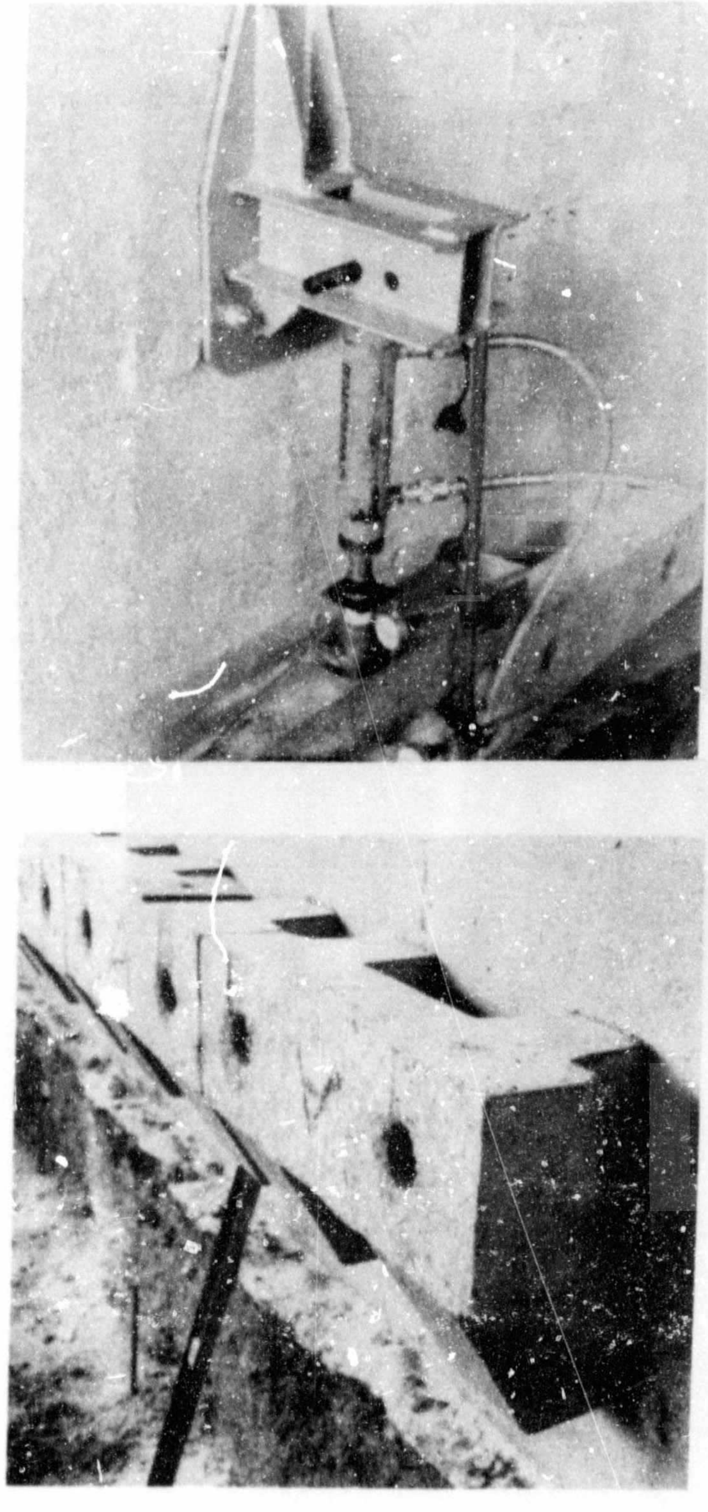
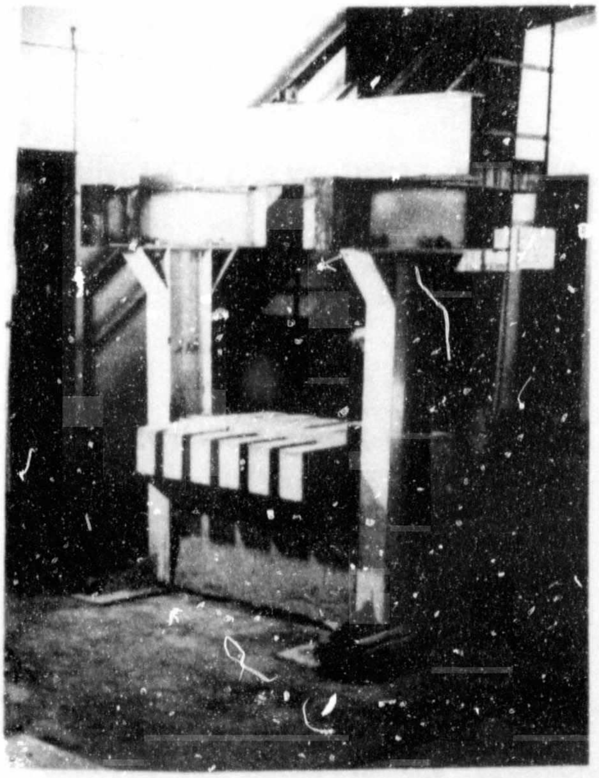


FIGURE 3.1 PHOTOGRAPHS SHOWING MOBILE TEST RIG AND TYPICAL PRECAST CORBEL SPECIMEN



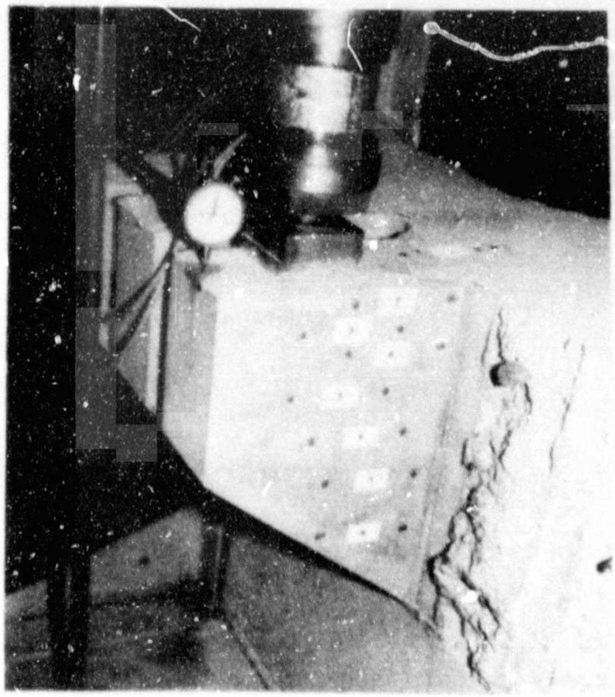


FIGURE 3.2 PHOTOGRAPHS SHOWING LABORATORY TEST RIG AND TYPICAL IN-SITU HAUNCHED CORBELS

3.3 CORBELS WITH HAUNCHED ELEVATION

3.3.1 DETAILS OF THE SPECIMENS

Consistent with the recognised approach to research in the field of shear evaluation, specimens were designed as being both reinforced for shear and unreinforced for shear in the traditional sense. Fo r types of corbels thus had to be considered in the implementation of the haunched corbel test series. These were precast specimens, unreinforced and reinforced for shear, and in-situ specimens, unreinforced and reinforced for shear.

3.3.1.1 PRECAST BOLT-ON CORBELS

As the bolt-on precast specimens were designed for a specific application in industry, their geometry and reinforcement configuration were predetermined by the designers for this project. Some variation in the "shear reinforcement" was considered, however, as relevant to this research programme. The geometry and reinforcement for these specimens are shown in Figure 3.3 and a

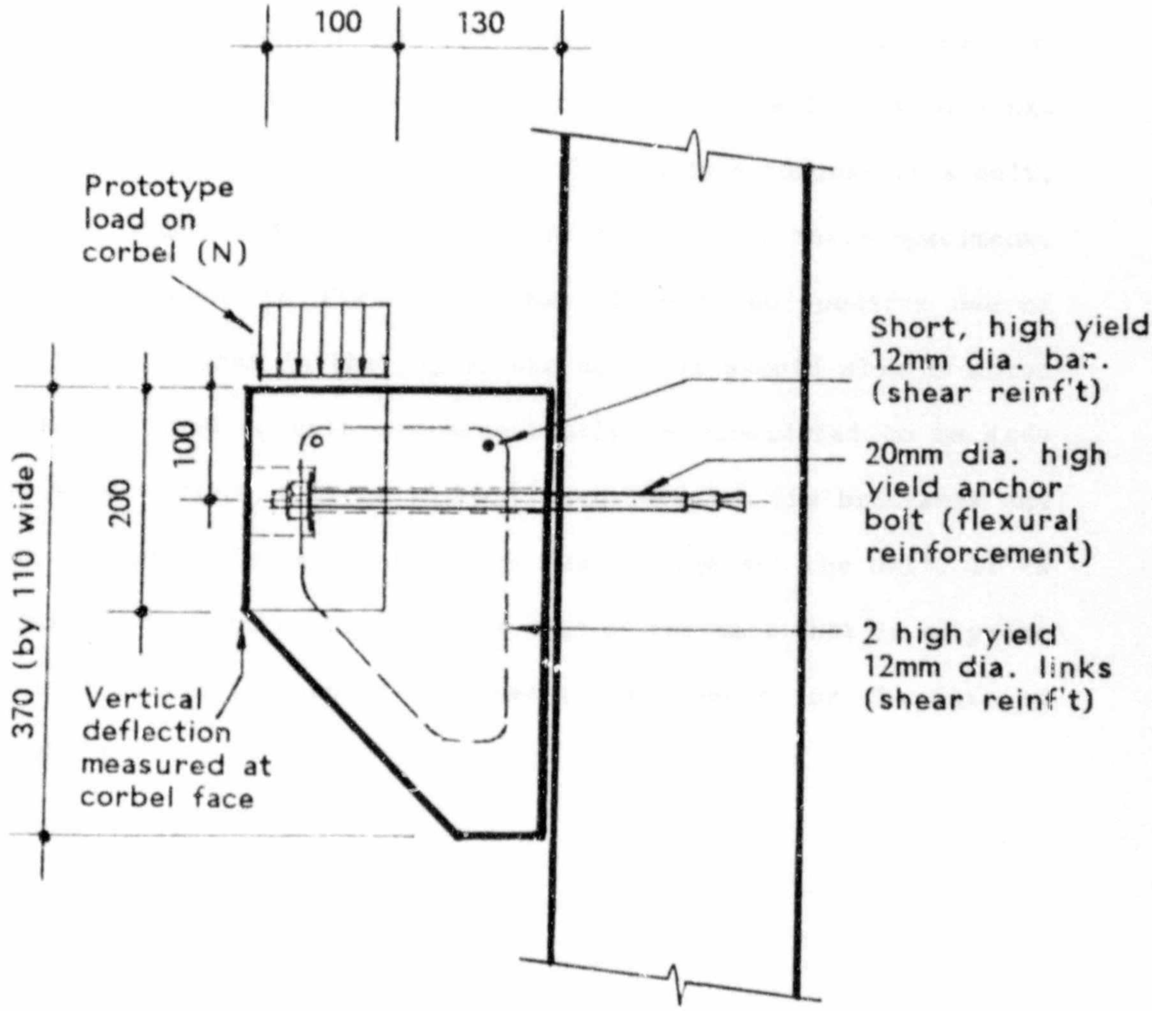


FIGURE 3.3 DETAIL OF A SINGLE LEG OF THE PRECAST BOLT-ON SPECIMENS TESTED.

general view is also given in Figure 3.1. Since the precast corbel was attached to the wall by a self-locking, 20mm diameter, high tensile anchor bolt for the prototypes and for the test specimens, this can reasonably be considered to be the the main flexural reinforcement for the precast specimens. Additional reinforcement in the corbel, particularly vertically placed links in the "legs" of the precast unit that fully encompass this bolt, can be considered to be shear reinforcement for these specimens. It is evident in Figure 3.3 that there is no specific use of horizontal links in the leg of the unit. It should also be noted that the precast unit can essentially be considered to be made up of a horizontal beam element that supports the brickwork and two corbel or leg elements which bear up against the wall. It is thus the reinforcement in these legs of the unit that is of prime consideration in respect of shear reinforcement for the unit as a whole.

3.3.1.2 IN-SITU CORBELS

In order to eliminate the effects of scale, the in-situ corbel specimens were chosen so that their geometry would match that of a single leg of the precast specimens and the details are shown in Figure 3.4. All the in-situ corbels were cast in the laboratory

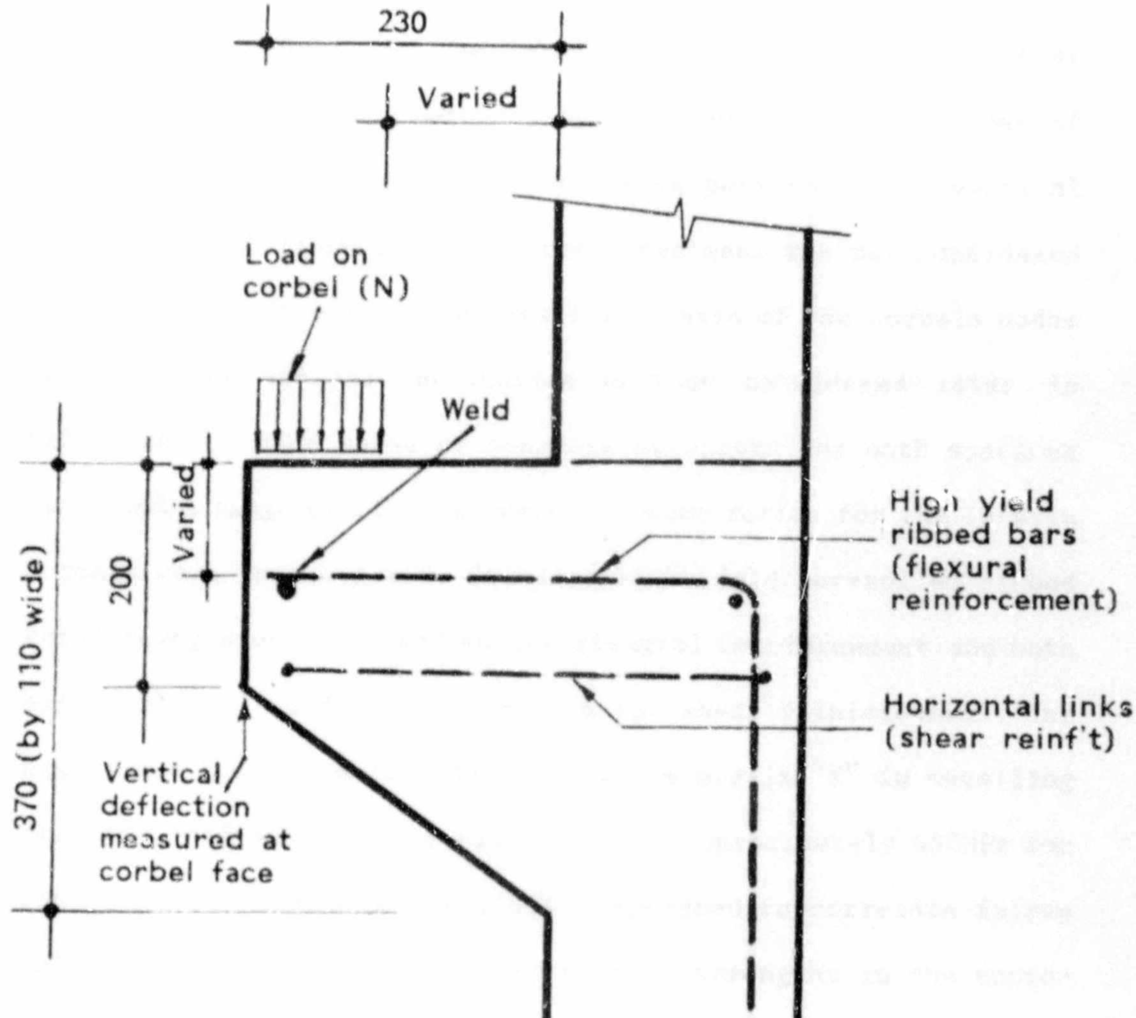


FIGURE 3.4 DETAILS OF THE IN-SITU CORBEL SPECIMENS TESTED.

and Figure 3.2 gives a general view of these specimens. Test results refer to the single leg capacity for both corbel types. Shear reinforcement for the in-situ corbels was in the form of horizontal links as recommended by a number of current codes of practice. An evaluation of the relative performance of vertical and horizontal links for the corbel specimens was not considered to be feasible in view of the small a/d ratio of the corbels under consideration and this evaluation is thus considered later in this work. A wide range of concrete strengths for both specimen types and a range of flexural reinforcement ratios for the in-situ corbels were investigated. Standard high-yield hot-rolled ribbed reinforcing steel was used as the flexural reinforcement and both high-yield and mild steel were used for shear reinforcement. The high-yield reinforcement, denoted by the prefix "Y" in detailing practice, had an average yield stress of approximately 450MPa for the range of diameters used, which appeared to correlate fairly well with the recommended characteristic strengths in the appropriate codes. The quantity of flexural reinforcement in the in-situ corbel specimens tested varied from 0,4% to 1,3% and the mean cube strength of the concrete from 20MPa to 40MPa. Adequate end anchorage to the flexural reinforcement was obtained by welding a transverse bar of equal diameter to this steel at the front face of the corbel prior to concreting.

3.3.2 TEST RESULTS

3.3.2.1 BOLT-ON PRECAST CORBELS

The following modes of unserviceability and failure were observed in these tests:

1. 'Dowel cracking' of the concrete. This mode was always the first to form and was caused by the fracture of a triangular wedge of of concrete immediately adjacent to the wall and above each of the bolts as illustrated in Figure 3.5, and which is also evident in the photographs of the corbels after failure. For the bolt-on corbels unreinforced for shear, the occurrence of this dowel crack coincided with abrupt non-ductile shear failure of the corbel unit as a whole, as illustrated in the form of the load-deflection curve of Figure 3.5. For the bolt-on corbels reinforced for shear the load at which dowel cracking occurred did not necessarily represent total shear failure of the unit, and in certain cases, depending on the shear reinforcement present, subsequent additional modes of failure were observed.

2. 'Front-shear' failure or front splitting. This mode, which occurred only after dowel cracking, was a typical consequence of

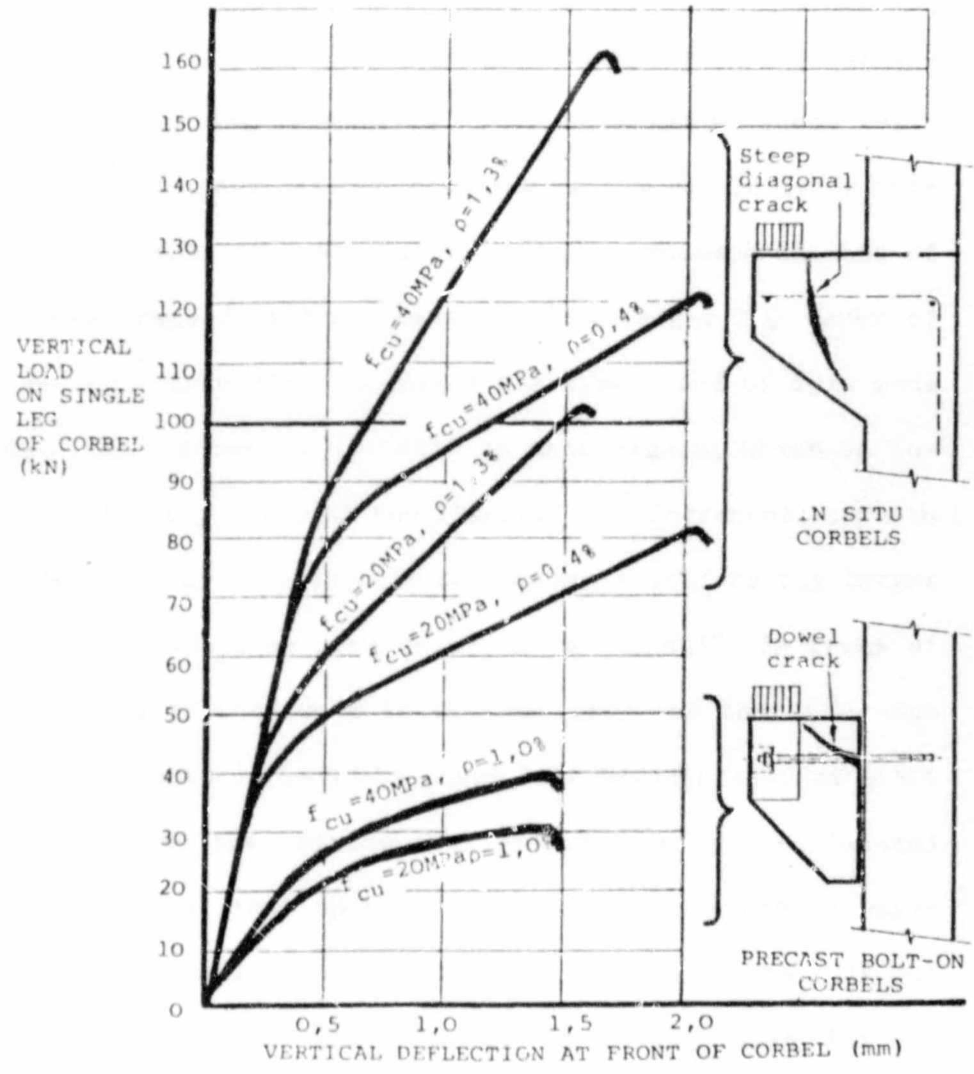


FIGURE 3.5 TYPICAL LOAD DEFLECTION CURVES FOR IN-SITU AND SINGLE LEG OF PRECAST BOLT-ON CORBELS UNREINFORCED FOR SHEAR.

poor detailing of the steel reinforcement in both the precast and in-situ specimens, in that the load caused the front portion of the concrete beyond the steel to fail in shear (or splitting), as indicated in Figure 3.6. It is evident that this mode of failure generally precluded additional anticipated shear failure modes from manifesting themselves, such as diagonal shear failure. This mode of shear failure occurred fairly regularly in this test series and this is a strong case for the recommendations of certain codes[5] regarding load position. The larger top cover of the precast unit generally increased the likelihood of this mode occurring. The criterion of CP110 in this regard, which is independent of the top cover to the flexural reinforcement, appears to be inadequate for corbels with top covers significantly larger than those which would be condidered to be "normal". In terms of the test results of this work it is thus proposed that the edge of the load on a corbel should be moved back horizontally at least a cover depth from the extreme straight portion of the flexural reinforcement. This front splitting mode of failure was accompanied by large deflections of the unit and tearing of the bolt out of the top of the leg for the specimens where the links did not fully encompass the bolt. A flexural mode of failure, in the form of a rotation of the unit about the heel, could not be achieved, nor was the pull-out value of the anchor bolt achieved in any test. The typical load-deflection curves for bolt-on precast corbel units reinforced for shear are indicated in Figure 3.7, plotted in common with the in-situ test results. For specimens

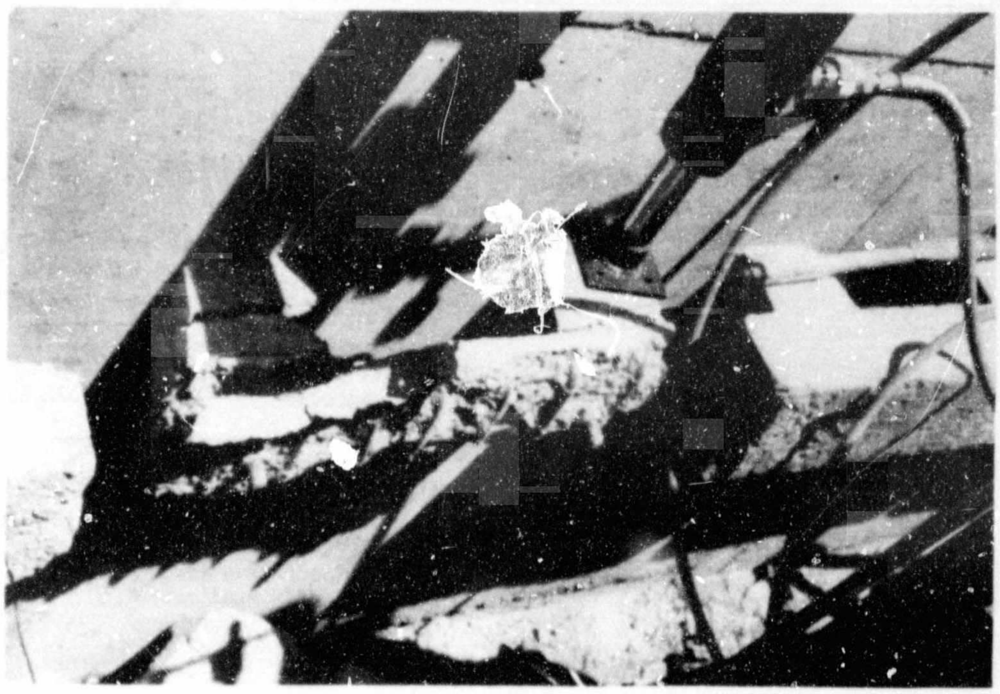


FIGURE 3.6 PHOTOGRAPH SHOWING TYPICAL FRONT SHEAR FAILURE (OR VERTICAL SPLITTING) OF THE PRECAST BOLT-ON CORBEL SPECIMENS.

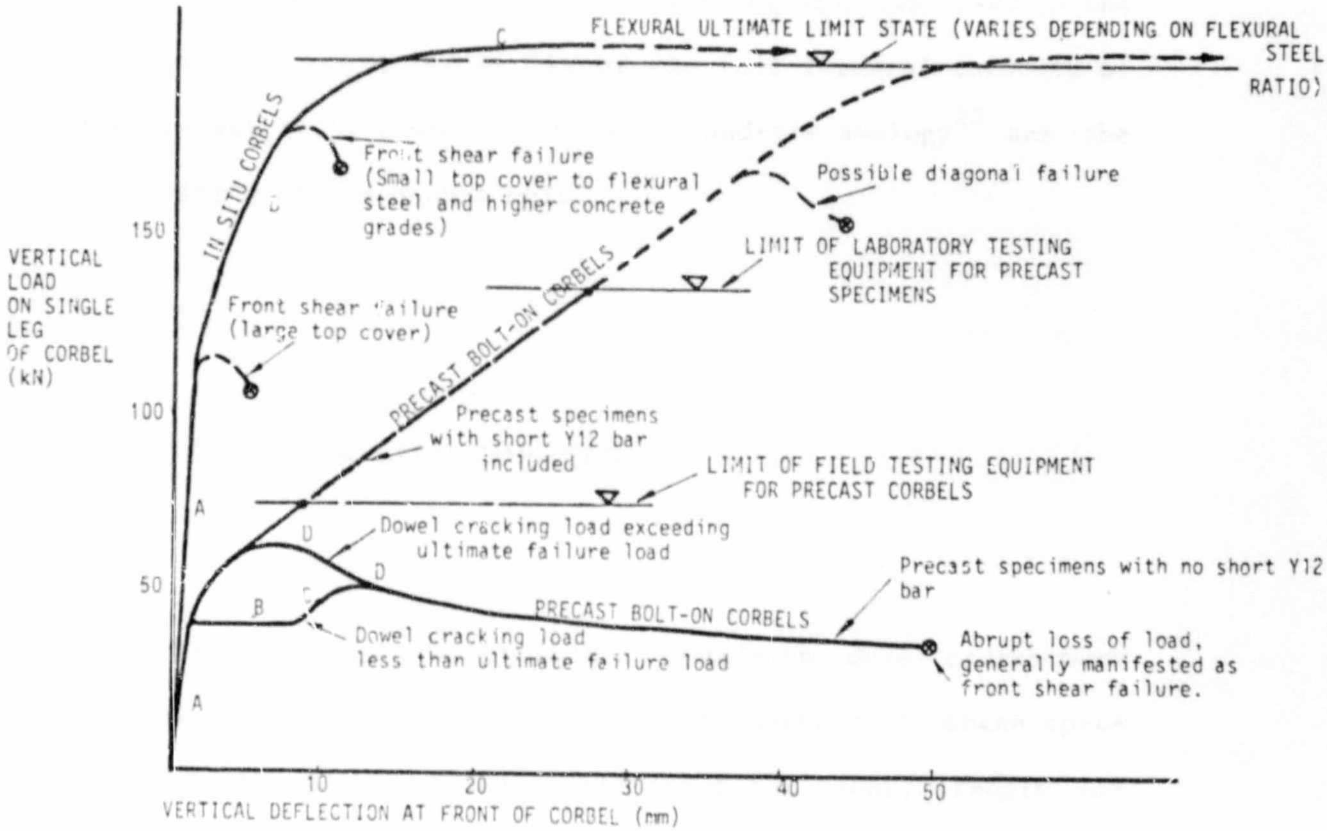


FIGURE 3.7 TYPICAL LOAD-DEFLECTION CURVES FOR IN-SITU AND SINGLE LEG OF PRECAST BOLT-ON CORBELS REINFORCED FOR SHEAR.

where the links encompassed the anchor bolt fully, and where the load was repositioned in order to eliminate the occurrence of 'front-shear' failure, the capacity of the testing equipment was reached without shear failure occurring, but with the anchor bolts extremely distorted in bending, and carrying a major proportion of the load by the vertical component of the tension in the bolt. The load capacity of the testing equipment used in the field was of the order of 25% of the full flexural capacity of the precast unit, based on the strut-and-tie analogy[22] and the yield force of the anchor bolt.

3.3.2.2 IN-SITU CORBELS

In contrast to the precast bolt-on specimens, dowel cracks were not observed during tests on the in-situ corbels. In these specimens, both reinforced and unreinforced for shear, steeply inclined diagonal shear cracks were the first to form, in a position indicated in Figure 3.5. Although these were often accompanied by flexural cracks at the interface between the corbel and the wall, especially noticeable on the flexurally lightly reinforced specimens, these flexural cracks did not lead to shear failure for any specimens. It is important to note that the in-situ corbels generally accepted considerably more load after the observation of the first diagonal crack. This increase in additional load is larger for specimens with higher flexural steel ratios in general and the load at which cracking was first observed appeared to be relatively independent of this steel ratio but dependent only on grade of concrete. The implication is that the diagonal shear crack, for this "constrained" situation of reduced a/d ratio, will tend to form between the edge of the load and the heel of the corbel in the compression zone, as indicated in the figure, independently of the shear or flexural reinforcement in the element. It is also important to note that it is this particular diagonal shear crack that finally precipitates the shear failure. It is thus this particular crack that must be examined closely at the ultimate limit state. Other cracks in the element, of any origin, assume lesser importance if they do not precipitate the ultimate limit state.

After the steep diagonal cracks were first observed by eye, they grew, firstly in length, and then also in width, terminating in abrupt shear failure of the corbel. Just prior to the instant of sudden shear failure, the diagonal crack extended from the edge of the load right up to the compression zone at the heel, and appeared in general to have a maximum crack width near the neutral axis of the element. This is illustrated in the photograph of Figure 3.8. The demec targets used for strain and crack width measurement on the corbel are also clearly evident in this figure.

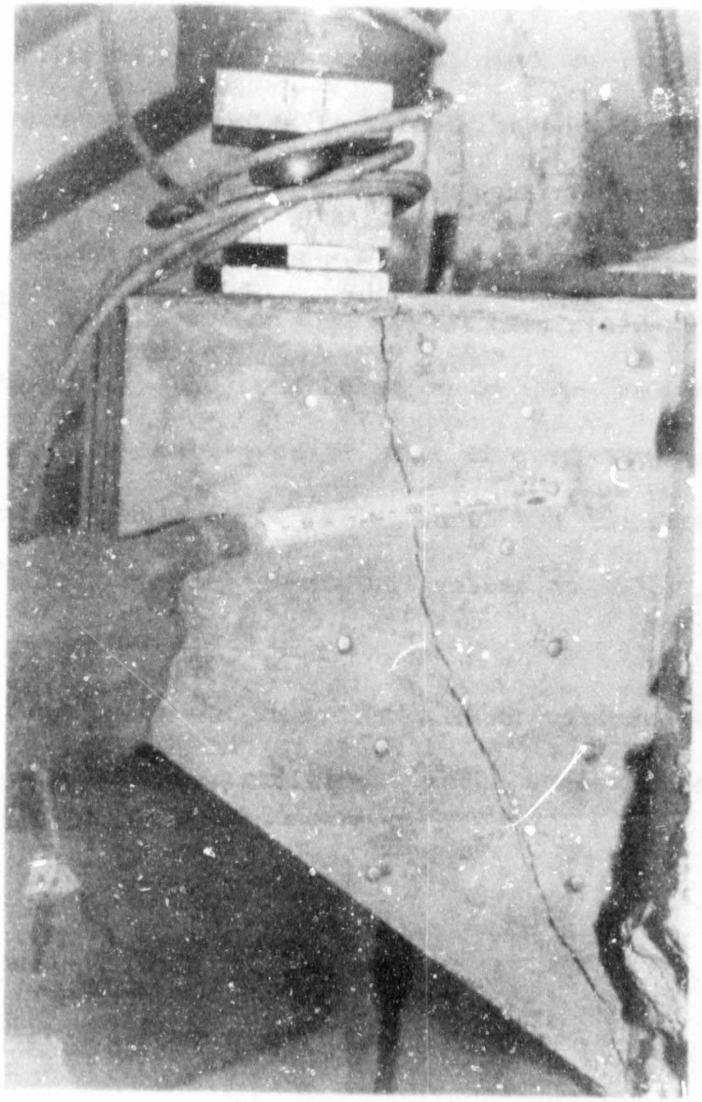


FIGURE 3.8 PHOTOGRAPH SHOWING THE DETAILS OF THE DIAGONAL SHEAR CRACK JUST PRIOR TO SHEAR FAILURE OF THE CORBEL. (THE DEMEC TARGETS FOR STRAIN AND CRACK WIDTH MEASUREMENT ARE ALSO EVIDENT)

The test results for the specimens unreinforced for shear, summarised in the curves of Figure 3.5, clearly require evaluation in terms of results of other researchers, both on corbels and relating to the more usual results for simply supported beams subjected to two-point loading. These results appear to be reasonably consistent with the observations of other researchers of corbel behaviour[28,38]. The comparison with normal beam evaluation is probably best achieved by the consideration of code values, as these represent an acceptable average of a wide spectrum of results. The code thus referred to is CP110, which has identical values to SABS 0100[6], and virtually identical values to BS0000.

For the corbel test results in the figure, the shear performance can be evaluated in terms of the traditional shear stress, such that:

$$v = \frac{V}{bd} \qquad \text{(MPa)}$$

where

V is the recorded maximum shear load accepted by the corbel (N)

b is the width of the corbel (mm)

d is the effective depth of the corbel (mm)

In order to take into account the proximity of the load to the support, this stress must be adjusted. Test results of various researchers indicate that the shear strength is enhanced as the a/d ratio is reduced[28,38]. A reasonable representation of this phenomenon, which also appears in certain of the codes considered in this thesis, is that the resistant shear stress for elements unreinforced for shear increases by the factor 2d/a as the a/d ratio drops below 2. The stress calculated above can thus be adjusted accordingly. Further adjustments are of course generally required before a representative figure for design shear stress can be obtained. This shear stress must therefore be further adjusted by a partial material factor and by change of mean cube strength of the laboratory tests to "grade of concrete" values. The partial material factor in the codified shear stress values for BS0000 is given as 1,25[13]. The corbel test results, after the adjustments considered above, are compared with the code results of CP110 in Figure 3.9. The parametric trends are further evident in examination of the test results for corbels in Appendix A. This comparison reveals that the in-situ corbels of this geometry, unreinforced for shear, follow the anticipated parametric trends of this code. The increase of shear resistance with increase in flexural steel ratio appears to be slightly smaller than that predicted by CP110. This increase does vary between codes, however, with the CEB FIP 78 model code also predicting smaller

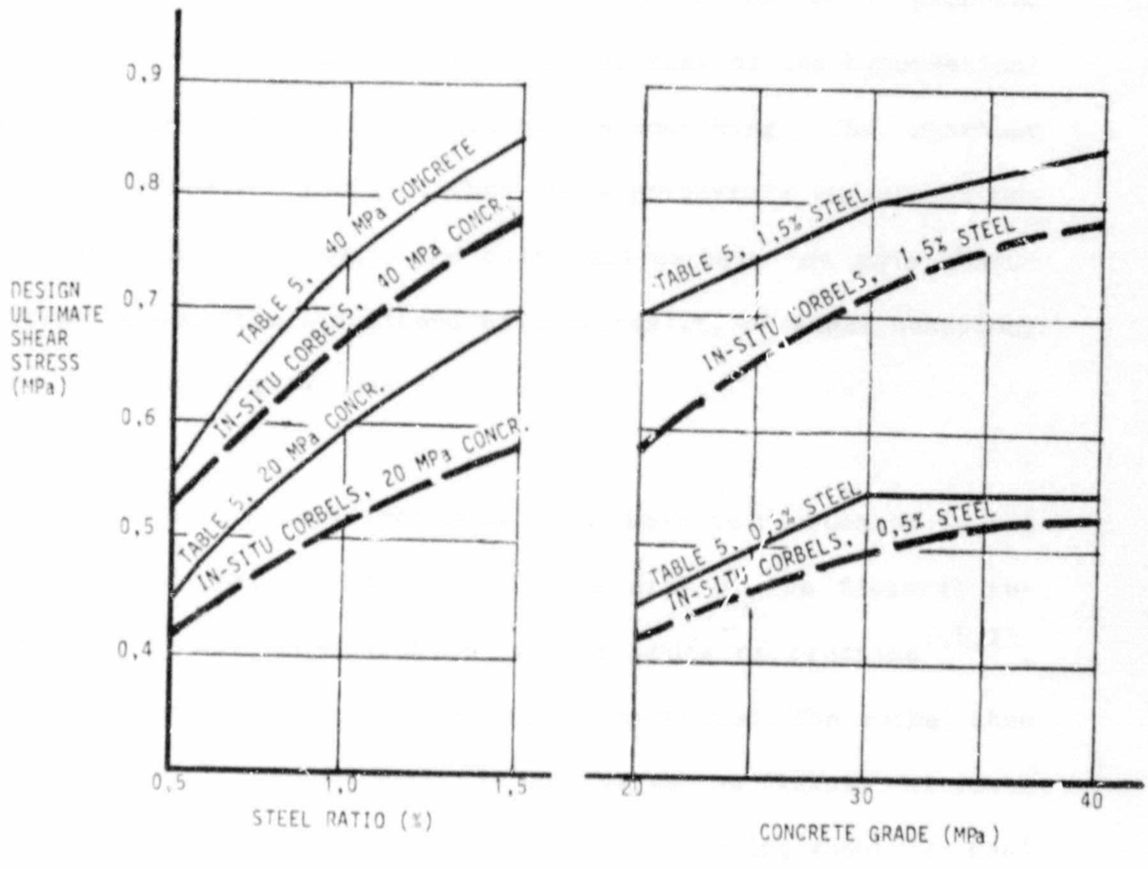


FIGURE 3.9 COMPARISON OF VARIATION OF CORBEL RESULTS WITH STEEL RATIO AND GRADE OF CONCRETE WITH SIMILAR VARIATIONS FOR CP110

increases of shear resistance with steel ratio than CP110. The number of tests undertaken in this series is insufficient to assume any significant variation between the test results and the code predictions. Examination of Appendix A, however, reveals that the measured resistant stress for haunched corbels appears to be consistently slightly below that of the prismatic corbels. This can be explained in terms of the model for shear proposed in Chapter 8 in terms of a reduced stiffness of the hypothetical beam of the model, as a result of the haunching. The important observation of the tests is that these parametric variations occur for both simply supported beams and corbels and this observation implies the likelihood of universality of shear behaviour in most structural elements.

For the in-situ corbel specimens with shear reinforcement in the form of horizontal links of half the area of the flexural reinforcement, as recommended by certain codes of practice[1,5,13], diagonal shear failure was effectively prevented. The corbel then failed in an alternative mode. In some cases the flexural ultimate limit state was achieved, with the corbel rotating about the heel and the flexural reinforcement at yield. In general, however, with the details and position of the load on the specimens as indicated, the mode was manifested as "front shear failure" of the corbel, with the concrete splitting around the flexural and shear reinforcement. This mode is of course again attributable to poor detailing, where the load is permitted to bear too near to the front of the corbel relative to the reinforcement positioning and the recommendations mentioned previously, resulting from these observations, are equally applicable to the detailing requirements for in-situ corbels.

Typical load-deflection curves for in-situ corbels reinforced for shear are shown with those for the precast units for comparative purposes in Figure 3.7. It is relevant to note in this figure that the vertical deflections at maximum load of the corbels reinforced for shear are an order of magnitude greater than those for corbels unreinforced for shear. The ductility thus achieved by reinforcing for shear, in conjunction with judicious positioning of the load on the specimen at an appropriate distance from the front face, is clearly evident for both corbel types from the vertical deflections recorded in this figure.

3.3.3 SHEAR DESIGN PHILOSOPHY FOR CORBELS

As for shear design in general reinforced concrete construction, good shear design for corbels (both haunched and prismatic) should aim at the achievement of the appropriate ductility. In the case of corbels in particular, with the steep disposition of the diagonal shear crack and the consequent general use of horizontal links for shear reinforcement, the implication is that this ductility is only achieved by raising the shear capacity of the corbel to a value in excess of the flexural capacity. The ductility is then obtained by flexural rotation of the corbel unit about the heel. For both the precast and in-situ specimens it is evident from the test results that adequate ductility is only achieved if attention is given to the inclusion of appropriate shear reinforcement. Especially if redistribution in the structural system is a prerequisite of the structural design philosophy, there is no decided advantage in the use of in-situ over bolt-on corbels unreinforced for shear, as both lack adequate ductility. The results of the in-situ corbel tests are plotted relative to Kani's results for simply supported beams in Figure 3.10. Both results of course refer to specimens unreinforced for shear. Although the use of high yield steel for the flexural reinforcement in the in-situ corbel specimens causes a shift relative to Kani's results, in which mild steel flexural reinforcement was used, this plot nevertheless indicates that shear failure prior to flexural failure is highly likely for corbels, even with a/d ratios less than unity. This is an extremely important observation as Kani's results in general indicate that this is unlikely to be the case for his specimens for a/d ratios less than unity and could therefore be regarded as an area where detailed consideration of reinforcing for shear is not warranted. It is thus somewhat in contrast to Kani's results that it is evident that the valley for corbels is still likely to be very much

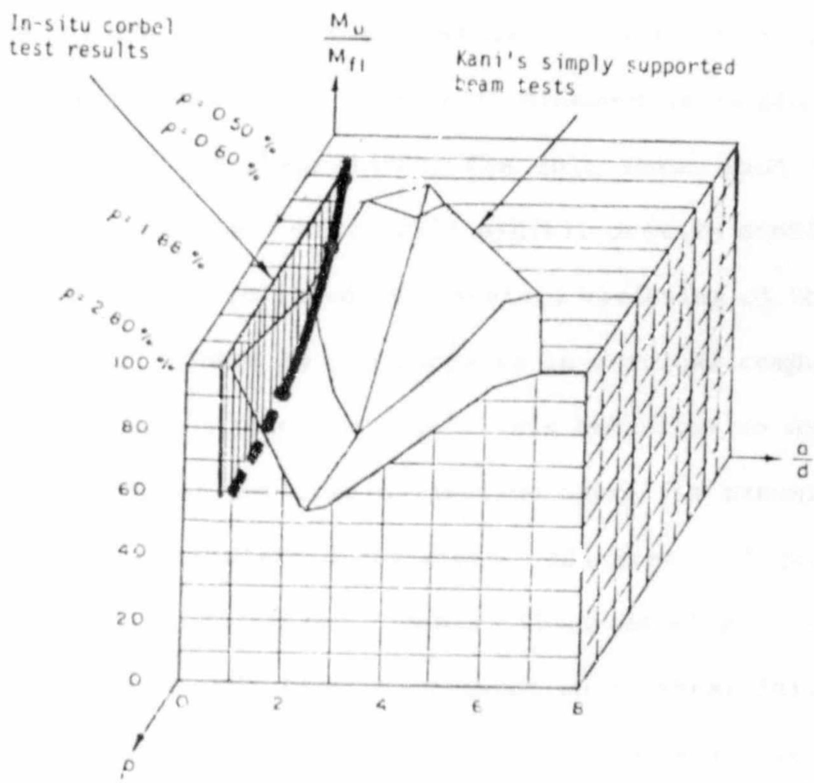


FIGURE 3.10 COMPARISON OF IN-SITU CORBEL TEST RESULTS WITH KANI'S SIMPLY-SUPPORTED BEAM TESTS. (ALL SPECIMENS UNREINFORCED FOR SHEAR.)

in existence and thus careful consideration should be given to reinforcing for shear so as to eliminate this valley. The philosophy and approach to reinforcing for shear depends on the observed mode of shear failure and thus differs for in-situ and bolt-on corbels. For the case of the bolt-on corbels where dowel cracks develop which are nearly normal to the direction of the applied shear force and which are the first manifestation of shear failure, the most effective use of shear reinforcement is to place it in the direction of the shear force. The full shear load is then transferred to the dowel, which will exhibit ductile double curvature hinge formation, followed by immediate hardening of the load-deflection curve, caused by the increase in vertical component of the tension in the deformed dowel. This behaviour is evident upon examination of the load-deflection curve for precast bolt-on corbels "fully" reinforced for shear, indicated in Figure 3.7. For the case of the in-situ corbels, which develop steep diagonal shear cracks as the first manifestation of shear failure, the most effective means of reinforcing for shear is to place links as nearly normal to this crack as is practically possible, in such a way that opening of the crack is inhibited and aggregate interlock is thus maintained. Horizontal links generally fulfil this requirement for corbels. This form of shear reinforcement was shown to be very effective in this test series and a model for shear which quantifies the effect of these horizontal links is developed later in this thesis. The performance of vertical links relative to horizontal links is also quantified in this model, and confirmed by tests on deep beams reinforced for shear with both vertical and horizontal links later in this work.

It is important to note that the horizontal links do not exhibit specific plastic ductility at the ultimate limit state for shear, in that the combined ductile dowel action contributions of of all the shear reinforcement is likely to be only a fraction of the peak shear resistance. The peak shear resistance is thus considerably enhanced by horizontal shear reinforcement but the mode of shear failure specifically remains non-ductile. This behaviour is observed in tests and should be reflected in a universal model for shear. Existing truss analogy models, regardless of their sophistication, do not generally quantify the performance of horizontal links. In contrast to the mode for horizontal links, the failure mode in shear for the truss analogy model, for beams with vertical links, can be plastic ductile, although it is not necessarily always the case. For horizontal link shear reinforcement, ductility must of necessity be provided by flexural ductility, with the reinforcement having the specific task of raising the shear capacity to a value greater than the flexural capacity. Current codes[1,5,6] require a proportion of the area of flexural reinforcement as horizontal shear reinforcement for corbels. This is a reasonable approach as the shear capacity and flexural capacity of the element are thus linked, which is consistent with the above observations. Nevertheless, there is no technique available for quantifying the performance of the horizontal links themselves within the scope of these codes or cuurent literature, neither is it certain that an area of horizontal links of half that of the flexural reinforcement will always ensure attainment of the flexural ultimate limit state, and a new model for shear should aim at providing simple techniques for such evaluations.

The general philosophy of providing horizontal shear link reinforcement for structural elements with steeply inclined diagonal shear cracks implies that the technique can be used in a number of situations where the a/d ratio of the shear crack is somewhat less than unity, such as pad and raft foundations and pilecaps. Punching shear in deep slabs with span/depth ratios that are unusually small could also fall into this category and is consequently given attention later in this work.

In observing the form of the first shear crack in the bolt-on corbel specimens, it is evident that the first requirement is to provide a vertical link that encompasses the anchor bolt near the face of the corbel leg. Having eliminated this form of shear failure, however, the next mode to manifest itself is likely to be diagonal shear failure as for the in-situ elements, and thus horizontal links in the bolt-on units would also be required if flexural ductility was to be ensured.

3.3.4 PREDICTING CORBEL BEHAVIOUR

3.3.4.1 PREDICTED BEHAVIOUR OF BOLT-ON UNITS

Two stages of failure are apparent for bolt-on corbels which are reinforced for shear, namely dowel cracking load and ultimate failure load, where dowel failure necessarily precedes ultimate failure. A detailed evaluation of the load-deflection curves presented in Figure 3.7, using the alphabetic notation evident in the figure, indicates the curves to comprise the following:

(A) LINEAR ELASTIC UNCRACKED

In this stage a significant proportion of the load is carried by dowel action of the anchor bolts. The balance of the load is carried by friction between the legs of the unit and the wall. Previous research[12] has shown that a steel dowel embedded in a concrete matrix can be analysed as a beam on elastic foundation for this linear zone of the load-deflection curve. The subgrade modulus for the concrete is then given by the approximate relationship:

$$k_c = 130 \sqrt{f_{cu}}$$

where

k_c is the subgrade modulus for normal concrete. (N/mm^3)

f_{cu} is the mean cube strength of the concrete. (MPa)

The beam on elastic foundation analysis does not have particular relevance at the the ultimate limit state of a dowelled connection except in determining the point of formation of plastic hinges in the dowel. The relative difference in stiffness between the steel dowel and the concrete embedding matrix results in the evaluation generally being in terms of the flexural capacity of the dowel, associated with some subgrade plasticity, or tensile splitting of concrete, especially where covers to the dowel are inadequate. In normal reinforced concrete construction, the contribution of dowel action will thus generally be ductile, resulting from the effect of the plastic double curvature. In some cases, where the diagonal shear crack forms remote from a support in a structural element which is unreinforced for shear, the contribution of dowel action could be dependent only on tensile splitting of the concrete along the flexural reinforcement and would thus be non-ductile.

The linear behaviour generally terminated with dowel splitting as described above. The dowel cracking occurred visibly independently of the presence of shear reinforcement in the corbel. The dowel force at first cracking or dowel splitting is, however, dependent on the cover to the dowel, usually in relation to the dowel diameter[12]. It is also dependent on the grade of concrete of the embedding matrix. While it is thus evident that dowel behaviour is generally dependent on a large number of parameters[12,14], the assessment for the particular prototype specimens tested, in which only the concrete strength was varied, of the capacity of the dowels at dowel cracking is based on the tensile strength of the concrete, which in turn can be assumed to be approximately proportional to the square root of the concrete cube strength[5]. In order to obtain the total load at which dowel cracking occurs for the bolt-on unit, any other contributions to shear resistance must be added to this dowel resistance for overall equilibrium. This is consistent with the approach adopted in the formulation of a new model for shear later in this thesis. The contribution of friction between the wall and the unit must therefore be added to the dowel action contribution to obtain the bolt-on specimen failure loads indicated in Figure 3.5, or the limits of the principally elastic zones of the bolt-on specimen curves in Figure 3.7. A summarized tabulation of the test results for the precast specimens, which also gives an indication of the variation measured, is given in Appendix A.

(B) LIMITED DUCTILE BEHAVIOUR

If the precast specimen is reinforced for shear, dowel cracking does not necessarily represent the ultimate shear capacity of the unit, as development of the crack may be resisted by the shear reinforcement present. Under these conditions, the bolts are free to kink in double curvature, with plastic hinges forming at a approximately the position of the shear link in the corbel leg nearest the interface and at a point within the wall determined from an assessment of the dowel in the wall as a beam on an elasto-plastic foundation. For the specimens tested, the value was just under 20kN per bolt, based on the assessed double curvature span and the fully plastic moment of resistance of the bolt. The balance of the shear load is carried by friction between the precast unit and the wall.

(C) STRAIN-HARDENING BEHAVIOUR

As the bolts deform further from the position of kinked double curvature, a considerable additional component of vertical load is carried by direct tension in the bolt.

(D) ULTIMATE SHEAR FAILURE

This was usually reflected as front shear failure, or ront splitting, of the precast unit, as described previously. The load at which this mode occurred appeared to depend on the geometry of the corbel, the tensile strength of the concrete and the position of the the load on the specimen relative to the front cover to the flexural reinforcement. The theoretically conceived consecutive modes of failure that have been identified as a result of this test series can thus be summarised as follows:

Mechanism of potential failure	Mode of failure	Remedial action
1. Dowel cracking	Shear. (Value determined by tensile strength of concrete)	Vertical link encompassing the dowel, near face of corbel. Link to carry full load.
2. Front cover (or front splitting)	Abrupt, thus can be classified as splitting shear.	Reduce top cover, move load back from front of corbel.
3. Steeply inclined diagonal cracks	Abrupt, "typical diagonal shear failure"	Reinforce with horizontal links to inhibit opening of diagonal crack.
4. Pullout of anchor bolt.	Limited ductility. Analogous to average bond failure.	Ensure adequate average bond by detailing attention.
5. Yield of bolt in tension; rotation of unit about heel or ductile vertical slip.	Ductile flexural ultimate limit state or ductile vertical shear ultimate limit state.	Design philosophy should aim at this being the lowest ultimate limit state.

With the implementation of each consecutive remedial action, the applicable potential mode of failure or ultimate limit state is eliminated and the failure is forced to reflect in the next mode. The achievement of structural ductility would thus be considerably aided by the use of mild steel flexural reinforcement, all other parameters remaining constant. This is particularly true for anchor bolts which rely on wedging or similar techniques for adequate average or anchorage bond.

3.3.4.2 PREDICTED BEHAVIOUR OF IN-SITU CORBELS

In order to investigate the performance of of in-situ corbels reinforced for shear with horizontal links in detail, the load-deflection curves shown in Figure 3.7 can be assessed as follows:

(A) LINEAR ELASTIC PORTION

Up to the instant of first cracking of the concrete in any form, the load is carried by the homogeneous structural element subjected to the state of plane stress resulting from the applied load. Flexural cracks are the first to occur, at the root of the top of the corbel at the point of maximum bending moment. These do not affect this portion of the curve markedly and opening is inhibited by the presence of flexural reinforcement.

(b) DIAGONAL CRACKING

As the load increases relative to the load at which first flexural cracking occurred, steep diagonal cracks develop in the corbel, with their point of origin near the neutral axis of the element. An associated loss of stiffness is evident in the load-deflection curve, although this is more noticable for corbels unreinforced for shear, as indicated in Figure 3.5. For corbels unreinforced for shear, from the instant of the formation of this steep diagonal shear crack to the point of shear failure, the load is carried by the contributions of aggregate interlock, dowel action and compression zone. For specimens reinforced for shear with horizontal links, the load is increased significantly above that of shear failure for specimens unreinforced for shear. In certain specimens, the flexural ultimate limit state is attained as a result of the enhanced shear capacity of the corbel. Front shear, or splitting, modes occurred in many specimens as a result of poor detailing, independently of the presence of horizontal links, prior to flexural failure.

(C) DUCTILE FAILURE

For specimens where both the above modes of shear failure were precluded by the use of horizontal links and good detailing procedures, the ultimate limit state coincided with flexural ductility, the corbel unit rotating about a point near the heel of the corbel. Only if this mode of failure is achieved, can the unit be analysed using the strut and tie analogy. A ductile shear mode, as opposed to the ductile flexural mode observed in appropriate specimens, cannot be attained. The theoretical consecutive modes of failure that could occur in the in-situ corbels can be summarised as follows:

Mechanism of Potential Failure	Mode of Failure	Remedial Action
1. Steep diagonal tension cracks.	Shear (diagonal tension).	Provide horizontal links to enhance shear capacity above flexural capacity.
2. Front cover (or front splitting)	Abrupt, thus classified as splitting shear.	Attention to detail. Move load from front of corbel.
3. Pull-out of flexural reinforcement.	Average bond, or anchorage failure.	Attention to anchorage of flexural reinforcement both sides of flexural crack.
4. Yield of flexural reinforcement, rotation of unit about heel.	Ductile flexural ultimate limit state.	Design philosophy should aim at this being the lowest ultimate limit state

3.4 PRISMATIC CORBELS

In 3.3 it was observed that the resistant shear stress for the haunched in-situ corbels unreinforced for shear appeared to be slightly lower than the generally predicted code values. Although this observation was made, insufficient tests were undertaken to assess its significance. For this reason, a series of prismatic, in-situ corbels, unreinforced for shear, was tested in parallel with a series of rectangular, prismatic, simply-supported beams of identical geometry.

3.4.1 DESCRIPTION OF THE SPECIMENS

Six rectangular, prismatic corbels, unreinforced for shear and six rectangular, prismatic, simply-supported beams, subjected to normal two-point loading, were tested in this series. A general view of the corbels and a typical diagonal shear crack for one of the specimens are shown in Figure 3.11. All twelve specimens were designed to have a concrete cube strength of 30MPa at the time of the test. The grade of concrete was thus not intended to be investigated in this test series. All specimens were of the

FIGURE 3.11 PHOTOGRAPHS SHOWING GENERAL VIEW OF THE PRISMATIC CORBELS TESTED AND A DETAILED VIEW OF A TYPICAL DIAGONAL SHEAR CRACK.

same sectional dimensions, in order to eliminate scale effects. All depths were thus 250mm and all widths 150mm. The flexural steel ratio, however, was varied between 0,5% and 3%. The range of flexural steel ratio investigated was thus intentionally increased relative to the previous tests, in order to assess the influence of flexural steel ratio on the shear resistance for these specimens unreinforced for shear, over the full range of steel ratio likely to be encountered in practice. The cover to the flexural reinforcement was not varied in this test series, and could be considered to be "average".

3.4.2 TEST RESULTS

With the exception of one of the lightly reinforced specimens which failed in flexure, all the prismatic corbels tested failed in shear, developing classical diagonal shear cracks which eventually precipitated the shear failures. In the evaluation of the ultimate shear capacity of the specimens, it was found that the parametric trends for the variation of shear resistance with change in flexural steel ratio were consistent with the previous test results of haunched corbels, the test results being tabulated in Appendix A. The shear resistance of these specimens unreinforced for shear was observed to increase with increasing flexural steel ratio consistently throughout the range considered. The relative increase measured in the tests between 0,5% and 3% steel ratio was, however, slightly less than the codified increases of CP110, although again insufficient tests were undertaken to assess the significance of this observation. A distinct parametric trend was, however, clearly evident.

The shear resitance of the six prismatic corbels was not significantly or consistently different to that of the simply-supported beams subjected to two-point loading. The modes of shear failure were also entirely consistent for the two specimen types, in that distinct diagonal shear cracks occurred in both, and these cracks in turn eventually precipitated the shear failure for both specimens. These observations are consistent with the hypothesis of the universal nature of shear behaviour relative to certain parameters.

3.4.3 GEOMETRIC EXTREMES

In addition to the prismatic corbels tested above, consideration was given to the evaluation of prismatic corbels of significantly deeper section and reduced a/d ratio compared with any of the corbel specimens tested previously. These tests were conducted in order to assess the influence of extreme reduction in a/d ratio and thus also to simulate web-flange interface conditions and evaluate specimens of geometry approaching those used by Walraven and Reinhardt[44], with respect to a/d ratio.

In order to achieve this, eight additional prismatic corbels were constructed and tested in shear. These specimens were 500mm in depth and 75mm in width. Grade of concrete was not varied for these tests and was constant at a mean cube strength of approximately 20MPa. The a/d ratio for all the specimens was of the order of 0,2. Flexural steel ratio was varied, and horizontal shear reinforcement was included i certain of the specimens. The specimen geometry and the test procedure are indicated in Figure 3.12. The significantly reduced a/d ratio for these corbels is also clearly evident in the figure.

Test results indicated that fc corbel specimens that were unreinforced for shear, s occurred on a steeply in-