Because the shape of the diagonal shear crack is not influenced by the shear reinforcement, it will adopt a similar shape to that described for the case of vertical links, and the influence of the horizontal links on the component of aggregate interlock, by inhibiting the opening of the diagonal shear crack, will be in a manner essentially identical to that for vertical links.

The manner in which the aggregate interlock stress component, v_a, of total shear resistance is defined in terms of the horizontal links is again entirely consistent with the approach used for structural elements unreinforced for shear. For the case of horizontal link shear reinforcement, the hypothetical prismatic concrete beam is subjected to the restraining influence of the links in addition to the opening distributed l-ad, q. From Figure 8.10, it is evident that the equivalent uniformly distributed load, r, resulting from the effects of the horizontal links, is given by:

$$r = \frac{F}{s_h} \sin^2\theta$$

where

F is the average force per link (or pair of legs). (N)

s_h is the spacing of the links up the depth of the structural element. (mm)

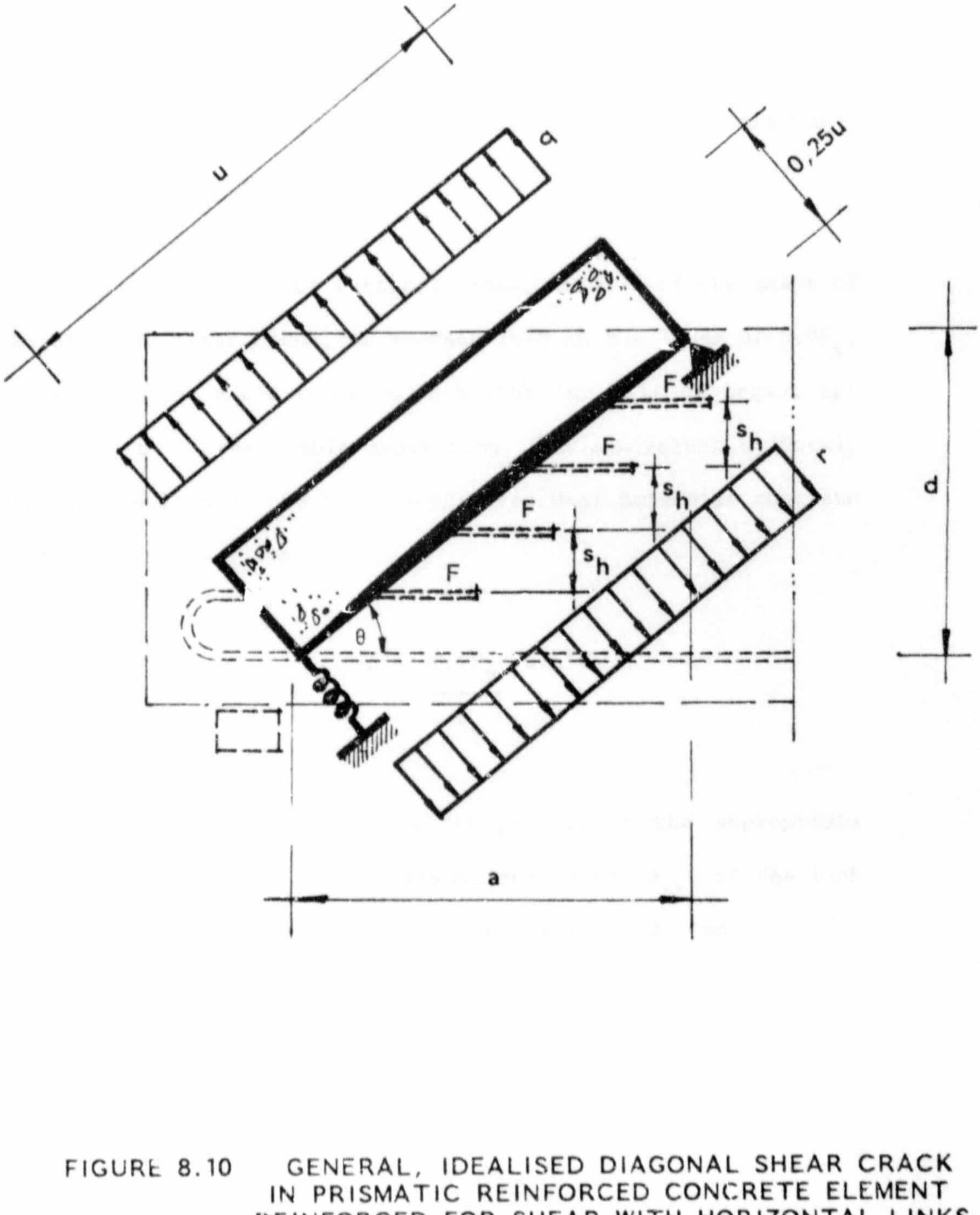

FIGURE 8.10 GENERAL, IDEALISED DIAGONAL SHEAR CRACK IN PRISMATIC REINFORCED CONCRETE ELEMENT REINFORCED FOR SHEAR WITH HORIZONTAL LINKS

θ is the slope of the diagonal shear crack to the axis of the element as defined previously. The derivation of the expression $\sin^2\theta$ is identical to that of the evaluation of reinforcement performance for yield line theory.

As for the evaluation of vertical links, because of the shape of the diagonal shear crack, an average load in the links of $0,9F_y$, where F_y is the yield force in each link (or pair of legs), appears to be a reasonable assumption. The equivalent uniformly distributed load on the hypothetical residual prismatic concrete beam can thus be given as:

$$r = \frac{0,9F_y}{s_h} \sin^2\theta$$

Substituting the yield force by the product of the appropriate yield stress, f_y, and the cross-sectional area, A_{sh}, of the link (usually 2 legs), gives the following distributed load:

$$r = 0,9f_y \frac{A_{sh}}{bs_h} b\sin^2\theta$$

thus

$$r = 0,9f_y \; \mu \; b\sin^2\theta$$

where μ is the (horizontal) link reinforcement ratio.

From the previously derived model for the aggregate interlock component of shear stress, and using the previously defined equivalent uniformly distributed load resulting from the residual tensions of the aggregate interlock component, the enhanced aggregate interlock component can again be derived. Reconstituting this expression exactly as before, under the influence of the revised equivalent uniformly distributed load, q-r, results in the relationship identical to that derived previously:

$$v_{a'} = \frac{0{,}65}{10d/E_c + 0{,}05/\rho} \cdot \frac{\sin^2\theta}{\cos\theta} + \frac{r\tan\theta}{b}$$

Making the appropriate substitution for the uniformly distributed load due to the links, r, results in the following relationship for the contribution of aggregate interlock stress to total shear resistance:

$$v_{a'} = \frac{0{,}65}{10d/E_c + 0{,}05/\rho} \cdot \frac{\sin^2\theta}{\cos\theta} + 0{,}9f_y\mu \frac{\sin^3\theta}{\cos\theta}$$

where

$v_{a'}$ is the enhanced aggregate interlock shear stress component of total shear resistance in MPa.

E_c is the Elastic Modulus of the concrete of the structural element in MPa.

ρ is the flexural steel ratio of the structural element expressed as a percentage.

ω is the slope of the idealised diagonal shear crack to the axis of the element.

f_y is the mean yield stress of the horizontal link shear reinforcement in MPa.

μ is the (horizontal) link reinforcement ratio expressed as a ratio.

This equation defines the contribution of the aggregate interlock component of resistant shear stress for structural concrete elements which are reinforced for shear with horizontal links. The effect of the horizontal links in this regard is thus to inhibit opening of the diagonal shear crack in terms of the proposed model, thereby enhancing aggregate interlock along this crack. The overall shear resistance of the structural element is still composed of the contributions of the four components; dowel action, compression zone, aggregate interlock and the sum of the dowel action contributions of the horizontal links crossing the diagonal shear crack. The value of aggregate interlock in this case is again enhanced relative to elements unreinforced for shear. Upon examination of the form of the enhanced aggregate interlock shear stress component equation, it is evident that the enhancement portion of this equation is related to the expression $\sin^3\theta/\cos\theta$, and thus the enhancement is particularly significant as θ becomes steep. The enhanced value thus becomes significantly larger than that for vertical link reinforcement as the a/d ratio reduces significantly below unity.

The proposed model for shear for elements reinforced for shear with horizontal links is thus described fully by the following equations:

The peak ultimate shear load accepted by the structural element prior to loss of aggregate interlock at a maximum idealised crack width of 1,3mm is given by:

$$V_{peak} = V_{a'} + V_d + V_c \qquad (= V_R)$$

where

$V_{a'}$ is the enhanced aggregate interlock component. (N)

V_d is the dowel action component of the horizontal links and the flexural reinforcement crossing the diagonal shear crack, prior to loss of aggregate interlock. (N)

V_c is the unchanged contribution of compression zone derived previously. (N)

After the peak shear value has been attained, the component of aggregate interlock is abruptly lost as before (together with the major proportion of the compression zone component), and the ultimate shear resistance of the structural element is then given by the following:

$$V_{duct} = V_d$$

where

V_{duct} is the small residual ductile plateau of shear resistance resulting from the contributions of dowel action of all the reinforcement which crosses the diagonal shear crack. (N)

Because the residual ductile plateau does not have a significantly larger value than that for structural elements unreinforced for shear, the shear failure mode in this case cannot be considered to have ductility. For structural elements reinforced for shear with horizontal links, overall structural ductility must therefore, of necessity, be obtained by the attainment of flexural ductility. The function of the horizontal links is thus specifically to raise the shear resistance to a value greater than the flexural capacity of the section. This is achieved by the enhancement of the aggregate interlock component of total shear resistance. The model thus not only predicts the qualitative behaviour of structural elements reinforced for shear with horizontal links, but can also be used to quantify the anticipated enhancement of the shear capacity of such elements.

The model stress-deflection curves for the standard beam are shown in Figure 8.11. The stress values are derived from an evaluation of the standard beam with varying a/d ratio and varying horizontal link reinforcement ratio, using mild steel links of average yield stress 280MPa. The equation for the enhancement of the aggregate interlock component indicates that horizontal links are only efficient for a/d ratios less than unity. The good correlation between these predicted curves and the curves observed in the test programme, shown in Figure 6.3, is evident. This applies particularly to a qualitative comparison of the curve shapes. (The low level of the ductile "dowel action" plateau will not necessarily be maintained indefinitely, since increased load will be carried as the dowels deform in double curvature and accept increased axial tension.) The quantitative comparison is further demonstrated in the tabulated values of Appendix D. The model thus predicts that structural elements which normally have small a/d ratios, such as corbels, are more effectively reinforced with horizontal links than with vertical links.

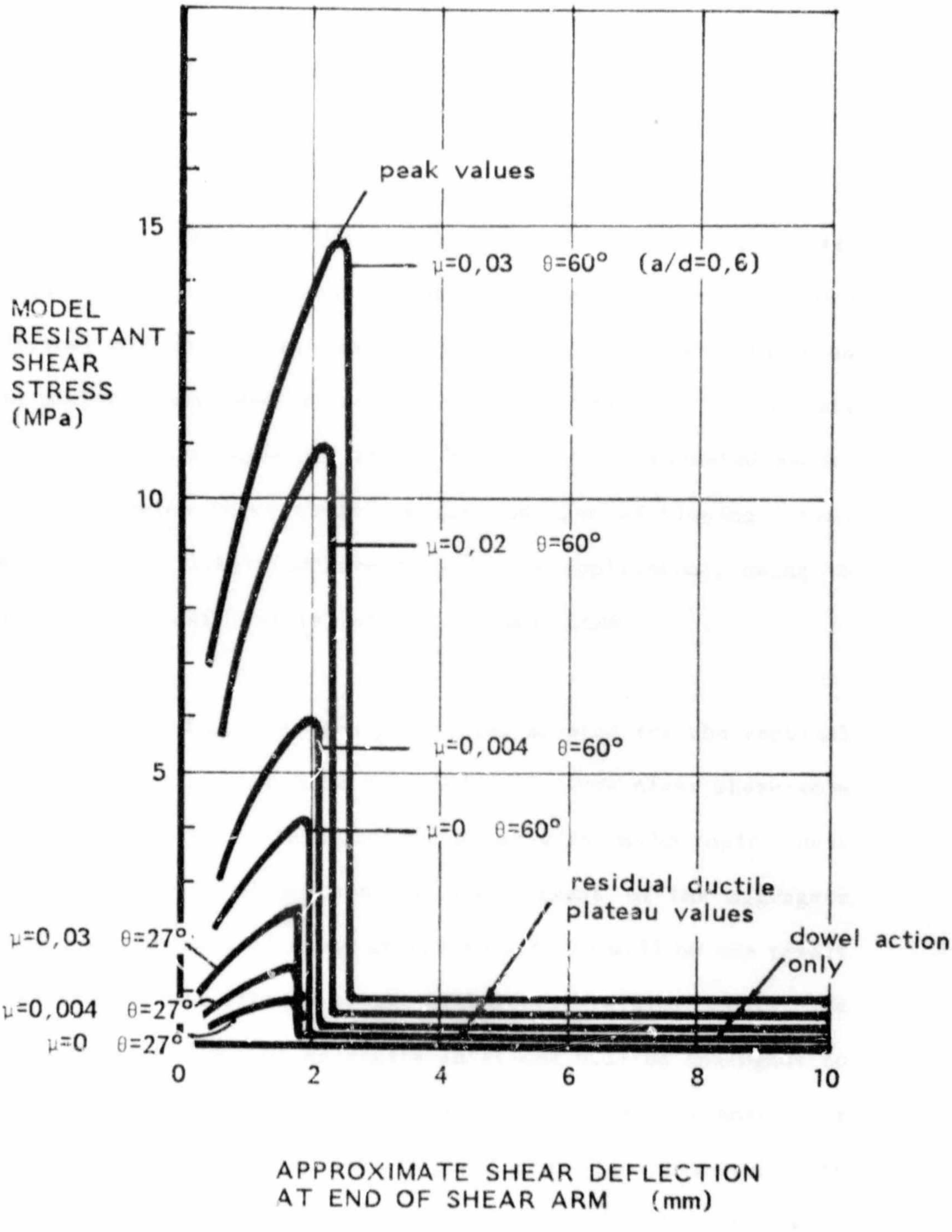

FIGURE 8.11 MODEL STRESS-DEFLECTION CURVES FOR SHEAR FAILURE MODES FOR THE "STANDARD BEAM" REINFORCED FOR SHEAR WITH MILD STEEL HORIZONTAL LINKS OF VARYING PERCENTAGE

8.2.3 LINK REINFORCEMENT IN GENERAL

It is evident from the derivations of the enhancement of the aggregate interlock component of shear stress resistance for both vertical and horizontal link reinforcement that the principles can easily be extended to cater for links at any angle to the axis of the structural element. It is, however, not considered necessary to develop these equations for the case of sloping links, as this is not likely to have much direct application, owing to the practical difficulties of fixing such links.

It is also evident that the principles adopted for the vertical and horizontal links can be extended to include cases where webs or shear walls are reinforced for shear with orthotropic shear reinforcement. In this case the enhancement of the aggregate interlock component of shear stress resistance will be the result of the sum of the effects of the orthotropic reinforcements. The behaviour after loss of aggregate interlock will be analogous to the formulation for performance of flexural reinforcement for yield line theory. This will have particular significance in very deep sections such as shear walls where the peak shear value will be similar to the residual ductile value, because the concrete contribution to the enhanced aggregate interlock component is small, owing to the large depths involved. Specific shear ductility is also a design prerequisite for shear walls in general.

8.3 MODEL FOR PRESTRESSED ELEMENTS

As for normal reinforced concrete construction, the evaluation of prestressed concrete elements should be considered for elements both unreinforced and reinforced for shear.

For prestressed concrete structural elements unreinforced for shear, the model is developed as before, but with an additional component resulting from the effects of the prestress. For reinforced concrete elements, only cases of reinforcing with either vertical or horizontal links were considered in detail, because these had the most practical significance, even though the model is easily extended to any form of shear reinforcement. For similar reasons, only axial prestress is considered here, even though the model is applicable generally.

Axial prestress in the structural element can be considered to be a uniformly distributed load on the diagonal shear crack in a manner analogous to that used for the evaluation of horizontal link reinforcement. The implication of such an approach is that the shear crack must have developed before the model becomes applicable. This requirement is entirely consistent with the conditions of applicability of the model to ordinary reinforced concrete elements, where under certain unusual circumstances, shear failure occurs simultaneously with the first diagonal crack. While the presence of axial compressive prestress will influence the directions of the principal tensile stresses in a homogeneous section prior to first diagonal cracking, once the diagonal shear crack has formed, it will develop further to assume the idealised shape shown in Figure 8.1 at the shear ultimate limit state independently of the prestress present. The shape of the idealised diagonal shear crack at the ultimate limit state is thus relatively independent of the reinforcement present or the prestress present. This qualitative assessment of the diagonal shear c[illegible]rved consistently in tests. Thus for prestressed sp[illegible]ere shear resistance increases after the initial formation of the diagonal shear crack, the model can be used directly and the manner in which the prestress affects shear performance is again by the enhancement of the aggregate interlock component of total shear resistance. For prestressed elements which have the tendons or secondary reinforcement positioned consistent with the location of the flexural reinforcement in the models for shear formulated thus far, the diagonal shear crack can be evaluated exactly as before. For specimens which have the tendon placed near the centroid of the sections at the point of shear assessment, a similar evaluation can be made, using a reduced effective depth. In both cases, the diagonal shear crack opens in a similiar qualitative manner to ordinary reinforced specimens, for elements which accept increased load after the initial formation of the diagonal shear crack.

It is thus evident that for the case of prestressed elements unreinforced for shear, the hypothetical equivalent prismatic beam of the mathematical model is subjected to the restraining influence of the prestress in addition to the opening load due to applied diagonal tensile stresses or crack opening forces. It is thus postulated that the manner in which the axial prestress influences the diagonal shear crack is analogous to that for the case of horizontal link shear reinforcement. The uniformly distributed restraining load resulting from the effects of the prestress is thus given by:

$$r = c \sin^2\theta$$

where

c is the average axial compressive prestress in the structural element in MPa

As for the case of horizontal links, the effective uniformly distributed load on the residual prismatic concrete beam is given by the difference between the applied load, q. and the restraining load, r. The component of aggregate interlock resistant shear stress can, as before, thus be derived to be:

$$v_{a'} = \frac{0,65}{10d/E_c + 0,05/\rho} \cdot \frac{\sin^2\theta}{\cos\theta} + c\,\frac{\sin^3\theta}{\cos\theta}$$

where

v_a' is the enhanced aggregate interlock shear stress component of total shear resistance in MPa. (The enhancement in this case is a result of the presence of axial prestress in the element)

E_c is the Elastic Modulus of the concrete of the structural element in MPa.

ρ is the true flexural steel ratio of either the prestressing tendon or the secondary flexural reinforcement crossing the diagonal shear crack remote from the compression zone. (%)

θ is the slope of the idealised diagonal shear crack to the axis of the element.

c is the mean axial compressive prestress in the structural element in MPa.

This equation defines the contribution of the aggregate interlock component of resistant shear stress for structural prestressed concrete elements which are subjected to an average compressive prestress of c MPa in the vicinity of the diagonal shear crack.

Examination of this relationship for the enhanced aggregate interlock component of shear stress resistance reveals that the increase in shear resistance capacity with reducing a/d ratio is far more rapid than for ordinary reinforced concrete elements. This trend is evident in the test results in Figure 4.15.

For prestressed concrete elements which are reinforced for shear, the aggregate interlock component of shear stress resistance is further enhanced by the presence of the shear link reinforcement as before. The overall equilibrium equation for the element would also be changed as for ordinary reinforced concrete elements.

8.4 CRITICAL APPRAISAL OF THE PROPOSED MODEL

1. The model as proposed in this chapter is in the most simplistic form such that the observed major parametric behaviour of resistant shear stress is reasonably well reflected. The manner in which the model is formulated, however, is such that a more detailed evaluation of any aspect of the conceptual model could easily be incorporated. For example, the behaviour of dowel action depends on a large number of parameters[12], and a more sophisticated assessment of its contribution to the model for specific cases might be warranted. The evaluation of the flexural reinforcement spring support in the aggregate interlock part of the model could also be made considerably more sophisticated, for the same reasons, but this has the danger of complicating the fundamental aggregate interlock parametric equation to such an extent that it becomes difficult to apply as a practical design aid.

Nevertheless, for elements unreinforced for shear, the four major parameters, grade of concrete, flexural steel ratio, depth of section and a/d ratio, are all well modelled if shear stress is the criterion used for ultimate shear capacity evaluation. The formulation of the influence of the flexural reinforcement indicates that local bond is unlikely to be a parameter of any significance with respect to the ultimate limit state of shear. Although the model does not define the mechanics of shear behaviour or predict resistant shear stress values for a/d ratios of 0 accurately, being asymptotic to this value, a reasonable estimate of the values predicted for these cases[44] is obtained if a cut-off at an a/d ratio of 0,4 is applied. The model could thus be used to estimate likely approximate resistant shear stress for cases such as checks at support faces or web-flange interfaces, in the absence of more sophisticated evaluations being readily available. The model also has some qualitative merit in this zone of shear behaviour as it predicts the effectiveness of "horizontal" link reinforcement for such cases and also predicts that the dowel action of the reinforcement will be a significantly reduced proportion of total shear resistance than for "normal" beams.

2. For structural elements reinforced for shear with vertical links, the observation from tests that the specific ultimate limit state of shear can still remain essentially non-ductile, especially for elements lightly reinforced for shear, is satisfactorily explained by the model. The form of the load-deflection curves observed in tests, with a peak resistance value followed by a residual ductile plateau, is well modelled. An evaluation of the importance of specific shear ductility in such elements is related to their design performance requirements.

3. The observation by many researchers, that the traditional Ritter and Mörsch 45° truss analogy models are conservative for elements lightly reinforced for shear with vertical links, is predicted by the model. The "contribution of the concrete" referred to in this regard is the maintenance of aggregate interlock due to the influence of the shear reinforcement.

4. The influence of horizontal link reinforcement, used in certain structural elements such as corbels, is well modelled. The effect of this form of shear reinforcement is also quantified by the model, and the observation in tests that this mode of shear failure remains essentially non-ductile is predicted by the model. The model is unique in this form of general application.

5. The model is capable of predicting the anticipated variation in mode of shear failure for structural elements unreinforced for shear. Observations of tests on beams of similar proportions to the standard beam indicate that the mode of shear failure for such a beam is likely to be such that the diagonal shear crack forms prior to the attainment of the ultimate limit state in shear. The tests undertaken in this programme indicate that this is likely to be of the order of 80% of the ultimate shear load for the standard beam. An accurate assessment of this value is complicated by the fact that shrinkage stresses are likely to influence the load at which diagonal cracking first occurs[32].

It is likely that the tensile stress at which cracking occurs is related approximately to the square root of the mean concrete cube strength[5]. Based on the fact that first cracking occurs at 80% of ultimate for the standard beam (ultimate for this beam being 1MPa), an approximate relationship for stress at first cracking can be assumed to be:

$$v_t = 0.16 \sqrt{f_{cu}}$$

where

v_t is the average shear stress at which first diagonal cracking occurs. (MPa)

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

This simplistic equation takes into account the influences of shear shape factor and shrinkage stresses for the homogeneous element. Because the proposed model for shear for elements unreinforced for shear depends on four parameters, a simple evaluation of the parametric limitations is difficult to present. However, in practical terms, the parameter of flexural steel ratio is generally of the most interest in this regard. Calculating the ultimate mean resistant shear stress for the standard beam with varying flexural steel ratio, and equating it to the above expression, indicates that the proposed model will no longer be applicable for the standard beam with flexural steel ratios less than approximately 0,25%. For steel ratios lower than this the shear capacity of the section is likely to be determined simply by the tensile strength of the concrete, which in turn is related approximately to the square root of the concrete cube strength. This assessment would obviously vary for elements with parameters differing from those of the standard beam.

6. The proposed model for shear explains why small diameter bars at close centres perform better as shear reinforcement than large bars at large centres in many situations. Because the model is derived from an evaluation of the diagonal shear crack as a deflection-related phenomenon, the shear reinforcement enhances the aggregate interlock component by inhibiting the opening of the crack. If the shear reinforcement is spread such that the centres between links approaches the effective depth dimension of the element, it is evident in the model formulation that there is a high probability that only two links would intersect the diagonal crack, and these could be near the supports of the proposed hypothetical prismatic beam, where they would be inefficient in inhibiting the opening of the crack. The enhancement of the aggregate interlock component would thus be considerably impaired, resulting in a reduction of the peak shear stress resistance value. This argument is equally valid in the evaluation of the performance of bent-up bars as shear reinforcement, where they would be relatively inefficient unless they intersected the diagonal shear crack near the midspan. The model thus aids in the assessment that the maximum link spacing should be less than

half the effective depth of the section, for it to be consistently effective. The model also predicts that horizontal links should replace vertical links as the a/d ratio reduces below unity.

7. The model predicts that for very deep sections, such as shear walls, the contribution of the concrete will generally be small at the ultimate limit state. This phenomenon is caused by the prediction of the model that the diagonal shear crack will open to a hypothetical crack width of 1,3mm under a very low stress, for unconstrained diagonal shear cracks, and the shear resistance is thus almost entirely a function of the shear reinforcement present.

Conversely, for very thin structural elements, the greatly enhanced resistant shear stress for elements unreinforced for shear, predicted by the model, results in reduced relative benefit from shear reinforcement. This effect can be aggravated according to the phenomena expounded in 6 above, as a result of the physically small dimensions of the diagonal shear crack for thin sections relative to the practical link size and spacing in normal construction.

8. From an evaluation of the parametric equati[illegible]or the aggregate interlock component of shear stress resis[illegible]ce, it is evident that the average absolute slope of the diagonal shear crack, θ, has a fundamental influence. In order to have a consistent

approach for elements which have unusually large covers, the shear arm for all cases for the proposed model should be defined as the dimension along the element axis of the diagonal shear crack, between the compression zone and the point of intersection with the flexural reinforcement. The a/d ratio used in the parametric equation of the proposed model (in the form of θ), has limits between 0,4 and 2,0, regardless of the traditional assessment of the true a/d ratio of the structural element. The concept of the effective depth is used for the model average stress evaluation, where v = V/bd, since this is fundamental to the derivation of the span of the equivalent residual concrete beam in the formulation of the model for shear, and this influence is also confirmed by test results in this work on specimens of variable cover, and thus variable effective depth.

9. It is evident in the formulation of the universal model in this thesis that the contribution of the flexural reinforcement to the aggregate interlock component of shear stress resistance is based on the assumption that the flexural reinforcement is unyielded in axial tension at the ultimate limit state of shear. This is generally a legitimate assumption, as the onset of yield in the flexural reinforcement implies that the ultimate limit state of flexure has been attained, which is the desired mode of failure, usually with adequate ductility. In general, for hinges which form near the midspan of structural elements subjected to largely distributed loads, the hinge can rotate virtually unim-

peded by shear considerations, since the contributions of compression zone and dowel action alone are sufficient to withstand the relatively low shear forces in these zones. The hinge in this case will have no limits of ductility imposed as a result of the presence of shear forces. For certain structural situations, in particular hogging regions in flat plate structures where the hinge forms at the face of the support column, the presence of large shear forces may impair ductility of this hinge, if significant strain hardening occurs after the attainment of the flexural ultimate limit state. The detailed investigation of such an eventuality is beyond the specific scope of the proposed model. The evaluation of rotation limits of plastic hinges in reinforced concrete construction is perceived as being an avenue of further research.

10. The model can be applied directly to cases of punching shear in slabs by considering the stress evaluation at 0,85 of the perimeter where the diagonal shear crack intersects the flexural reinforcement remote from the compression zone of the shear crack. The stress evaluation can be applied virtually directly because the support to the residual equivalent beam from the flexural reinforcement is obviously unchanged at the perimeter, and the compression zone is likely to be enhanced by biaxial compressions. For rectangular column heads, the loss of biaxial enhancement coincides with an equivalent trend to the normal model for beam elements, and the model is thus also applicable

to punching in general, if the appropriate perimeter is used. A more detailed evaluation of the hypothetical beam in the model, as applied to the portion of the punching cone with its more complex shape and hypothetical supports, is perceived as an area of further research which could explain, and possibly replace, the affine surface at 0,85 of the shear arm, currently used in the model.

11. The proposed model for elements unreinforced for shear can be critically appraised by direct comparison with Kani's valley of diagonal failure. Kani's valley, in three dimensional format, gives an excellent assessment of the relative shear capacity of the beams tested, and is easily interpreted because it is presented in a clear, visual form. The equation developed in this work for the aggregate interlock component of shear stress resistance, being dependent on four major parameters, and not two as in Kani's valley, can unfortunately not be depicted in the same three dimensional graphical plot. The relative failure load, rather than relative moment as in Kani's valley, has been plotted in a qualitative sense against a/d ratio in Figure 8.12, for a reinforced concrete beam with two-point loading. The influences of the various parameters are indicated on the curves, and the resultant danger zone where flexural capacity generally exceeds shear capacity, equivalent to Kani's valley, is clearly depicted in the diagram.

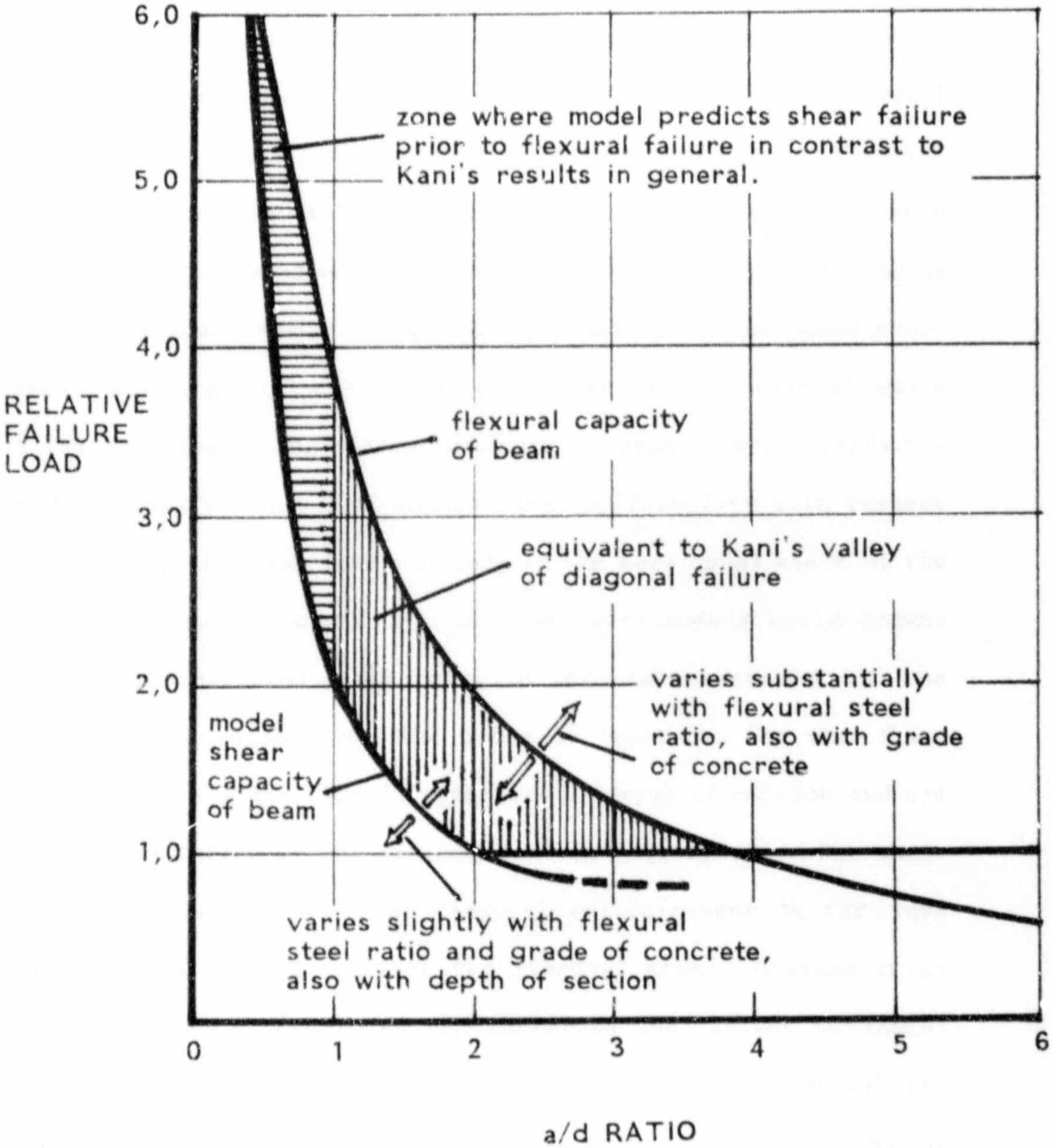

FIGURE 8.12 REPRESENTATION OF KANI'S VALLEY OF DIAGONAL FAILURE FOR TWO POINT LOADING ON BEAMS UNREINFORCED FOR SHEAR ACCORDING TO THE PROPOSED MODEL.

12. The model for shear developed within the scope of this work can be compared directly with the rectangular stress block model and associated couple for the assessment of the ultimate flexural capacity of a standard structural element. For the flexural model, tensile flexural capacity of the concrete must be exceeded before this flexural model becomes applicable. This is analogous to the diagonal shear crack forming prior to the proposed shear model becoming applicable. In both cases, if the load at which first cracking in the appropriate tensile zone occurs exceeds the model capacity, then both models lose applicability with respect to defining the peak load reached. In the more usual event of the model resistance exceeding this load, both models would become applicable, performing according to the characteristics of the reinforcement present and the grade of concrete. For the shear model, however, additional parameters of depth of section and a/d ratio also influence the performance. In addition, for the shear model, the influence of the flexural reinforcement is far less marked than in the case of the flexural model, because it is relatively badly positioned and therefore inefficient in inhibiting opening of the diagonal shear crack. This flexural reinforcement is obviously well positioned to inhibit opening of the flexural crack.

Any shear reinforcement present assists the flexural reinforcement in inhibiting opening of the diagonal shear crack, and if adequate knowledge of the form of this shear crack exists, the shear reinforcement can be positioned such that it is very efficient in inhibiting the opening of this diagonal shear crack.

Both models refer essentially to a single crack in the relevant stressed zone in determining the specific ultimate maximum resistance value of the structural element. The concept of bending or shear stress in the homogeneous, uncracked structural element has no relevance in either model, once the models become applicable. The concept of average stress as an aid to the determination of the ultimate resistance of the structural section is retained for both models, the criteria for flexure based on a concrete compressive strain limitation and moment equilibrium and the criteria for shear based on a crack width limitation and vertical force equilibrium.

Both models have upper limits to the quantity of reinforcement which can be included prior to the crushing strength of the concrete being attained in the appropriate compression zone of the system.

The models can be considered to be independent of one another, reflecting the absence of moment-shear interaction generally in the design approach.

9 CONCLUSION

A wide variety of structural types has been investigated and evaluated, leading to a better qualitative and quantitative assessment of the general phenomenon of shear failure in reinforced and prestressed concrete structural elements.

A model for shear stress resistance has been developed based on the test results and a rational approach. This model not only takes cognizance of the major parameters influencing shear performance, but also takes into account transition zones and peripheral areas of shear behaviour, in particular the transition between structural elements unreinforced for shear and those reinforced for shear. The formulation of the parametric equations of the model indicates clearly where and when specific aspects of shear resistance will become negligible relative to others over the broad spectrum of structural element types likely to be encountered in practice. It is believed that the model is unique in this regard. The model thus has virtually universal application and as such is a meaningful contribution towards the better understanding and evaluation of shear behaviour in general reinforced concrete construction.

The model can be used in a code or design application virtually unchanged. Generally all the parameters of the structural element will be easily selected. In certain structural applications the model can be used in a parametric sense to minimise the shear collapse load function.

Although presented in the most simple form which still reflects observed parametric shear behaviour adequately for most structural elements, the conceptual formulation of the model is such that a more detailed evaluation of any aspect of shear performance is easily incorporated in an entirely consistent approach.

APPENDIX A

TEST RESULTS PERTINENT TO CHAPTER 3

A summary of the test results for Chapter 3 is included in this Appendix. Examination of this test data indicates the existence of certain parametric trends, which have been summarised and compared with code and other research results in Chapter 3.

The identification and evaluation of these parametric trends is complicated by the response of the structural specimen to four major parameters for elements unreinforced for shear and five if shear reinforcement performance is included in a suitably simplistic manner. The difficulty in controlling certain parameters, even in laboratory conditions, aggravates this procedure. Scatter in test results is partly attributable to unintentional variations in parameters not being investigated, which are either difficult to control, or are not initially perceived by the researcher as having a significant influence on the performance of the specimens under consideration. The influences of effective depth and a/d ratio can be particularly significant in this regard.

For this reason, the test results have been recorded against the major parameters which have been perceived within the scope of this research as having the major influence on the shear performance of the reinforced concrete structural elements.

A new, universal model for the evaluation of resistant shear stress is proposed in Chapter 8. The predicted resistant shear stress for each structural element according to this model is presented adjacent to the test result for direct comparison.

The notation used in this table is consistent with the remainder of this thesis. In particular, v_R is the mean measured ultimate resistant shear stress of the specimen in MPa and v_{Rm} the model value of the resistant shear stress, also in MPa, for the same specimen. As indicated in the list of symbols, μ is the link reinforcement ratio, which may refer to either vertical,(v) or horizontal,(h) links. A link ratio of zero implies that the structural element is unreinforced for shear.

Unless otherwise noted in the tables, all results refer to stresses measured at an ultimate limit state for which the mode was classical diagonal tension type shear failure.

TABLE A1 SCHEDULE OF MEASURED AND MODEL MEAN PEAK RESISTANT SHEAR STRESS FOR SPECIMENS IN CHAPTER 3

Specimen Type	d (mm)	a/d	f_{cu} (MPa)	ρ (%)	μ	v_R (MPa)	v_{Rm} (MPa)
Haunched In-situ corbel	270	0,6	20	0,4	0	3,0	3,7
"	270	0,6	20	1,3	0	3,7	5,0
"	270	0,6	40	0,4	0	4,2	4,6
"	270	0,6	40	1,3	0	5,4	7,3
"	270	0,6	20	0,4	0	3,1	3,7
"	270	0,6	40	1,3	0	"front" @5,6	7,3
"	340	0,4	20	0,6	0	5,8	6,3
"	340	0,4	20	0,4	0	5,2	5,5
"	340	0,6	20	0,4	0	3,1	3,5
"	340	0,6	20	1,1	0	4,1	4,7
"	340	0,4	40	0,6	0	6,5	7,8
"	340	0,4	40	0,5	0	5,8	7,3
"	270	0,6	20	1,3	0,007h	"front" @5,5	6,6
"	270	0,6	20	0,4	0,002h	"flex yld" @3,6	4,6
"	270	0,6	40	1,3	0,007h	"front" @5,8	9,6
"	270	0,6	40	0,4	0,002h	"flex yld" @3,6	4,0

TABLE A2 SCHEDULE OF MEASURED AND MODEL MEAN PEAK RESISTANT SHEAR STRESS FOR SPECIMENS IN CHAPTER 3							
Specimen Type	d (mm)	a/d	f_{cu} (MPa)	ρ (%)	μ	v_R (MPa)	v_{Rm} (MPa)
Haunched In-situ corbel	340	0,4	20	0,6	0,003h	"flex yld" @5,1	9,1
"	340	0,4	20	0,4	0,002h	"flex yld" @3,5	7,2
"	340	0,6	40	0,6	0,002h	"front" @5,2	5,5
"	340	0,6	40	1,1	0,006h	"front" @5,4	9,2
Prismatic Corbel	220	1,0		0,5	0	"flex yld" @2,7	2,8
"	220	1,0	30	0,5	0	2,7	2,8
"	220	1,0	30	1,0	0	3,6	3,7
"	220	1,0	30	2,0	0	4,3	4,6
"	220	1,0	30	3,0	0	4,7	5,0
R C Beam (comparative)	220	1,0	30	0,5	0	2,7	2,8
"	220	1,0	30	1,0	0	3,8	3,8
"	220	1,0	30	2,0	0	4,3	4,7
"	220	1,0	30	3,0	0	4,7	5,1

TABLE A3 SCHEDULE OF MEASURED AND MODEL MEAN PEAK RESISTANT SHEAR STRESS FOR SPECIMENS IN CHAPTER 3

Specimen Type	d (mm)	a/d	f_{cu} (MPa)	ρ (%)	μ	v_R (MPa)	v_{Rm} (MPa)
Deep Prismatic corbel	470	0,2	20	0,2	0,004h	"flex yld" @3,7	5,7
"	470	0,2	20	0,5	0,003h	6,0	6,5
"	470	0,2	20	1,0	0	4,8	5,6
"	470	0,2	20	1,0	0	5,2	5,6
"	470	0,2	20	0,5	0	4,7	4,9
"	470	0,2	20	0,5	0	4,3	4,9
Precast Corbel*	270	0,4	70	1,0	0	1,5	-
"	270	0,4	40	1,0	0	1,2	-
"	270	0,4	20	1,0	0	0,8	(0,4)
"	270	0,4	40	1,0	0,01v +0,008h	"front" @1,7	(7,8)

*Included here are the summarised results of 26 specimens in total. The coefficient of variation for ten tests of specimens with identical parameters was approximately 15%. Only concrete strength was varied, as indicated above, because this was the only potential variable in the prototypes. The results for the precast corbels refer to dowel cracking modes of failure. A more comprehensive evaluation of dowel action would entail a test programme where all of the potential parameters would have to be given consideration.

APPENDIX B

TEST RESULTS PERTINENT TO CHAPTER 4

A summary of the test results for Chapter 4 is included in this Appendix. Examination of this test data indicates the existence of distinct parametric trends, which have been summarised and compared with code and other research results in Chapter 4.

The test results are again recorded against the five parameters which have been perceived within the scope of this research as having the major influence on the shear performance of the reinforced concrete structural elements.

A new, universal model for the evaluation of resistant shear stress is proposed in Chapter 8. The predicted resistant shear stress for each structural element according to this model is presented adjacent to the test result for direct comparison.

The notation used in this table is consistent with the remainder of the thesis. As indicated in the list of symbols, μ is the link reinforcement ratio, which may refer to either vertical,(v) or horizontal,(h) links. A link ratio of zero implies that the structural element is unreinforced for shear

TABLE B1 SCHEDULE OF MEASURED AND MODEL MEAN PEAK RESISTANT SHEAR STRESS FOR SPECIMENS IN CHAPTER 4

Specimen Type	d (mm)	a/d	f_{cu} (MPa)	ρ (%)	μ	v_R (MPa)	v_{Rm} (MPa)
R C Slab (punching)	40	0,8	35	1,0	0	9,4	9,7
"	40	0,8	35	0,5	0	5,6	5,8
"	40	0,8	35	1,0	0	9,5	9,7
"	40	0,8	20	0,5	0	5,1	5,5
"	40	0,3	20	1,0	0	17,6	19,9
"	40	0,8	20	1,0	0	6,8	8,8
"	40	1,4	20	1,0	0	3,2	3,8
"	40	2,5	20	1,0	0	2,3	2,3
"	85	0,8	20	1,0	0	5,9	7,5
"	85	0,8	20	1,0	0	5,5	7,5
"	85	0,8	20	0,5	0	4,0	4,7
"	85	0,8	20	0,5	0	4,6	4,7
"	85	2,5	20	1,0	0	1,7	2,0
"	85	2,5	20	0,5	0	1,3	1,4
"	85	2,0	35	0,5	0	1,5	1,6
"	85	2,0	35	1,0	0	1,9	2,2
"	85	2,5	20	1,0	0,005h	1,8	2,1
"	85	0,3	20	0,5	0,003h	14,0	12,6
"	85	0,8	20	0,5	0,003h	5,6	5,4

TABLE B2 SCHEDULE OF MEASURED AND MODEL MEAN PEAK RESISTANT SHEAR STRESS FOR SPECIMENS IN CHAPTER 4

Specimen Type	d (mm)	a/d	f_{cu} (MPa)	ρ (%)	μ	v_R (MPa)	v_{Rm} (MPa)
R C Slab (punching)	85	1,6	40	0,5	0	1,7	2,0
"	85	2,0	20	0,2	0	0,6	0,7
"	85	2,0	20	0,2	0	0,7	0,7
"	85	1,0	40	1,0	0	5,3	6,1
"	85	1,5	40	1,0	0	3,0	3,2
"	135	0,8	20	0,5	0	4,0	4,1
"	135	0,8	35	0,5	0	5,1	4,7
"	135	0,8	35	1,0	0	6,3	6,8
"	135	0,8	20	1,0	0	4,5	5,8
"	135	1,4	20	0,5	0	2,2	2,0
"	135	1,4	20	1,0	0	2,5	2,7
"	135	0,8	20	0,5	0,003h	4,3	4,7
"	135	0,8	40	0,5	0,003h	5,2	5,3
"	135	0,8	40	1,0	0,005h	6,6	8,0
"	185	0,8	20	0,5	0	3,5	3,6
"	185	0,8	20	1,0	0	4,1	4,9
"	185	0,8	40	0,5	0	4,7	4,3

TABLE B3 SCHEDULE OF MEASURED AND MODEL MEAN PEAK RESISTANT SHEAR STRESS FOR SPECIMENS IN CHAPTER 4

Specimen Type	d (mm)	a/d	f_{cu} (MPa)	ρ (%)	μ	v_R (MPa)	v_{Rm} (MPa)
R C Slab (punching)	185	0,8	40	1,0	0	6,1	6,0
"	185	0.8	20	0,5	0,003h	3,8	4,2
"	185	0,8	20	1,0	0,005h	5,1	5,8
"	185	0,8	40	1,0	0,005h	7,2	7,0
"	185	0,8	40	0,5	0,003h	4,7	4,9
"	280	0,8	20	0,5	0	2,6	2,9
"	280	0,8	20	1,0	0	3,2	3,7
"	280	0,8	20	0,5	0	2,8	2,9
"	280	0,8	30	0,5	0	3,1	3,4
"	280	0,8	30	1,0	0	3,7	4,4
"	280	0,8	20	0,5	0,003h	3,2	3,5
"	280	0,8	20	0,5	0,003h	3,0	3,5
"	280	0,8	20	1,0	0,005h	3,7	4,7

TABLE B4 SCHEDULE OF MEASURED AND MODEL MEAN PEAK RESISTANT SHEAR STRESS FOR SPECIMENS IN CHAPTER 4

Specimen Type	d (mm)	a/d	f_{cu} (MPa)	ρ (%)	μ	v_R (MPa)	v_{Rm} (MPa)
R C Beam (2 pt load'g)	125	1,0	25	0,5	0	3,3	3,3
"	125	1,0	30	0,5	0	3,3	3,5
"	125	1,0	25	1,5	0	5,5	5,8
"	125	1,0	40	1,5	0	6,0	6,6
"	125	1,0	45	1,5	0	6,4	6,8
"	175	1,0	25	0,5	0	2,8	2,9
"	175	1,0	25	0,5	0	2,7	2,9
"	175	1,0	25	1,5	0	4,0	4,7
"	175	1,0	40	1,5	0	4,3	5,4
"	175	1,0	45	1,5	0	6,0	5,9
"	175	0,8	20	1,5	0	4,6	5,8
"	175	0,8	20	1,5	0	4,8	5,8
"	175	1,0	20	1,5	0	3,6	4,2
"	175	1,5	20	1,5	0	1,8	2,1
"	175	2,0	20	1,5	0	1,4	1,5
"	175	2,5	20	1,5	0	1,4	1,5
"	175	3,0	20	1,5	0	1,3	1,5

TABLE B5 SCHEDULE OF MEASURED AND MODEL MEAN PEAK RESISTANT SHEAR STRESS FOR SPECIMENS IN CHAPTER 4

Specimen Type	d (mm)	a/d	f_{cu} (MPa)	ρ (%)	μ	v_R (MPa)	v_{Rm} (MPa)
R C Beam (2 pt load'g)	270	0,9	25	0,5	0	2,9	2,8
"	270	0,9	40	0,5	0	3,4	3,2
"	270	1,0	45	0,5	0	3,5	2,9
"	270	1,0	25	1,5	0	3,8	3,6
"	270	1,0	25	1,5	0	3,4	3,6
"	270	1,0	40	1,5	0	4,1	4,1
"	270	1,0	45	1,5	0	4,1	4,3
"	220	3,0	20	2,0	0	1,3	1,4
"	220	3,0	25	2,0	0	1,4	1,5
"	220	3,5	25	2,8	0	1,3	1,5
"	220	3,5	25	2,8	0	1,5	1,5
"	220	3,8	20	2,8	0	1,4	1,4
"	570	0,9	20	0,5	0	2,6	1,9
"	570	0,9	40	0,5	0	2,9	2,3
"	570	1,0	20	1,5	0	2,3	1,9
"	570	0,9	50	1,5	0	3,9	3,1

TABLE B6 SCHEDULE OF MEASURED AND MODEL MEAN PEAK RESISTANT SHEAR STRESS FOR SPECIMENS IN CHAPTER 4

Specimen Type	d (mm)	a/d	f_{cu} (MPa)	ρ (%)	μ	v_R (MPa)	v_{Rm} (MPa)
R C Beam (1 pt load'g)	1170	0,9	20	0,5	0	1,4	1,3
"	1170	0,9	20	0,5	0	1,6	1,3
"	1170	0,9	45	0,5	0	1,9	1,7
"	1170	0,9	25	1,5	0	1,8	1,5
"	1170	0,9	35	1,5	0	1,9	1,8
"	1170	0,9	50	1,5	0	2,4	2,2