Shear behaviour of structural lightweight concrete T-beams without web reinforcement

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I am surprised by the comment in § 35 that, 'It is seen that the method in the British code is more conservative than the American method and significantly underestimates the ultimate shear strength'.

46. The comparison of the two codes seems to be made on rather false premises as it would appear that British design ultimate stresses, including the partial safety factor for materials γ_m have been compared with unfactored American ultimate values. To obtain a truer comparison, I considered the gravel concrete beam with the lowest ratio of main steel ($100A_s/b_wd = 4.94$) and $a/d = 6.0$. From CP 110⁴ the design ultimate stress is 1.00 N/mm^2 . From ACI 318-71,³ with the cylinder strength f'_c of the concrete taken as 32 N/mm^2

$$v_c = 0.158\sqrt{f'_c} + 17.24\rho_w(V_u d/M) = 1.04 \text{ N/mm}^2 \quad (1)$$

Thus in this instance for any reasonable value of γ_m for concrete CP 110 is clearly less conservative than the ACI code.

47. It is true that with equation (1) the American values would increase with decreasing shear spans but CP 110 would give no such increase until $a < 2d$. However, ACI 318-77²⁷ adopts the same treatment of span/depth effects as CP 110. The American treatment of all lightweight concretes is slightly more conservative than the British as the reduction factor applied to the basic dense concrete stresses is 0.75 compared with 0.8 in CP 110.

48. More important perhaps is the suggestion that CP 110 significantly underestimates the shear resistance available in members of the type in question. Again the issue is confused by the implied comparison of factored design values and actual experimental strengths.

49. The Authors use very small beams. Tests by Walraven²⁸ on lightweight aggregate beams of varying depths indicate a continuous scale effect for overall depths in the range 150-750 mm (the effect probably extends further but these were the limits of the sizes tested). Walraven demonstrates that good agreement with his own and other test data can be obtained using a scale factor proportional to $d^{-0.4}$. On this basis the Authors' test strengths would need to be reduced by 29% to apply to members with $d = 300 \text{ mm}$ and by 46% for $d = 600 \text{ mm}$.

50. For series S3 and O3, with $a/d = 6$, shear cracking as defined by the Authors would seem to have occurred with $V/bd \approx 1.2 \text{ N/mm}^2$, and the ultimate shear stresses appear to have been about 1.65 N/mm^2 and 1.9 N/mm^2 for the Solite and gravel beams respectively. I doubt whether cracking at about two thirds of the ultimate load in such long spans would normally be regarded as shear cracking and prefer to work in terms of the ultimate values.

51. Realistic prototype members of this type would probably have effective depths of up to about 300 mm, and for them the strengths could be expected to be reduced by about 29% to 1.17 N/mm^2 and 1.35 N/mm^2 . If these values are compared with the values in CP 110 of 0.8 N/mm^2 and 1.0 N/mm^2 for the lightweight and gravel members, the partial safety factor for materials seems to be about 1.4, which is reasonable.

52. It is true that for members with a/d less than about 4 CP 110 can err on the safe side in relation to ultimate loads, but any allowance of higher stresses would be reliant both on the behaviour of members in a seriously damaged state and on very good conditions of bearing and end anchorage of reinforcement, which are easier to ensure in laboratory tests than in many practical applications.

53. Another factor which has at least some influence is the very high ratios of main steel used in the tests. Values of $100A_s/b_wd$ vary from 4.14 to 8.11 and are all beyond the upper cut-off of table 5 in CP 110. It would not be surprising if the provision of such large amounts of main steel had a beneficial effect, and it is questionable whether or not these heavily reinforced specimens are representative of normal structural members.

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We do not agree with Dr Regan's comment on the comparison of CP 110 and ACI codes with our experimental results. When designing for shear, the shear stress carried by the concrete can be based only on the values of v_c as given in the codes, irrespective of the partial or other safety factors implied in the derivation of the v_c values. These are the values available to the designer.

55. Figure 7 is based on the experimental shear stresses as measured from the tests for a whole range of a/d values and the values of v_c as given in the British and American codes. The conclusions we have drawn based on this figure are thus clear and unambiguous. Comparisons based on a single value of a/d are meaningless as shown in Fig. 7. A truer comparison can be obtained only by comparing the actual shear reinforcement required for a given set of beam conditions because, apart from safety factors on concrete and steel stresses, the ACI code includes a capacity reduction factor ϕ in calculating v_u . We therefore regard Dr Regan's arguments as untenable.

56. The effect of size on shear strength is well established. Although the beams we tested are small, they are based on extensive similitude tests¹⁹ and the results are therefore valid for larger laboratory size beams. All laboratory tests are bound to be based on small test specimens compared with those in situ.

57. There have been two sets of data on the influence of beam depth on the shear capacity of lightweight concrete beams. Dr Regan has quoted Walraven,²⁸ but his results are not directly applicable to the data in the Paper because his tests were carried out using a single a/d ratio and the geometry of the beams in two of the series (namely $200 \text{ mm} \times 150 \text{ mm}$ and $200 \text{ mm} \times 750 \text{ mm}$) is not truly representative of practical sizes. Moreover, the equation for shear capacity

is based on tensile splitting strength (cubes) whereas the test data showed not only comparatively high values of tensile splitting strength but also increases of about 12% and 33% in tensile splitting strength for a decrease in compressive strength of about 8% in series B and C respectively. The effects of depth as quoted by Dr Regan are therefore of doubtful validity, despite the size effect.

58. In beams of large shear span (e.g. $a/d = 6$), the initial shear cracking as distinct from flexural cracking occurs at a higher proportion of the ultimate load. It is immaterial whether or not one considers this as shear cracking load because designs are based on the ultimate limit state. However, such information gives a better appreciation of the state of the beam at service loads.

59. The ratios of steel reinforcement in the beams are not high when based on flange width. If the presence of the compression zone is ignored, the reinforcement ratios will be high.¹²

60. Dr Regan implies that the provision of adequate anchorage for reinforcement and bearing for the member cannot be relied on in practical designs. These are other aspects of construction which may also be unreliable; the most important is the variability of the nature of loading. The influence that these factors should have on design stresses is not discussed in the Paper.

References

27. AMERICAN CONCRETE INSTITUTE. *Building code requirements for reinforced concrete*. American Concrete Institute, Detroit, 1977, ACI standard 318-77.
28. WALRAVEN J. C. *The influence of depth on the shear strength of lightweight concrete beams without shear reinforcement*. Stevin Laboratory, Delft University of Technology, 1974, Report 5-8-74.