A survey of the problem of lateral instability in reinforced concrete beams

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In collecting together the available evidence of lateral instability in reinforced concrete beams, Professor Marshall has drawn attention to a little investigated topic of great interest. His analysis emphasizes the important variables of the problem, but the following discussion may indicate a need to revise his suggested critical values.

39. Professor Marshall suggests that the beams in Series 1, of the tests by Smith (Table 3), may have failed by flexure and not instability. The reinforcement of these beams had a 0.2% proof stress of 81 000 lb/sq. in. and, based on an average cube strength of 4840 lb/sq. in., the calculated moment of resistance is 10 560 in. lb, which would give a flexural failure load of 595 lb. Moreover, there was an additional beam specimen with the same details and test conditions, except that it was restrained against lateral buckling by small rollers at midspan. It failed at an ultimate moment of 10 830 in. lb. This is considered conclusive evidence that the other beams of Series 1 did not fail in flexure.

40. Although these beams have on average lower values of ML/ub^3d_1 than any others reported, there is no conflict between the results and those of the companion Series 2 beams or of the Building Research Station tests according to the concept of an interaction between flexure and stability. This is illustrated by the comparison of Rankine interaction moment against test moment given in Fig. 3. Of the two very low test results (beams 13 and 14) which are marked on the plot, one was loaded with lateral restraint up to 530 lb before it was retested without restraint and allowed to buckle. Thus it had sustained much flexural damage. The cause of the other low failure must be attributed to some unknown imperfection of the specimen. The average values of M/ubd_1^2 for Series 1, 2 and the BRS tests, respectively, are 0.233, 0.140 and 0.109, which indicate the differing degrees of 'flexural deterioration' at buckling. This affects the quantity (EI_yGJ) in equation (1b) of which ML/ub^3d_1 is a convenient measure.

41. There is an error in the values of ML/ub^3d_1 given in Table 2. These should all be multiplied by $\frac{2}{3}$ which makes them on average less than those of Series 2, in spite of lower M/ubd_1^2 . This is due to the K -factor for third point loading (equal to 3.5) used in these tests, compared with 4.24 for central point load.

42. The Rankine interaction is also plotted in Fig. 3 for the tests given in Tables 3 and 4. Sant and Bletzacker's beams 36-1, 2 and 3 and 30-3 would fit well into the first plot, but Massey's beams give higher test values, along with the other beams by Sant and Bletzacker. The possibility of restraint offered by the kind of end support arrangements used in the latter tests has already been described.¹³ In the case of Massey's tests, which alone show any consistent difference, the only clue would appear to be the absence of shear force with this manner of loading.

Professor Marshall

43. I am grateful to **Professor Smith** for the interest he has shown in the Paper.

44. In calculating the flexure failing load for the beams of Series 1 of the tests by Smith, I assumed mild steel with a yield point of 40 000 lb/sq. in. The use of high tensile steel of the properties mentioned by Professor Smith will certainly give a higher

Fig. 3. Comparison of Rankine interaction moment against test moment for beams failing by lateral instability

M_T reported failure load

M_u flexural capacity

M_{cr} buckling load calculated from Michell's equation, based on initial torsional and flexural stiffnesses ($E = 4 \times 10^6$ lb/sq. in., $G = 1.8 \times 10^6$ lb/sq. in.)

$$M_R = M_u M_{cr} / (M_u + M_{cr})$$

value of the flexure failing load. This means that the ML/ub^3d_1 factor for such beams should have been included in the summary given in Table 6 reducing the average value of this factor.

45. The use of the Rankine interaction approach and the results obtained therefrom is interesting. This seems to be in conflict with the work done by Cowan⁸ but appears to give good results in this case.

46. Professor Smith is quite right when he states that the values for the factor ML/ub^3d_1x given in the BRS tests should all be multiplied by $\frac{4}{3}$. The reason however is not because of the difference in the K -factor for third point loading but simply because the bending moment for third point loading is only $\frac{2}{3}$ of the value for central point loading—I confess to overlooking the fact that this was the type of loading used.

47. The result of the changes resulting from Professor Smith's contribution is to reduce the average value of M_{cr} of all the tests from the value of $156 ub^3d/L$ given in the Paper to $140 ub^3d/L$, not a very significant difference.

Reference

13. SMITH R. B. L. Discussion of a paper by Hansell W. and Winter G. Lateral stability of reinforced concrete beams. *J. Am. Concr. Inst.*, 1960 (March).