Title No. 63 32

AN ACI TWO-PART PAPER

Basic Facts Concerning Shear Failure

By G. N. J. KANI

Reports on tests of rectangular beams performed to determine the influence of the three basic parameters in Eq. (12-2) and (17-2) of ACI 318-63. The results showed: (1) The influence of compressive strength, f_c', on so-called shear strength was insignificant and could be ignored in the analysis of diagonal failure load or allowable shear stress. (2) The influence of the percentage of main reinforcement, p, on "shear strength" was considerable. (3) The minimum value of bending moment at failure for beams of identical cross section was obtained in the vicinity of a shear arm ratio, a/d, of 2.5, and this was not influenced by p or f_c'. However, flexural load capacity varied considerably with percent of main reinforcement. (4) There exists a clearly defined region bounded by limiting values of p and a/d inside which diagonal failure is imminent and outside which full flexural strength is attained.

Key words: beams; diagonal tension; reinforced concrete; reinforcement; shear failure; strength.

■ AN EXCELLENT REVIEW OF RESEARCH in the field of diagonal tension failure through 1951 has been written by Hognestad.[1] The recently published ACI Bibliography No. 4, "Shear, Diagonal Tension, and Torsion,"[2] listing 469 publications which appeared from 1897 to 1960, shows that this research has been intensified from year to year.

In the "Foreword" to Bibliography No. 4 it was stated: "With this number of tests, one would expect the understanding of the problem to be quite complete. However, this is not the case . . . , there is still much to be learned before the problems may be considered solved." In fact, when the author tried to collect, from published test reports, a systematic presentation of the influence of the basic parameters: concrete strength, f_c'; percentage of reinforcement, p; and the shear arm ratio $a/d - M/Vd$ [see Eq. (1)]; for the whole range of practically-important beams, this turned out to be impossible. The specimens described by the different authors were, in general, so different with regard to depth, reinforcement, concrete strength, etc., that no reliable interpolation was possible. Thus, the extensive test program described in this paper became necessary.

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The primary reason for this limited understanding of the problem of diagonal failure is the great number of parameters influencing the beam strength: grade of steel, percentage of steel, grade of concrete, shape of the cross section (e.g., rectangular, T-section), absolute values of the dimensions of the cross section (e.g., d, b, b'), shear arm ratio, type of web reinforcement (e.g., bent-up bars, vertical or inclined stirrups), the quantity, arrangement, and location of web reinforcement, the type of loading (e.g., point load, two-point loading, uniformly distributed load, symmetrical or unsymmetrical loading), the type of beam (simply supported, continuous, etc.), and prestress in the longitudinal, transverse, and vertical directions which, of course, create additional parameters.

THE TEST PROGRAM

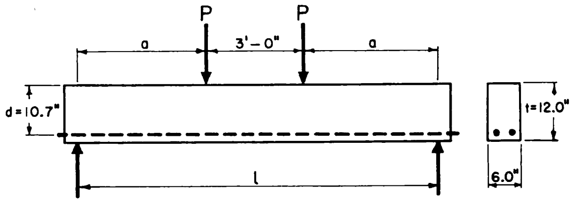
It should be noted that 133 test beams were required to establish the influence of only three parameters; f_c', p, and a/d. To study the influence of a single parameter on the shear stress at failure (v_u), a beam series of at least a dozen test specimens, in which all other parameters remain unchanged, is necessary. Therefore, in a test project designed to study the influence of three different grades of concrete (e.g., $f_c' = 2500$, 3750, and 5000 psi), in conjunction with four different percentages of main reinforcement, the number of test specimens needed would be $12 \times 3 \times 4 = 144$. Such a project would present the behavior of only one type of cross section, only one type of loading, and no web reinforcement. If the influences of these other parameters are to be established with a similar degree of reliability, it is obvious that several thousand specimens are required.

This project was an attempt at such a systematic investigation. This particular report concerns the experimental evidence of the influence of those three parameters which appear in Eq. (17-2) of the 1963 ACI Building Code:[3]

$$v_u = 0.85 \left(1.9\sqrt{f_c'} + 2500p \frac{Vd}{M} \right) \quad (1)$$

The ranges of the parameters were chosen as: f_c' between 2500 and 5000 psi, p between 0.50 and 2.80 percent, and a/d between 1.0 and transition point, T.

Fig. 1 — Typical loading arrangement and cross section of beams



Three different concrete strengths; $f_c' = 2500$ psi, 3800 psi, and 5000 psi; were used in combination with four different steel percentages; $p = 0.50$, 0.80, 1.88, and 2.80 percent; within the a/d range of 1.0 to T, for each test series.

However, for $f_c' = 2500$ psi and $p = 2.80$ percent, an over-reinforced cross section would be produced, i.e., flexural failure would occur prior to the attainment of the yield stress in the reinforcement. Since it was not the intention to investigate the behavior of over-reinforced beams in this program, this particular series was omitted. Thus, the total number of test series was $(3 \times 4) - 1 = 11$.

Because anchorage failure produces a crack which is similar to the diagonal crack of the so-called shear failure, this particular type of failure had to be excluded. Therefore, all reinforcing bars obtained anchor plates at the ends of the beam.

All beams had the same concrete cross section (6 x 12 in.) and two-point loading arrangement (see Fig. 1), no web reinforcement, but were of varying spans, and, therefore, had varying a/d ratios.

Using the system of beam designation ($f_c' - p - t$ P), (where P indicates "point" loading), the following series have been tested:

2.5-0.50-12P	3.8-0.50-12P	5.0-0.50-12P
2.5-0.80-12P	3.8-0.80-12P	5.0-0.80-12P
2.5-1.88-12P	3.8-1.88-12P	5.0-1.88-12P
—	3.8-2.80-12P	5.0-2.80-12P

TEST RESULTS

For the two-point loading condition illustrated in Fig. 2a, M/Vd in Eq. (1) is equal to a/d. Fig. 2b presents the test results of four beam series having four different percentages of main reinforcement showing v_u versus a/d. For comparison, the ACI Code formula is also represented on the diagram, from which considerable differences become apparent.

The fact that v_u varies considerably if a/d is changed (see Fig. 2b) has been known for a long time. As early as 1907, Talbot[4] had shown the influence of span length on ultimate shear stress. But only much later, in 1951, was the important parameter a/d introduced by Clark,[5] who incorporated it into his empirical formula. Unfortunately, this formula did not agree well with the behavior of beams without web

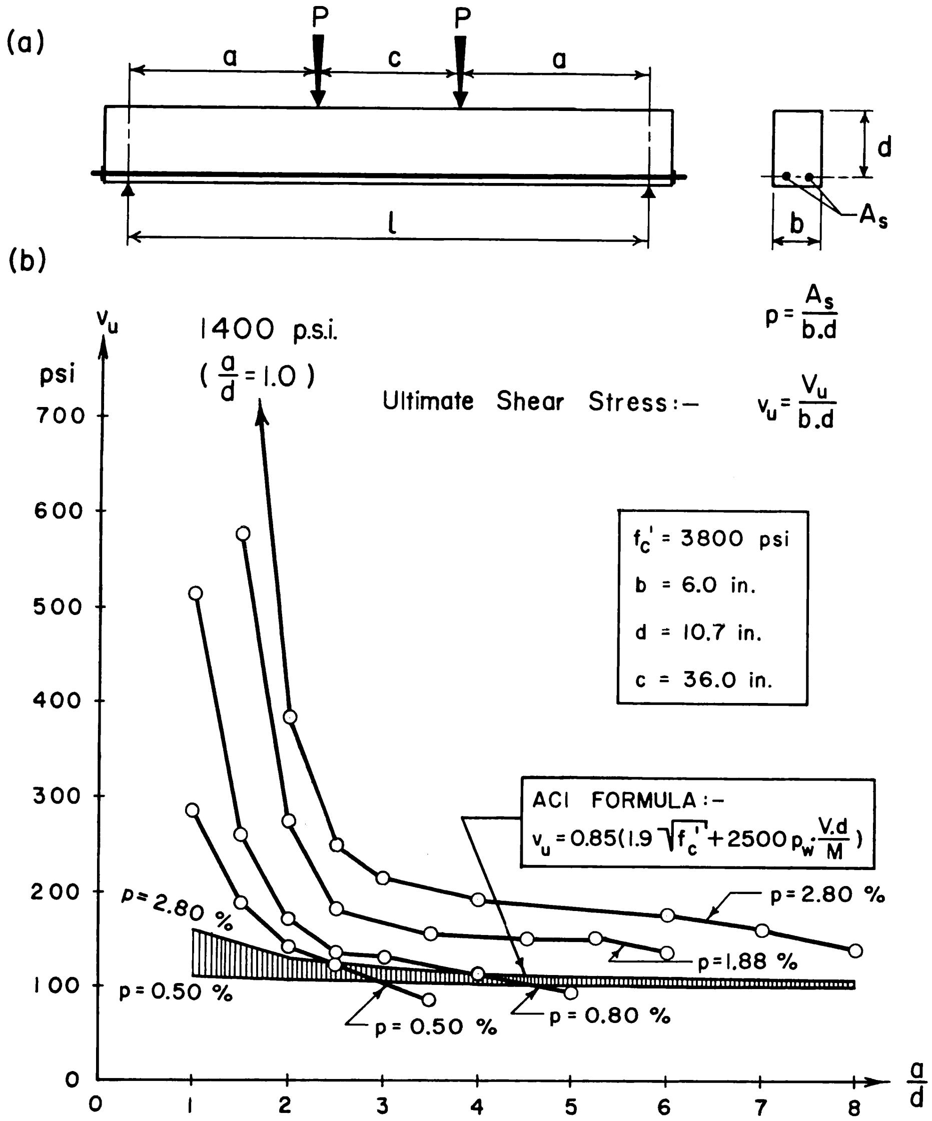


Fig. 2 — Shear stress at failure versus a/d

reinforcement. (For more details see the report of ACI-ASCE Committee 326, Shear and Diagonal Tension.[6])

Presentation of test results

The full extent of the variation of v_u, if only the two parameters, a/d and p, are varied, can be seen in Fig. 2b. For all four series $f_c' = 3800$ psi. The values of v_u for $a/d = 1.0$ are up to 700 percent higher than the corresponding values for $a/d = 5.0$. In addition to this variation, a change of p from 0.80 to 2.80 percent produced a further increase in v_u in the order of 100 percent. It can be seen that the value of $v_u = 1400$ psi for $p = 2.80$ percent and $a/d = 1.0$ is 388 percent higher than $v_u = 287$ psi for $p = 0.50$ percent and $a/d = 1.0$. Eq. (1) indicates an expected variation, for the same two beams, of only 44.5 percent (see hatched area in Fig. 2b).

Initially, it was believed, particularly by Mörsch,[7] that v_u was a property constant of concrete and depended solely on the concrete strength. Therefore, v_u was chosen as an indicator of the so-called shear failure. An allowable constant shear stress, depending only on f_c', is still the accepted basis in most of the building codes.

However, as discussed previously, tests have shown that the above assumption has not proved to be true. In particular, the tests under discussion here showed that v_u for heavily-reinforced, short beams was in the order of 15 times greater than for long beams with a low percentage of reinforcement, although the concrete strength remained constant. Thus, v_u is far from being a constant quantity. On the other hand, a change in f_c', as will be shown later, produced negligible variation in shear strength.

The author is convinced that the other possibility, which is to use the maximum bending moment at failure, M_u, as an indicator of diagonal failure, appears to be a more logical approach:

1. The upper value of M_u is the flexural strength, M_{fl}, which depends on few parameters and, therefore, necessitates a simple calculation

2. The lowest values of M_u for all the 133 beams tested in this program were in the vicinity of 0.50 M_{fl}. Thus, all values of M_u range between 50 and 100 percent of M_{fl}, instead of the 1500 percent variation in v_u evident in Fig. 2b

3. The prevention of premature failure by the formation of a diagonal crack is the very problem of "shear failure." When we obtain a diagonal failure at 70 percent of the flexural failure load, this means, of course, that we are just 30 percent short of our goal, i.e., the full flexural capacity of the cross section

4. The purpose of the web reinforcement is, of course, to increase the strength of the beam to 100 percent of M_{fl}. Thus, a result of $M_u = 0.70$ M_{fl} for a beam without web reinforcement expresses the requirement:

"a web reinforcement which increases the capacity of the beam by 30 percent of M_{fl}"

Since 1952, when Ferguson[8] turned attention to the fact that v_u varies considerably with a/d, several authors (e.g., References 9 and 10) have presented their test results in terms of diagrams of M_u versus a/d. It seems to be especially convenient to present the results as dimensionless quantities such that 100 percent represents the full flexural capacity of the cross section.

An example of test results of the series 3.8-1.88-12P is presented in Fig. 3. The shaded area is bounded by the upper and lower values of the test results. The characteristic behavior of such a series is:

(a) At $a/d = 1.0$, the full flexural capacity is attained

(b) With increasing a/d values, the beam strength declines sharply, reaching a minimum value in the vicinity of $a/d = 2.5$

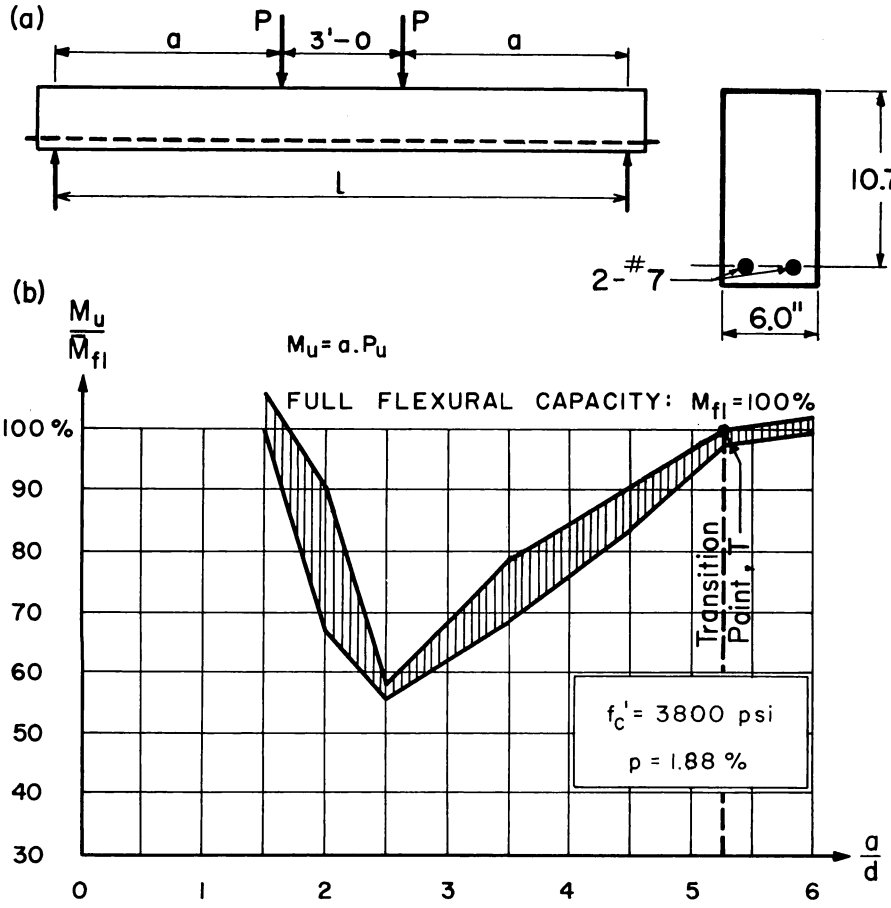


Fig. 3 — Relative beam strength $M_u/\overline{M}_{fl}$ versus a/d

(c) An increase in a/d beyond 2.5 corresponds to increasing beam strength, which reaches 100 percent of M_u/M_{fl} (i.e., full flexural capacity of the cross section) at the "transition point," T

(d) Beyond point T, no diagonal failure can be expected

Comparative flexural failure, $\overline{M}_{fl}$

The dimensionless indicator of beam strength, M_u/M_{fl}, requires a clearly defined M_{fl}. The actual M_{fl}, calculated on the basis of the actual f_y of the reinforcement in each individual beam, was found to be unsuitable for the comparison of M_u/M_{fl} between various beams because of significant differences in f_y of the various reinforcing bars used.

The flexural failure moment, $M_{fl} = f_y A_s z$, where $z = jd$, is directly proportional to the yield strength, f_y. However, diagonal failure occurs regularly *before* the yield strength of the reinforcement has been reached. Therefore, f_y does not influence M_u at all, when the beam capacity is governed by the diagonal failure strength. Two beams, equal in every respect except for f_y, and having the same M_u, would result in different M_u/M_{fl} values because M_{fl} is different.

Available reinforcing steel usually has considerable differences in f_y. In these tests, the #3 bars, used for beams with $p = 0.50$ percent, had nearly a 20 percent higher f_y than the #8 bars, used for the beams with $p = 2.80$ percent. Therefore, it was decided that the "required yield strength," $\overline{f}_y$, would be used instead of the actual f_y, and that the "comparative flexural strength," $\overline{M}_{fl}$, would be synonymous with the "required flexural strength," in the sense of the ACI Building Code.

Deformed bars of the category ASTM A 16, which specify a minimum yield stress of 50 ksi, were used as the main reinforcement for the 11 test series. Thus, the "comparative flexural moment," $\overline{M}_{fl}$, has been calculated with $\overline{f}_y = 50$ ksi:

$$\overline{M}_{fl} = 50 \text{ ksi} \times A_s z \qquad (2)$$

In Fig. 4, the difference between the "comparative" stress-strain curve, a, and an "actual" stress-strain curve, b, is illustrated. As long as the failure of the test beam occurs before the stress in the reinforcement reaches 50 ksi, the actual value of f_y is of no influence, and $M_u/\overline{M}_{fl}$ is less than 100 percent. However, if the actual steel stress at failure is greater than 50 ksi, the failure load will be affected and will accordingly produce a value of $M_u/\overline{M}_{fl}$ greater than 100 percent. Therefore, it can be assumed that the same beam, if it had been reinforced with a steel having $f_y = 50$ ksi, would have carried $50/f_y$ times the measured load. To bring all of the test results to the same basis of $f_y = 50$ ksi, the failure load for those beams which had $M_u/\overline{M}_{fl}$ greater than 100 percent were rectified by the factor, $50/f_y$. Such a rectification, of course, should not

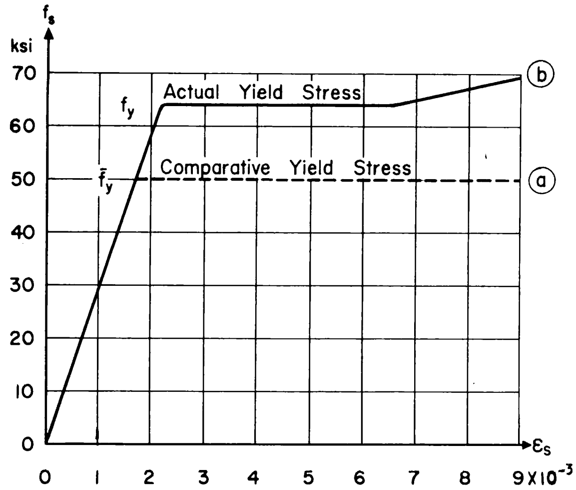


Fig. 4 — Comparison of actual and comparative stress-strain relationship for reinforcing steel

result in a value lower than 100 percent (flexural failure) since the basis for the rectification was that the yield strength had been reached. Where the rectification produced a value lower than 100 percent, the value of 100 percent was taken instead.

Where a/d and p deviated from the nominal values, the failure loads were also rectified. With test results available for the whole field of p and a/d, the rate of change of $M_u/\overline{M}_{fl}$ with respect to p and a/d could be easily determined at any combination of p and a/d values. Using the rates of change obtained from tests, the results $M_u/\overline{M}_{fl}$ were rectified accordingly to obtain their values for the exact nominal values of p and a/d. Thus, the values presented in the diagrams correspond to beams having exact nominal values of a/d and p, and having a steel reinforcement with $f_y = 50$ ksi.

The test beams with a high or medium percentage of reinforcement failed at a relatively low steel strain (i.e., between 1.7×10^{-3} and 4.0×10^{-3}) producing, at that stage, a relatively small beam deflection. On the other hand, those beams with a low percentage of reinforcement (e.g., $p = 0.50$ percent) have, prior to failure, a very shallow compressive zone, and the steel strain is a multiple of the concrete strain at the extreme compressive fiber of the beam. Prior to final collapse of the beam, the strain in the main reinforcement is often in excess of 20×10^{-3}, producing large curvature and deflection, the bearing plates at that stage being far from horizontal. With $\varepsilon_s = 20 \times 10^{-3}$, the cracks in the tensile zone of the beam are in the order of ⅛ in. wide. Since it is generally accepted that a beam is considered as having failed before such a stage of excessive cracking and deflection has been reached, the following definition of beam failure has been applied to this test program.

"The failure load of a test beam has been attained either when the steel strain of the main reinforcement reaches 6.0×10^{-3}, or when the beam collapses. The lower value of the two is taken as the failure load."

To determine the failure load of the specimens, with $p = 0.50$ percent, it was necessary to take strain readings at the level of the main reinforcement.

Diagrams presenting the test results

The test results are illustrated in Fig. 5, 6, 7, and 8 as obtained from the 133 beams of the 11 test series, after applying the considerations for failure load and rectifications for a/d, p, and f_y.

The test results presented in Fig. 5 show the characteristic variation of beam strength versus a/d of the beam series with a high percentage of reinforcement. At $a/d = 1.0$, either full flexural failure was attained with a relative strength of about 100 percent, or, due to high compression in the vicinity of the support, a web compression failure destroyed the beam. With increasing values of a/d, the beam strength decreased and was governed by its diagonal failure capacity, which was as low as 51 percent of $\overline{M}_{fl}$. At $a/d = 6.5$, the beams again attained full flexural strength. Between the two high points, $a/d = 1.0$ and $a/d = 6.5$, a "valley of diagonal failure" of reduced strength exists, whereas, outside of this region, failure occurs only after the flexural capacity of the cross section has been reached. It appears that the graphs of $M_u/\overline{M}_{fl}$ versus a/d are made up of two different functions, of which $a/d = 2.5$ is the point of intersection. Thus, the laws governing the strength of a beam for a/d less than 2.5 and for a/d greater than 2.5 are totally different and unrelated. The preceding discussion of test results substantiates the author's rational theory described in Reference 11.

Compressive destruction of the concrete in the vicinity of the supports, which is often observed in T-beams, was absent in all specimens of the 11 test series of beams with rectangular cross sections except for two beams with $p = 2.80$ percent and $a/d = 1.0$ which had the highest reaction forces at the supports.

Fig. 6 illustrates the strength diagrams of the three series ($f_c' = 2500$, 3800, and 5000 psi) of beams with $p = 1.88$ percent. The behavior of these beams was essentially the same as the beams with $p = 2.80$ percent (Fig. 5). However, a shift in the transition point, T, from $a/d = 6.5$ for the $p = 2.80$ percent series, to a lower value, $a/d = 5.5$, for the $p = 1.88$ percent series has been observed.

This trend was even more obvious for the $p = 0.80$ percent series (Fig. 7) where the "valley of diagonal failure" is greatly reduced to the region of $a/d = 1.5$ to 3.5, and the minimum point, which appears here at $a/d = 2.5$, is not lower than $M_u/\overline{M}_{fl} = 84$ percent.

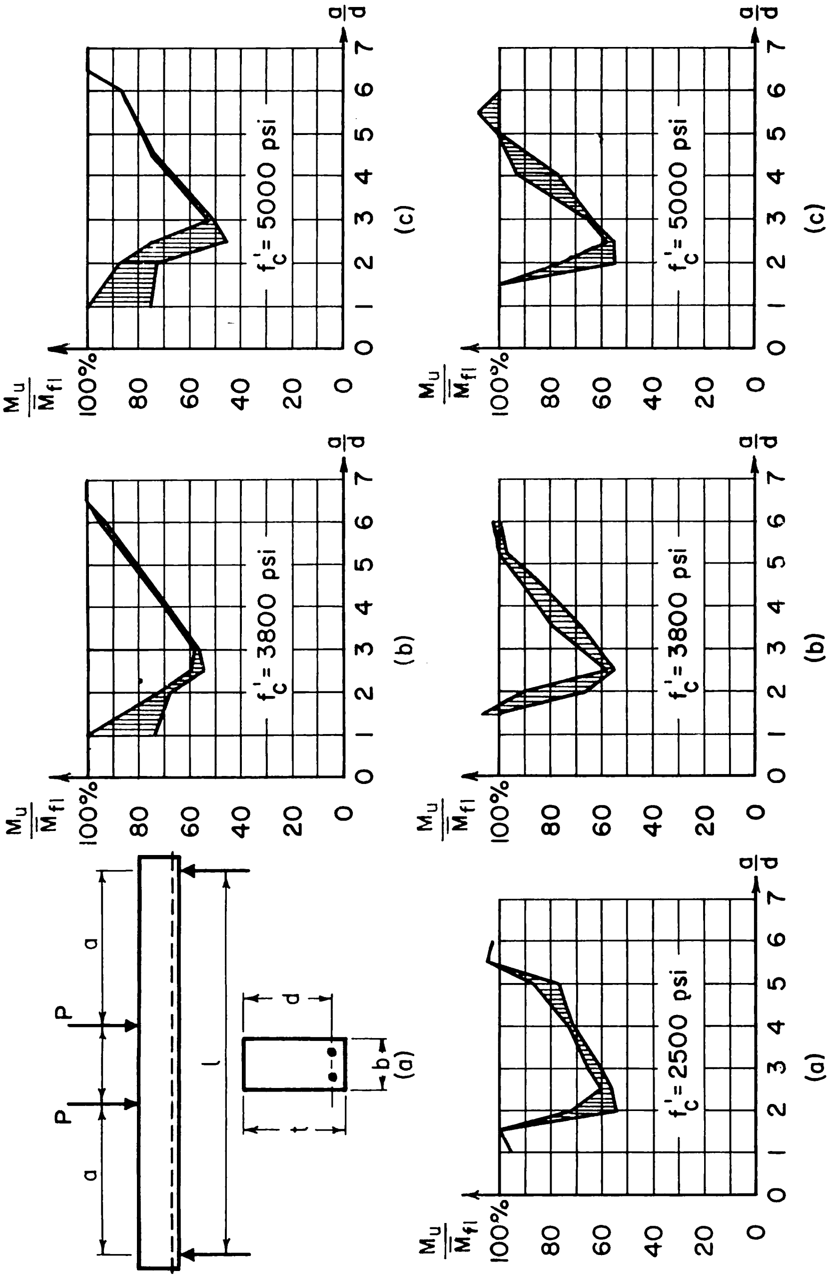
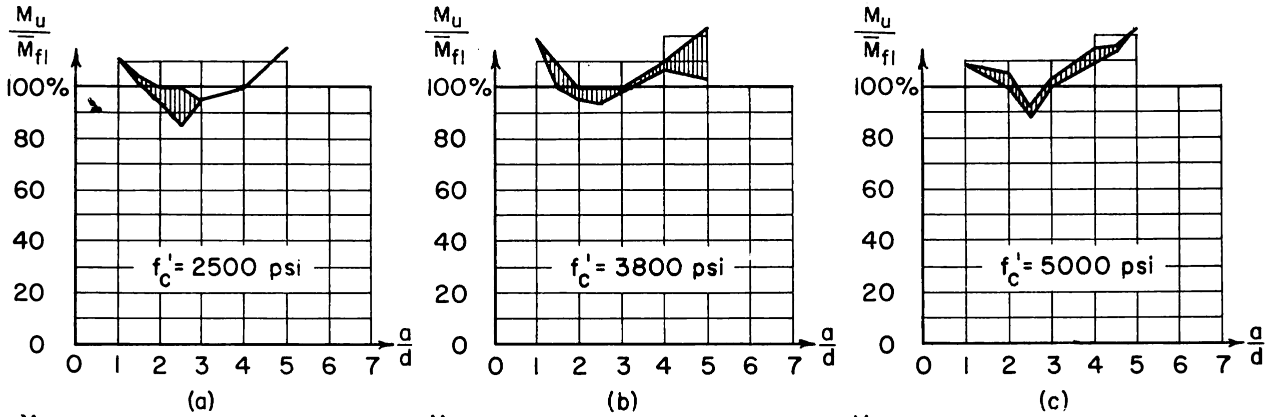


Fig. 5 (top) — Influence of the basic parameters, f'_c and a/d, on the relative beam strength for $p = 2.80$ percent

Fig. 6 (bottom) — Influence of the basic parameters, f'_c and a/d, on the relative beam strength for $p = 1.88$ percent



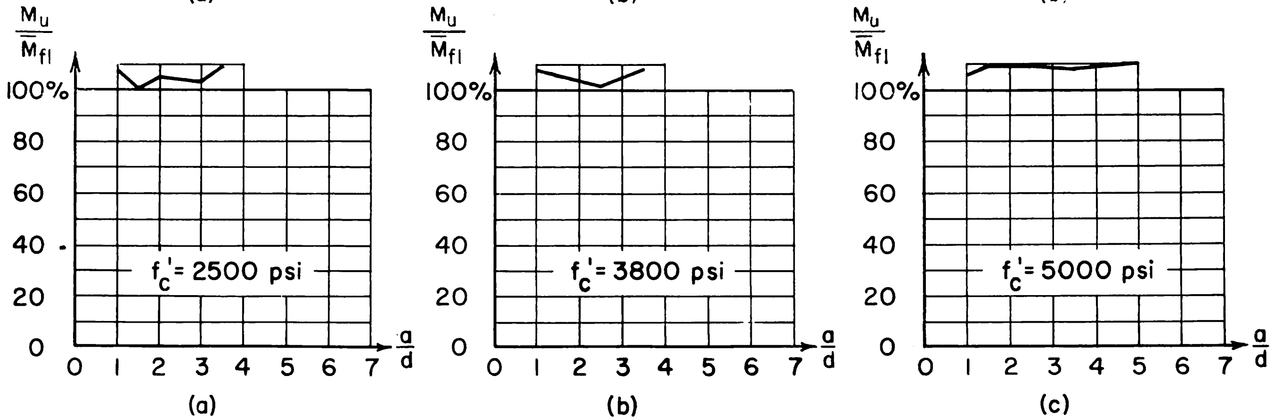


Fig. 7 (top) — Influence of the basic parameters, f_c' and a/d, on the relative beam strength for $p = 0.80$ percent

Fig. 8 (bottom) — Influence of the beam strength, f_c' and a/d, on the relative beam strength for $p = 0.50$ percent

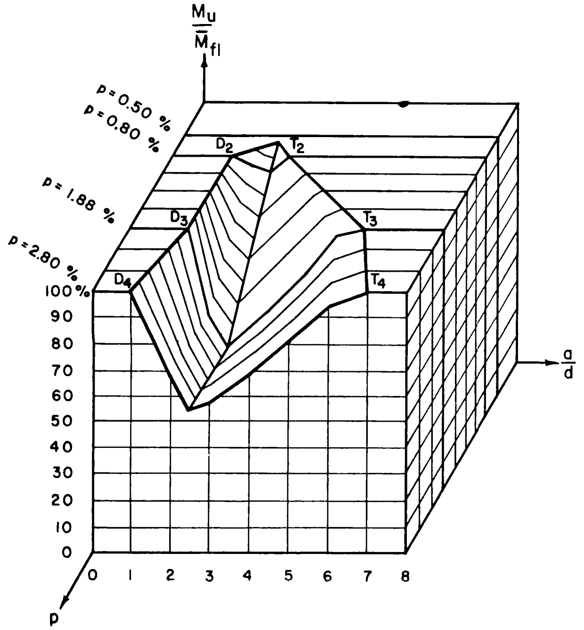


Fig. 9 — Relative beam strength, $M_u/\overline{M}_{fl}$, versus a/d and p.

For the three beam series with $p = 0.50$ percent (Fig. 8), the "valley of diagonal failure" disappeared completely. In every case, the relative beam strength, M_u/M_{fl}, was 100 percent or more.

The region of diagonal failure may be clearly visualized in a three-dimensional coordinate system, with axes of a/d, p, and $M_u/\overline{M}_{fl}$. Fig. 9 presents the test results of the four series with $f_c' = 3800$ psi, illustrating that for $p = 0.50$ percent or less, no "shear failure" exists. With increasing values of p, the "valley of diagonal failure" deepens rapidly, with its range always limited by the two characteristic boundary points, D and T. If the a/d axis is considered, it can be seen that the reduced beam strength, due to "shear failure" occurs only between $a/d = D$, which is nearly a constant equal to 1.0, and the transition point, T, which varies with p. Therefore, all the considerations of web reinforcement should be limited to this region.

Although Fig. 9 presents only those test results for $f_c' = 3800$ psi, the diagrams for those beams with $f_c' = 2500$ psi and $f_c' = 5000$ psi are similar since the concrete strength has a negligible influence on diagonal failure, as long as over-reinforced beams are excluded.

UNIFORMLY DISTRIBUTED LOAD

The behavior of beams under a uniformly distributed load was not included in the Toronto test program. However, the behavior of such a complete beam series can be derived from the Leonhardt-Walther paper[10] in which beams were used of nearly the same depth (see Fig. 10d) as in our tests.

The definition for shear arm ratio, as used for point loading cannot, of course, be used for a uniformly distributed load, and has to be modified. This can be done, as suggested in Fig. 10a, 10b, and 10c, by starting with two-point loading at the quarter points. The load is gradually distributed until the uniformly distributed load of Fig. 10c is obtained, whereas the resultants of the distributed loads remained, in all three cases, at $a = l/4$. Thus, it appears that a uniformly distributed load compares best with a point loading arrangement of two point loads at the

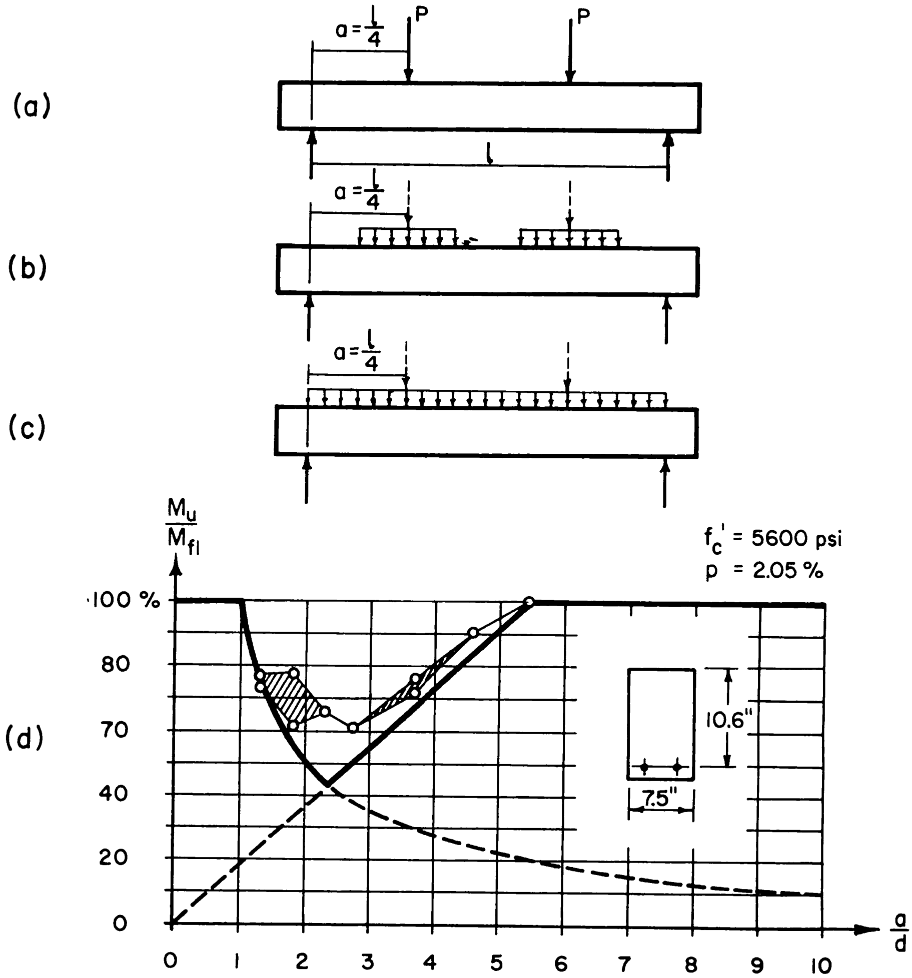


Fig. 10 — Relative beam strength, $M_u/\overline{M}_{fl}$, under uniformly distributed load, as obtained from the tests described in Reference 10

quarter points. *Consequently, the shear arm of a beam with a uniformly distributed load has been defined by* $a = l/4$. This definition has the important advantage that a "critical cross section of diagonal failure," which cannot be reliably determined, is eliminated. Also, the otherwise suggested definition of $a = M/V$ agrees with the proposed definition of M_{max} and V_{max}:

$$a_{UDL} = \frac{M_{max}}{V_{max}} = \frac{l}{4}$$

With this definition for the shear arm, a, in the case of a uniformly distributed load, the diagram of relative strength in Fig. 10d has been plotted, using the test values as reported in Reference 10. The behavior of reinforced concrete beams under a uniformly distributed load appears to be essentially the same as under point loadings, including the observation that a minimum point in the relative strength diagram exists. Thus, the relative strength of short beams, which increases with decreasing a/d ratios, cannot be attributed to high pressure under point loads since the same effect is observed under a uniformly distributed load.

A comparison of Fig. 10d with a corresponding diagram for point loading tests (e.g., Fig. 6b) shows, as could be expected, that a uniformly distributed load produces somewhat more favorable results. Thus, it is slightly conservative if the design requirements for beams under point loadings are extended to beams under uniformly distributed loads.

CONCLUSIONS FROM THE TESTS

1. The most significant conclusion from these 11 test series is that the "shear strength" of rectangular, reinforced concrete beams *does not depend on concrete strength* within the entire range of $f_c' = 2500$ to 5000 psi and $p = 0.50$ to 2.80 percent.

From Fig. 6, for example, it appears that the relative beam strength is practically the same for all three grades of concrete, and therefore, independent of the concrete strength. The same can also be said for beams with $p = 2.80$ percent (Fig. 5), $p = 0.80$ percent (Fig. 7), and $p = 0.50$ percent (Fig. 8).

Although this appears to be true only for the relative beam strength, $M_u/\overline{M}_{fl}$, it also applies to the absolute beam strength, M_u, itself, and the shear stress at failure, v_u, if we consider the factor j in $z = jd$ to be a constant. The attitude that the difference in j can be neglected has already been accepted by the ACI Building Code in the analysis of "shear strength" by taking $v_u = V_u/bd$ instead of $v_u = V_u/bjd$. By taking $j = 7/8$, the comparative flexural moment,

$$\overline{M}_{fl} = \frac{7}{8} d\overline{f}_y A_s \qquad (3)$$

depends only on the amount of reinforcement. Therefore, for any given steel percentage, p, $\overline{M}_{fl}$ is the same for all grades of concrete. Then, if the $M_u/\overline{M}_{fl}$ values in this group are equal, the M_u values are likewise identical. Since $v_u = M_u/abd$, then the ultimate shear stress is also independent of concrete strength.

2. The second significant conclusion arises from the comparison of the beam series with the same grade of concrete, e.g., for $f_c' = 3800$ psi, but with different percentages of main reinforcement. Fig. 5b, 6b, 7b, and 8b illustrate the considerable influence of the amount of main reinforcement on the relative beam strength, $M_u/\overline{M}_{fl}$. For those beams with a high percentage of reinforcement ($p = 2.80$ percent), the "valley of diagonal failure" has a low point in the vicinity of $M_u/\overline{M}_{fl} = 50$ percent, whereas for those beams with a low percentage of reinforcement ($p = 0.50$ percent with $M_u/\overline{M}_{fl} = 100$ percent), the "valley of diagonal failure" disappears. Since $\overline{M}_{fl}$, for $p = 2.80$ percent, is approximately $2.80/0.50 = 5.6$ times greater than for $p = 0.50$ percent, M_u itself, for $p = 2.80$ percent, is about $5.6 \times 0.50 = 2.8$ times greater at the low point of the "valley" than for $p = 0.50$ percent. At all other a/d values, the ratio is greater, ranging between 2.8 and 5.6.

3. While the shear stress at failure, v_u, for small a/d ratios, is up to 760 percent higher than for large a/d values, the variation of the relative beams strength, $M_u/\overline{M}_{fl}$, exhibits considerably smaller variations over the same range; in the order of 100 percent rather than 1000 percent. Contrary to initial expectations, the "shear strength," v_u, turned out to be very remote from being a "constant," characteristic of a particular grade of concrete. It varies up to 1000 percent and does not appear to be influenced by the concrete strength at all. Therefore, the author suggests that the "relative beam strength," $M_u/\overline{M}_{fl}$, is a much more suitable indicator of the beam strength than the "ultimate shear strength," v_u. (Compare Fig. 2 and Fig. 5 to 8).

4. Fig. 9 illustrates that the beam strength varies between 50 and 100 percent of the flexural capacity of the cross section, the exact strength depending on the combination of values of a/d and p. If we are to express the ultimate strength of a beam by

$$M_u = r\overline{M}_{fl} \qquad (4)$$

the reduction factor, r, would vary between 0.50 and 1.00. The reduction factor, r, which depends on p and a/d (or M/Vd) may be determined either by a formula or from a table. Thus, the problem of "shear strength" would become an investigation of, and a search for, the type and quantity of web reinforcement required to increase the reduction factor, r, to 1.00.

Although the reinforced concrete beam without web reinforcement is the exception rather than the rule in modern structural practice, it is the author's belief that, prior to considering additional parameters such as other shapes of cross sections (eg., T-beams) and web reinforcement, the basic facts concerning the behavior and strength of rectangular beams without web reinforcement should be clearly understood and established for the entire range of beams which have practical applications. Only by reducing the number of variables to a minimum are we able to attribute the results of our investigations to the individual parameters inherent in a reinforced concrete beam. The investigation of the effect of only three parameters necessitated the extensive test program outlined in this paper.

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Sinopsis—Résumé—Zusammenfassung

Hechos Básicos Concernientes al la Falla por Cortante

Se reportan ensayes de vigas rectangulares efectuados para determinar la influencia de los tres parametros básicos en las ecuaciones, (12-2) y (17-2) del ACI 318-63. Los resultados muestran lo siguiente: (1) La influencia de la resistencia a compresión, f_c' sobre la llamada resistencia a cortante fue insignificante y puede ser ignorada en el análisis de cargas diagonales de falla o de esfuerzo cortante permisible. (2) La influencia del porcentaje del refuerzo principal, p, sobre "la resistencia a cortante" fue considerable. (3) El valor mínimo de la capacidad de carga para vigas de sección idéntica correspondió a una relación de claro de cortante, a/d, de 2.5, y este valor no influenciado por p, o f_c'. Sin embargo, la capacidad de carga flexionante varió considerablemente con el porcentaje de refuerzo principal. (4) Existe una región claramente definida confinada por valores límites de p y a/d dentro de la cual la falla por tensión diagonal es imminente y fuera de la cual se obtiene una resistencia flexionante completa.

Les Facteurs à la Base de la Rupture par Cisaillement

Cet article traite de certains essais auxquels on a soumis des poutres à section rectangulaire afin d'établir l'influence de chacun des trois paramètres de base contenus dans les équations (12-2) et (17-2) de la Norme 318-63 de l'ACI. Ces essais ont démontreé que: 1) L'influence de la résistance à la compression f_c' sur ce qu'on a convenu d'appeler la résistance au cisaillement, est négligeable et il n'y a pas lieu d'en tenir compte dans le calcul de la charge de rupture en diagonale ou de la contrainte permise en cisaillement. 2) L'influence de la proportion d'armature principale p sur la résistance au cisaillement est considérable. 3) La valeur minimum de la capacité portante des poutres de même section est atteinte lorsque le rapport a/d entre l'effort tranchant et son bras de levier approche 2.5; cette valeur minimum n'est pas affectée par p ou f_c'. On a noté toutefois que la capacité portante des poutres en flexion varie considérablement suivant le pourcentage d'armature principale. 4) Il existe une zône nettement délimitée par des valeurs définies de p et a/d, à l'intérieur de laquelle la rupture par cisaillement est inévitable mais à l'extérieur de laquelle la résistance à la flexion de la poutre peut être développée à sa pleine valeur.

Die Grundtatsachen des Schubbruches

Es wird über Versuche berichtet, die mit der Absicht durchgeführt wurden, den Einfluss der drei Grundparameter, d.h. der Betonfestigkeit, des Bewehrungsgrades und des Schubarm-Verhältnisses festzustellen, die in den Festigkeits-

formeln (12-2) und (17-2) der neuen amerikanischen Stahlbetonnorm ACI 318-63 aufgenommen wurden. Die Versuchsergebnisse zeigten: (1) Der Einfluss der Betondruckfestigkeit f_c' auf die sog. Schubbruchfestigkeit ist unbedeutend und könnte in der Berechnung der Schubbruchfestigkeit oder der Bestimmung der zulässigen Schubspannungen unberücksichtigt bleiben. (2) Der Einfluss des Bewehrungsgrades auf die Schubbruchfestigkeit ist bedeutend. (3) Der Mindestwert des Bruchmomentes M_u jeder Balkenserie gleicher Querschnitte wurde jeweils in der Nähe des Schubarmverhältnisses $a/d = 2{,}5$ festgestellt, wobei weder der Bewehrungsgrad, noch die Betonfestigkeit von Einfluss war. Allerdings erwies sich das Tragfähigkeitsmoment M_u als sehr vom Bewehrungsgrad p abhängig. (4) Es besteht ein klar bestimmbares Gebiet, das durch Grenzwerte von p und a/d gekennzeichnet ist, innerhalb welcher Schubbrüche entstehen, während ausserhalb dieses Gebietes volle Biegetragfähigkeit des Querschnittes erreicht wird.