is subjected to shearing force; the reinforcement usually being normal to this plane. Dowel action has been extensively investigated by a number of researchers [12,14,17] and the contribution to the total ultimate shear resistance of reinforced concrete beams attributable to dowel action of the flexural reinforcement is generally considered to be of the order of 20%. The assessment of dowel action, in the traditional sense analytically, has been in terms of a beam on elastic foundation, when considering behaviour in the elastic range, and the formulation of this theory relies extensively on a realistic assessment of the subgrade modulus of the concrete surrounding the embedded dowel or reinforcement. At the ultimate limit state, however, the evaluation of the dowel as a beam on elastic foundation becomes less meaningful, except in determining the distance into the concrete matrix at which the dowel is likely to form a plastic hinge in bending. Two distinct ultimate modes of failure for a dowelled connection or a dowel-type phenomenon in a normal reinforced concrete structural element in the post diagonal cracking phase are observed. These are the formation of plastic hinges and subsequent kinking of the dowel, or tension cracking, splitting and spalling of the concrete cover to the dowel. The former mode can be considered to be a ductile ultimate limit state and the latter a brittle, non-ductile mode.

The likelihood of the latter mode occurring is dependent on the ratio of dowel diameter to the concrete cover to the dowel, the material properties of the dowel and matrix, and the presence of additional reinforcement such as links. For dowels having infinite cover the predicted ductile ultimate limit state would generally be considerably in excess of 20% of the total ultimate shear capacity of normally proportioned reinforced concrete beams. Taking into account realistic covers and load situations in normal reinforced concrete beams, however, it would appear that the contribution of dowel action of 15% to 20% of total ultimate shear resistance is quite reasonable. In considering the ductile type of ultimate limit state for a dowel in a reinforced concrete structural element, the plastic double curvature evaluation, taking into account distances between links and observed splitting modes along reinforcement, gives results which are consistent with those mentioned above. Taylor[40,41] has evaluated the contributions of dowel action, aggregate interlock and compression zone to the total ultimate shear resistance of a range of beam specimens and summarises these contributions very effectively, as shown in Figure 2.5.

2.1.2.2 AGGREGATE INTERLOCK

Considerable effort has been dedicated to an evaluation of the contribution of aggregate interlock to the total shear resistance

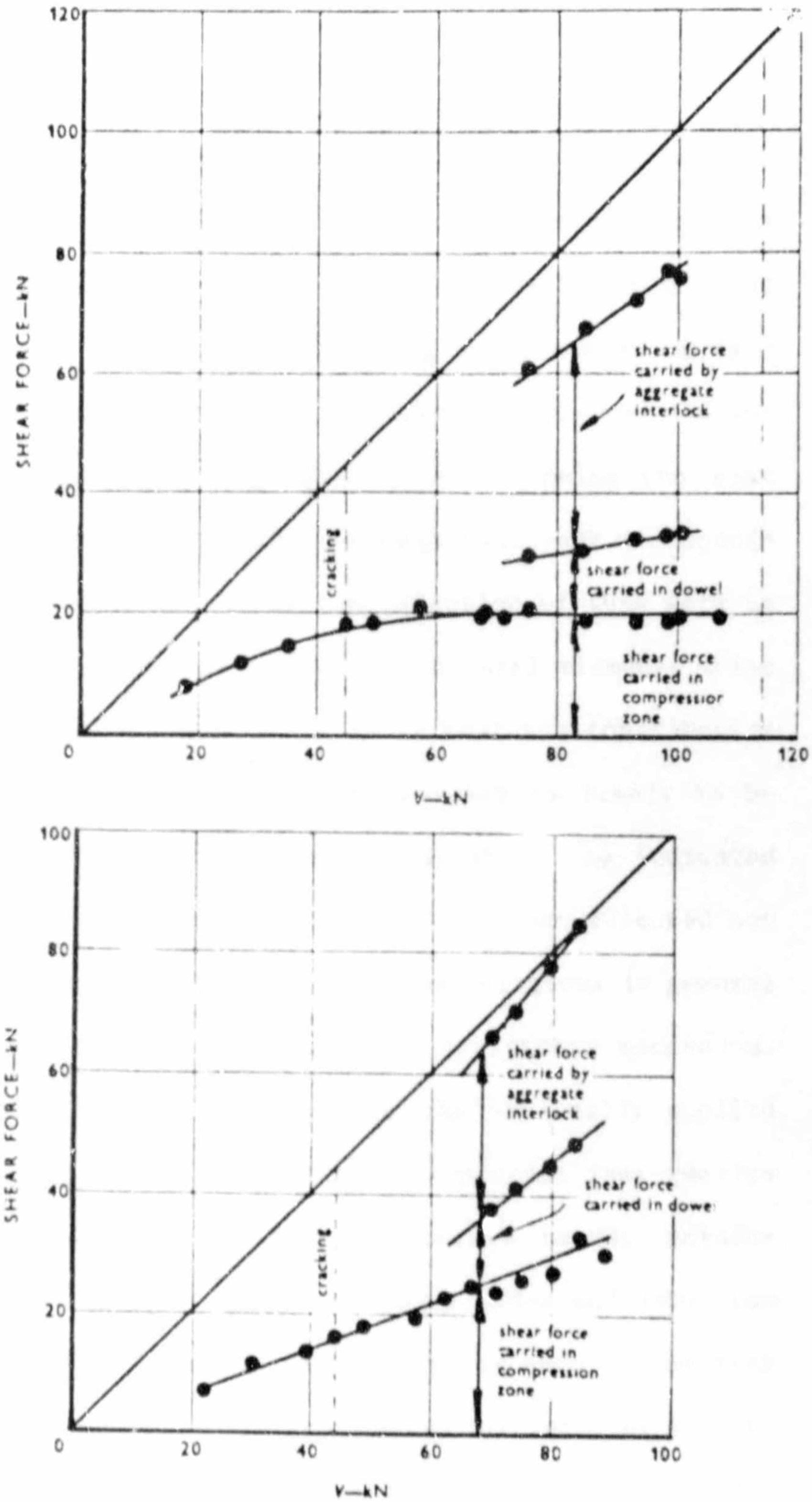


FIGURE 2.5 DIAGRAMMATIC REPRESENTATION OF TAYLOR'S TEST RESULTS SHOWING RELATIVE CONTRIBUTIONS TO TOTAL SHEAR CAPACITY BY DOWEL ACTION, AGGREGATE INTERLOCK AND COMPRESSION ZONE FOR BEAMS UNREINFORCED FOR SHEAR.[41]

of beams unreinforced for shear and to an understanding of this behaviour[41,44]. This phenomenon is considered to be an important contributor to the assessment of the total shear resistance of reinforced concrete, usually concentrating on specimens unreinforced for shear. In contrast to the generally accepted assessment of the contribution of aggregate interlock being the most significant contributor by an order of magnitude only for specimens unreinforced for shear, it is the intention of this work to propose that this is also true for many structural elements which are reinforced for shear. Taylor has shown that the contribution of aggregate interlock to total shear capacity is likely to be in excess of 50% for beams unreinforced for shear, as indicated in Figure 2.5. Two of Taylor's diagrams have been selected and are shown in this figure. The intent of these diagrams in general is to match the sum of the internal shear resistance mechanisms with the diagonal correlation line of the externally applied shear force, as indicated. Both diagrams represent test results on "complete" rectangular, reinforced concrete beams, unreinforced for shear, of overall depth just over 400mm and subjected to single point loads. The upper diagram is for such a beam with a/d ratio of 3,2 and the lower diagram for an a/d ratio of 4. Both diagrams can thus be considered to represent "unconstrained" diagonal shear cracks at the ultimate limit state, in that the diagonal cracks are not forced to form steeply owing to the proximity of the load to the support.

Possibly the most significant work undertaken recently in evaluating of the mechanics of aggregate interlock in reinforced concrete is that of Walraven and Reinhardt[44]. In a detailed examination of the phenomenon of aggregate interlock, a theory based on the concept of particle interference is developed. For any particular shear crack in concrete two stresses are derived, these being the shear stress tangential to the sliding area (or crack), τ, and a normal stress, σ, created by the movement of the particles over one another and over the matrix as a result of the application of a shear force parallel to the crack under consideration. Movement parallel to the shear crack (or slip) is denoted by Δ and the crack width of the shear crack is denoted by W. Two test series were undertaken by the authors, these being either embedded bars crossing the shear crack or external restraints being applied to the specimens. The primary function of these external supports was to faciltate the measurement of the normal stress developed in the crack as a consequence of particle interference during the shearing process. The former tests generally commenced with a nominal crack width W (of the order of 0,02mm) and further opening of the shear crack was a consequence of particle (or aggregate) interference under the influence of the shear force which was applied parallel to the crack as is evident from the specimen configuration in Figure 2.6. This phenomenon was referred to as the crack opening path.

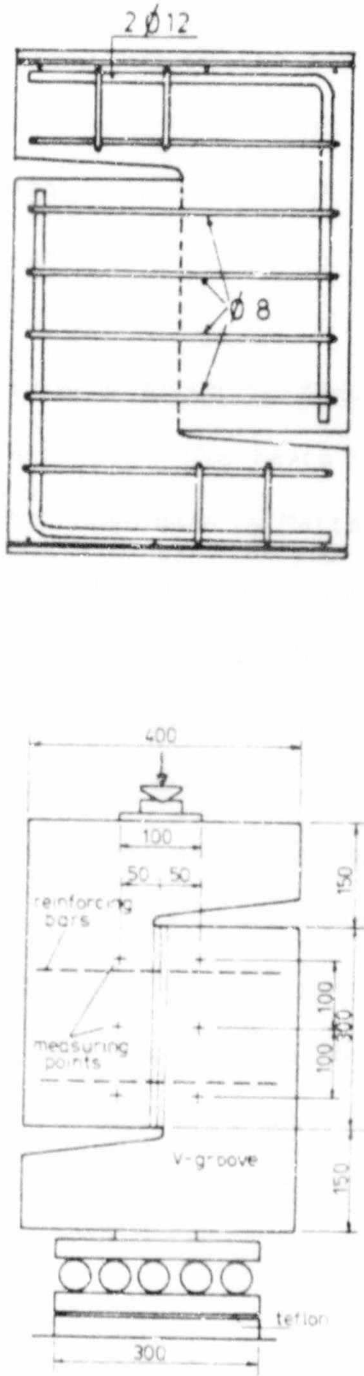
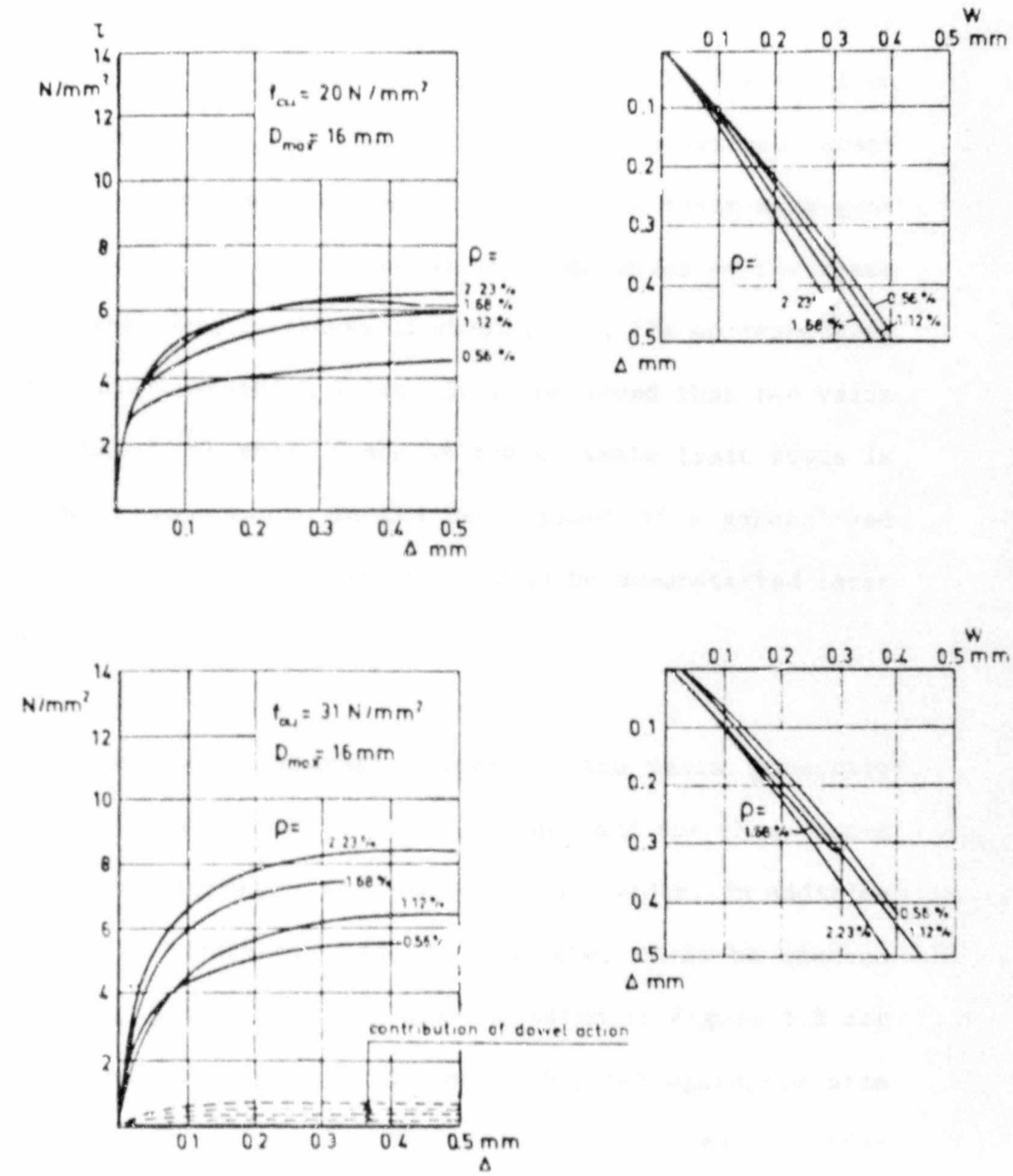


FIGURE 2.6 GEOMETRY OF SPECIMENS USED BY WALRAVEN AND REINHARDT FOR DETAILED INVESTIGATION OF THE MECHANICS OF AGGREGATE INTERLOCK.[44]

For the latter the shear crack was opened to varying amounts, by a transverse splitting process, prior to the application of the vertical shear force. The maximum initial crack width was 0,4mm and additional opening was again the result of of the applied shear force parallel to the crack.

The configuration of the specimens shown in Figure 2.6 is considered to be an important aspect of the tests undertaken. The specimens chosen are similar to those used by Mattock and Dulacska previously[14,19] and it is evident that these specimens simulate the specific shear situation which corresponds to an a/d ratio of 0 in practice. The implication is that a specific point in the generalised relationship between ultimate shear resistance capacity and a/d ratio has been established and can therefore be used with considerable confidence. The crack opening path mentioned by the authors is, however, believed to be affected by additional parameters as the a/d ratio changes for changing situations of shear in various structural elements, and not purely by particle interference, which is the case for the specimen geometry selected by the authors.

For the specimen geometry chosen and for those types having embedded bars, the relationship between τ along the shear crack and the slip along the shear crack, Δ, for varying concrete grade, f_{cu}, and varying steel ratio, ρ, is indicated in Figure 2.7. It can be seen that the ultimate shear stress achieved is of the



D_{max} is the aggregate size.

FIGURE 2.7 INFLUENCE OF REINFORCEMENT RATIO AND GRADE OF CONCRETE ON τ, Δ RELATIONSHIPS AND CRACK OPENING PATHS FOR TESTS BY WALRAVEN AND REINHARDT.[44]

order of 5MPa to 8MPa, this value being only moderately sensitive to either concrete grade or steel ratio for strengths normally encountered in practice. It is also evident that dowel action contributes very little to this ultimate shear resistance capacity. The crack widths at the commencement of the tests were generally of the order of 0,02mm and at the completion of the tests the slip, Δ, was never in excess of about 2,5mm, the corresponding crack width never exceeding 1,5mm. It is believed that the value of crack width of the shear crack at the ultimate limit state is of fundamental importance in the development of a generalised model for shear resistance capacity as will be demonstrated later in this work.

For the specimens with external restraint, the tests generally commenced with larger initial crack widths, and for these tests the influences of aggregate scaling and crack width, in addition to the effects of grade of grade of concrete, could be studied in some detail. Typical results are indicated in Figure 2.8 for concrete of cube strengths of 13 and 38MPa and aggregate size 16mm. In essence, the curves plotted from the test results indicate that for normal concrete the specimen can still sustain a shear stress of the order of 6MPa before attaining the ultimate limit state even at the largest crack width recorded of approximately 1,2mm. In addition to the results for 16mm aggregate shown here, tests were also undertaken on specimens of 32mm maximum aggregate size. The results were not very sensitive to these

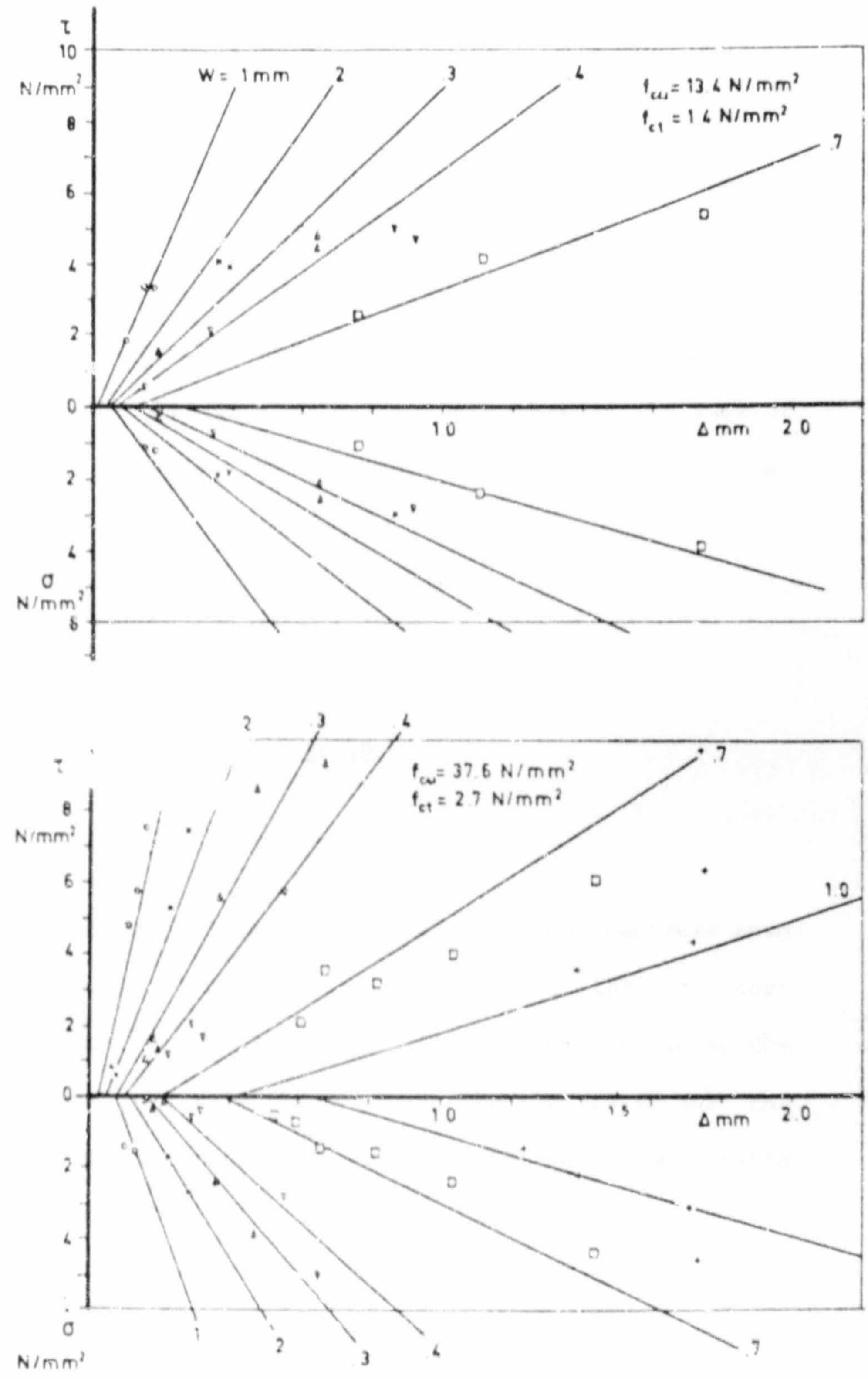


FIGURE 2.8 NORMAL STRESS- AND τ-Δ RELATIONSHIPS ESTABLISHED BY WALRAVEN AND REINHARDT[44] FOR SPECIMENS WITH EXTERNAL RESTRAINT.

changes although the larger aggregate performed slightly better at larger crack widths. Within the range of aggregates that could reasonably be anticipated in practice, it seems unlikely that scaling of the aggregate would be significant in respect of practical design. It is important to note that in the case of this series of tests the crack width on completion of the test never exceeded 1,5mm.

2.1.2.3 COMPRESSION ZONE

From tests conducted by Taylor[40,41] on reinforced concrete beam specimens unreinforced for shear, it is apparent that the contribution to total ultimate shear resistance attributable to the compression zone is likely to be of the order of 20 to 30% The specimens tested in this evaluation generally had an a/d ratio of between 2 and 4.

In conclusion it would thus appear that as a reinforced concrete beam unreinforced for shear approaches the ultimate limit state with respect to shear, one specific diagonal shear crack, out of a number of potential shear or shear/flexural cracks, will actually precipitate the shear failure. With respect to an evaluation of this specific shear crack, the total shear resistance of the beam is made up essentially of contributions from three sources. In general terms, the results of researchers such as Taylor indicate that the contribution of dowel action is of the order of 15% to 20% of the total shear resistance, the contribution of the compression zone about 20% and that of aggregate interlock the most significant at approximately 50% to 60% of the total resistance, for beams unreinforced for shear with "unconstrained " diagonal cracks. The fact that the contribution of aggregate interlock is significantly larger than the other contributions is considered to be of fundamental importance, as loss of aggregate interlock will generally coincide with abrupt shear failure of the section as a whole, if the ultimate mode is diagonal failure.

2.2 MODELS FOR ELEMENTS REINFORCED FOR SHEAR

The models considered thus far, although giving some attention to techniques for reinforcing for shear, tended to concentrate on, and attempt to describe adequately, the phenomenon of shear behaviour in structural elements unreinforced for shear in the conventional sense.

The models examined here move essentially into a different realm with respect to the evaluation of shear behaviour in that they are derived from principles relating to the necessary inclusion of shear reinforcement, generally in the form of vertical links. The current models for shear evaluation of structural elements reinforced for shear generally owe their derivation to some extent to the "45° truss analogy" developed around the turn of the century by Ritter and Mörsch[9]. This theory postulates that the diagonally cracked reinforced concrete beam reinforced for shear acts as a truss with parallel longitudinal chords and with a web composed of diagonal concrete compression struts inclined at 45°, the web reinforcement acting as transverse tension ties. The model is generally defined by a shear arm zone which is subjected to a large number of diagonal shear cracks spread over the full zone. Strains are thus generally evaluated as average strains, spread over the full zone under consideration. The fact that this model is generally considered to be "conservative", particularly for beams with relatively small amounts of web reinforcement, has led to a correction term, normally regarded as the "concrete contribution", being incorporated in most code formulations. The acknowledgement of this shortcoming in the truss analogy model is of considerable importance in the formulation of the model proposed in this work. It is believed that the truss analogy and many of the derivatives of the theory cannot adequately explain the shear behaviour, observed consistently in tests, of structural elements lightly reinforced for shear in the web region.

The current models which owe their derivation to the truss analogy generally improve the evaluation of the phenomenon of shear failure by consideration of the variability of the slope of the diagonal tension crack as opposed to fixing it at 45° as in the Morsch model, and the development of related shear flow criteria. Among those who have contributed significantly to the development of sophisticated plastic flow truss models to simulate the behaviour of shear and torsion in reinforced concrete elements which are reinforced for shear or torsion are Braestrup[3], Collins[9] and Thürlimann[42]. With respect to shear specifically, models have been developed both for a generalised element of a shear web subjected to pure shear and also for the more typical case encountered in beams of the combined effects of shear and bending. The models proposed in this field certainly display reasonable similarities and that of Thürlimann is considered to be fairly representative of this class of model.

2.2.1 THÜRLIMANN'S MODEL FOR SHEAR AND BENDING

As a static model, the truss model shown in Figure 2.9 is used, with upper and lower stringers as chords, the links as posts and the concrete as a diagonal compression field under a variable angle θ. The effective depth, d, of the section can approximate to the dimension between the chords and the width of the section is given as b. The mechanical reinforcement ratio, ζ_c, of the vertical links for the reinforced concrete element under consideration is given by the expression:

$$\zeta_c = \frac{A_{sv}}{bs_v} \cdot \frac{f_y}{f_c}$$

where

A_{sv} is the cross-sectional area of the link (this would usually be 2 legs of a link). (mm^2)

s_v is the spacing of the links. (mm)

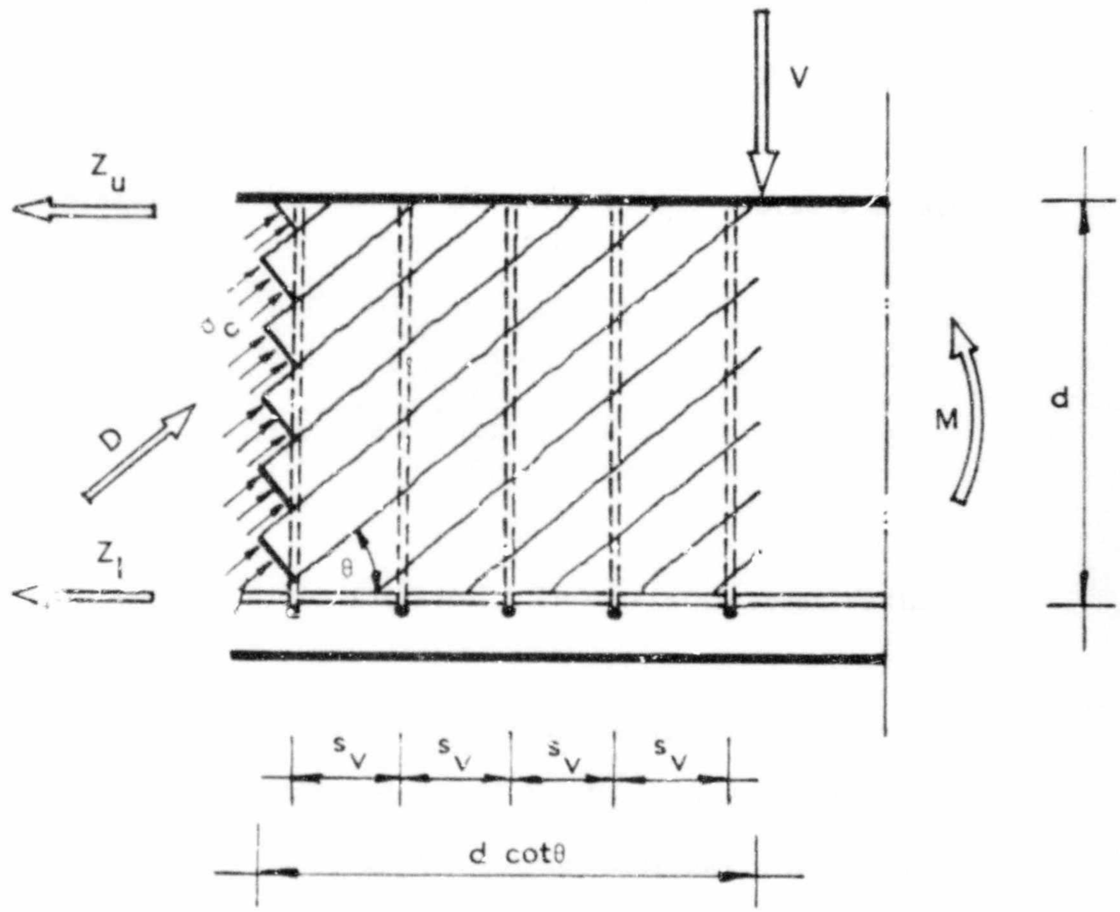


FIGURE 2.9 THÜRLIMANN'S TRUSS MODEL FOR THE EVALUATION OF BEAMS REINFORCED FOR SHEAR WITH VERTICAL LINKS.[42]

f_y is the yield stress of the link reinforcement. (MPa)

f_c is the crushing strength of the concrete in the reinforced concrete element. (MPa)

The expression for the mechanical reinforcement ratio can also be written as:

$$\zeta_c = \mu \frac{f_y}{f_c}$$

where μ is the (vertical) link reinforcement ratio.

The portion of beam under consideration is subjected to a shear force of V and a bending moment of M. From considerations of vertical equilibrium in Figure 2.9, the diagonal compression D is given by:

$$D = V/\sin\theta$$

From considerations of horizontal equilibrium, the forces in the upper and lower chords of the truss model are given by:

$$Z_u = -M/d + 0,5V\cot\theta$$

$$\text{and} \qquad Z_1 = +M/d + 0.5V\cot\theta$$

The concrete stress, σ_c, in the compression field in the web zone can thus be derived as being:

$$\sigma_c = -D/bd\cos\theta$$

$$\text{thus} \qquad \sigma_c = -V/bd\sin\theta\cos\theta$$

The stirrup reinforcement is constant along the axis of the beam. The lower stringer reinforcement varies such that it will reach a yield value of P_1 at a critical section. Excluding concrete failure the link reinforcement will also reach a yield value of $F_y = f_y A_{sv}$ for each link.

From vertical equilibrium it is also evident that for the model:

$$V = F_y(d\cot\theta/s_v)$$

From consideration of the above equilibrium equations and through appropriate manipulation, the interaction equation between applied shear force and applied bending moment at any point in a reinforced concrete beam, shown diagrammatically in Figure 2.10, can be derived as follows:

$$\frac{M_p}{M_{po}} + \left(\frac{V_p}{V_{po}}\right)^2 = 1$$

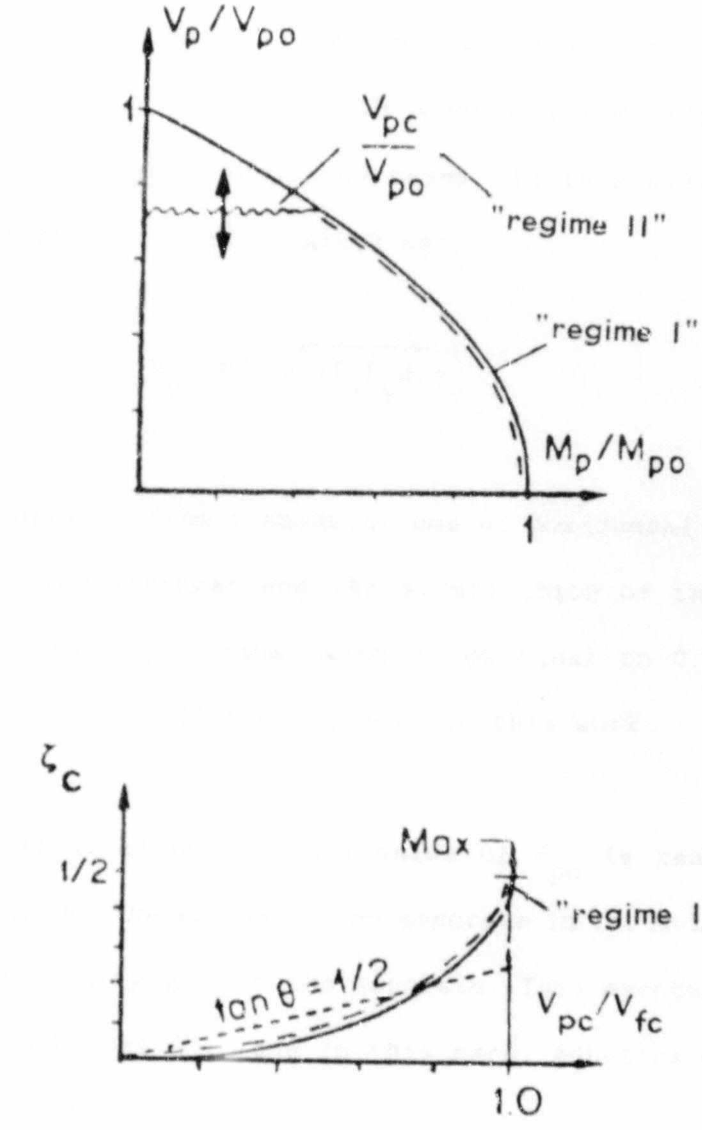


FIGURE 2.10 ULTIMATE SHEAR-ULTIMATE MOMENT INTERACTION CURVE AND INFLUENCE OF MECHANICAL REINFORCEMENT RATIO FOR THÜRLIMANN'S TRUSS ANALOGY MODEL FOR REINFORCED CONCRETE BEAMS REINFORCED FOR SHEAR WITH VERTICAL LINKS.[42]

where the terms are as defined in the list of symbols.

In many cases the occurrence of maximum shear force in a reinforced concrete beam will coincide with a support condition for which M is zero for simply supported cases. In this event the limiting "plastic shear force" is given as:

$$V_{po} = \sqrt{2P_1 F_y d/s_v}$$

This equation is derived from considerations of horizontal equilibrium for the lower stringer and the substitution of the requirement for vertical eqilibrium, with M put equal to 0. The symbols are defined in the list of symbols in this work.

A cut-off can be attained before this value of V_{po} is reached, however, if the compressive stress in the concrete in the web zone reaches the crushing strength of the concrete. This eventuality is also depicted in Figure 2.10 and in this case, equating σ_c to $-f_c$, results in the expressions:

$$\zeta_c = \sin^2\theta$$

and

$$\frac{V_{pc}}{bdf_c} = \sqrt{\zeta_c(1-\zeta_c)}$$

This expression has been maximised by appropriate differentiation, such that:

$$\frac{\partial V_{pc}}{\partial \zeta_c} = 0$$

The reference value of the maximum value V_{fc} governed by the concrete strength alone is introduced, such that:

$$V_{fc} = 0{,}5 f_c bd$$

The ratio thus becomes:

$$\frac{V_{pc}}{V_{fc}} = \sqrt{\zeta_c(1-\zeta_c)}$$

This expression is depicted graphically in Figure 2.10. The practical interpretation of the curve in Figure 2.10 is that for mild steel links and average grade of concrete (in the region of Grade 25 to Grade 30 concrete), concrete crushing of the web will occur for a link reinforcement ratio, μ, of approximately 5%. For high yield links this would be about 2 to 3%. Realistically, it is probably impossible to construct a normal beam with a link reinforcement ratio in excess of about 4%. Codes are generally formulated such that even less link reinforcement is used as a maximum.

In considering the trends represented by these figures it is evident that various regimes of shear failure are applicable to this truss analogy model. The diagrammatic representation of Figure 2.10 indicates the zones of "regime I", in which yield of the link reinforcement occurs simultaneously with yield of the flexural reinforcement of the lower stringer and the onset of "regime II", where the yield of the link reinforcement coincides with the crushing of the diagonal compression field in the web zone. Owing to the fact that diagonal shear cracks first form at 45° to the horizontal near the neutral axis in the web region, it is argued that radical redistribution of the general slope of the shear crack is unlikely to occur and the angle θ in the model is thus limited to values between $\tan^{-1}0,5$ and $\tan^{-1}2$. The implications of the limit of $\tan^{-1}0,5$ are indicated in Figure 2.10. It is also recognised within the scope of the model that it is likely that a zone of uncracked concrete and a transition zone exist before the model becomes fully applicable. When these constraints are considered together with the practical construction constraints regarding normal link reinforcement, the truss model is probably only fully applicable, without transition effects, for mild steel link reinforcement ratios between 2% and 3% (and about half these percentages for high yield links). A significant number of structural elements are reinforced for shear with link ratios considerably smaller than this range.

2.3 FURTHER OBSERVATIONS IN SHEAR TESTS

In addition to the work done by various researchers mentioned above in deriving the models under consideration, certain parametric trends have been identified by recent research and which do not generally conform to certain of the current models.

In addition to Kani, other researchers have recently recorded the influence of scale on the ultimate shear stress of reinforced concrete beams, particularly those unreinforced for shear. The results of an investigation by Chana[4] are shown graphically in Figure 2.11. There is, however, a lack of similar test data for slab specimens of significant depth variation subjected to punching shear and this is considered to be justification for a series of tests of this nature. It is also generally noted that the effect of reduced shear stress capacity with increased depth of section becomes significantly less noticeable for specimens which are reinforced for shear. Some codes of practice attempt to take the phenomenon of the variation of ultimate shear stress capacity with depth in to account, notably the British and European codes and their derivatives. The American code, however, appears to take no cognizance of this phenomenon. The most recent evaluation of this phenom ified terms appears to be the proposal of the draft BS0000[13], which gives a re-

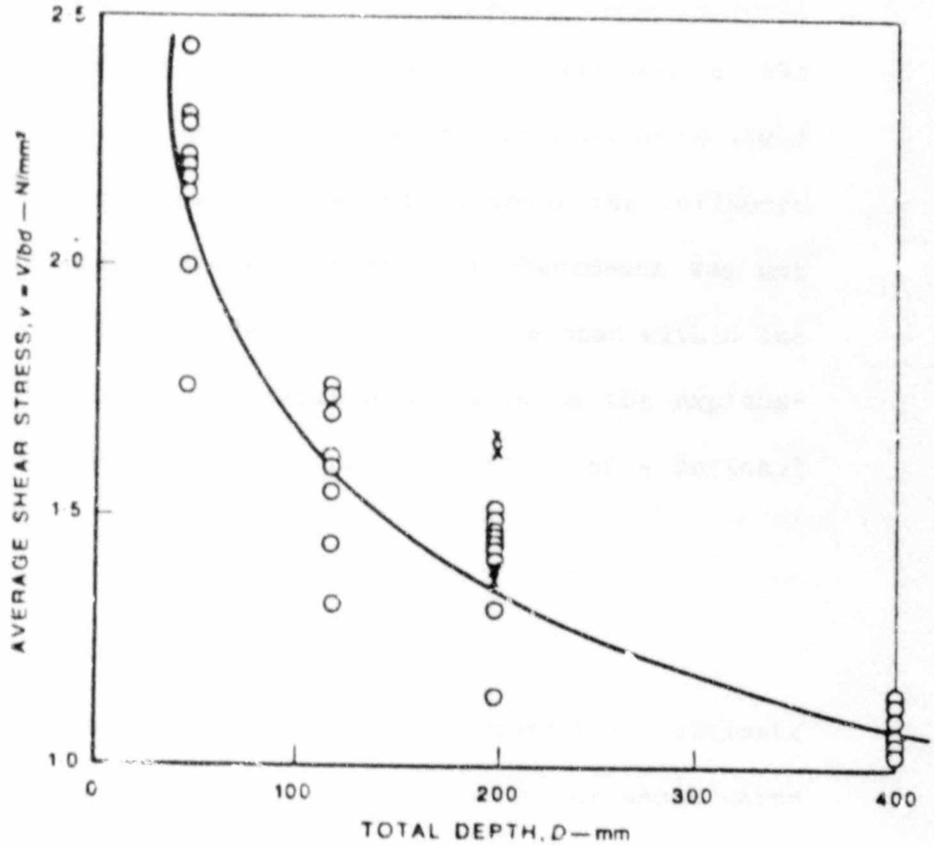


FIGURE 2.11 INFLUENCE OF DEPTH OF SECTION ON RESISTANT SHEAR STRESS OF BEAMS UNREINFORCED FOR SHEAR, ACCORDING TO CHANA.[4]

lationship for the variation of ultimate shear stress with depth of section as the fourth root of the inverse of the depth. Various explanations are given in the literature for this phenomenon, among them being change of strain rate, but with the exception of Kani's evaluation, no other current models appear to explain this phenomenon adequately in terms of their rational formulation. On the basis of the test series undertaken at the time, Kani was of the opinion that no simple factor could be added to the ACI code[1] provisions to take into account the influence of depth. He did state, however, that this phenomenon was not unexpected in terms of his model. The model developed within the scope of this work places considerable emphasis on the explanation of this phenomenon within the constraints of a rational formulation.

In addition to the observed behaviour of variation of ultimate shear stress resistance with depth, particularly for beams unreinforced for shear, it is generally recognised that there is an apparent increase in shear resistance with decreasing shear-arm to depth ratio (usually referred to as a/d ratio). Kani's valley of diagonal failure reflects this phenomenon quite clearly. It has certainly also been observed and documented by other researchers, notably those involved in tests on specimens which generally tend to have small a/d ratios owing to specific geometric requirements, such as corbels. There is an attempt to reflect this increase in certain codes, by adjusting the the ultimate resistant shear stress by a factor 2d/a, for a/d ratios less than 2. Some codes do not take cognizance of this parameter, although there is usually a concession in terms of an empirical distance from the support that shear performance should be calculated. Current models for shear, especially those based on the contributi to shear resistance by aggregate interlock, compression zone and dowel action and those based on the truss analogy do not generally appear to explain or model this phenomenon adequately in terms of their rational formulation. The model developed within the scope of this thesis attempts to incorporate this phenomenon.

It is also evident from the assessment of the various models for shear considered thus far in this chapter that there are two broad approaches which can be adopted in the evaluation of the shear performance of reinforced concrete structural elements. The zone of the structural element subjected to the transfer of shear force can either be evaluated on the basis of the average or "smeared" strain behaviour of the whole zone subjected to a large number of diagonal shear cracks, or closer scrutiny can be given to a single, specific shear crack. On the basis of the observed modes of shear failure recorded in the following chapters, this work concentrates on the specific, single diagonal shear crack which eventually precipitates the shear failure of the structural element at the ultimate limit state.

2.4 SUMMARY OF CODE APPROACHES

Codes of practice generally tend to be formulated by committee such that they reflect both state of the art research results and state of the art practice or the perceived requirements of practitioners. In order to gain an overview of the variety of approaches which can be adopted in the evaluation of shear performance, a representative sample of codes has been considered. Each code is assessed in terms of its recognition of the parameters which appear to affect the shear performance of various reinforced concrete structural elements. The codes considered are:

2.4.1 CP114

The measure of the shear performance of reinforced concrete structural elements adopted in this code[7] is that of shear stress, which is calculated by dividing the service shear force by the appropriate effective cross-sectional area of the element under consideration. This shear stress must be less than or equal to an allowable shear stress which is dependent only on grade of concrete. Thus the only parameter recognised in the assessment of the shear resistance is grade of concrete. The position where the section is to be evaluated is not stated specifically except for the case of punching shear which must be considered on a square perimeter (for square columns) at a distance of half the thickness of the slab from the column face.

If shear reinforcement is required, the link requirements are derived from a simple truss analogy model with no contribution to the shear resistance from the concrete.

2.4.2 ACI 318-77

The reinforced concrete structural element is assessed in terms of the ultimate shear stress which is obtained by dividing the ultimate shear force by the appropriate effective cross-sectional area of the section under consideration[1]. The resistant shear stress is dependent only on the grade of concrete. This is thus the only parameter recognised as influencing shear resistance. The shear evaluation is considered at an effective depth from the support face for both beams and punching shear in flat slabs, where a rounded punching perimeter is used.

In the event that shear reinforcement is required, the link requirements are derived from a simple truss analogy model but cognizance is taken of the "contribution of the concrete" to the total shear resistance.

Transfer of moment from a flat slab to a support column is considered to influence the punching shear capacity of the slab.

The use of a prescribed quantity of horizontal link shear reinforcement is specified as a detailing requirement for corbels, but there is no specific means of quantifying the performance of this reinforcement within the scope of the code.

2.4.3 CP110

The structural element is generally assessed in terms of ultimate shear stress[5]. The resistant shear stress is dependent on four parameters, namely grade of concrete, steel ratio of the flexural reinforcement, the depth of the section under consideration and the shear arm (or a/d) ratio. The section where the shear is to be evaluated is not stated specifically for beams, but for punching shear in flat slabs the perimeter is considered at a distance of 1,5 times the slab thickness from the column face, using a rounded perimeter shape.

In the event that shear reinforcement is required, a simple truss analogy approach is used, with the contribution of the concrete being taken in to account.

In BS0000, the draft code which is intended to succeed CP110, the same parameters are recognised as influencing shear resistance. The variation of resistant shear stress with depth has been increased, however, and some other amendments have been made such as the use of a square perimeter for punching shear and a check on shear stress levels at the column face in addition to the evaluation at the perimeter at 1,5 times the slab thickness from the column face.

In both codes, transfer of moment from a flat slab to a support column is considered to reduce the punching shear capacity of the slab.

In the evaluation of the punching shear capacity of prestressed slab elements, the design handbook Report 25[11] recognises the same parameters as influencing the shear resistance, but takes prestress into account by considering it to be an adjusted contribution to the flexural steel ratio.

The use of a fixed quantity of horizontal links is specified as shear reinforcement for nibs and corbels, but there is no stated method of quantifying the influence of this reinforcement on the shear performance of the element within either code.

2.4.4 CEB FIP 78

The reinforced concrete structural element is again assessed in terms of ultimate shear stress in this code[20]. The resistant shear stress is dependent on four parameters; the grade of concrete, steel ratio of the flexural reinforcement, the depth of the section under consideration and the a/d ratio.

The shear evaluation is generally considered at an effective depth from the face of the support in the case of beams and at half an effective depth from the face of the support in the case of the evaluation of the perimeter for punching shear calculations, using an enhanced resistant shear stress.

In the event that shear reinforcement is required, a simple truss analogy is generally used, with cognizance being taken of the "contribution of the concrete". The option is allowed, however, of making use of a more sophisticated truss analogy model derived primarily from the work of Thürlimann. This method is referred to as the "accurate method" and the additional parameter of the slope of the diagonal shear crack, within specified limits, is introduced.

The presence of prestress is taken in to account generally by adjusting the resistant shear stress by an appropriate factor derived from an evaluation of the prestress present.

There is no requirement for the use of horizontal links in consoles or corbels with small a/d ratios, although there is an empirical assessment of the (similar) reinforcement requirements for web-flange connections.

2.5 APPARENT REASONS FOR CONFUSION

It is evident from a study of state of the art codes and current research and developments in the field of the evaluation of the shear performance of reinforced concrete structural elements that there are a number of reasons for the confusion that exists amongst practitioners today. The roots of the confusion can probably be traced historically to the early concept that shear performance of reinforced concrete depended on the parameter of shear stress, which in turn was dependent only on concrete grade. It is now realised that resistant shear stress is in fact dependent on a number of parameters. The problem is aggravated by the the manner in which these parameters affect the shear performance of a specific structural element in that the shear capacity may be either highly sensitive or virtually totally insensitive to a specific parameter depending on the state of other parameters and whether the element is reinforced or unreinforced for shear. This unusual manner in which the shear performance varies with these parameters has led to certain codes choosing to ignore the effect of certain parameters. The fact that different codes choose to ignore different parameters has certainly not alleviated the confusion amongst practitioners. The problem thus appears to be attributable in part to attempts at generalisation. It appears that none of the current models for shear are applicable throughout the full spectrum of parameters or specimen types normally encountered in practice. Although linked historically as indicated above, numerous situations exist currently that are potential areas of confusion regarding the interpretation of shear behaviour. Current models can also tend to exacerbate this situation as they generally appear to concentrate on specific aspects or regimes of the evaluation of overall shear performance. For example, models for reinforced concrete structural elements reinforced for shear based on the the truss analogy are not capable of predicting the behaviour of specimens unreinforced for shear or specimens very lightly reinforced for shear. It cannot be argued that a model for elements unreinforced for shear has no practical structural significance, since elements of this nature are used frequently in normal construction practice, notably slabs. Conversely, models for specimens unreinforced for shear, and the trends observed in the behaviour of these specimens, are not directly applicable through appropriate extension of the model to elements which are reinforced for shear.

It thus appears that there is a need for a model that will not only link existing models, but will also take cognizance of observed parametric trends which are not normally reflected in current models and concentrate on the transition area of elements lightly reinforced for shear.

For structural elements which are lightly reinforced for shear, the ultimate limit state of shear failure specifically is still expected to be abrupt and non-ductile, with the necessary ductility being provided by the attainment of the flexural ultimate limit state. It is thus evident that two distinct philosophies can be adopted with respect to reinforcing for shear. Either the element is reinforced for shear solely to achieve the flexural ultimate limit state or the element is reinforced to achieve specific shear ductility. The truss analogy models generally appear to model the latter more specifically and it thus remains essential that a model for shear should be able to quantify the reinforcement requirements for both situations.

The truss analogy models for beams reinforced for shear are generally derived on the basis of vertical links at constant spacing along the span of the beam. If the links were placed horizontally in the beam, as occurs in reinforcing for shear in short cantilevers, consoles or corbels, or where large point loads are applied in the vicinity of a support, the truss analogy models would generally not quantify the shear reinforcement requirements. It thus appears that the model should also be capable of evaluating and quantifying the shear reinforcement requirements of such elements. As in the case of concrete beams very lightly reinforced for shear with vertical links, it is anticipated that the ultimate limit state in shear of specimens reinforced with horizontal links will again be of a brittle nature, and the model should th[illegible] quantify and qualify the shear reinforcement requirements in this situation such that the flexural ultimate limit state can be reached prior to shear failure.

It is traditionally accepted that the contribution of dowel action to the total ultimate shear resistance of normal beams unreinforced for shear is of the order of 15% to 20% and somewhat smaller for beams reinforced for shear. It is thus generally reasonable to neglect dowel action in the evaluation of shear resistance for beams reinforced for shear, as is done in the truss analogy models. The dowel action contribution can, however, become extremely significant in certain structural applications, such as precast connections or normal beams unreinforced for shear with relatively high flexural reinforcement ratios. It is thus evident that in general, dowel action should be considered a variable contribution, which should be adequately represented in a general model for shear. If the contribution of dowel action is a variable quantity, it is likely that aggregate interlock and compression zone contributions are also variables and should also be reflected as such in the formulation of a universal model for shear.

It was thus with the observations of areas of concern amongst practitioners as mentioned previously and the acceptance of the undeniable influence of a wide variety of parameters that the test programme was conceived. In addition to tests on normally proportioned beams, generally subjected to two point loading, tests have been undertaken on specimens which are considered to be at the fringes and limitations of the models evaluated above. Tests have also been carried out on specimens both unreinforced for shear and reinforced for shear. Short cantilevers and corbels, both precast and cast-in-situ, and of varying geometry, have been considered. For structural elements of this nature, the diagonal shear crack is usually very steeply inclined and the parameter of a/d ratio thus assumes considerable importance. The precast corbels present an ideal specimen type for the evaluation of the contribution of dowel action to total ultimate shear resistance. In order to study the influence of scale and depth of section, beams and slabs varying from 50mm thick to 1200mm deep have been tested in shear, the specimens considered comprising both those reinforced and unreinforced for shear. A wide variety of slab specimens have been tested in punching shear and the results correlated with those of beam tests such that the relevance of the proposed model to punching shear can be determined. Beams have been tested which have had the local bond eliminated as far as is practically possible in order to determine whether the local bond evaluation is a significant parameter in the formulation of a model for ultimate shear resistance.

In all, over a hundred and fifty specimens have been evaluated in the test programme which follows, representing almost forty tons of reinforced and prestressed concrete.

Each test programme is evaluated in order to both verify and identify the parameters which appear to affect shear performance, to extend existing m s and to develop a new model appropriate to the research result

3 THE SHEAR PERFORMANCE OF CORBELS

The corbel (console, nib or short cantilever) has numerous structural applications, but is generally intended to transfer principal vertical load into a major structural element such as a column or wall. For practical reasons, the principal vertical load usually has its line of action somewhat displaced from the centre-line of the column or wall and this necessitates the use of a corbel or nib.

In slip-formed reinforced concrete construction, for example, it is frequently necessary to transmit vertical load from floors, diaphragms or protective brickwork into the slid wall. This also frequently applies to the requirements for crane girder supports on columns. The transfer of load is generally achieved through the application of the load to a corbel or nib which in turn is attached to the wall. The manner in which the corbel transmits this load to the wall is a shear-related issue which demands significant attention. Two fundamentally different approaches can be adopted by the designer in this regard in certain situations.

A conceptual decision regarding the use of either cast-in-situ or precast, bolt-on corbels can be made by the designer. For the former, the corbel can be cast monolithically with the supporting structural element in some instances, or can be cast into a recess in the supporting element, together with the appropriate returned reinforcement. For the bolt-on precast corbel the unit can be bolted to the smooth, off-shutter face of the wall by means of drilling into the wall and making use of self-locking type anchor bolts. This technique has some construction merits as it eliminates the need for the recess in the slid wall, which is both difficult to construct and has the potential of weakening the supporting wall.

Tests on cast-in-situ and precast corbels of virtually identical geometry, both being of a haunched profile, have been carried out in both the field and in the laboratory to assess the parameters affecting their shear performance. The precast specimens considered were actually designed initially for the specific purpose of supporting chimney-flue brickwork, and the physical size was thus determined by these requirements. Because of knowledge of the influence of scale[4], the in-situ corbels were constructed in the laboratory with virtually identical geometric parameters.

The possibility existed that the haunched profile of these corbels would in some way influence their shear performance and 6 rectangular, prismatic corbels of similar size to the haunched specimens were therefore tested concurrently for control purposes.

The corbels tested in this programme can be considered to represent practical examples of a relatively extreme zone of the a/d ratio spectrum, generally having small a/d ratios. They are also unusual relative to the majority of specimens tested in laboratories in that they have a single support at which the maximum moment and maximum shear force occur simultaneously.

3.1 GENERAL OBJECTIVES OF THE TEST PROGRAMME

The primary function of all the tests undertaken in this work was ultimately a parametric investigation of general shear performance and this was also the case for the corbels considered. The objectives of the corbel tests was thus to identify the following:

To compare the relative performance of precast and in-situ corbels in general terms.

To assess the influence of steel ratio and grade of concrete on the shear performance of corbels unreinforced for shear, taking cognizance of the low a/d ratios associated with this particular type of structural element.

To evaluate the influence of geometry of the corbel. The "traditional" haunched corbels and corbels rectangular in elevation were thus tested on a comparative basis.

To establish and evaluate the performance of shear reinforcement for both in-situ and precast corbels. The reinforcement which can be considered to be "shear reinforcement differed substantially for the precast and in-situ corbels.

3.2 DESCRIPTION OF THE TESTS

Special test rigs were designed and fabricated in order to carry out the required tests on both the in-situ and precast corbel specimens. The field test equipment for the precast elements is indicated in the photograph of Figure 3.1 and the rig used for the laboratory corbel tests is shown in Figure 3.2. Load was generally applied through the use of hydraulic jacks which reacted against the test rig as is evident in these figures. Load was recorded through the use of a variety of load cells manufactured in the laboratory. Vertical and horizontal deflection were generally measured at the front face of the corbels and on certain of the specimens, strain and diagonal shear crack width were monitored on the lateral face of the element by using demountable mechanical (demec) strain gauge targets and the appropriate demec gauges. The rather unusual support conditions peculiar to corbels necessitated the use of the specialist rigs described above.