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Critical Appraisal of the Moment-Shear-Ratio

Considérations critiques sur le rapport moment fléchissant-cisaillement

Kritische Betrachtungen über das Momenten-Schubverhältnis

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1. Introduction

In the last few years an extensive investigation into the problem of shear in reinforced concrete has been carried out by Prof. Dr.-Ing. F. LEONHARDT and the author at the *Otto-Graf-Institut* of the Technische Hochschule Stuttgart. The results of these tests conducted so far have been reported in several publications [1, 2, 3]. The limited space allotted to the present publication does not permit here a detailed description of all these tests or findings. We therefore treat only one of the many problems dealt with, namely, the question whether it is the bending moment or the shear force or both together which decisively govern the shear strength of reinforced concrete.

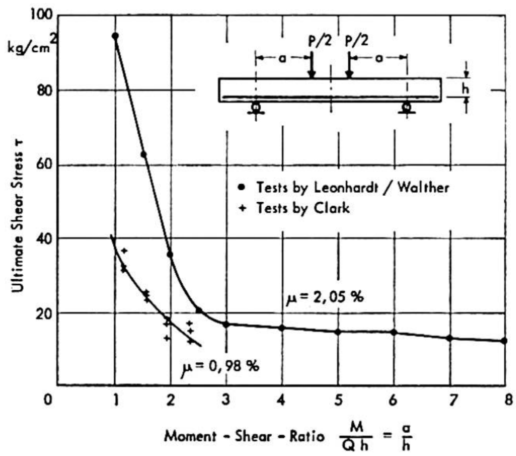

Fig. 1. The influence of $\frac{M}{Q\,h} = \frac{a}{h}$ on the ultimate shear stresses of simple span rectangular beams without web-reinforcement (μ = percentage of longitudinal tension reinforcement).

As is well known, mainly from early investigations in the U.S.A. the ultimate shear force of a given member is by no means a constant (Fig. 1), but increases very rapidly with decreasing shear span a (i.e. distance of the load from the support). As a result it was and is often concluded, that the shear force Q or the shear stresses τ alone could not be a suitable criterion for the shear strength. Many investigators, therefore, have formulated the shear strength as a moment capacity, sometimes even completely disregarding the magnitude of the shear force. In fact the ultimate shear moment of a given member varies less than the shear force, but it is also not a constant (Fig. 2).

Opposed to this point of view, warning voices were raised in many places, particularly in Germany, by those who wanted to adhere to the traditional truss analogy founded by Mörsch, according to which moment and shear are to be treated independently and the shear stresses are the only criterion for the dimensioning of the web and the web reinforcement. The high shear strength for loads near the supports are considered as local effects, which occur so pronounced only in the case of rectangular members and only for loads applied from the top and near the supports. These effects are believed to be insignificant for practical purposes.

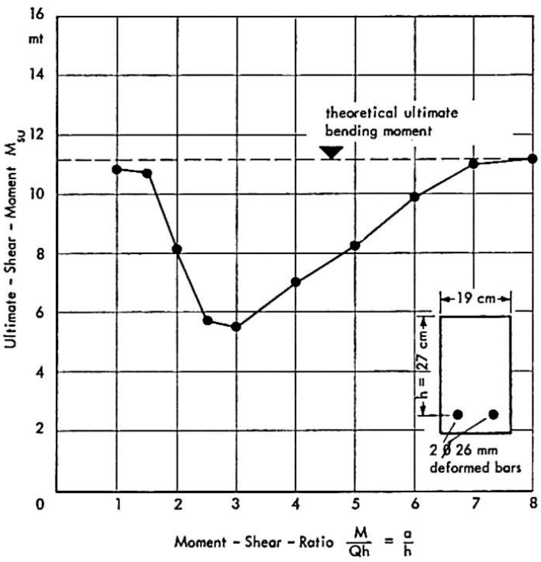

Fig. 2. Evaluation of the same test results as given in Fig. 1 (upper curve), but in terms of ultimate shear-moment.

Concrete strength: $\beta_w \cong 350$ kg/cm^2
Yield strength of steel: $\beta_{0,2} = 4740$ kg/cm^2

As is often so in the case of such extreme views, both overlook a few significant aspects important for the general understanding and the practice. Undoubtedly some premature and too far reaching conclusions were drawn from recent investigations, but there are, just as definitely, many cases in which the new findings can positively be applied. To derive these and to limit the possible fields of application shall be the task of this paper.

In place of shear span, which after all is clearly defined only for the members with symmetrical two point loading, predominant in shear testing, the more general term "moment-shear-ratio" $\frac{M}{Q\,h}$ is used mostly nowadays.

2. Simply-supported Rectangular Beams

2.1. Experimental evidence

The results of some test series on rectangular beams without web reinforcement are summarized in Fig. 1. They all show that in case of loads near the support, i.e., for small shear spans or for small moment-shear-ratios the relative shear stresses $\tau_{0,u} = \frac{Q_u}{b\,z}$ at ultimate are very high and attain sometimes ten times the value of the allowable stresses. Undoubtedly this phenomenon has led to the conclusion, that the shear stresses are not a suitable criterion for the shear strength. Therefore many mostly empirical formulae were proposed which give the shear strength as a function of $\frac{M}{Q\,h}$.

The same test results are plotted over again in Fig. 2 as a function of $\frac{M_{su}}{b\,h^2\,\beta_p}$ (ultimate shear-moment / cross. sectional properties). In case of loads close to the support M_{su} corresponds to or exceeds the ultimate bending moment even though the mode of failure was that of shear. In all the test series the ultimate shear-moments show a minimum for $\frac{M}{Q\,h}$ values between 2 and 3. After that, M_{su} increases continually up to the point where failure is due to bending. The transition from shear to flexural failure depends largely on the percentage of longitudinal reinforcement.

Similar conclusions were observed for beams with uniformly distributed load, where the ultimate shear resistance decreases with increasing slenderness l/h.

2.2. Interpretation and objections

The results of such tests were hardly ever doubted, but they were interpreted quite differently. Some take it as a clear proof that the shear failure results from the combined action of moment and shear, a notion which is decidedly disputed by others.

Let us first consider some arguments of the latter.

2.2.1. Vertical load stresses. The most important is probably the fact that loads and support reactions acting directly on the beam produce vertical compression stresses σ_y, which in turn reduce the inclined principal tensile stresses according to the well known relationship

$$\sigma_{I/II} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\frac{(\sigma_x - \sigma_y)^2}{2} + \tau_{xy}^2}.$$

Thus they delay the formation of shear cracks and consequently the shear failure. Since these σ_y may be pretty big close to the supports or load points and fade away with increasing distance from these points, it is conceivable to attribute the high shear strength for small values of $M/Q\,h$ to this favorable effect of local stress concentration.

As a further proof for this school of thought, an investigation by FERGUSSON [4] is often cited. In these tests it was observed, that the ultimate load drops considerably, when the loads are not applied from the top as usual, but over crossmembers. In the extreme case, where not only the loads but also the support reactions are introduced indirectly over crossmembers, the shear strength was found to be only about one third of that for normal load and support conditions; this may be traced back to the lack of the favourable vertical load stresses σ_y [1]).

These considerations seem to substantiate the view, that beams without web reinforcement fail in shear, when the principal tensile stresses reach the tensile strength of concrete. Since these tensile stresses outside the region of loads or supports are equal to τ_0 in the neutral axis of cracked sections, one can conclude, that the shearing stresses are after all the governing factor for shear failures, and that there is no need or justification of considering the bending moment.

2.2.2. The influence of the percentage of longitudinal reinforcement. There are however other aspects of the problem, which cannot be explained by the favourable effect of vertical loads stresses σ_y mentioned above. Fig. 3, for

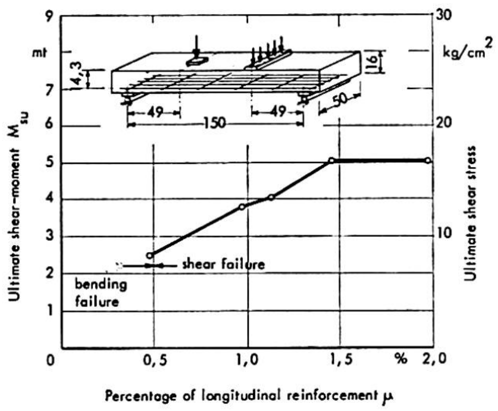

Fig. 3. Influence of the percentage of longitudinal tension reinforcement on the ultimate shear-moment of simply supported slabs.

example, shows the results of a test series of simply supported slabs without web reinforcement. The only parameter of this series was the percentage of longitudinal reinforcement μ. Accordingly the shear strength can increase by more than 100%, when μ increases from 0.5% to 1.5%. One can thus presume, that the bending moments do have an influence on the shear strength and that it might be appropriate to relate the carrying capacity to a moment rather than to the somewhat uncertain shearing stresses.

2.2.3. The dowel-action. However, also this argument has lately been discredited, owing to recent investigation into the dowel-action of the longitudinal reinforcement.

It has always been sensed, that the longitudinal bars crossed by an inclined crack carry a certain amount of shear force, which however was considered negligible compared with the total shear. Yet according to tests by W. J. Krefeld and C. W. Thurston [5] this does not seem to be the case.

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[1]) Fergusson's test specimens had, however, no reinforcement in the cross-members and very short anchorage lengths of the longitudinal main reinforcement, thus precipitating failure for other reasons than shear.

In these tests (see Fig. 4) a central concrete block, separated from the rest of the concrete beams, permitted to apply the loads to be applied directly on the longitudinal reinforcing bars; the whole load had thus to be carried by dowel action. In order to give an idea about the magnitude of this dowel-action, we have evaluated the results of these tests in terms of nominal shearing stresses at ultimate. Fig. 4 shows that they amount to about one third of the ones obtained in normally loaded beams. The dowel-action increases with the

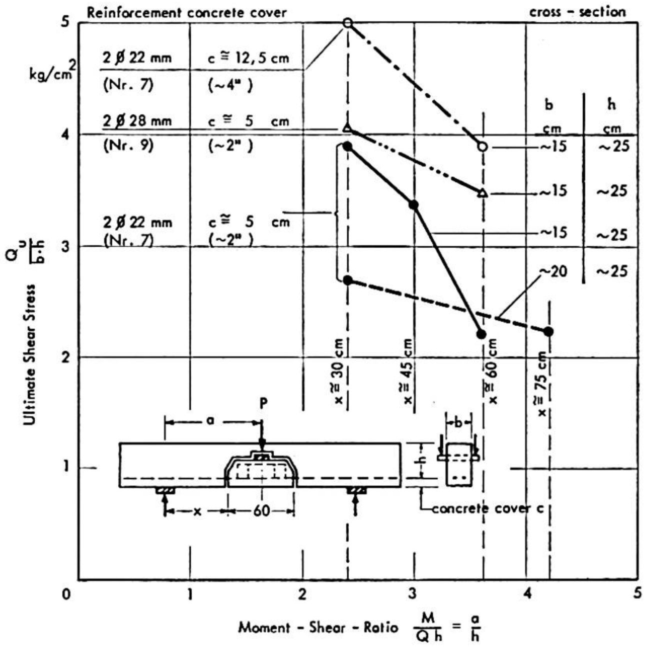


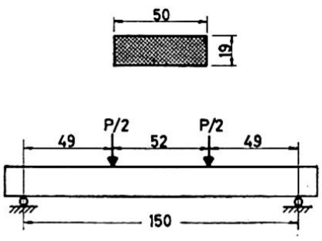
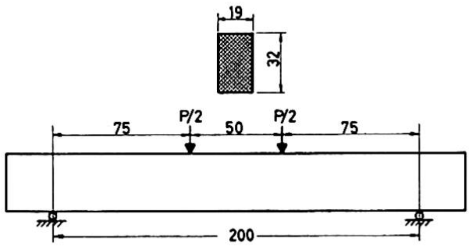
Fig. 4. Tests by KREFELD and THURSTON [5] to determine the dowel-action of the longitudinal tension reinforcement.

(In order to allow a comparison with standard shear tests, the results are given in terms of ultimate shear stresses of the cross-section $b.\,h$.)

thickness of the concrete cover, the percentage of the longitudinal reinforcement and last but not least with decreasing shear span or M/Qh. The dowel-action follows, therefore, quite closely the tendencies of the phenomena mentioned so far, and one could be tempted, to consider it as their major cause.

2.2.4. The influence of bond. However, other points of view have still to be considered. In the course of our investigations two beam series were tested with the purpose to check the influence of the bond of the longitudinal reinforcement (see [1], Ch. 3 and 5). The variation of bond was obtained by placing only a few but thick bars in one set of beams or slabs and more but thinner bars in the others in such a way that both sets had the same total area of steel. Consequently the distributed reinforcement had a higher ratio of surface to area than the concentrated one and thus a better bond. The results are summarized in table 1. It shows that the shear strength in case of distributed reinforcement is up to 28% greater than in case of concentrated reinforcement. In our opinion this increase can only be the result of better bond because the effect of dowel-action, if any, should exhibit an inverse trend since the spacing and the proper stiffness of the bars are smaller for distributed than for concentrated reinforcement.

Table 1. Influence of Bond

Beam	Dimensions of test beams	Long. steel	p (%)	P_U (Mp)
P 4		9 ⌀ 12 mm	1,40	20,0
P 6		4 ⌀ 18 mm	1,43	17,4
P 7		2 ⌀ 26 mm	1,48	15,0
EA 2		3 ⌀ 16 mm + 2 ⌀ 14 mm	1,87	15,2
EA 1		2 ⌀ 24 mm + 1 ⌀ 16 mm	1,88	11,9

Now if the bond of the longitudinal reinforcement does play an important role, which can hardly be doubted, then so do the deformations of the tensile zone and consequently the bending moment.

2.2.5. Cut-off-bars. This can also be seen from our tests with simple and continuous slabs [6] with partially cut-off main reinforcement. The most essential data is presented in Fig. 5. The reinforcement consisted of two layers of welded wire meshes; one layer extending over the support, while the other was cut-off in the tension zone. The main variable of that series was the anchorage length v, i. e., the distance between the point where the second layer is theoretically not anymore needed as bending reinforcement and the last lateral wire at the end of the mesh.

As follows from Fig. 5, the strength of these slabs decreases appreciably with decreasing anchorage length v even though there was everywhere sufficient reinforcement to cover the flexural stresses. It is further to be noted, that the ultimate shearing stresses can be much lower than in comparable beam tests with the same concrete strength of $\beta_w \cong 300$ kp/cm^2 and without bar cut-off. This is especially true for continuous slabs, where even for ample bond length the flexural capacity was not attained. Therefore, the deformations in the tension zone do have an important influence on the shear strength. It proves once more, that the shearing stresses alone are indeed a very uncertain criterion for the prediction of shear failure.

Aside from the problem treated here, careful attention should be payed to the low shearing stresses causing shear failure, which were even smaller than the ones presently allowed in the German Code for service load [1]. Since it is fairly common practice in slab construction to have the wire mesh partly overlapping similarly to these tests, a reduction of the allowable shearing stresses is recommended for such slabs.

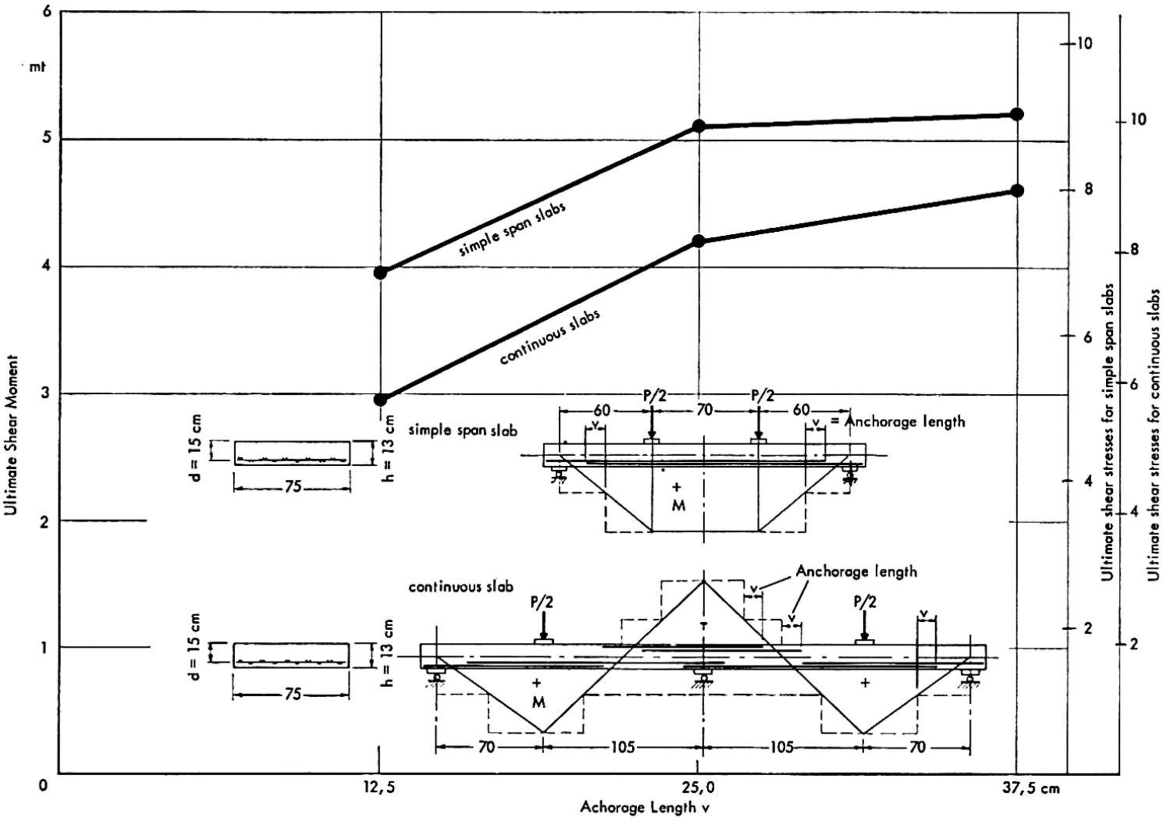


Fig. 5. Shear strength of simple span and continuous slabs as a function of the anchorage length of the tension reinforcement (welded wire mesh). All slabs without shear reinforcement; percentage of tension reinforcement $\mu = 0.69\,\%$; concrete strength $\beta_w \cong 270$ kg/cm^2. Ref. [6].

2.3. Consideration of the mode of failure

We distinguish here the following modes of failure:

1. flexural failure,
2. shear-compression failure,
3. failure of the web,
   a) by yield or rupture of the stirrups,
   b) by destruction of concrete due to inclined compression

In case of rectangular cross sections the so-called shear-compression failure predominates, with shear cracks generally originating from the bottom and progressing continuously towards the compression zone.

For slender beams without web reinforcement this type of failure often occurs suddenly, in all other cases the shear-cracks propagate gradually whereby reducing the depth of the compression zone and collapse finally takes place by destruction of this weakened compression zone. In our opinion, a close observation of the development of this common failure mode indicates, that it is the magnitude of the moment rather than that of the shear which ultimately causes failure, i.e. the destruction of the shear-compression zone.

2.4. The Strength of the Shear-Compression-Zone

For beams without web reinforcement the whole shear force has to be carried by the compression zone and the dowel-action of the longitudinal reinforcement after the formation of the shear crack. Since the latter accounts for only about one-third of the total shear force, as described in 2.2.3, considerable shear stresses have to be sustained by the compression zone in addition to the flexural normal stress σ_x. Consequently there is a biaxial state of stress which, according to the theory of the strength of material reduces the capacity of the compression zone to resist the normal stresses σ_x. We have denoted this resistance by the term shear compression strength $\beta_{p\tau}$ which is always smaller than the monoaxial compression strength β_p (cylinder strength). This fact was derived theoretically, e.g., by H. Bay [7], S. Guralnick [8], Goschy [9] and the author [10]. Since the mean shearing stresses in the compression zone are about $\tau_x = Q/bx$ (x = depth of the compressive zone) and the normal bending stresses may be estimated as $\sigma_x = M/hbx$, the expression M/Qh stands approximately for the ratio of normal to shearing stresses in this zone. This explains why all these authors have found that $\beta_{p\tau}$ depends theoretically and experimentally on M/Qh. In spite of considerable differences in the approaches all found about the same relationship as shown in Fig. 6. The consideration of the moment-shear-ratio can thus theoretically be justified.

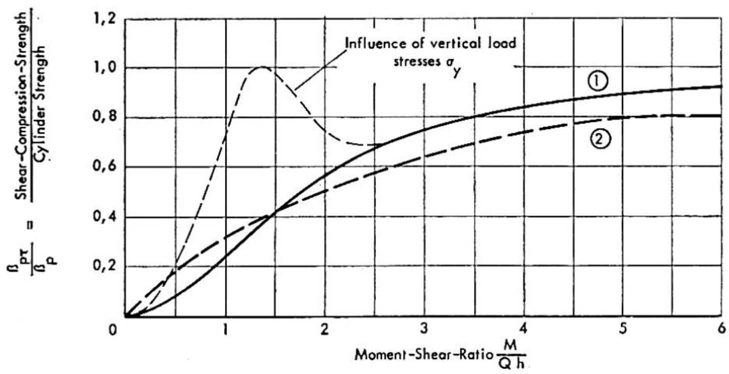


Fig. 6. Strength of the shear-compression zone $\beta_{p\tau}$ as a function of the moment-shear-ratio.
(1) Derived by the author in [10]
(2) Derived by Bay in [7]

This holds also true for rectangular beams with web reinforcement as long as this mode of failure occurs. However, because a part of the shear force is taken by the web reinforcement. M/Qh has to be replaced by

$$\frac{M}{Q(1-\eta)h}$$

where η is the relative portion of shear carried by the stirrups. For $\eta = 1$, i.e., for full shear reinforcement this expression becomes equal to ∞, which means that we have the same condition as for pure bending and a flexural failure is to be expected.

2.5. *The Depth of the Shear-Compression-Zone*

The theoretical investigations just mentioned have indeed produced significant information about the combined action of moment and shear, yet an even more important question concerns the effective depth of the shear-compression-zone. It has already been stated that with increasing load the diagonal cracks propagate towards the compression edge of the beam more rapidly than bending cracks, thus reducing the effective depth of the compression zone and consequently precipitating shear failure. This propagation is to a large extent a function of the deformation of the tension chord, which in turn depends on the magnitude of the bending moment, the percentage of longitudinal reinforcement and the bond. Obviously the moment-shear-ratio alone does not give any pertinent information here, if not coupled with the deformation characteristics of the tendon. This is certainly the reason why in the new ACI-Building Code [1] the term M/Qh is replaced by

$$\frac{M}{1200\,A_s d}\ ^{2)}$$

which is the ratio of the approximate steel stress $M/A_s d$ (A_s = area of longitudinal tension reinforcement, d = effective depth of the beam) to some arbitrarily chosen tensile stress of 1200 psi. In the shear theory proposed by the author in [10] M/Qh, μ and the bond characteristic are included.

3. T- and Similar Cross-sections

Beams with strong chords and relatively thin web show an entirely different shear behavior than the rectangular beams treated so far. The shear failures occur here in general by destruction of the web zone due to one or mostly a combination of the following causes:

a) the inclined principal compression stresses exceed the concrete cylinder strength;
b) the stirrups reach the yield or rupture strength;
c) slip in the anchorage of the stirrups.

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2) The allowable shear stress v_c is specified in the ACI-Code by

$$v_c = \sqrt{f_c'}\left(\frac{f_s}{f_s - 1500\,\text{psi}}\right) \quad \text{or} \quad \frac{\sqrt{f_c'}}{1 - \frac{1200\,A_s d}{M}},$$

where f_c' is the concrete cylinder strength and f_s the working stress of the tension reinforcement.

Both b) and c) show large deformation and eventual destruction of the concrete in the web zone, while failure of the type a) may occur suddenly without excessive deformations.

Our tests have shown that it is primarily the magnitude of the principal compression stresses σ_{II} which determine the failure of the web. These principal stresses in turn are a function of the shear stresses and the inclination of the web reinforcement.

The following relationship was experimentally observed (see 1 and 3):

$$\sigma_{II} = 1.5 \text{ to } 1.8\,\tau_0 \text{ for inclined stirrups,}$$
$$\sigma_{II} = 2.1 \text{ to } 2.5\,\tau_0 \text{ for vertical stirrups.}$$

Fig. 7 gives the distribution of the stirrup stresses for a beam series with the web thickness as the sole variable. Evidently the stirrup stresses are con-

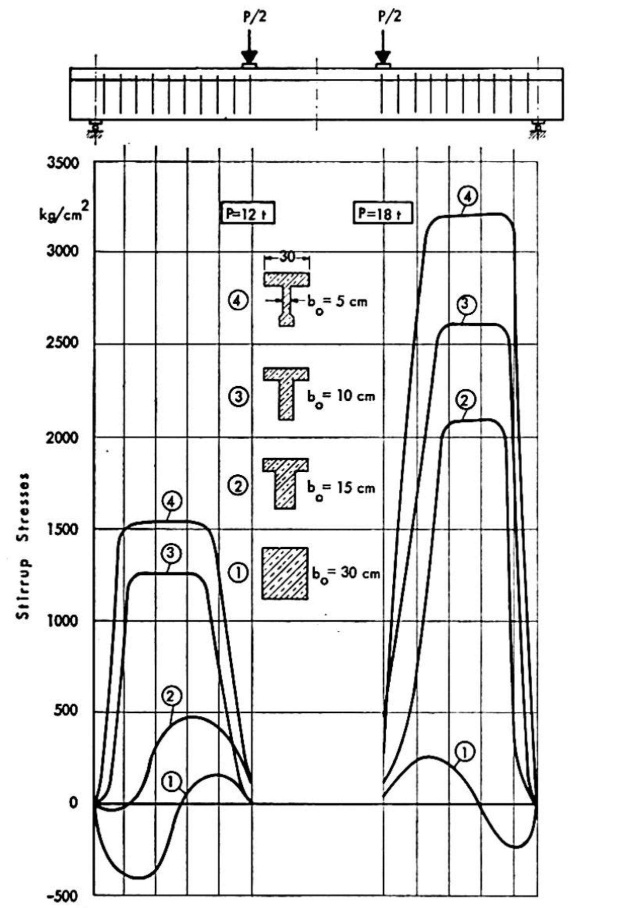

Fig. 7. Distribution of stirrup stresses in beams of various web thickness for $P = 12$ t and $P = 18$ t.

Contrary to the usual assumption (truss analogy) the stirrup stresses depend greatly on the web thickness and thus on the magnitude of the shear stresses. Ref. [1].

siderably more important for beams with thin web than for rectangular beams, even though they should theoretically be equal according to the truss analogy. Since the shear strength and the stirrup stresses depend so much on the web thickness, i. e., the shear stresses, it is obviously quite incompatible with reality to ignore the shear stresses in shear problems, as is so often done. The numerous empirical formulae for the shear strength, which do not include the shearing stresses or similar expressions cannot generally be valid and stem mostly from evaluating tests on rectangular beams only, where compared with other influencing factors the magnitude of τ is indeed often of secondary importance.

On the other hand it may be seen from Fig. 7 that for beams with thick webs the maximum stirrup stresses occur in the vicinity of maximum moment, whereas the moment has little influence on the stirrup stresses of beams with thin webs[3]). This is in agreement with the fact that thin webs fail always in the zone of maximum shear stresses.

While the shear stresses are an important criterion for such beams they definitely do not represent a really existing stress after cracking has taken place, but they give a very simple and practical indication of how much the web is strained. It is therefore perfectly justified and even recommendable to replace the theoretically derived term $\tau_0 = Q/bz$ by $\tau = Q/bh$, thus eliminating the problem of whether the lever arm z of internal forces has to be computed for working load or for ultimate load.

4. Continuous Members

Further complications but also valuable information result from investigation of continuous beams. The crack and failure pattern for a two span continuous beam of rectangular cross-section is shown in Fig. 8. This picture alone proves that the bending moment really has a significant influence because no shear cracks at all developed in the region of the point of contraflexure (0) although the ultimate shear stresses $\tau_0 \cong 25\,\mathrm{kp/cm^2}$ in this region were approximately twice that observed in similar but simply supported beams without web reinforcement (concrete strength $\beta_w \cong 350\,\mathrm{kg/cm^2}$). Conversely the distinct shear cracks in the vicinity of the load points and near the middle support have formed already at about half the ultimate load, i.e., at $\tau_0 \cong 12\,\mathrm{kg/cm^2}$ because not only the shear force but also the moment is simultaneously important. The increased shear resistance at the point of contraflexure can neither be traced back to the effect of vertical loads stresses nor to the dowel-action but only to the absence of initial flexural cracks which precipitate the formation of the shear cracks. It can therefore hardly be denied that shear failures of continuous rectangular beams result from the combined action of moment and shear.

It is however another question whether or not one should express this correlation in terms of the moment-shear-ratio. The evaluation of our tests has shown that for continuous members, there exists a similar relationship between shear strength and M/Qh as for simply supported members (Fig. 9),

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[3]) However T-beams with cut-off bars failed at considerably lower loads than similar beams with the entire tendon extending over the supports.

yet both here and there, secondary phenomena like dowel-action and vertical load stresses are involved which do not causally depend on M/Qh.

As expected, the statement made earlier according to which, mainly the shear stresses are to be governing in case of beams with thin webs, holds also for continuous members. It has to be kept in mind, however, that T-beams without lower flange act essentially as rectangular beams over intermediate supports and that the danger of shear-compression-failure is very pronounced in this zone, especially due to the bad bond characteristics of bars placed at the top.

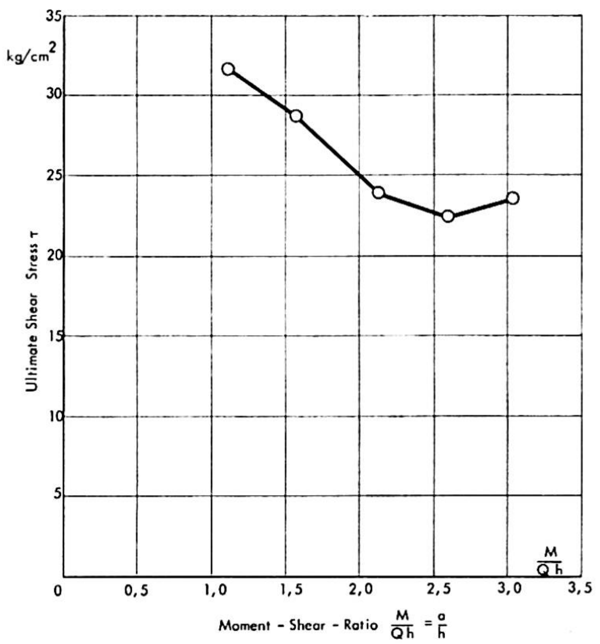


Fig. 9. Ultimate shear stresses of two span continuous beams of rectangular cross-section as a function of the moment-shear-ratio $\frac{M}{Q\,h}$.

(Beams with moderate amount of web reinforcement.)

5. Conclusions

It was attempted to clarify how the two cross-sectional forces M and Q influence the shear strength of reinforced concrete beams and whether it is appropriate to express their combined effect as a function of the moment-shear-ratio $\frac{M}{Q\,h}$.

Theoretical and experimental evidence leads to the conclusion that the bending moment does exert a significant influence on the shear strength particularly in case of beams and slabs with rectangular solid cross-sections, and that the shear stresses alone are indeed a very unsuitable and uncertain failure criterion. Even outside the region of vertical normal stresses σ_y in the vicinity of loads and supports the ultimate shear stresses can fluctuate more than 100% depending on whether the simultaneously acting moment is great or small. Contrary to the often accepted implication, the scatter of the concrete tensile strength is not the major cause for these differences. In case of very

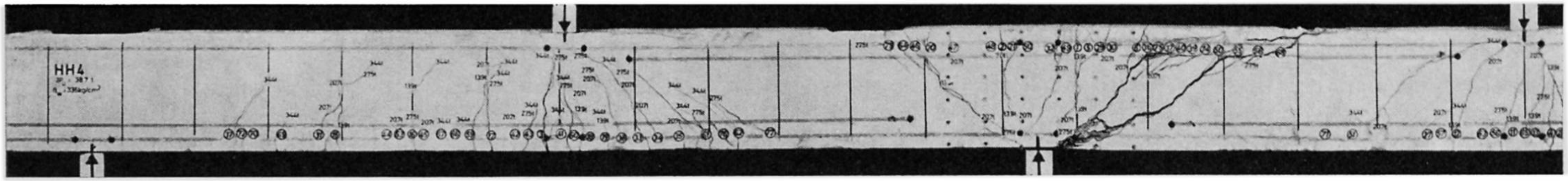
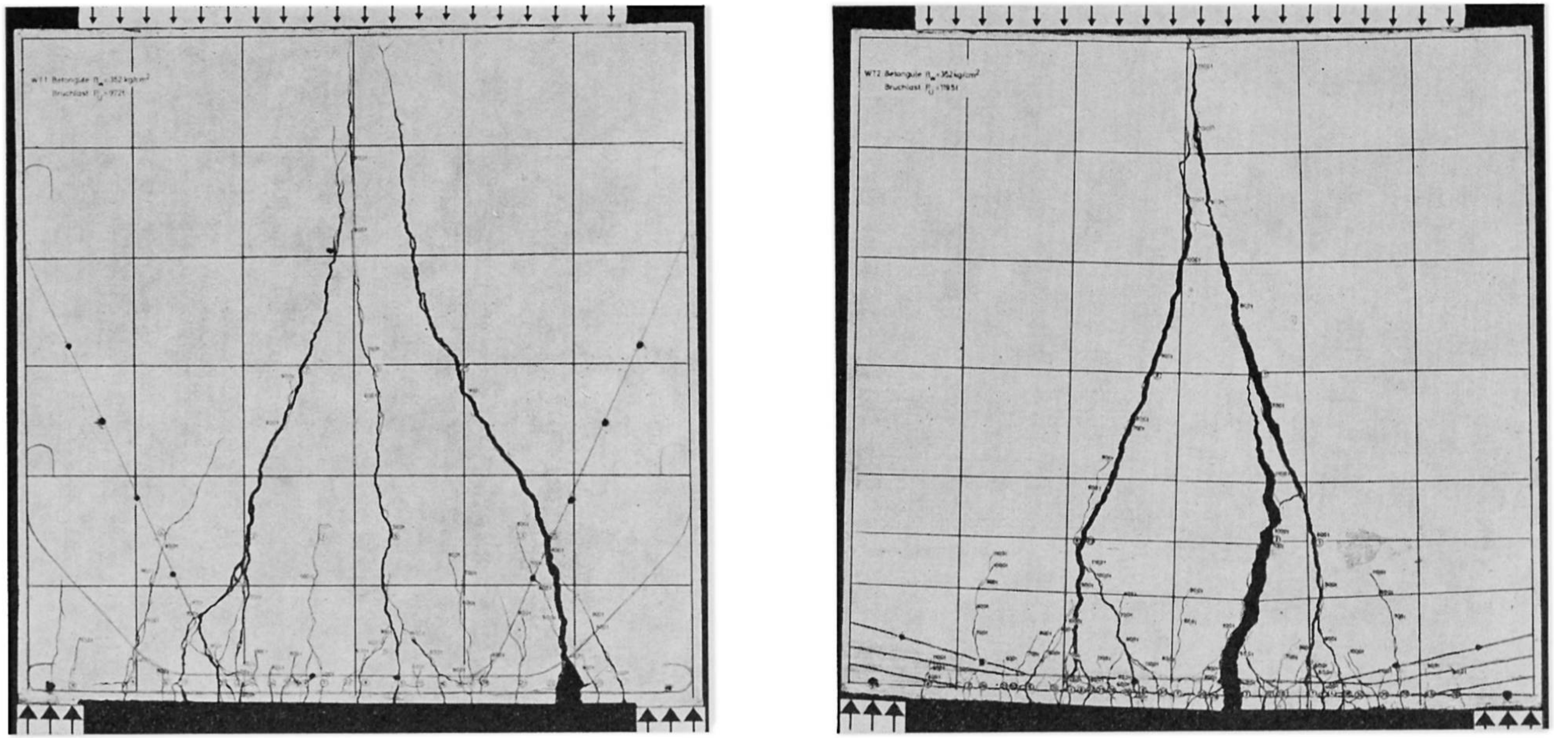


Fig. 8. Crack pattern of continuous rectangular beam with web reinforcement.

No shear cracks developed in the vicinity of the point of contraflexure, whereas shear cracks near the loads and near the intermediate support occured already at half the ultimate load.



Concrete strength:	$\beta_w = 350$ kg/cm^2	$\beta_w = 350$ kg/cm^2
Ultimate load:	$P_u = 97.2$ t	$P_u = 119.5$ t
max. crack width at $P = 80$ t:	0.48 mm	0.35 mm

Fig. 10. Tests on deep beams (walls) with different disposition of reinforcement.

short beams the ultimate shear force can rise to very high values, owing to the favourable effect of the dowel-action and the load stresses σ_y mentioned before. For continuous rectangular members, the combined action of moment and shear must be taken into account, because it can lead to a critical state of stress over the intermediate supports causing premature shear compression failure.

On the other hand in the case of beams with thin webs, the shear stresses τ represent the most important criterion as well for determining the shear strength as well as for dimensioning the web reinforcement, while the magnitude of the bending moment is here of minor importance. The widespread opinion that the notion of shear stresses should be abandoned in shear problems cannot possibly be maintained for a generally valid approach. The numerous formulae giving the shear strength without regard to the shear stresses or similar quantities are therefore inadequate.

A direct and theoretically provable influence of $\frac{M}{Q\,h}$ could only be found with respect to the strength of the shear-compression-zone. Other influencing factors, which are difficult to derive theoretically, such as dowel-action, vertical load stresses, percentage and bond of longitudinal reinforcement may in part empirically be expressed as a function of the simple and practical notion of the moment-shear-ratio, but one has to keep in mind, that this is a rather indirect approach depending greatly on the special conditions of the tests from which pertaining conclusions are drawn.

In view of these circumstances one may thus question, whether it is beneficial and feasible in practice to consider such complex implications or whether one is not better off to stick to the old concept of allowable shear stresses. In our opinion this is not the case, on the contrary, the knowledge broadened by recent investigation can lead to a more practical and more ecconomical design.

This is, for example, the case for deep beams (Fig. 10), where our tests have shown that the standard shear reinforcement by means of bent up bars is quite useless since these bars are subjected to compression and not to tension as expected from the hitherto accepted theory, whereas a similar deep beam (W 2) where the same bars extended straight over the supports developed a 20% higher strength and showed smaller crack widths and depths. Similar conditions occur in cantilevers, short beams, foundation slabs etc. In all these cases it is of primary importance to have strong and well anchored tendons, whereas one can often dispense with web reinforcement, and the concept of allowable shear stresses is of little help. Since such members constitute a large portion, if not the majority, of all the cases where shear is a problem, it would be inappropriate not to take advantage of the findings of recent research. The many investigations of the last decade certainly permit these cases in modern design without undue complications while for the other cases they open the way for more slender and daring constructions.

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Summary

The question is treated whether the shear strength of reinforced concrete beams and slabs is determined by the moment or the shear force or a combination of both and whether it is appropriate to express their interaction by means of the moment-shear-ratio M/Qh.

For members of rectangular cross-section the moment is indeed of significant influence and the shearing stresses alone do not constitute a suitable criterion for the determination of the shear strength. For beams with thin webs, however, the shearing stresses are of primary importance both for predicting the shear strength and for dimensioning the web reinforcement.

A direct and theoretically given influence of M/Qh could only be found with respect to the strength of the shear compression zone, while other phenomenae, usually attributed to the influence of M/Qh are a function of other factors such as vertical normal stresses due to support and load conditions and the dowel action of the longitudinal reinforcement.

Résumé

Il est examiné si la résistance au cisaillement de dalles et de poutres en béton armé est déterminée par le moment, l'effort tranchant ou une combinai-

son des deux, et s'il est légitime d'exprimer leur action commune par le rapport moment — cisaillement M/Qh.

Il est montré que pour les pièces de section rectangulaire le moment a une importance prépondérante et que les efforts tranchants ne peuvent constituer à eux seuls un critère sûr pour la détermination de la résistance au cisaillement. Pour les pièces à âme mince, par contre, les contraintes de cisaillement sont déterminantes pour l'évaluation de la résistance au cisaillement et pour le dimensionnement de l'armature de cisaillement.

Une influence directe, justifiable théoriquement, du rapport M/Qh n'a été constatée que sur la résistance de la zone comprimée soumise au cisaillement, tandis que d'autres phénomènes, généralement attribués à l'action du rapport M/Qh, résultent d'autres facteurs tels que contraintes verticales normales dues aux appuis et aux charges et effet de cheville des armatures longitudinales.

Zusammenfassung

Der Beitrag behandelt die Frage, welchen Einfluß das Moment und die Querkraft bzw. die Schubspannungen auf das Schubbruchverhalten von Stahlbetonbalken und -platten haben und ob es zweckmäßig sei, deren gemeinsame Wirkung gegebenenfalls durch das Momenten-Schub-Verhältnis M/Qh zu erfassen. Es wird gezeigt, daß das Biegemoment vor allem bei Rechteckquerschnitten von maßgebender Bedeutung ist und daß die Schubspannungen allein hier ein sehr ungeeignetes Maß für die Abschätzung der Schubbruchgefahr darstellen. Für Balken mit dünnen Stegen hingegen ist die Größe der Schubspannungen das wesentlichste Kriterium, sowohl für die Beurteilung der Schubtragfähigkeit als auch für die Bemessung der Schubbewehrung.

Ein direkter, theoretisch begründeter Einfluß von M/Qh konnte nur bezüglich der Festigkeit der Schubdruckzone gefunden werden. Die bei kleinen M/Qh-Werten beobachtete hohe Schubtragfähigkeit ist dagegen weniger eine Folge der gegenseitigen Wirkung von Moment und Querkraft, sondern es spielen hier andere Einflüsse wie Lasteintragungsspannungen und Verdübelungswirkung der Längsbewehrung eine maßgebende Rolle.