complications of diagonal tension or flexural cracking, and therefore the calculated horizontal shear stresses are the ultimate values. These ultimate stresses compared favorably with values obtained using Zia's failure envelope in the manner recommended by the authors for the initially uncracked concrete. An inevitable question is: how could the strength be closely predicted when the behavior in dowel action is different? Well, it may be that we are victims of our definitions. A change of classification from "initially cracked" to "cracked along the interface" would lead to a compromise concerning the correlation of dowel action in beams and push-off specimens. Another question is: can one predict the strength of the initially uncracked beams, that develop cracks along the interface, using Zia's failure envelope? Apparently, yes.

AUTHORS' CLOSURE

By his discussion, Mr. Nosseir appears to have misunderstood the nature of the shear-friction hypothesis. It has never been seriously proposed as a true representation of the conditions at failure. Rather it is a simplified physical model for use in design, which corresponds to the equation representing a lower bound to experimental data relating shear transfer strength to the shear reinforcement parameter pf_y. It is a simplified model of behavior in that it neglects cohesion effects, and compensates by using a fictitious value for the coefficient of friction which is greater than the actual value. This was noted by the authors and earlier by Mast.[3]

The shear-friction hypothesis may be compared with the equivalent rectangular stress distribution used in ultimate strength design. In this case also, a simplified physical model of behavior is used for purposes of design calculations. No one believes that the concrete stress distribution is actually rectangular, but use of this simplified physical model enables one to calculate the resultant concrete compressive force and its center of action, both with reasonable accuracy. Similarly, the actual conditions at ultimate strength in shear transfer are somewhat different from the shear-friction representation, but its use in design leads to a reasonably conservative estimate of shear transfer strength.

In view of the above comments on the nature of the shear-friction hypothesis, it does not appear appropriate to use it as Mr. Nosseir has used it in the first part of his discussion.

The authors would be interested to learn how Mr. Nosseir isolated and identified the dowel action he reports as observed in his tests of prestressed composite beams. It may be that the cross section of his beams was such that the horizontal shear cracks could not spread vertically, but were constrained to travel horizontally at the interface, and linked up with one another. A continuous crack would then exist along the interface at failure, and conditions would be similar to those existing in the authors' initially cracked push-off specimens. In this case it is conceivable that some dowel action may have occurred.

The authors wish to thank Mr. Nosseir for his discussion.

Discussion of a paper by

Disc. 66-14

G. I. N. ROZVANY and J. F. WOODS

Sudden Collapse of Unbonded Underprestressed Structures*

DISCUSSION BY: KENNETH B. BONDY, M. F. SHAIKH and A. Q. KAHN, and AUTHORS

By KENNETH B. BONDY†

The method proposed by the authors for preventing sudden collapse of unbonded members at the cracking load is unnecessary and will, in typical structures, create more of a problem at service loads than it solves at overloads. Providing an average prestress higher than the modulus of rupture would require average compressive stresses in the order of 500 psi. American practice over the past 16 years has shown that members compressed to this high level, especially slabs, are often associated with excessive shortening and camber problems, mainly due to the exaggerated effects of elastic shortening and axial creep in the highly compressed member.

A much more practical and effective method of solving this problem is one which has been in practice in the United States for years, the addition of relatively small amounts of mild reinforcing steel in combination with the minimum amount of unbonded tendons, the mild steel being located in the areas of the member undergoing flexural tension. This mild steel distributes cracking, increases the ultimate capacity of the section, and assists in providing the necessary ductility. Properly designed members using a combination of unbonded prestressed tendons and bonded mild steel perform at least as well as members with bonded tendons from every behavioral standpoint including ductility, crack distribution, and strength.

The adequacy of the "mild steel" technique has been verified not only in actual construction covering mil-

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lions of square feet and hundreds of buildings, but it has also been experimentally verified. Lin, Scordelis, and Itaya, for example, studied the behavior of a continuous concrete slab prestressed in two directions with unbonded tendons.[2] The average compression in the slab was considerably less than the modulus of rupture, and the slab contained a small mat of mild reinforcing steel over each column. Lin, Scordelis, and Itaya state in the conclusion of their paper, "The physical behavior of the slab was ideal as a structure. Deflections and cracks were small up to a live load of 347 lb per sq ft. Ample warning was given of impending failure by the opening of large cracks and a large increase in the magnitude of the deflections."

Recommended minimum amounts of bonded mild steel in unbonded prestressed members may be found in Sections 3.1 and 3.2 of the Reference 3. The addition of this mild steel effectively and economically prevents the instability problem which the authors attribute to "underprestressed" unbonded structures, and results in safe, serviceable structures with reasonable compression levels.

A further assurance against a sudden collapse of a member with unbonded tendons is found in Section 2609(c) of ACI 318-63.[4] This Code provision requires that the ultimate flexural load in a prestressed member be at least 1.2 times the cracking load. A member designed in accordance with this provision will crack at a moment considerably less than its ultimate moment, providing a visual warning of overload and preventing a sudden failure. It is interesting to note that the flat plate structure constructed in Melbourne, Australia, and used by the authors as an example on page 134 would not have satisfied this American Code requirement had the tendons been unbonded. The cracking moment for this slab, according to ACI 318-63, is 43.6 ft-kips per foot. Section 2609(c) would therefore require an ultimate slab moment of 1.2 times 43.6 or 52.32 ft-kips. The actual allowable ultimate moment capacity of the slab with unbonded tendons is 49.2 ft-kips, less than the required 52.32 ft-kips. Thus, had this structure been constructed with unbonded tendons, the ACI Code would have required the addition of mild reinforcing steel to satisfy the ductility requirement alone, and the resultant structure would not have been subject to the instability to which the authors refer.

In summary, proper ductility in unbonded structures can be effectively realized merely by designing the structures in accordance with ACI 318-63 and Standard American Practices, as expressed in Reference 3. Resorting to the high compression levels recommended by the authors is not only unnecessary but it would result in structures with highly undesirable performance characteristics.

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3. ACI-ASCE Committee 423, "Tentative Recommendations for Concrete Members Prestressed With Unbonded Tendons," ACI JOURNAL, *Proceedings* V. 66, No. 2, Feb. 1969, pp. 81-86.

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By M. F. SHAIKH* and A. Q. KHAN*

The writers appreciate the authors' attempt to establish the conditions of crack instability in unbonded prestressed concrete beams. However, the writers disagree with the authors' concept of instability based on the sign of the M (c) curve only at $c = 0$. It should be pointed out that Eq. (4) which gives the relationship between the moment capacity and the depth of the crack is a quadratic in c, so that it is possible that the slope dM/dc may become positive for certain values of

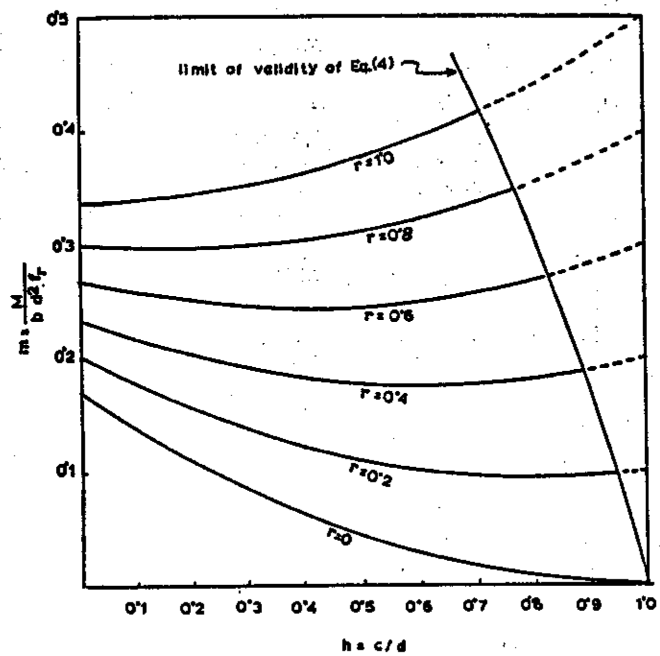

Fig. A—$m(h)$ curves for various values of r

$c > 0$, even though its initial value at $c = 0$ is negative. Introducing the notations $r = F/bdf_r$, $h = c/d$ and $a = e/d$, Eq. (4) can be written in the dimensionless form:

$$m = \frac{M}{bd^2f_r} = \left[1 + \frac{r}{1-h}\right]\frac{(1-h)^2}{6} + r\left(a + \frac{h}{2}\right)$$

which has two parameters a and r. For concentric tendons $a = 0$ (which is the case for the tests reported by the authors) a family of curves is plotted in Fig. A. for values of $r \leq 1$. It can be seen that even for $r < 1$ there exists a value of h after which the slope becomes positive. This value can be obtained from the above equation as follows:

$$\frac{dm}{dh} = \frac{-(1-h)}{3} - \frac{r}{6} + \frac{r}{2} = -\frac{(1-h-r)}{3}$$

so that when $dm/dh = 0$; $h_o = (1 - r)$.

Hence for values for r such that $0.5 < r < 1$ the slope dM/dc becomes positive before the crack reaches mid-depth. This means that the moment capacity of the section reaches a minimum value after an initial drop at the onset of cracking and further cracking takes

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