in this regard, summarised in Figure 4.9, is self evident. The relationship between punching shear capacity and yield-line capacity depicted in this figure is directly analogous to Kani's valley of diagonal failure for simply-supported beams subjected to two-point loading.

In regard to the manner in which the large discrepancies of the deep slab case study are redressed by a check at the column face, it is apparent from Figure 4.10 that the test results are only more severe than this check of BS0000 or the value at the 1,5h perimeter for very deep slabs which also have a span/depth ratio less than 7. In Figure 4.10, the test results of the previous section have been used to obtain a variable function of ultimate