A critical look at the recommendations regarding shear in the Draft Unified Code

R. TAYLOR & H. SEKI

Dr H. P. J. Taylor, Cement & Concrete Association

Mr Taylor has presented the results of six tests which he says demonstrate that in some particular cases, T beam floors designed in accordance with the Draft Code have inadequate shear resistance.

25. I have been privileged to see a copy of the latest version of the Code and, in it, the current stresses are lower than those shown in Fig. 5 although not as low as Mr Taylor would require. The Code stress at 3% of steel is 1·0 N/mm^2, not 1·1, at 2% it is 0·95, not 1·0 and at 1% it is 0·75, not 0·8, all for 40 N/mm^2 concrete. This is presumably in part because of the experience that Mr Taylor reported.

26. Looking at the tests themselves, it is noteworthy that certain of the Code regulations with regard to cover and bar spacing are violated and this probably explains why the beams with large steel percentages had failures influenced by bond. I would be interested to know if allowance has been made for the dead load of the beams and test rig in assessing the diagonal cracking load as, if this was ignored, it could also have the effect of apparently reducing the strength of the beams.

27. Mr Taylor will no doubt agree that the results of his six tests will have to be considered with the experience we have of many thousands of square feet of flooring designed to the stresses in CP 114, stresses that are significantly higher than any of those discussed here, with no serious ill effects.

Dr R. N. Swamy, Department of Civil and Structural Engineering, University of Sheffield

Messrs Taylor and Seki have highlighted the problem of defining the parameters that need to be considered in the design of reinforced concrete beams against shear failure. As pointed out by the Authors, it is now well established that concrete strength, the percentage of longitudinal reinforcement and the moment–shear ratio (a/d_1 or M/Qd_1) are the most significant parameters that influence the shear resistance of rectangular beams. With T sections, however, the shear strength and the mode of failure are also influenced by a fourth parameter, namely, the size of the compression flange.

29. It is true that an increase in the a/d_1 ratio produces a decrease in the diagonal cracking and ultimate shear strength; however, in the range of long shear spans this decrease due to the moment–shear ratio is insignificant, just as the effect of the percentage of reinforcement at constant a/d_1 ratio is also relatively small. It is therefore only rational that the effect of the variable a/d_1 should be eliminated in design by adopting safe limiting values for the nominal shear stress as pointed out by the Authors.

30. There are, however, other considerations with regard to the percentage of longitudinal reinforcement apart from its influence on shear strength. With T beams, there is the further point that this change in shear resistance with a/d_1 ratio and percentage of longitudinal steel should be considered in conjunction with the size of the compression flange, as this is an additional parameter which influences shear strength.

31. To illustrate these comments, I would like to quote the results of two groups of tests which form part of an extensive investigation on the shear resistance of T beams.[6] Group 1 consisted of 13 T beams with three different percentages of

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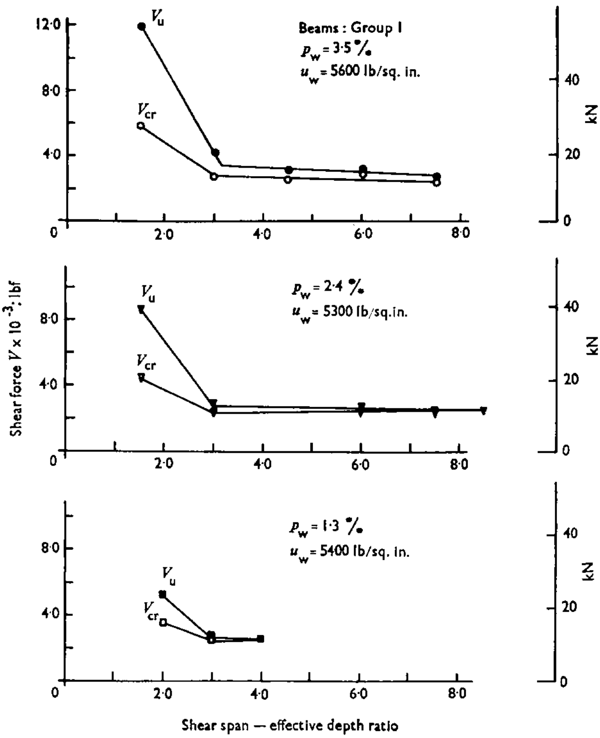

Fig. 6. Influence of shear span–effective depth ratio on the shear cracking and ultimate shear strength of T beams without web reinforcement

longitudinal steel but without web reinforcement and tested at various a/d_1 ratios. Group 2 consisted of four beams without web reinforcement, with varying percentages of longitudinal steel but tested at the same a/d_1 ratio of 3·5.

32. All the beams of Groups 1 and 2 were of identical cross-section and had constant flange width–web width ratio (3·0), effective depth–flange thickness ratio (3·75) and flange thickness–overall depth ratio (0·222). The beams all had the same effective depth, and the longitudinal steel was provided in one layer. The average concrete strength was 37·5 N/mm². The beams were all of the same length, and tested at the same span under two point loads applied over the web width at various a/d_1 ratios.

33. The results of the tests of Group 1, showing the influence of the a/d_1 ratio, are shown in Fig. 6. It is clearly seen that the effect of a/d_1 ratio is significant for beams of short shear span only ($a/d_1 < 3{\cdot}0$), but almost non-existent for beams of large shear span ($a/d_1 > 3{\cdot}0$). This is true for both the shear cracking strength and the ultimate strength. The reasons for not including the a/d_1 ratio as a variable in the design recommendations of the Draft Code of Practice are clear and seen to be well justified.

34. The influence of the percentage of longitudinal steel for various moment–shear ratios is shown in Fig. 7. It is seen that the influence of the tension steel is

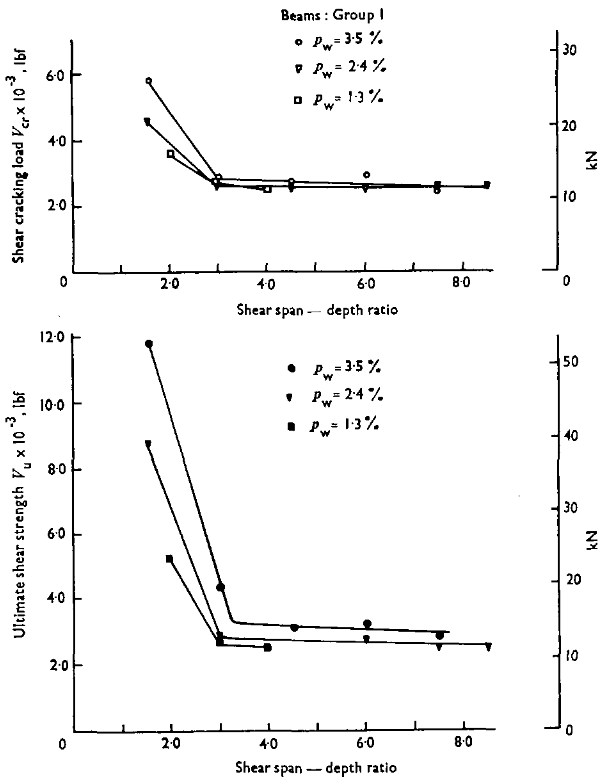

Fig. 7. Influence of main steel ratio on the shear cracking and ultimate shear strength of T beams without web reinforcement

significant for beams of short shear span, at both the diagonal cracking and ultimate stages. For beams of long shear span, it is seen that at the diagonal cracking stage (assuming a constant definition for all the tests, namely, the load at which an independent shear crack in the web became visible), the influence of the longitudinal steel is again almost nil. At failure, however, there is a small but measurable influence of the longitudinal steel on the ultimate shear strength, and this increase is much more clearly distinguishable than the decrease due to the a/d_1 ratio. It may also be noticed from Fig. 7 that the range of a/d_1 ratio in which shear failure occurs depends upon the amount of longitudinal steel.

35. Assuming the shear strength to be relatively constant for a/d_1 ratios greater than 3·0, tests of Group 2 were carried out at $a/d_1 = 3{\cdot}5$ to represent the shear behaviour of beams of large shear span, the only variable in these tests being the percentage of longitudinal steel. The results of these tests are shown in Table 2 which shows clearly an increase in the ultimate shear strength with an increase in the amount of longitudinal steel.

36. In all the discussion here, emphasis is laid on the ultimate shear stress rather than the shear stress at the shear or diagonal cracking stage. Apart from the well known variability of this load stage because of the numerous definitions adopted by the various investigators, there is probably only one single overriding parameter

Table 2. Results of tests on T beams, Group 2

Beam no.	Tensile steel ratio $p_w\%$*	a/d_1	Concrete cube strength, *lb/sq. in.*	Shear cracking strength, V_{cr}, *lbf*	V_{cr}/V_{crm}†	Ultimate shear strength V_u, *lbf*	Ultimate shear stress v_u, *lb/sq. in.*	V_u/V_{um}†	V_u/V_{cr}	Mode of failure‡
ST 1	0·87	3·50	5300	2600	1·00	2600	231	1·00	1·00	DT–FT
ST 2	1·95	3·50	5300	3250	1·25	3500	311	1·35	1·08	DT
ST 3	3·50	3·50	5300	3375	1·30	4000	356	1·54	1·18	DT
ST 4	5·50	3·50	5300	3375	1·30	4500	400	1·73	1·33	DT

*Based on web width
† Strengths compared to those with the lowest $p_w\%$
‡ DT—Diagonal tension
FT—Flexural tension

Conversion factors
1 lb/sq. in. = 0·0069 N/mm²
1 lbf = 4·448 N

which influences the initial shear resistance of concrete, namely, its tensile and compressive strength.

37. The a/d_1 ratio primarily distinguishes the beams of short shear span from those of large shear span, i.e. between the two predominant structural actions involved, namely, the arch action and the beam action. All other components which carry shear such as aggregate-interlock, dowel action and, indeed, the compression zone at the head of the diagonal crack, all become effectively significant only after diagonal cracking as failure is approached. What one ought to look at, from a design point of view, is therefore not the diagonal cracking stress but the ultimate shear stress.

38. The results of the tests of Groups 1 and 2 are shown in Fig. 8 for a/d_1 ratios equal to or greater than 3·0. In Fig. 8 the ultimate shear stress is plotted for reasons already discussed, and the permissible stress according to the Draft Unified Code is also shown for comparison. It may be seen that the influence of the longitudinal reinforcement is clearer than that of a/d_1 ratio shown in Fig. 6. In spite of the scatter, it seems appropriate that the percentage of longitudinal reinforcement should be considered as a variable in the design against shear failure.

39. I have mentioned that, in T beams, the effect of the percentage of longitudinal steel and the a/d_1 ratio should be considered in relation to the size of the compression flange. Fig. 9 shows the influence of the longitudinal steel on the ultimate shear stress of T beams without and with vertical stirrups for various sizes of compression flange. The beams shown in Fig. 9 were all tested at a/d_1 ratio = 3·5 and had constant effective depth–flange thickness ratio (2·13) and flange thickness–overall depth ratio (0·30). The beams had all the same length and effective depth, and were tested at the same span under two point loads applied over the web width. The designed concrete strength was about 34·5 N/mm², and the results for beams without web steel have not been corrected for variations in concrete strength. However, the influence of the percentage of longitudinal steel is unmistakable.

40. Another important contribution of the longitudinal steel to shear resistance comes through its dowel action. In the tests from which Fig. 9 was derived, it was

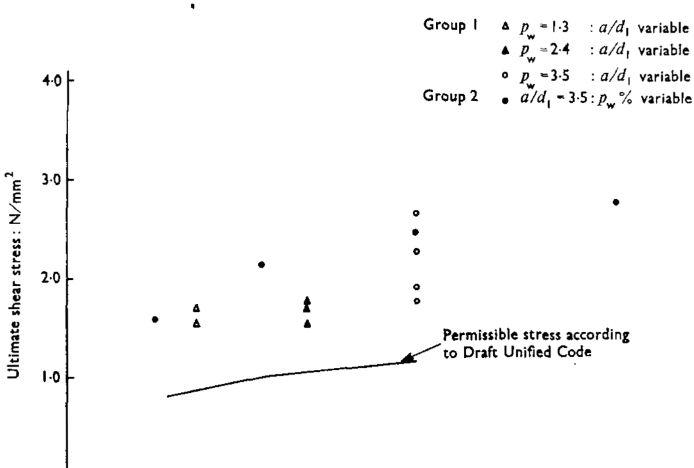

Fig. 8. Effect of the amount of reinforcement on the ultimate shear strength of T beams without web reinforcement

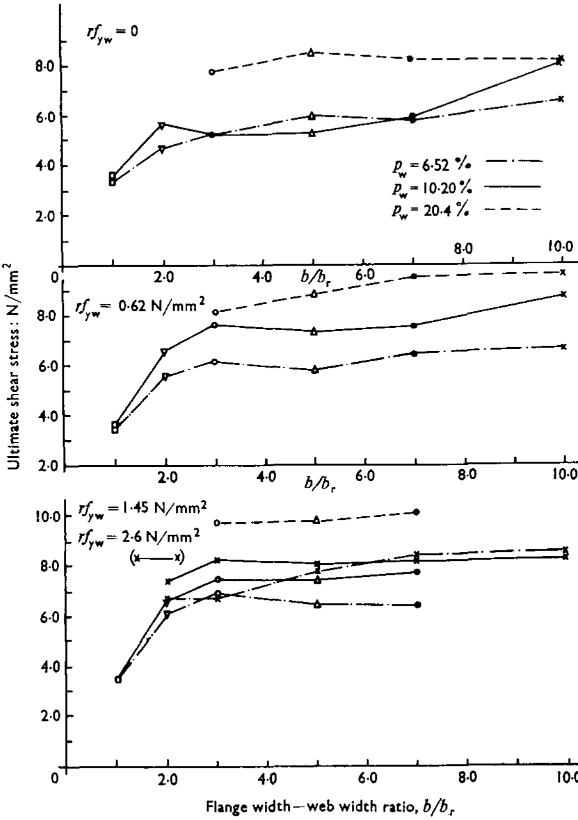

Fig. 9. Influence of longitudinal steel ratio on the ultimate shear strength of T beams of various flange widths

found that the diagonal crack occurred at higher loads with higher percentages of longitudinal steel when it was provided in two layers. The effect of dowel action was more noticeable after diagonal cracking due to the shear rigidity of the multi-layered tensile reinforcement, both in respect of bond cracking and the ultimate strength. Such beneficial effects due to dowel action with increased percentages of longitudinal steel become more evident even when a small amount of web reinforcement is provided (Fig. 9). This increased dowel activity because of the rigidity of the multi-layered reinforcement has been observed by other investigators.[7]

41. It has been argued that the flexural design stress of the longitudinal steel, rather than the percentage of longitudinal reinforcement, should be considered in deciding the permissible shear stresses. This is only partly true since part of the shear is carried by dowel action and aggregate interlock—and the former is dependent more on the percentage of tension steel (among other parameters) than on the stress in the reinforcement. In any case it is not possible to predict the steel stress in the vicinity of the diagonal crack since it no longer follows the variation of bending moment after the diagonal cracking stage. Further, assumptions of uniform steel

strain in the failure zone are equally invalid. In any case, the diagonal crack does not always develop from flexural cracks.

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The Paper is of considerable interest as it relates directly to structural safety, and gives a very unfavourable impression of the Draft Code's recommendations. Although I would favour the provision of shear reinforcement in all T beams, whether or not they form hollow-tile floors, I cannot agree with majority of the Authors' views.

43. In regard to the particular type of member tested, the overall trend of available data seems to support the Code approach, which also appears to be more rational than the alternative suggested in the Paper. The numerical levels of permissible stresses are more debatable but are certainly not so simply inappropriate as implied by Taylor and Seki.

44. Figure 10 shows results from three series of tests of T beams.[8] It can be seen clearly that their trend supports the Code approach, according to which the influence of the main steel ratio on the nominal ultimate shear stress is much greater

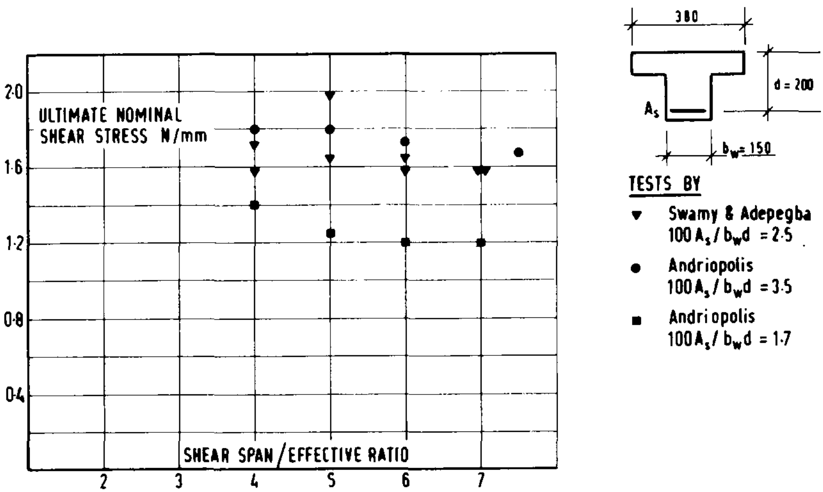

Fig. 10. Results from tests of T beams

than that of the moment–shear ratio, for all but the shortest spans. The relatively small effect of the latter can best be accounted for by the Shear Study Group's principle, that safe results will be obtained if the shear stress is related to the highest moment–shear ratio, at which shear cracking can precede flexural failure.

45. The shear stresses obtained in Taylor and Seki's tests lie below those of Fig. 10, and there are various possible explanations for this. First, the detailing of their beams did not comply with the Code's minimum requirements for cover and bar spacing. Second, and perhaps more significantly, although the strength of the main reinforcement is not reported it seems likely that the beams reached their full flexural moments, since the maximum steel stresses of up to 500 N/mm^2 are in excess of normal characteristic values.

46. The proposed alternative basis of determining design shear stresses as a function of design stresses for the main reinforcement is not satisfactory. If the reinforcement stress to be used is the permissible value related to the yield stress, it will be totally inappropriate where, as in simply supported beams, the sections of

maximum shear and maximum bending are not coincident. On the other hand, if actual stresses are to be used, the need for their determination will make shear calculations unreasonably complex, particularly in statically indeterminate structures with variable configurations of loading.

47. The final point at issue is the numerical level of the permissible stresses. It is true that in many cases the Draft Code will give low materials safety factors—even as low as 1—thus making the overall factor equal to the load factor of 1·4 to 1·6. This factor is higher than the corresponding one given by the present Code CP114, and not very different from that of Taylor and Seki's proposal. Their comments in § 18 are misleading in this context, since the 'permissible values' of Fig. 5 involve a materials factor of only 1·2 and there is no further factor 'applied to the characteristic strength of the concrete'.

48. The use of these low safety factors is somewhat worrying, but seems reasonable in the context of Limit State Design. As is well known, shear strength is only slightly affected by concrete strength, so the factor required to be applied to a characteristic shear strength, to allow for a one-third drop in cube strength, is only about 1·15.

49. In view of the preceding statements and Taylor and Seki's results, I would suggest a modest reduction of shear stresses by about 20% bringing them into line with the values for flat slabs, and the provision of nominal shear reinforcement in T beam and hollow-tile floors. Such reinforcement, which need not necessarily take the form of conventional stirrups, will be required in many cases, even with the present recommendations, as the low percentages of main steel commonly used in negative moment regions will allow only very low shear stresses.

Messrs Taylor and Seki

Dr Taylor has raised the philosophical argument that floors designed in the past to the shear stresses in CP 114 have proved satisfactory and that this is relevant to determining the shear stresses in the new codes of practice. We do not agree. The satisfactory behaviour of structures is certainly relevant to determining the overall load factors which should be adopted in the code, but having decided the load factors to be applied to strength in bending (and of course structures have been satisfactory in this way too), then the philosophy should be that the strength in shear should be at least as great. Only test data can determine this.

51. In considering the behaviour of structures for the determination of the overall load factors, it must be remembered that the floors in the past have had an extra in-built safety factor in that the assumed live loading has usually been much in excess of the actual live loading. Research into actual loadings is currently in progress and could well result in the design floor loadings being much reduced for many types of building. The extra safety factor will therefore disappear. The recommendations for the design of concrete structures should be well prepared for this eventuality.

52. It is heartening to learn from Dr Taylor that some reduction of the 'permissible' stresses has been made for structures generally from those originally proposed by the Shear Study Group. However, we understand that for the type of structure which our tests were intended to simulate, namely hollow block and voided slab floors, these general stresses are to be multiplied by a magnification factor (which varies and has a maximum of 1·2). Thus the stresses with which the test data should be compared are somewhat higher than those quoted by Dr Taylor.

53. It seems likely that in adopting these enhanced stresses for floor slabs the Code Committee has been influenced by the type of argument put forward by Dr Taylor. As already discussed, this is a false argument. It is also unfortunate that the Committee has related (by using the same magnification factors) the shear behavior in hollow-tile floor slabs to that of punching shear in flat slabs. The types of failure for the two conditions are quite different and there is no reason whatsoever why the design stresses should be the same.

54. Dr Taylor queries the method of calculating the shear stress for the beams. There was no test rig weight and it will be found that the shear stresses caused by the dead loads of the beams at the positions of the diagonal cracks are quite negligible.

55. **Dr Swamy** is to be thanked for presenting some additional data. However, our interpretation of his data would be different from his. We do not see how he can conclude from his Figs 7 and 8 that the effect of the percentage of longitudinal reinforcement is sufficiently important for it to be considered as a variable in design.

56. He appears not to have understood one of our main points. We contend that there is even less effect of the amount of reinforcement on shear strength when the beams are tested, such that at shear failure the reinforcement is at, or near, its maximum flexural stress. If this is applied to Dr Swamy's data in Fig. 7, the beam of $p_w = 1{\cdot}3$ which failed in shear at the $a/d = 4$ should be compared with the beams of $p_w = 2{\cdot}4$ at the a/d ratios greater than 7. The failure stresses are the same. The beams having $p_w = 3{\cdot}5$ would have to be tested at a/d ratios greater than those quoted to be comparable but extrapolating his existing data would again give similar stresses to before. The results quoted by Dr Swamy would therefore support our first conclusion that there is no justification for including the percentage of reinforcement as a design variable.

57. Our second conclusion, that the stresses permitted by the Draft Code are too high for T beams with high percentages of reinforcement, is not supported by Dr Swamy's results. Unfortunately he does not give sufficient details of his beams to enable a proper study to be made, but undoubtedly the difference is due to the size effect and the shape effect, in particular the size of compression flange in relation to the web. The beams in our tests were intended to simulate hollow-block floors where the flange is usually designed to be the minimum thickness possible. A larger flange would have produced higher strengths. Dr Swamy thinks that this is a parameter which should be taken into account in design. Our own feeling is that this would make design unnecessarily complicated.

58. Research workers should not attempt to cover all the possible variables that can occur. However, they should be aware of their effects and then in their tests adopt the worst cases that can occur in practice so that the simplified rules which must be adopted are on the safe side. The Shear Study Group were on the right lines by adopting values of shear strength from the tests of maximum shear spans. It was unfortunate that they did not realize that certain types of T beams can give worse effects than rectangular beams.

59. **Dr Regan** states that he disagrees with our views. Nevertheless he concludes that the permissible shear strength in the Draft Code should be reduced. There is therefore a good measure of agreement on this point, although we would not go so far as his suggestion that nominal shear reinforcement should be required in hollow-tile floors, a proposition which we feel industry would find difficult to accommodate.

60. Some criticism is made of our experimental work in an attempt to explain the low values of shear stress, even to the extent of suggesting that the beams really failed in flexure. It should be clear from the load–deflexion curves and the photographs at collapse that the beams failed in shear without reaching their ultimate moment capacity. That the reinforcement had a yield stress slightly higher than normal (but nevertheless less than that of some steels currently being used) is certainly one of the reasons for the low values and must be the final proof that, rather than the percentage of reinforcement being relevant to shear strength, the important factor is the maximum stress the reinforcement is required to attain.

61. Further test data are put forward by Dr Regan which are intended to support the viewpoint that the percentage of reinforcement is sufficiently important to include as a design variable. Again, we think that Dr Regan does not understand our point of view, particularly that of the difference in behaviour between rectangular beams and T beams. A further explanation is therefore necessary.

62. We indicated in § 14 of Technical Note 61 that the state of flexural cracking

is important in the development of diagonal cracking in a beam and that the maximum stress in the reinforcement is some measure of this flexural cracking. However, we acknowledge that for rectangular beams the flexural cracking is not truly represented by the stress in the reinforcement. As the percentage of reinforcement in a rectangular beam increases, the depth of the compression zone increases, and as the value of the percentage becomes high, with a consequent deep compression zone, the flexural cracks remain short and fine. With such over-reinforced beams, increases in the percentage of reinforcement produce only small changes in the ultimate resistance moment and flexural failure can occur while the stress in the reinforcement is well below the yield stress. Shear strength results have been obtained for such beams, forcing shear failure by testing at relatively short shear spans.

63. Contrast this behaviour with that of T beams having thin webs and wide, thin flanges. In such beams the depth of compression zone remains small for wide variations in the percentage of reinforcement. Thus the lever arm hardly changes and the ultimate resistance moment is approximately proportional to the amount of reinforcement. Suppose such a T beam has a percentage of reinforcement p and the maximum a/d_1 ratio causing shear failure is x. If there is a similar beam having a percentage of reinforcement $2p$, then it will be possible to obtain a shear failure by testing the beam at an a/d_1 ratio of approx. $2x$. The flexural cracking of the two beams will be similar with cracks up to the underside of the flange, the maximum flexural stresses will be similar, and the shear failure loads will be approximately the same. But for the T beam with the high percentage of reinforcement this shear failure load will be appreciably lower than the shear failure load of the comparable over-reinforced rectangular beam; it is tests on the latter from which permissible design stresses have been determined.

64. We have used the two extreme types of beam—the rectangular beam and the thin-webbed, thin-flanged beam—to illustrate the point. Of course, there will be T beams which are intermediate between these cases. The T beams quoted by Dr Regan come into this category.

65. Since the T beams used in hollow-tile floors can be, and usually are, of the thin-webbed, thin-flanged type we feel justified in reiterating our warning that for such beams with high percentages of reinforcement the stresses recommended by the Draft Code are too high and do not eliminate the possibility of sudden shear failure occurring before the flexural failure.

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