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SHEAR TRANSFER UNDER MONOTONIC LOADING, ACROSS AN INTERFACE BETWEEN CONCRETES CAST AT DIFFERENT TIMES

by

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SEATTLE, WASHINGTON

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16. Abstract
The total study is concerned with the shear transfer strength of reinforced concrete subject to both single direction and cyclically reversing loading, the latter simulating earthquake conditions. The primary topics studied were the influence of the existence in the shear plane of an interface between concretes cast at different times, and the influence of reinforcing bar diameter on shear transfer across a crack in monolithic reinforced concrete.

This Part I of the Final Report is concerned with shear transfer under monotonic, single direction loading, across an interface between concretes cast at different times. Push-off tests were made of both composite and initially cracked monolithic specimens made of normal weight concrete. The interface of the composite specimens was either roughened to comply with Sec. 11.15 of ACI 318-71, or was smooth. In each case, tests were made in which the interface was either bonded and initially cracked, unbonded and initially uncracked, or unbonded and initially cracked.

It was found that if the interface was roughened, then the shear transfer strength was very close to that of comparable initially cracked, monolithic push-off specimens, for all interface conditions. If the interface was smooth then the shear transfer strength was much reduced and was approximately equal to the shear yield strength of the reinforcement.

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composite construction, connections, precast concrete, reinforced concrete, shear transfer

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PREFACE

This study was carried out in the Structural Research Laboratory of the University of Washington. It was made possible by the support of the National Science Foundation, through grant No. ENG 74-21131.

Contributions were made to the execution of this project by graduate student Research Assistants, W. Chevapravatdumrong and K. Nakajima.

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I - INTRODUCTION

1.1 Background

A large proportion of connective distress found in precast concrete structures is centered around shear interfaces associated with corbels, bearing shoes, etc. For these concrete to concrete and concrete to steel interfaces, as well as along cracks in monolithic concrete, ordinary beam shear and flexure analyses do not apply. In such a case, we are concerned with the transfer of shear across a specific plane and with shear failure as slippage along the plane. This type of behavior is referred to as "shear transfer".

Situations where shear transfer across a definite plane must be considered in the design of precast concrete connections were discussed by Birkeland and Birkeland(1) and by Mast(2). Mast emphasized that dependence should not be placed on the tensile strength of concrete when considering shear transfer. He also pointed out the need to consider the effect on shear transfer behavior of a pre-existing crack in the shear plane. Such a crack could occur for a variety of reasons unrelated to shear, e.g. due to tension forces caused by restraint of long term deformations of concrete.

Interest in shear transfer across cracks in monolithic reinforced concrete has also arisen in connection with the design of reinforced concrete secondary containment vessels for nuclear reactors. These containment vessels are subjected to pressurization tests before acceptance, which result in circumfrential and vertical cracks in the wall of the vessel. It is therefore necessary to consider the transfer of shear across these cracks, should the vessel subsequently be subjected to lateral loads due to earthquake, wind, etc.

1.2 Previous Research

A continuing study of the factors affecting shear transfer strength has been made over the last decade at the University of Washington.(3 through 9) The most recent previous research on this topic (1972-74), was carried out under NSF Grant No. GK-33842X and was concerned with shear planes in monolithically cast concrete. Factors affecting shear transfer strength which were examined in that study included:

Aggregate type - normal weight and lightweight.

Shear transfer reinforcement parameter ρf_y.

The existence of a crack in the shear plane.

Concrete strength - low, medium and high.

Tension force across the shear plane.

Moment across the shear plane.

Type of loading - single direction and cyclic reversal of loading.

The various aspects of this prior study have been reported in detail elsewhere[6,7,8,9]. Based on the conclusions drawn in the study, code clauses for design for shear transfer in lightweight concrete [7] and for the design of reinforced concrete corbel brackets[10] have been submitted to ACI Committee 318, Standard Building Code. These proposals are currently being considered for inclusion in the ACI "Building Code Requirements for Reinforced Concrete," (ACI 318).

1.3 This Study

1.3.1 General - The studies previously undertaken have been concerned with the transfer of shear across shear planes in monolithically cast concrete. The results of those tests relate primarily to the design of precast members in the vicinity of connections.

Another type of problem is that involving transfer of shear across the interface between concretes cast at different times. This may be between precast and cast-in-place concrete in composite precast construction, or at a construction joint in monolithic concrete construction. (Shear transfer type failures of construction joints in shear walls have been observed in earthquakes, and engineers have asked whether the "shear friction" provisions of ACI 318-71 may properly be used to design construction joints against this type of failure.) Since the design of concrete to concrete interfaces for shear transfer is a frequently occurring problem in precast concrete design, further study of this problem, directed toward the development of design recommendations for both monotonically increasing and cyclically reversing shear, appeared to be desirable.

The previous shear transfer tests were made using specimens reinforced with bars of 3/4 in. diameter or less. Within this range of sizes, it has been found that shear transfer strength is only influenced by change in bar size to the extent that the reinforcement parameter ρf_y is changed, i.e. if both bar size and spacing are changed so that ρf_y is not changed, then the shear transfer strength is unchanged.

Although the bar size range covered by previous tests covers the bar sizes used in many situations in precast concrete construction, an increasing number of design situations are occurring where it is necessary to use larger bar sizes as shear transfer reinforcement. This is occurring in both precast concrete construction and cast-in-place concrete construction. An extreme case occurs in the design of nuclear reactor containment vessel walls, where it has been proposed by some engineers that the "shear friction" design concepts which are based on data from tests of specimens reinforced with relatively small sized bars, should be applied to the membrane shear design of a containment vessel wall reinforced with #14 or #18 rebars. Such extrapolation is questionable and could be dangerous.

The transfer of shear across a crack involves a combination of frictional resistance to sliding of one crack face over the other, resistance to shearing off of local "high spots" on the crack faces and dowel action of the reinforcing bars. The role of rebar dowel forces in the development of shear transfer strength, is analagous to the role of dowel forces in stud shear connector behaviour in composite steel and concrete construction. Research[11] has shown that the useful strength of stud shear connectors of diameter greater than about 1 in. is limited by splitting of the concrete as a result of dowel action, rather than by the yield strength of the stud. It was considered possible that a similar limitation might be found in the case of large size reinforcing bars used as shear transfer reinforcement. It therefore appeared desirable to establish the maximum bar diameter for which current shear transfer design procedures are valid.

1.3.2 Scope of Entire Study - Tests are being made of "push-off" type specimens and such modified versions of the simple push-off type type of specimen as will permit the desired loading condition to be imposed on the specimen. The following variables are being included in the test program:

The use of composite or monolithic specimens.

The strengths of the concretes cast against one another.

The condition of the face of the precast concrete against which other concrete is cast; i.e., smooth, deliberately roughened, bond deliberately broken or not.

The existence of a crack in the shear plane.

The shear transfer reinforcement parameter ρf_y.

The size of rebars used as shear transfer reinforcement.

The type of loading;- single direction and cyclic reversal of loading.

1.3.3 This Report - This part of the final project report describes that part of the study concerned with shear transfer across interfaces between precast and cast-in-place concrete, when subject to monotonically increasing shear.

II - EXPERIMENTAL STUDY

2.1 Scope

The experimental program reported here was designed to study single direction shear transfer across an interface between concretes cast at different times. Variables included in the tests were,

1. The condition of the face of the precast concrete against which other concrete was cast, i.e., whether it was smooth or was deliberately roughened to comply with the requirements of Sec. 11.15.7 of ACI 318-71[12].

2. The existence of bond between the precast and cast-in-place concrete.

3. The strengths of the concretes, i.e., whether the precast and cast-in-place concretes have the same or different compressive strengths.

4. The existence of a crack at the interface.

5. The shear transfer reinforcement parameter ρf_y.

2.2 The Test Specimens

The test specimens were of the "push-off" type shown in Fig. 2.1, with a shear plane of 50 $in.^2$ area. When loaded as indicated by the arrows, shear without moment is produced in the shear plane. The reinforcement crossing the shear plane was in the form of closed stirrups. This was to ensure the effective anchorage of the reinforcement on both sides of the shear plane. The composite specimens were cast in two stages, so as to cause an interface between the two concretes to occur in the shear plane. The specimens were cast on their sides, so that at the time of casting the shear plane was horizontal.

Eight series of push-off specimens were tested, as indicated in Table 2.1. The variables between test series are also detailed in Table 2.1. The variable within each test series was the reinforcement parameter ρf_y. The size and number of closed stirrups provided in each specimen are shown in Table 2.3.

2.3 Materials and Fabrication

The specimens were made from Type III Portland Cement, sand and 3/4 in. maximum size gravel in the proportions shown in Table 2.2. In all cases, water was added sufficient to produce a 3 inch slump. The gravel was a glacial outwash gravel obtained from a local pit. The concrete was mixed in a 5 cu. ft. capacity "Eirich Counter-Current Rapid Mixer" and was compacted in the forms using an immersion vibrator.

The deformed bar reinforcement used conformed to ASTM Specification 615. The bars used for the shear transfer reinforcement had a yield point of approximately 50 ksi. The actual yield point was determined for the reinforcement used in each specimen. The #6 bar longitudinal reinforcement had a yield point of 60 ksi. The stirrups were welded closed on one of the short sides. The reinforcement cages were tied with iron wire.

Reinforcement and concrete properties are shown in Table 2.3.

The specimens were cast in forms made of cold rolled steel plate. In the case of the composite specimens, half of the specimen was initially cast in an L shaped form. The surface which was to be in the shear plane in the completed specimen was finished in either of two ways; (a) trowelled as smooth as possible, or (b) deliberately roughened after having been "struck off" even with the edges of the form, so as to comply with the requirements of Sec. 11.15.7 of ACI 318-71. (By following this procedure the undulations formed in the interface were uniformly distributed on either side of shear plane in the completed specimen.) The half specimen was cured under polythene sheets for 3 days. It was then removed from the form, the reinforcement cage was completed, and it was placed in a form for a complete specimen. The remaining concrete was cast and the specimen was cured under polythene sheets a further 4 days before being removed from the form and tested.

In Series B, C and D, in which it was desired to obtain bond between the concretes of the two halves of the specimen, the interface was thoroughly wetted before the second half of the specimen was cast. In Series E, F, G and H, in which it was desired that there be no bond between the concretes of the two halves of the specimen, the interface was given a thin coating of bond breaker before casting the second half of the specimen. The bond breaker was a mixture of soft soap and talc, as used to prevent bond between "match-cast" parts in a local precast concrete plant. (By volume, 5 parts Flaxoap: 1 part talc.)

The specimens of Series A were cast in one piece and were cured in the forms under polythene sheets for four days before testing.

2.4 Testing Arrangements and Instrumentation

The push-off specimens were tested using a Baldwin hydraulic testing machine to load the specimen along the shear plane as indicated in Fig. 2.1. The actual arrangements for test are shown in Fig. 2.2. The specimen stood on the lower platten of the testing machine and was loaded through the spherically seated upper platten of the testing machine and a set of parallel plates and rollers. The rollers ensured that separation of the two halves of the push-off specimen was not restrained by the testing machine. A load cell was provided between the upper platten of the testing machine and the roller system to monitor the applied load continuously.

Both the slip (or relative motion parallel to the shear plane of the two halves of the specimen,) and the separation (or relative motion normal to the shear plane of the two halves of the specimen,) were measured continuously. Linear differential transformers were used as the sensing elements of slip and separation gages attached to reference points embedded in the concrete. The separation gage was located at the middle of the length of the shear plane and the slip gage 2 inches below it. The embedded reference points were located

1½ inches either side of the shear plane. Both the slip and separation gages, and the load cell were monitored continuously during the test by a Sanborn strip chart recorder. The slip and separation gages were calibrated directly before the test, using a micrometer head to impose predetermined displacements on the core of the linear differential transformer.

2.5 Testing Procedure

The specimens were subjected to a continuously increasing load until failure occurred, with short pauses as necessary to mark any cracks which may have appeared. The average length of time taken for a test was about 15 minutes. The ultimate load was defined as the maximum load that could be carried by the specimen. Measurements of slip, separation and load were continued as the load decreased from the ultimate load, in order to trace out the falling branch of the load-slip and load-separation curves.

As indicated in Table 2.1, certain of the push-off specimens were cracked along the shear plane before being tested in shear. The crack was produced by applying line loads to the back and front faces of the specimen along the line of the shear plane. To do this, the specimen was placed in a horizontal position and the line loads were applied through a pair of steel bars by the Baldwin testing machine, as shown in Fig. 2.3. The dilation of the specimen normal to the shear plane was measured during the cracking operation using dial gages attached to a reference frame surrounding the specimen. Little dilation occurred until the specimen cracked in the shear plane. Loading was continued until the dilation was slightly greater than 0.01 in. When the live loads were removed, a residual dilation of about 0.01 in. remained, this being the average width of the crack in the shear plane before the shear transfer test. The load required to produce an initial crack width of 0.01 in. was approximately proportional to the shear transfer reinforcement parameter ρf_y.

3.2.2 Composite Specimens with a Deliberately Roughened Interface, (Series B, D, E and F). The behavior of the initially cracked specimens of Series B and D, in which good bond was obtained at the interface, was similar to that of the initially cracked monolithic specimens of Series A. However, fewer diagonal tension cracks occurred than in Series A. In Series D, the diagonal tension cracks and the compression spalling at failure tended to be concentrated in that half of the specimen made of the lower strength concrete.

The behavior of the specimens of Series F in which the bond at the interface was destroyed, but the specimens were initially uncracked, was similar to that of Series B and D, except that the failure was a little less brittle and occurred at generally larger slips, as may be seen from the shear-slip curves. The shear stiffness at lower loads, of comparable specimens of Series F and B, was very nearly the same.

The specimens of Series E in which the bond at the interface was destroyed and which were also initially cracked, exhibited much lower shear stiffnesses than those of the comparable specimens of Series B and F, and the slip at ultimate load was greater.

The shear-slip and slip-separation curves for Series B, D and E are shown in Figs. 3.2, 3.4, and 3.5, and 3.10, 3.12 and 3.13 respectively.

3.2.3 Composite Specimens with a Smooth Interface, (Series C, G and H) In the specimens of all these three series, considerable slip occurred from the commencement of loading, but little or no separation was observed until just before failure. The shear stiffness of comparable specimens of all three series was similar at similar loads. A few diagonal tension cracks occurred adjacent to the shear plane in the most heavily reinforced specimens of Series C, but no diagonal tension cracks occurred in the specimens of Series G and H.

At ultimate load both slip and separation increased very rapidly. Failure was characterized by spalling of the concrete adjacent to the shear plane and the formation of cracks perpendicular to the shear plane. These cracks sometimes, but not always, coincided with the location of a shear transfer stirrup.

The shear-slip and slip-separation curves for Series C, G and H are shown in Figs. 3.3, 3.7 & 3.8, and 3.11, 3.15 & 3.16 respectively.

IV - DISCUSSION OF TEST RESULTS

4.1 Composite Specimens with a Deliberately Roughened Interface, (Series B, D, E and F.)

In Fig. 4.1 the ultimate shear transfer strengths of initially cracked composite specimens with a deliberately roughened interface are compared with those of initially cracked monolithic specimens. It can be seen that the strengths of the composite specimens are close to, but slightly less than those of comparable monolithic specimens. The difference in behavior is probably due to the difference in the minor roughness of the crack faces in the two cases.

The major roughness of the crack faces is caused by the crack propagating around the pieces of large aggregate lying in the shear plane. This would probably be about the same in both the monolithic specimen and the composite specimen. The minor roughness, caused by the crack passing around sand particles in a monolithic specimen, would probably be much less in a composite specimen. In this case the crack would probably lie at the interface between the precast and cast in place concretes. The deliberate roughening of the surface of the precast concrete would model the major roughness of the crack in monolithic concrete, but the minor roughness would be much less or even absent. The influence of the minor roughness of cracks on shear transfer behavior has been discussed elsewhere[7].

The shapes of the shear-slip curves of the initially cracked monolithic specimens and the initially cracked composite specimens with roughened interface are very similar; as are the magnitudes of the slips at ultimate shear in the two cases.

In Fig. 4.2 a comparison is made of the strengths of composite specimens with deliberately roughened interface, but with different interface conditions at the time of test. It can be seen that for low and medium values of the reinforcement parameter ρf_y, the shear transfer strength is almost the same whether the interface is bonded but initially cracked, unbonded but initially uncracked, or both unbonded and initially cracked. However, for large values of ρf_y, the shear transfer strength is less for the specimens in which the bond at the interface was broken.

The shear-slip relationships for the composite specimens without bond at the interface were much less brittle in character than were those of the cracked monolithic specimens and the cracked composite specimens in which good bond was obtained at the interface. The shear stiffnesses of the specimens which were both initially cracked and had the bond at the interface broken, were much less than those of the other specimens at all levels of shear, even when the ultimate strengths were not significantly different.

In Fig. 4.3 a comparison is made of the shear transfer strengths obtained when both halves of the composite specimens had the same concrete strength, (f'_c = 6000 psi,) and when the two halves of each composite specimen had different strengths, (f'_c = 6000 psi and 3000 psi). It can be seen that for low values of ρf_y the shear transfer strengths of the specimens made from 3000 psi and 6000 psi concretes were actually greater than those of the specimens made of 6000 psi concrete only. This may have been due to the crack having formed just inside the lower strength concrete, rather than at the interface. In such a case the minor roughness of the crack would be greater. This would result in a greater shear transfer strength, as long as concrete strength does not control shear transfer strength. This did become the case when ρf_y exceeded about 600 psi, so that specimens D3, D4 and D4A yielded very nearly the same shear transfer strength despite a substantial increase in ρf_y.

For values of ρf_y greater than 1000 psi, the shear transfer strength again increased, the strength being very nearly proportional to ρf_y. The strengths of these heavily reinforced specimens are approximately equal to the strengths which would be calculated assuming a simple truss action after formation of diagonal tension cracks. This perhaps indicates that the original crack "locked up," diagonal tension cracks formed and truss action developed. Heavy surface spalling occurred in these specimens adjacent to the shear plane at failure, so that it is difficult to be certain as to the exact mechanics of failure.

4.2 Composite Specimens with a Smooth Interface, (Series C, G and H.)

In Fig. 4.4 a comparison is made of the shear transfer strengths of composite specimens with a smooth interface, but with different interface conditions at the time of test. It can be seen that lower strengths resulted when bond at the interface was broken, as compared with the case in which good bond was obtained at the interface and then a crack was subsequently formed at the interface. The difference in minor roughness of the crack faces in the two cases is the probable reason for the difference in shear transfer strength.

It can also be seen that the shear transfer strength of the composite specimens with a smooth interface is only about half that of comparable composite specimens with a deliberately roughened interface. This result indicates that, if in a design it is assumed that the interface will be rough and appropriately high shear transfer stresses are allowed, then adequate inspection must be provided to ensure that the assumed roughness of the interface is in fact provided. If this is not done and the assumed roughness of the interface is not provided, then the actual shear transfer strength of the joint may be low and the safety of the structure may be impaired.

The shear transfer strength of the composite specimens with a smooth interface is approximately proportional to the strength of the shear transfer reinforcement and $v_u \simeq 0.6\ \rho f_y$. This fact indicates that the shear transfer reinforcement in these specimens did not develop its tensile yield strength, since the test results of Gaston and Kriz(14) showed that the coefficient of friction between smooth concrete unbonded contact faces is about 0.8.

The shear transfer strength of the composite specimens with a smooth interface is actually very close to the shear yield strength of the shear transfer reinforcement, i.e.,

$$V_u = \frac{f_y}{\sqrt{3}} A_{vf} = 0.58\ A_{vf}\ f_y \qquad (4.1)$$

or,

$$v_u = 0.58\ \rho f_y \qquad (4.2)$$

This appears to indicate that for an initially cracked smooth interface, the shear transfer strength is developed by dowel action of the shear transfer reinforcement, rather than by any shear-friction type mechanism. The large slips and small separations observed in these tests also support this conclusion.

4.3 Comparison of Measured and Calculated Shear Transfer Strengths of Composite Push-off Specimens

4.3.1 Using Shear Friction The shear friction theory is a simplified theory for design purposes. It is assumed that a crack exists in the shear plane and that reinforcement of area A_{vf} and having a yield point f_y, crosses the crack at right angles. It is hypothesized that when shear acts along the crack, slip will occur, accompanied by separation of the crack faces due to their roughness. This separation is assumed to stretch the reinforcement crossing the crack, and the tensile force so caused in the reinforcement results in a balancing compressive force across the crack. It is proposed that shear resistance in the shear plane is developed by the frictional resistance to sliding of one face of the crack over the opposite face.

At ultimate strength, it is assumed that the separation of the crack faces is sufficient to develop the tensile yield strength of the reinforcement crossing the crack. The normal compression across the crack is then equal to $A_{vf}f_y$ and the shear transfer strength V_u is given by,

$$V_u = \phi A_{vf}\, f_y\, \mu \qquad (4.3)$$

where μ is the assumed coefficient of friction and ϕ is the capacity reduction factor for shear (0.85) specified in ACI 318-71. The values of μ specified in Section 11.15 of ACI 318-71 are 1.4 for a crack in monolithic concrete, and 1.0 for a crack at the interface when concrete is placed against hardened concrete having a roughened surface. The ultimate shear stress is limited to the lesser of 800 psi or 0.2 f'_c. The shear friction equation (4.3) may also be restated as,

$$v_u = \frac{V_u}{\phi A_{cr}} = \rho f_y \mu \text{ but } \not> 0.2f'_c \text{ nor } 800 \text{ psi} \qquad (4.4)$$

In Sec. 6.1.9 of the PCI Design Handbook it is proposed that for values of ρf_y greater than 600 psi, the shear friction equations can continue to be used provided that the coefficient μ is multiplied by

$$\left(\frac{300}{\rho f_y} + 0.5\right).$$

That is, Eq. (4.4) now becomes

$$v_u = \rho f_y \mu \left(\frac{300}{\rho f_y} + 0.5\right) \tag{4.5}$$

This equation will be referred to as the PCI equation. Initially, no upper limit was specified for v_u calculated using the PCI equation, but subsequently an upper limit of the lesser of 0.25 f'_c or 1200 psi was proposed.

In Fig. 4.5 the measured shear transfer strengths of the composite push-off specimens with a deliberately roughened interface are compared with the shear transfer strengths calculated using the shear friction equations (4.4) and (4.5). (Note that in making comparisons of measured and calculated shear transfer strengths, the value of the capacity reduction factor ϕ is taken as 1.0, since the material strengths and specimen dimensions are accurately known.)

It can be seen that the shear transfer strengths of all these specimens are estimated conservatively by both shear friction equations, even if the value of μ = 1.4 (corresponding to a crack in monolithic concrete,) is used. If μ is taken as 1.0, as required by Sec. 11.15 of ACI 318-71 for the case of an interface when concrete is placed against hardened concrete, then the calculated strengths are over-conservative relative to the measured strengths.

The conditions of cracking and loss of bond at the interface in these specimens represent the worst conditions likely to occur at an interface in practise. If good bond is obtained and no crack occurs in the interface, the shear transfer strength will be higher than obtained in these tests. It appears therefore that it would be reasonably conservative to use μ = 1.4 in design, for the case of transfer of shear across an interface when concrete is placed against hardened concrete having a roughened surface as required by Sec. 11.15.7.

In Fig. 4.6 the measured shear transfer strengths of the composite push-off specimens with a smooth interface are compared with the shear transfer strengths calculated using the shear friction equations (4.4) and (4.5), assuming a value of 0.60 for μ, (ACI 318-71 does not specify a value of μ for this interface condition). It is seen that using this value of μ in the ACI equation (4.4), yields reasonably conservative results for the specimens of Series C and G. For the specimens of Series H, in which the bond at the interface was broken and the specimens were also deliberately cracked, the ACI equation (4.4) yields slightly unconservative results. The condition of the interface in the specimens of Series H would represent an extreme case. It is therefore proposed that μ should be taken as 0.6 for design purposes, for the case of concrete cast against hardened concrete with a smooth face.

It is seen that the shear transfer strength of the specimens with a smooth interface is approximately proportional to the reinforcement parameter ρf_y, for values of ρf_y up to 1400 psi. It does not therefore seem appropriate in this case, to use the PCI equation (4.5) which implies a change in the relationship between v_u and ρf_y at ρf_y equal to 600 psi. Also, the PCI equation is seen to yield quite conservative results for large values of ρf_y.

Section 11.15.7 of ACI 318-71 specifies a value of 0.7 for μ in the case of concrete cast against as-rolled, unpainted steel. This value of μ corresponds to the shear transfer strength obtained at a concrete-steel interface when headed stud shear connectors are used as shear transfer reinforcement. It is thought that the higher strengths obtained in this case than in the case of the smooth concrete to concrete interface, are due to the fact that the stress-strain curve for the headed stud shear connectors does not have a clearly defined yield point. It is therefore likely that at ultimate these studs would actually be stressed to a stress greater than their nominal yield point. Also, the cross-section of the stud in the shear plane is increased locally by the weld metal deposited around the stud where it is attached to the steel surface. The rebars used as shear transfer reinfrocement in the composite push-off specimens having a smooth interface had a clearly defined yield point, and it is considered that the yield stress was probably the maximum stress reached in this reinforcement.

It has been noted earlier, that for the case of a smooth concrete interface it is likely that the shear transfer strength is being developed by dowel action of the reinforcement, rather than by a shear friction type of action. Never-the-less it is convenient for design purposes to continue to use the shear friction equation for this case, using an artificially low value for μ. The shear friction equation (4.4) then becomes numerically the same as equation (4.2) corresponding to dowel action of the reinforcement.

4.3.2 Using Modified Shear Friction As noted in 4.3.1, the shear friction theory is a simplified theory for design purposes. It assumes that all the shear transfer resistance is developed by friction between the crack faces. This leads to a relationship in which the shear transfer strength is directly proportional to the reinforcement parameter ρf_y. In reality the resistance to shear is provided by a combination of frictional resistance, resistance to shearing off of asperities on the crack faces and dowel action of the reinforcement crossing the crack. In the case of a crack in monolithic concrete, the shear transfer strength is as a consequence, not directly proportional to ρf_y. For values of ρf_y greater than 200 psi, it has been shown[5] that, for normal weight concrete, the following equation corresponds more closely to the actual variation of shear transfer strength with ρf_y.

$$v_u = 0.8\ \rho f_y + 400 \text{ psi} \quad \text{but} \not> 0.3\ f'_c \tag{4.6}$$

This equation is referred to as the modified shear friction equation.

In Fig. 4.7 a comparison is made of the measured shear transfer strengths of the composite push-off specimens having a roughened interface, and the shear transfer strength calculated using the modified shear friction equation (4.6). It can be seen that the equation yields reasonably conservative values for shear transfer strength in this case. However, for the specimens in which bond was prevented at the interface, the maximum shear stress developed is approximately 0.2 f'_c rather than 0.3 f'_c. It is therefore proposed that for the case of concrete cast against roughened concrete, the shear transfer strength may be calculated using the modified shear friction equation (4.6) providing v_u is limited to 0.2 f'_c.

In the case of concrete cast against smooth concrete, it has already been shown that the shear transfer strength is closely predicted by the shear friction equation (4.4), if a value of 0.6 is used for μ. It is obvious therefore, that the modified shear friction equation (4.6) does not apply in this case.

4.4 Comparison of Measured and Calculated Shear Transfer Strengths of Initially Cracked, Monolithic Push-off Specimens of Normal Weight Concrete

In Fig. 4.8 a comparison is made of the measured shear transfer strength of initially cracked, monolithic, normal weight concrete push-off specimens, and the shear transfer strength calculated using the modified shear friction equation (4.6). Included in the figure are test data from Series A (f'_c=6000 psi) and data from previous tests(3) for which f'_c was 4000 psi or 2500 psi.

The modified shear friction equation was originally proposed on the basis of the data from tests(3) in which f'_c was 4000 psi or 2500 psi. It can be seen that the equation is conservative with respect to the data from tests in which f'_c was 6000 psi. It can also be seen that the upper limit of 0.3 f'_c for v_u is still valid for 6000 psi compressive strength concrete. The modified shear friction equation (4.6) can therefore be used with confidence in design for normal weight concrete strengths of up to 6000 psi. It is unlikely however that ultimate shear transfer stresses greater than 1200 psi would be used in design, because of problems of reinforcement congestion.

It is apparent from Fig. 4.8 that the modified shear friction equation could be made to reflect the test results more closely, by replacing the constant 400 with a function of the concrete compressive strength. [$4.50\ (f'_c)^{0.545}$ would be appropriate.] However, it is thought that for design purposes this would represent an unwarranted complication of the equation.

V - PRINCIPAL CONCLUSIONS

Based on the test data reported, the following conclusions may be drawn concerning shear transfer in normal weight concrete.

1. Roughening of the interface as prescribed by Sec. 11.15 of ACI 318-71, is essential if high shear stresses are to be transferred across the interface of concrete cast against hardened concrete.

2. If the interface is deliberately roughened, then the mechanics of shear transfer across the interface between concrete cast against hardened concrete, is essentially the same as in the case of shear transfer across a crack in monolithic concrete.

3. If the interface between concrete cast against hardened concrete is smooth, then shear is transferred across the interface primarily by "dowel action" of the shear transfer reinforcement.

4. If the shear friction equation contained in Sec. 11.15 of ACI 318-71 is used to calculate the shear which may be transferred across an interface between concrete cast against hardened concrete, then the following values of μ are appropriate in the case of normal weight concrete:

(a) If the interface is deliberately roughened as specified in Sec. 11.15.7 of ACI 318-71, $\mu = 1.4$,

(b) If the interface is smooth, $\mu = 0.6$.

5. If the interface is deliberately roughened, then the shear which may be transferred across an interface between concrete cast against hardened concrete can be calculated using the modified shear friction equation, providing v_u is limited to $0.2\ f'_c$.

i.e. $$v_u = 0.8\ \rho f_y + 400 \text{ psi} \quad \text{but} \ngtr 0.2\ f'_c$$

6. The modified shear friction equation can be used to calculate the shear

which may be transferred across a crack in monolithic, normal weight concrete, for concrete strengths of up to 6000 psi. In this case, the upper limit of $0.3\ f'_c$ on v_u is appropriate.

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TABLE 2.3A - PROPERTIES OF SPECIMENS AT TIME OF TEST (SERIES A)

Specimen No.	Stirrups No. & Size	Reinforcement Yield Strength (ksi)	Concrete Compressive Strength (1) (psi)	Concrete Tensile Strength (2) (psi)
A1	1- #3	51.64	6020	495
A2	2- #3	51.64	6020	495
A3	3- #3	55.45	5820	455
A4	4- #3	55.45	5880	480
A5	5- #3	51.27	6125	440
A6	4- #4	48.00	5900	465
A6A	4- $4	48.00	5970	495
A7	5- #4	48.20	5970	495

Note (1) Concrete compressive strengths measured on 6 x 12 inch cylinders.

(2) Concrete splitting tensile strengths measured on 6 x 12 inch cylinders.

TABLE 2.1 - PROGRAM OF PUSH-OFF TESTS

Series	Type of Specimen	f'_c at test (psi)	Type of Interface	Bond Condition of Interface	Initial Condition
A	Monolithic	6000	---	---	Cracked
B	Composite	6000	Rough	Bonded	Cracked
C	Composite	6000	Smooth	Bonded	Cracked
D	Composite	6000 & 3000	Rough	Bonded	Cracked
E	Composite	6000	Rough	Bond broken	Cracked
F	Composite	6000	Rough	Bond broken	Uncracked
G	Composite	6000	Smooth	Bond broken	Cracked
H	Composite	6000	Smooth	Bond broken	Uncracked

TABLE 2.2 - CONCRETE MIX PROPORTIONS
(lb./cu.yd.)

Concrete Strength f'_c (psi) and Age at Test (days)	3000 4	6000 7	6000 4
Type III Portland Cement	460	720	755
3/4 in. Gravel	1760	1665	1620
Sand	1415	1235	1230

Note: In all mixes, water was provided to produce a 3 inch slump.

TABLE 2.3B - PROPERTIES OF SPECIMENS AT TIME OF TEST (SERIES B AND C)

Specimen No.	Stirrups No. & Size	Reinforcement Yield Strength (ksi)	First Half		Second Half	
			Conc. Comp. Strength(1) (psi)	Conc. Tensile Strength (2) (psi)	Conc. Comp. Strength(1) (psi)	Conc. Tensile Strength (2) (psi)
B1	1- #3	51.27	6330	415	5840	410
B2	2- #3	50.55	6330	415	5840	410
B3	3- #3	51.27	6055	395	6225	470
B4	4- #3	53.82	6055	395	6225	470
B5	2- #3 +2- #4	53.82 49.25	6040	485	5895	405
B6	4- #4	49.25	6040	485	5895	405
C1	1- #3	50.91	6190	540	5870	450
C2	2- #3	50.91	6190	540	5870	450
C3	3- #3	50.55	5980	390	5980	400
C4	4- #3	51.64	5980	390	5980	400
C5	5- #3	52.73	6165	405	6185	380
C6	4- #4	45.25	6165	405	6185	380

Note (1) Concrete compressive strengths measured on 6 x 12 inch cylinders.

(2) Concrete splitting tensile strengths measured on 6 x 12 inch cylinders.

TABLE 2.3C - PROPERTIES OF SPECIMENS AT TIME OF TEST (SERIES D)

Specimen No.	Stirrups No. & Size	Reinforcement Yield Strength (ksi)	First Half		Second Half	
			Conc. Comp. Strength(1) (psi)	Conc. Tensile Strength (2) (psi)	Conc. Comp. Strength(1) (psi)	Conc. Tensile Strength (2) (psi)
D1	1- #3	51.27	6245	380	3770	350
D2	2- #3	51.27	6245	380	3770	350
D3	3- #3	56.00	5910	390	2940	240
D4	4- #3	56.00	5910	390	2940	240
D4A	4- #3	54.00	6085	460	2495	285
D5	2- #3 +2- #4	46.36 48.50	6200	440	2955	350
D5A	2- #3 +2- #4	53.64 46.20	6285	485	2795	295
D6	4- #4	48.50	6200	440	2955	350

Note (1) Concrete compressive strengths measured on 6 x 12 inch cylinders.

(2) Concrete splitting tensile strengths measured on 6 x 12 inch cylinders.

TABLE 2.3D - PROPERTIES OF SPECIMENS AT TIME OF TEST (SERIES E AND F)

Specimen No.	Stirrups No. & Size	Reinforcement Yield Strength (ksi)	First Half		Second Half	
			Conc. Comp. Strength(1) (psi)	Conc. Tensile Strength (2) (psi)	Conc. Comp. Strength(1) (psi)	Conc. Tensile Strength (2) (psi)
E1	1- #3	47.27	6490	490	5940	445
E2	2- #3	47.27	6620	465	6500	460
E3	3- #3	47.27	6140	490	5990	370
E4	4- #3	47.27	5860	—	5990	—
E5	2- #3 +2- #4	53.64 47.40	6650	455	6355	538
E6	4- #4	45.30	6125	451	6200	425
F1	1- #3	46.36	6090	410	6010	385
F2	2- #3	47.27	6490	490	5940	445
F3	3- #3	47.27	6620	465	6500	460
F4	4- #3	47.27	6140	490	5990	370
F5	2- #3 +2- #4	53.64 45.30	5860	---	5990	---
F6	4- #4	47.40	6650	455	6355	538

Note (1) Concrete compressive strengths measured on 6 x 12 inch cylinders.

(2) Concrete splitting tensile strengths measured on 6 x 12 inch cylinders.

TABLE 2.3E - PROPERTIES OF SPECIMENS AT TIME OF TEST (SERIES G AND H)

Specimen No.	Stirrups No. & Size	Reinforcement Yield Strength (ksi)	First Half		Second Half	
			Conc. Comp. Strength(1) (psi)	Conc. Tensile Strength (2) (psi)	Conc. Comp. Strength(1) (psi)	Conc. Tensile Strength (2) (psi)
G1	1- #3	50.91	6190	540	5870	450
G2	2- #3	50.91	6190	540	5870	450
G3	3- #3	50.55	5980	390	5980	400
G4	4- #3	51.64	5980	390	5980	400
G5	5- #3	52.73	6165	405	6185	380
G6	4- #4	45.25	6165	405	6185	380
H1	1- #3	55.45	6330	370	5825	400
H2	2- #3	55.45	6170	---	6080	---
H3	3- #3	55.45	6170	---	6080	---
H4	4- #3	53.64	6720	395	6075	375
H5	2- #3 +2- #4	53.64 46.80	6650	550	6180	465
H6	4- #4	46.80	6535	450	5900	370

Note (1) Concrete compressive strength measured on 6 x 12 inch cylinders.

(2) Concrete splitting tensile strengths measured on 6 x 12 inch cylinders.

TABLE 3.1A - TEST DATA (SERIES A AND B)

Specimen No.	Reinforcement Area, A_{vf} ($in.^2$)	Reinforcement Parameter ρf_y, (psi)	Ultimate Shear Stress v_u, (psi)	Slip at Ultimate (in.)	Separation at Ultimate (in.)
A1	0.22	227	760	0.0135	0.0126
A2	0.44	454	800	0.0227	0.0068
A3	0.66	732	1150	0.0090	0.0130
A4	0.88	976	1420	0.0213	0.0154
A5	1.10	1128	1500	0.0180	0.0089
A6	1.60	1536	1760	0.0147	0.0102
A6A	1.60	1536	1860	0.0177	0.0182
A7	2.00	1928	1940	0.0407	0.0260
B1	0.22	226	487	0.0120	0.0021
B2	0.44	445	700	0.0173	0.0017
B3	0.66	676	1054	0.0150	0.0083
B4	0.88	947	1276	0.0174	0.0062
B5	1.24	1262	1570	0.0213	0.0098
B6	1.60	1576	1700	0.0145	0.0100

TABLE 3.1B - TEST DATA (SERIES C AND D)

Specimen No.	Reinforcement Area, A_{vf} ($in.^2$)	Reinforcement Parameter ρf_y, (psi)	Ultimate Shear Stress v_u (psi)	Slip at Ultimate (in.)	Separation at Ultimate (in.)
C1	0.22	224	210	0.1073	0.0154
C2	0.44	448	360	0.0417	0.0031
C3	0.66	667	428	0.0245	0.0051
C4	0.88	908	600	0.0346	0.0018
C5	1.10	1160	780	0.0293	0.0036
C6	1.60	1448	882	0.0212	0.0064
D1	0.22	225	590	0.0099	0.0061
D2	0.44	451	920	0.0097	0.0045
D3	0.66	739	1010	0.0210	0.0092
D4	0.88	985	1002	0.0141	0.0092
D4A	0.88	950	994	0.0333	0.0140
D5	1.24	1184	1210	0.0168	-- (1)
D5A	1.24	1211	1250	0.0359	0.0209
D6	1.60	1552	1470	0.0299	0.0112

(1) Not recorded, instrument malfunction.

TABLE 3.1C - TEST DATA (SERIES E AND F)

Specimen No.	Reinforcement Area, A_{vf} ($in.^2$)	Reinforcement Parameter ρf_y, (psi)	Ultimate Shear Stress v_u (psi)	Slip at Ultimate (in.)	Separation at Ultimate (in.)
E1	0.22	208	420	0.0285	0.0125
E2	0.44	416	710	0.0435	0.0135
E3	0.66	624	850	0.0535	0.0135
E4	0.88	832	1040	0.0370	0.0110
E5	1.24	1230	1190	0.0422	0.0160
E6	1.60	1450	1250	0.0288	0.0100
F1	0.22	204	422	0.0420	0.0200
F2	0.44	416	674	0.0190	0.0087
F3	0.66	624	896	0.0340	0.0170
F4	0.88	832	1210	0.0203	0.0130
F5	1.24	1197	1250	0.0226	0.0145
F6	1.60	1517	1520	0.0207	0.0140

TABLE 3.1D - TEST DATA (SERIES G AND H)

Specimen No.	Reinforcement Area, A_{vf} ($in.^2$)	Reinforcement Parameter ρf_y, (psi)	Ultimate Shear Stress v_u (psi)	Slip at Ultimate (in.)	Separation at Ultimate (in.)
G1	0.22	224	160	0.0890	0.0034
G2	0.44	488	264	0.0600	0.0014
G3	0.66	732	384	0.0435	0.0056
G4	0.88	944	500	0.0400	0.0100
G5	1.24	1216	586	0.0240	0.0087
G6	1.60	1498	778	0.0350	0.0065
H1	0.22	240	188	0.0645	0.0115
H2	0.44	480	322	0.0240	0.0026
H3	0.66	720	460	0.0235	0.0058
H4	0.88	960	510	0.0168	0.0065
H5	1.24	1157	654	0.0195	0.0064
H6	1.60	1488	760	0.0350	0.0105

APPENDIX

NOTATION

A_{cr} = Area of shear plane, sq. in.

A_{vf} = Area of reinforcement crossing the shear plane, sq. in.

f'_c = Compressive strength of concrete measured on 6 x 12 in. cylinders, psi.

f_y = Yield point stress of reinforcement, psi.

V_u = Ultimate shear force, kips.

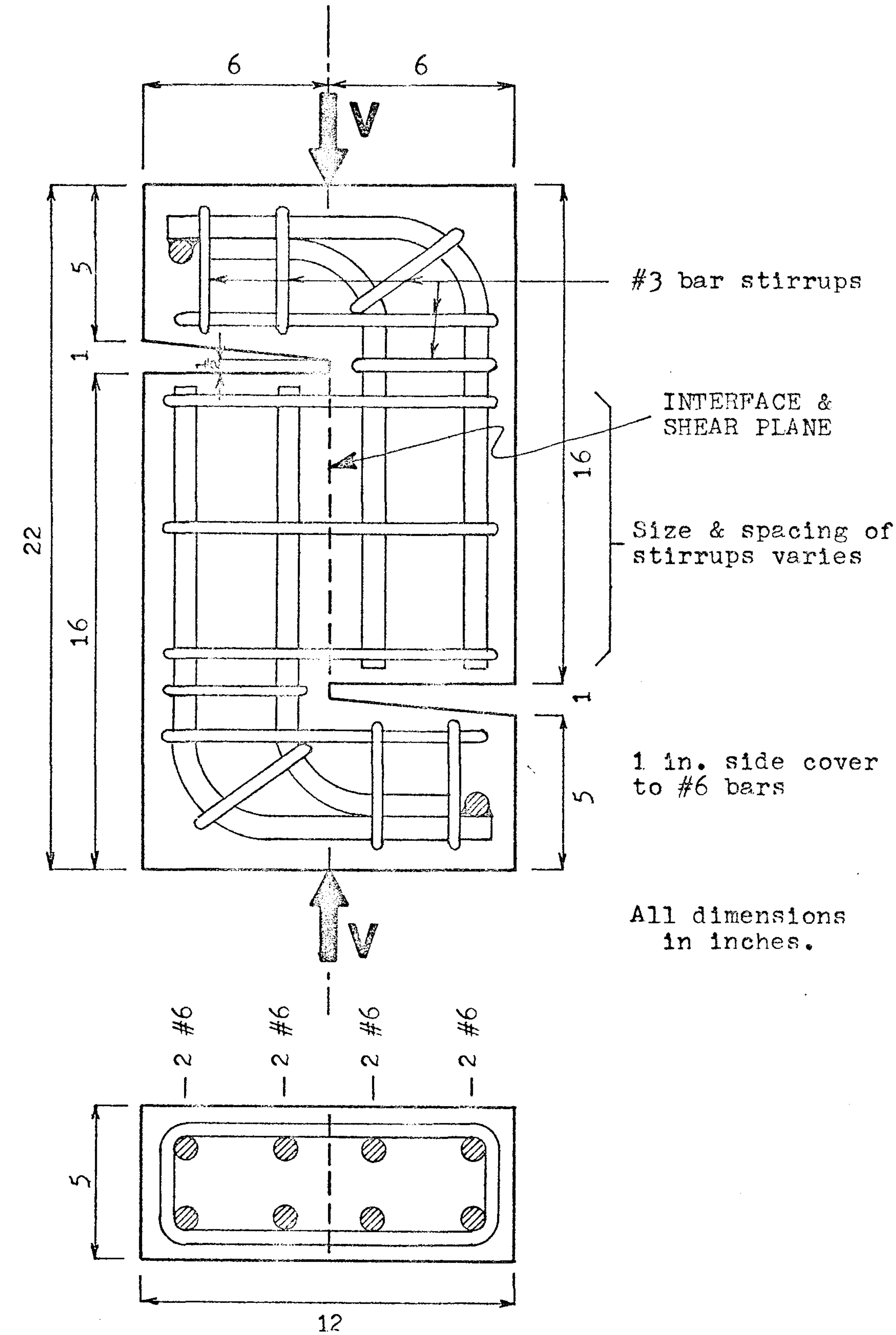
v_u = Nominal ultimate shear stress, psi.

= $1000\ V_u/A_{cr}$, psi.

μ = Coefficient of friction used in shear-friction calculations.

ρ = A_{vf}/A_{cr}

ϕ = Capacity reduction factor, as per Sec. 9.2 of ACI 318-71.



Section at mid-height

Fig. 2.1 - Push-off specimen

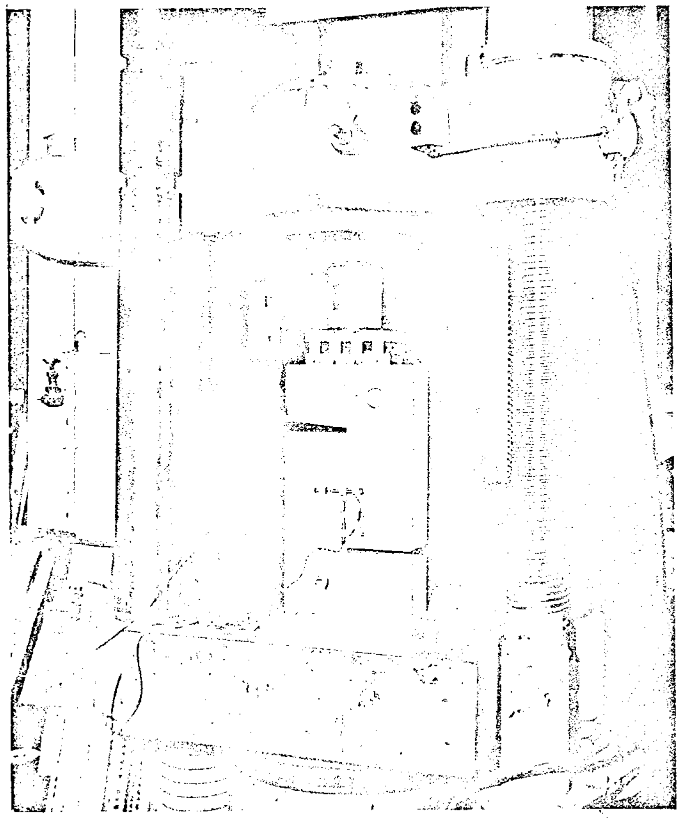


Fig. 2.2 - Arrangements for test of push-off specimen

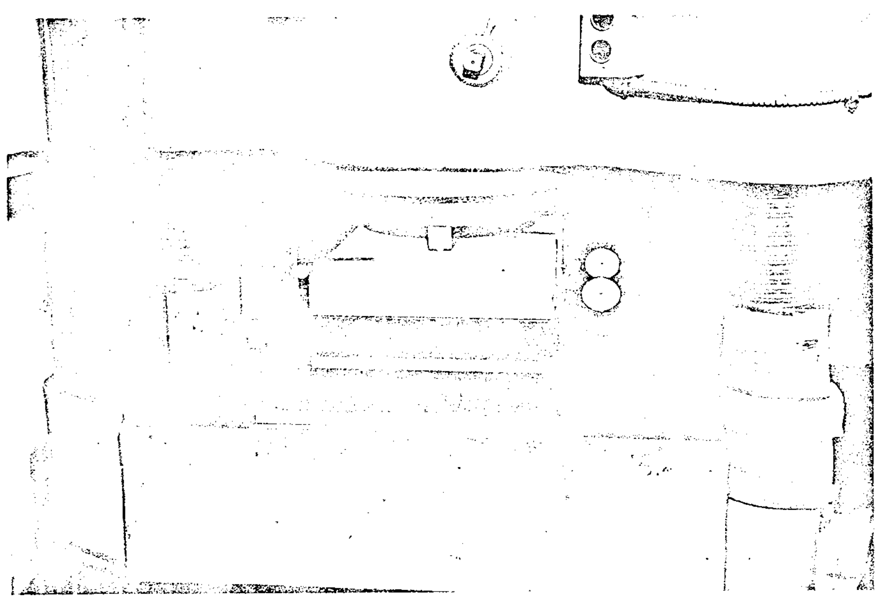


Fig. 2.3 - Cracking of an Initially Cracked Specimen
before shear transfer test

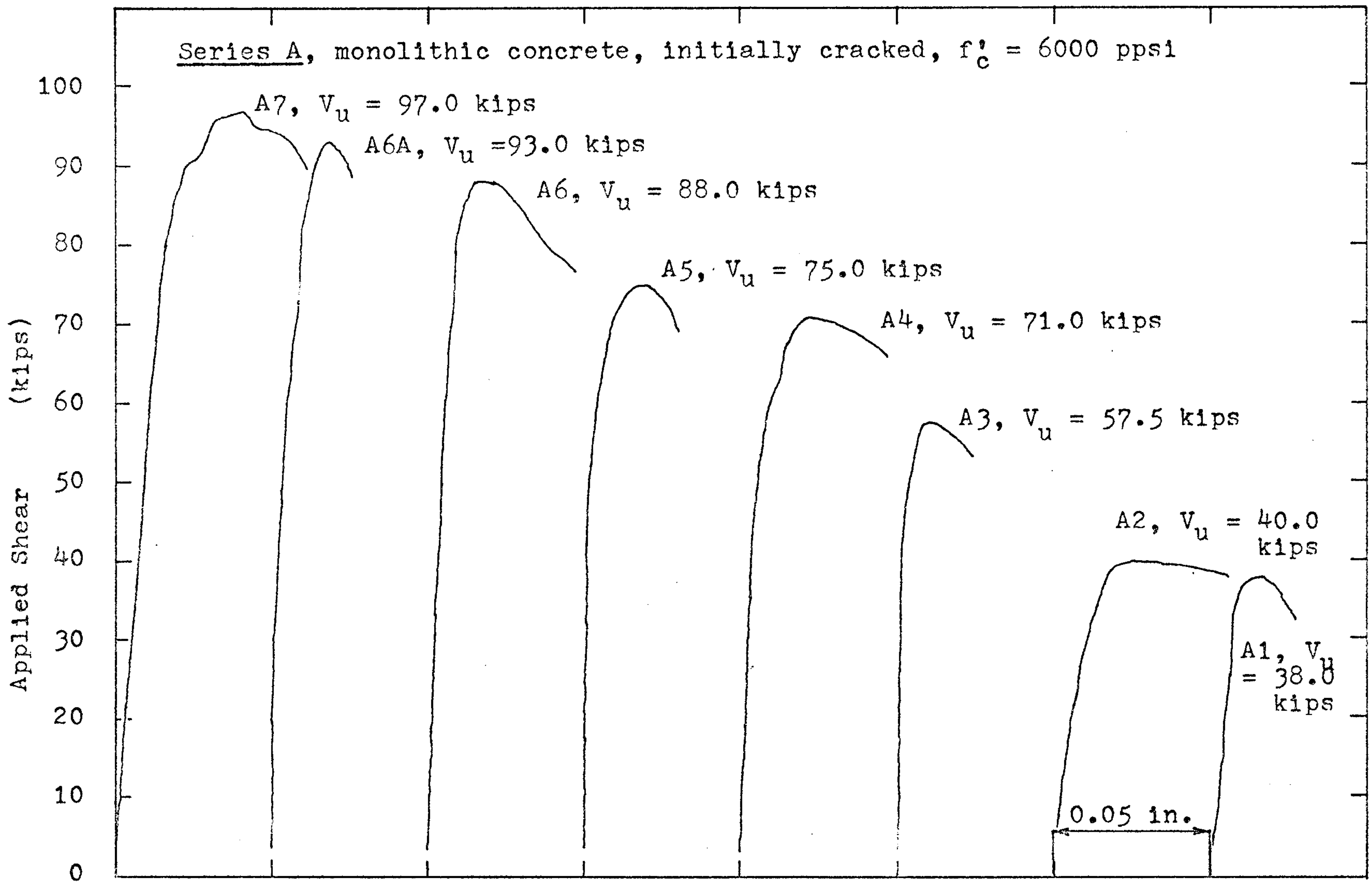


Fig. 3.1 - Shear-slip curves, Series A

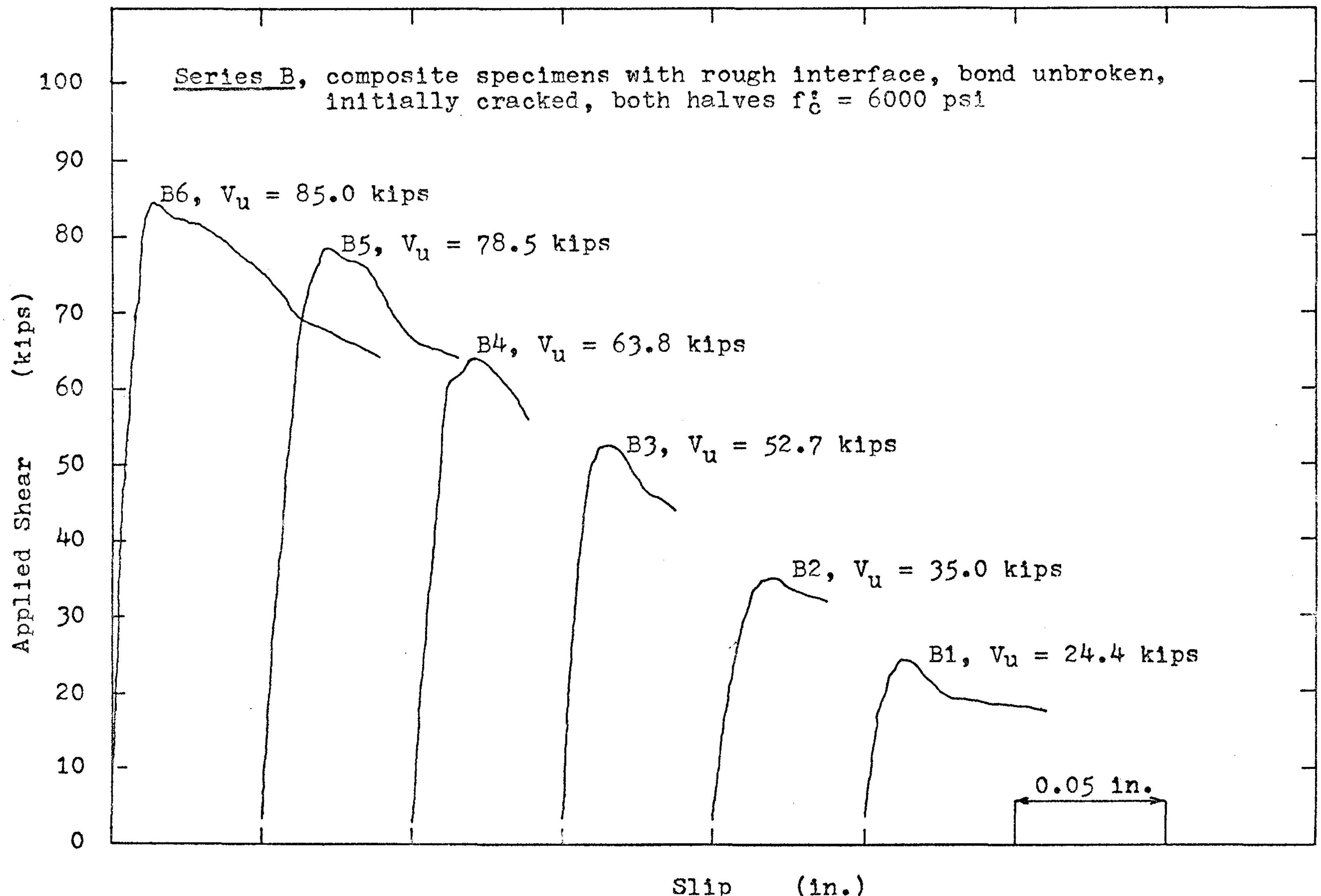


Fig. 3.2 - Shear-slip curves, Series B

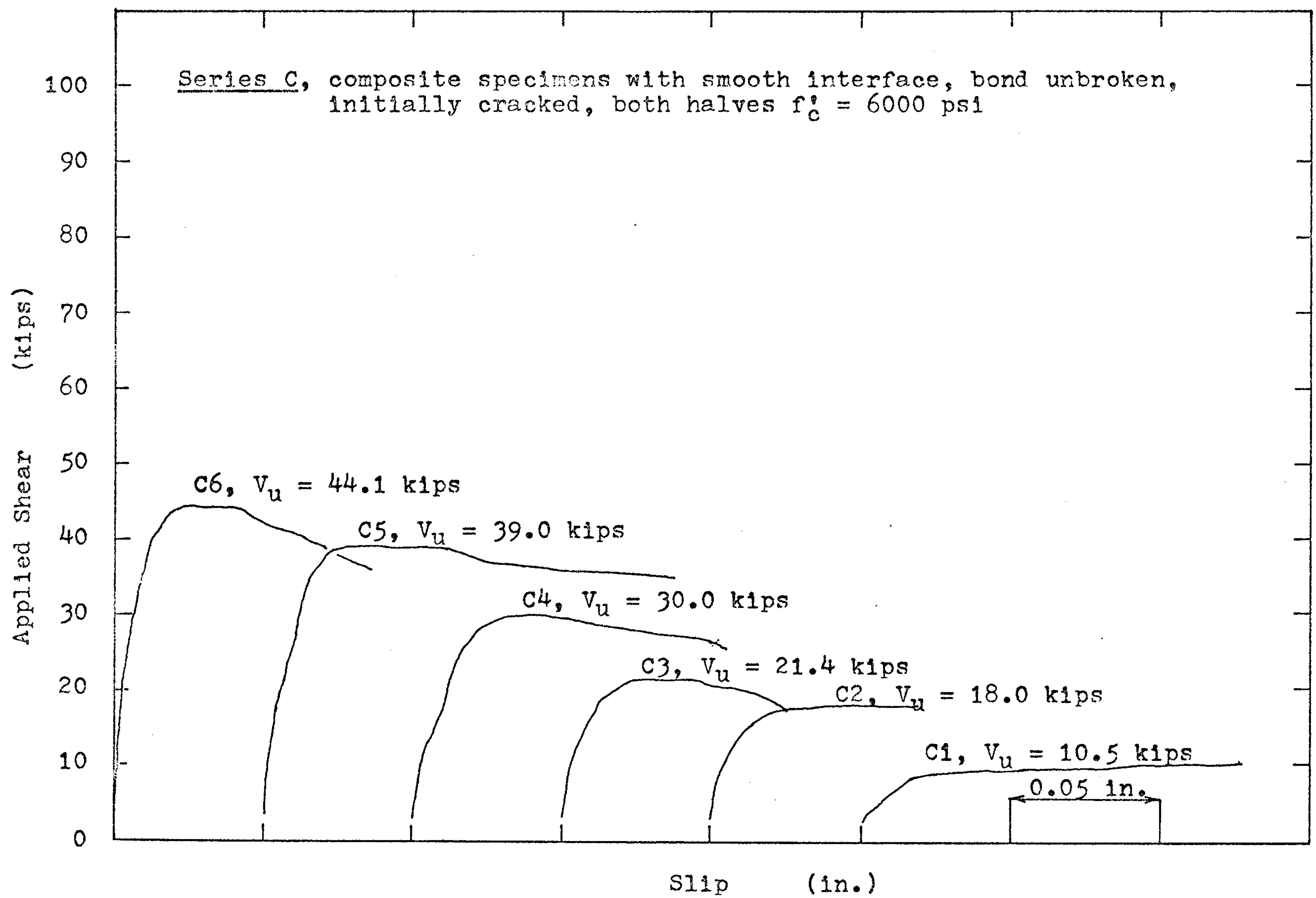


Fig. 3.3 - Shear-slip curves, Series C

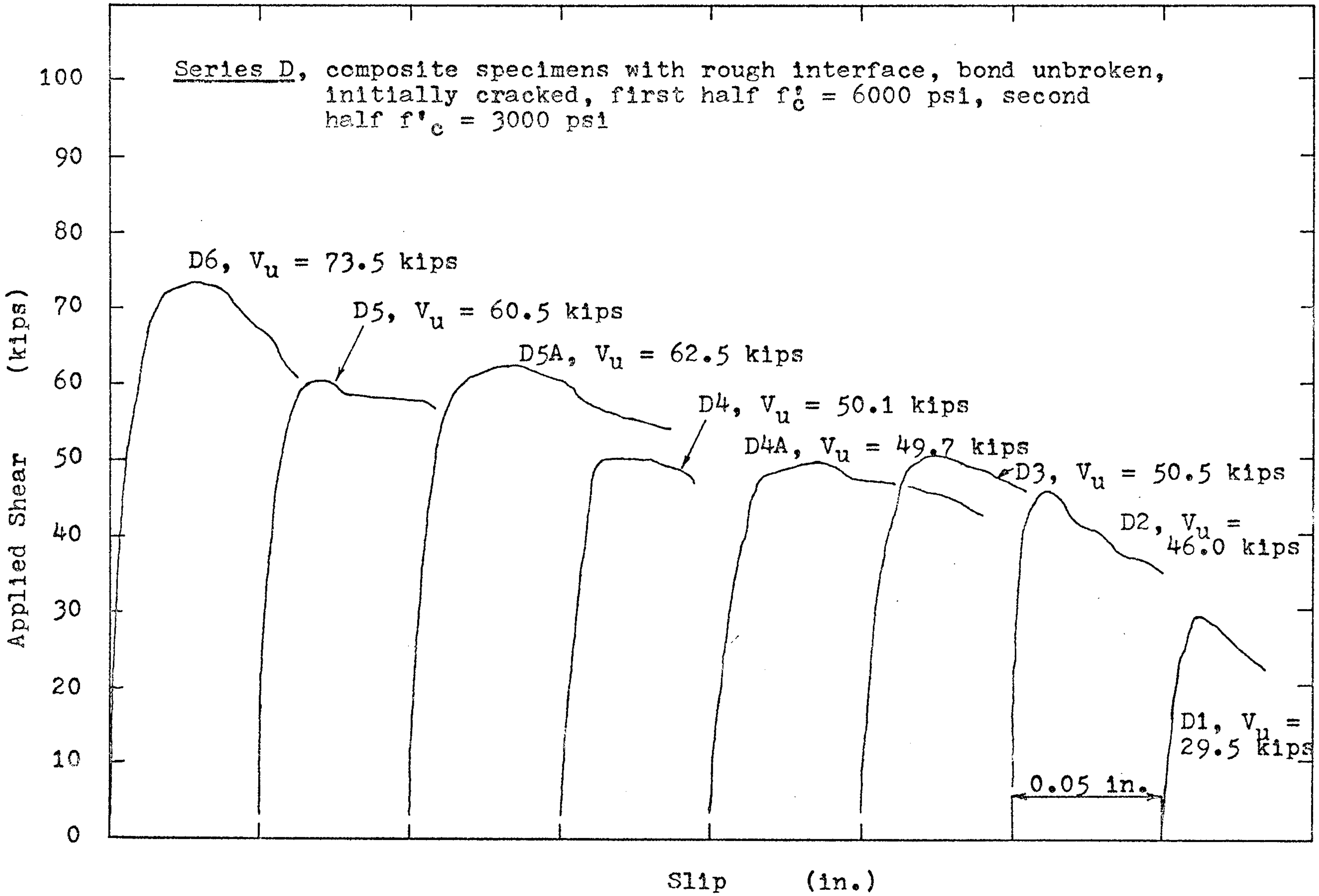


Fig. 3.4 - Shear-slip curves, Series D

41

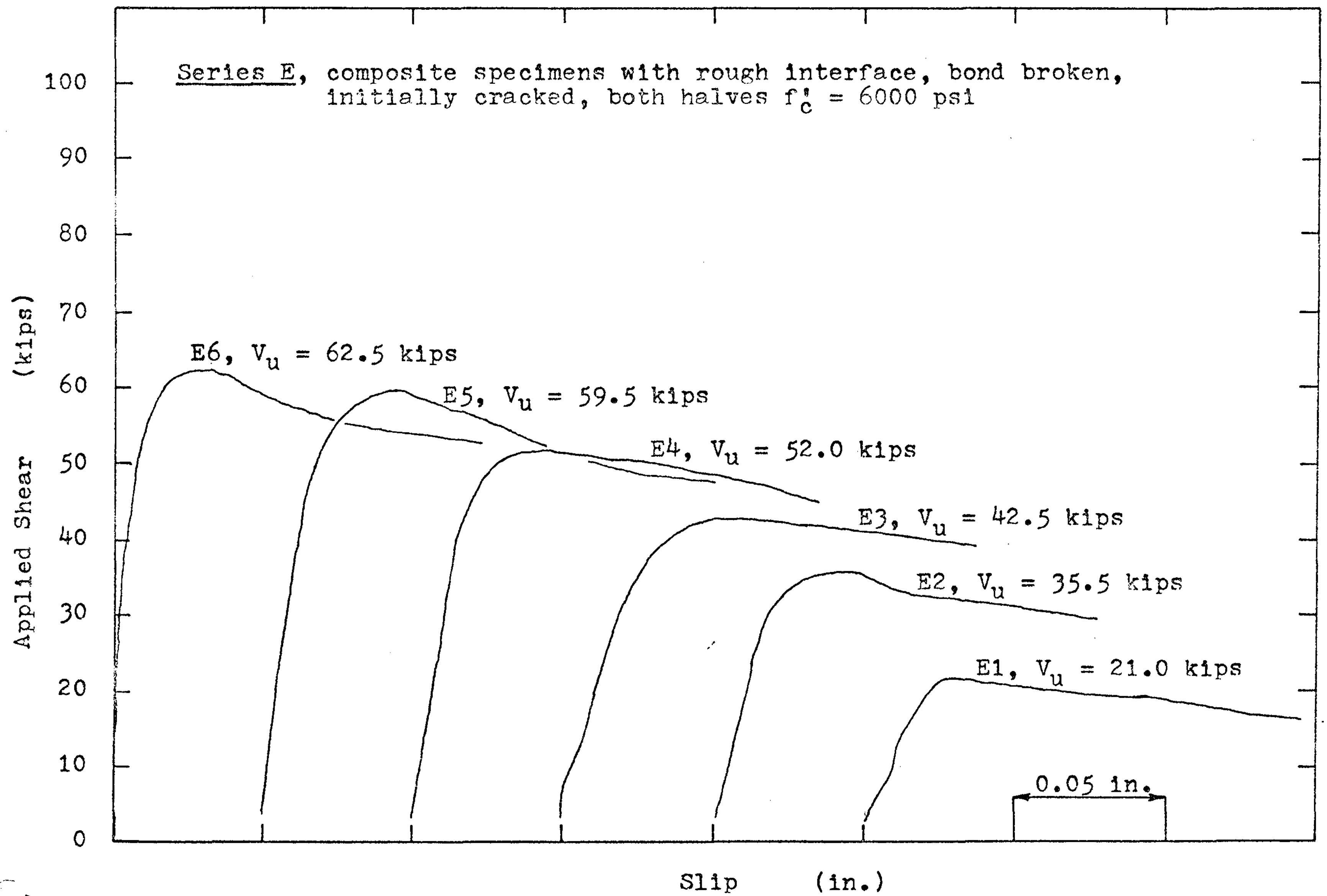


Fig. 3.5 - Shear-slip curves, Series E

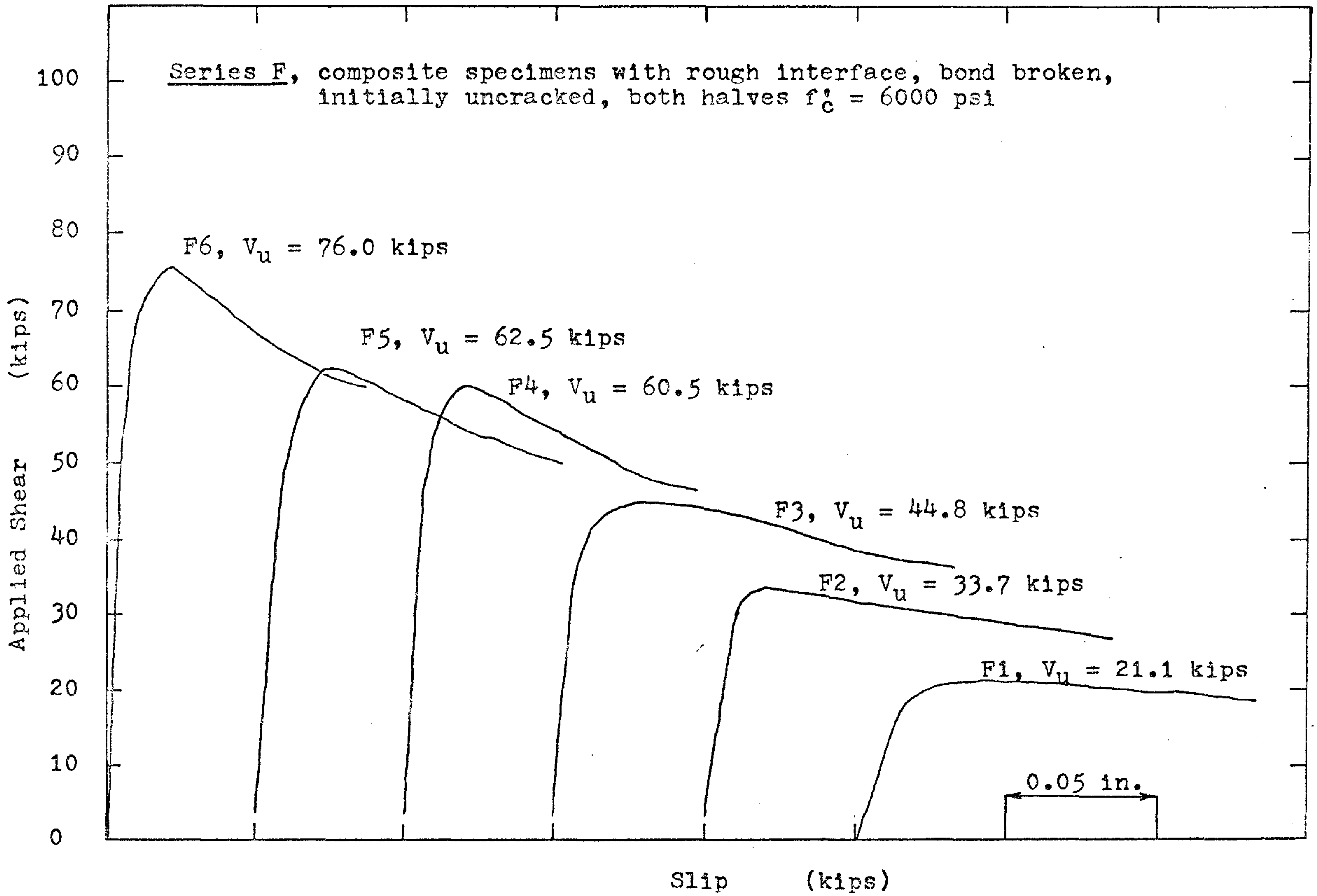


Fig. 3.6 - Shear-slip curves, Series F

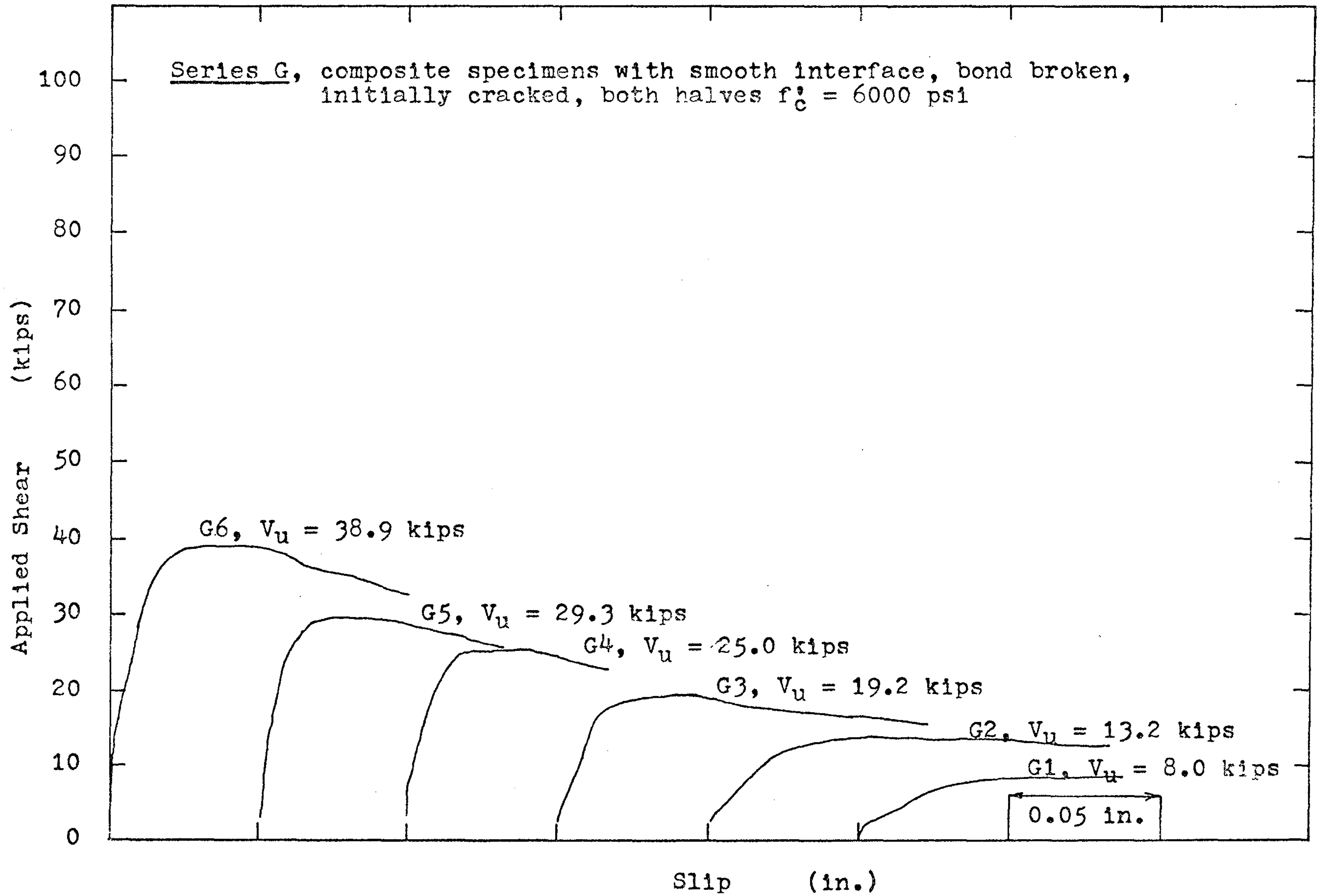


Fig. 3.7 - Shear-slip curves, Series G

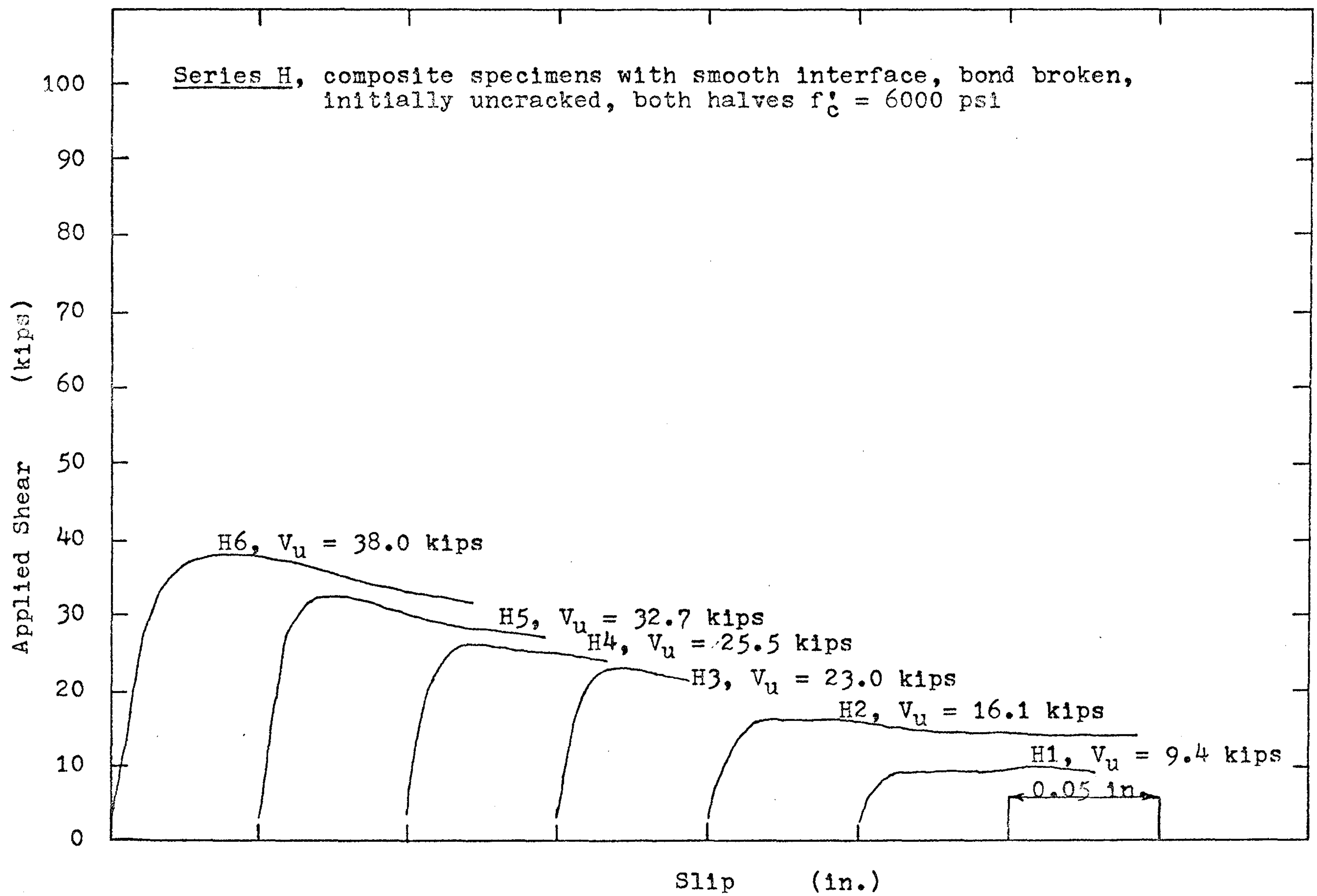


Fig. 3.8 - Shear-slip curves, Series H

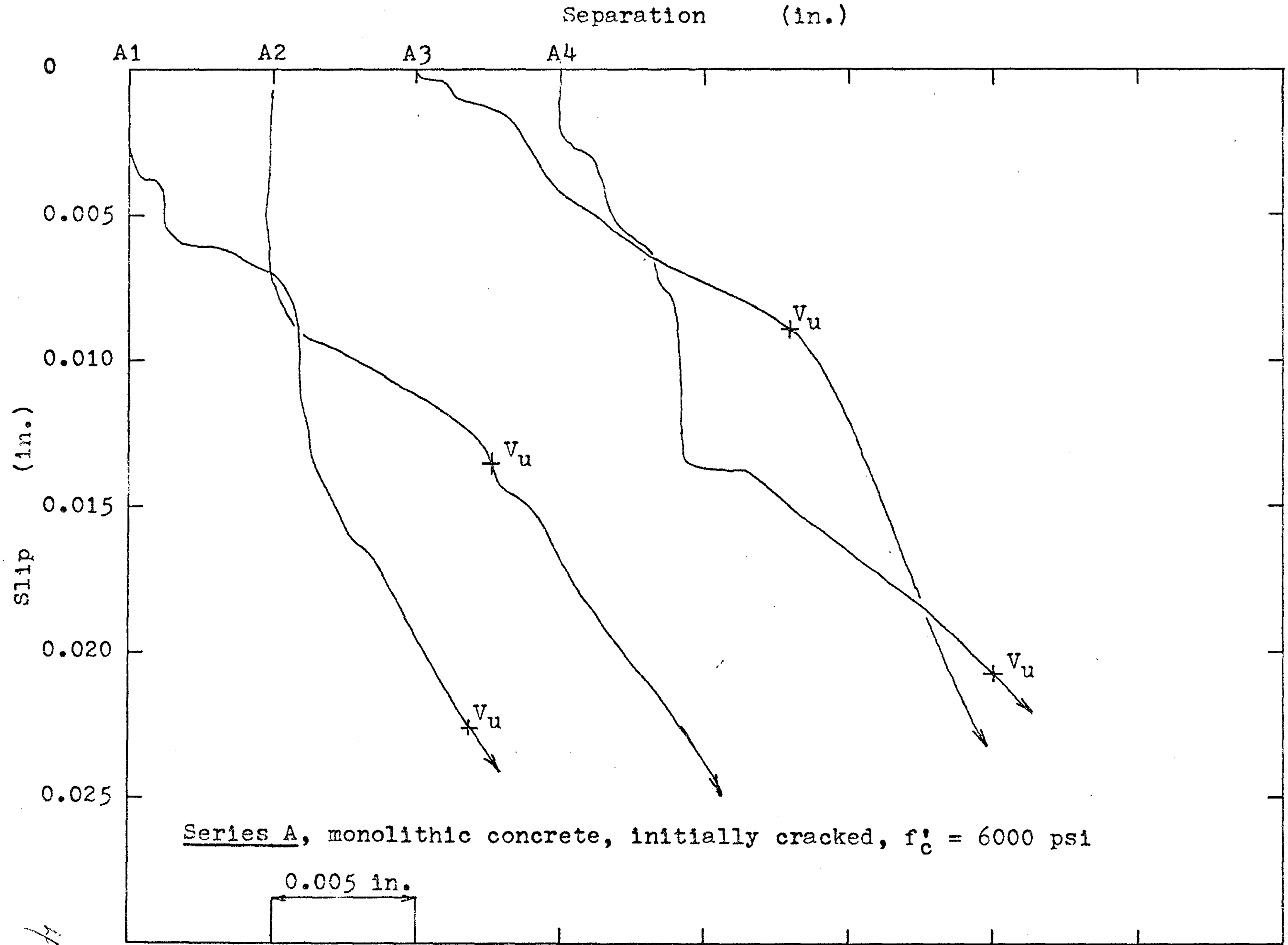


Fig. 3.9a - Slip-separation curves, Series A

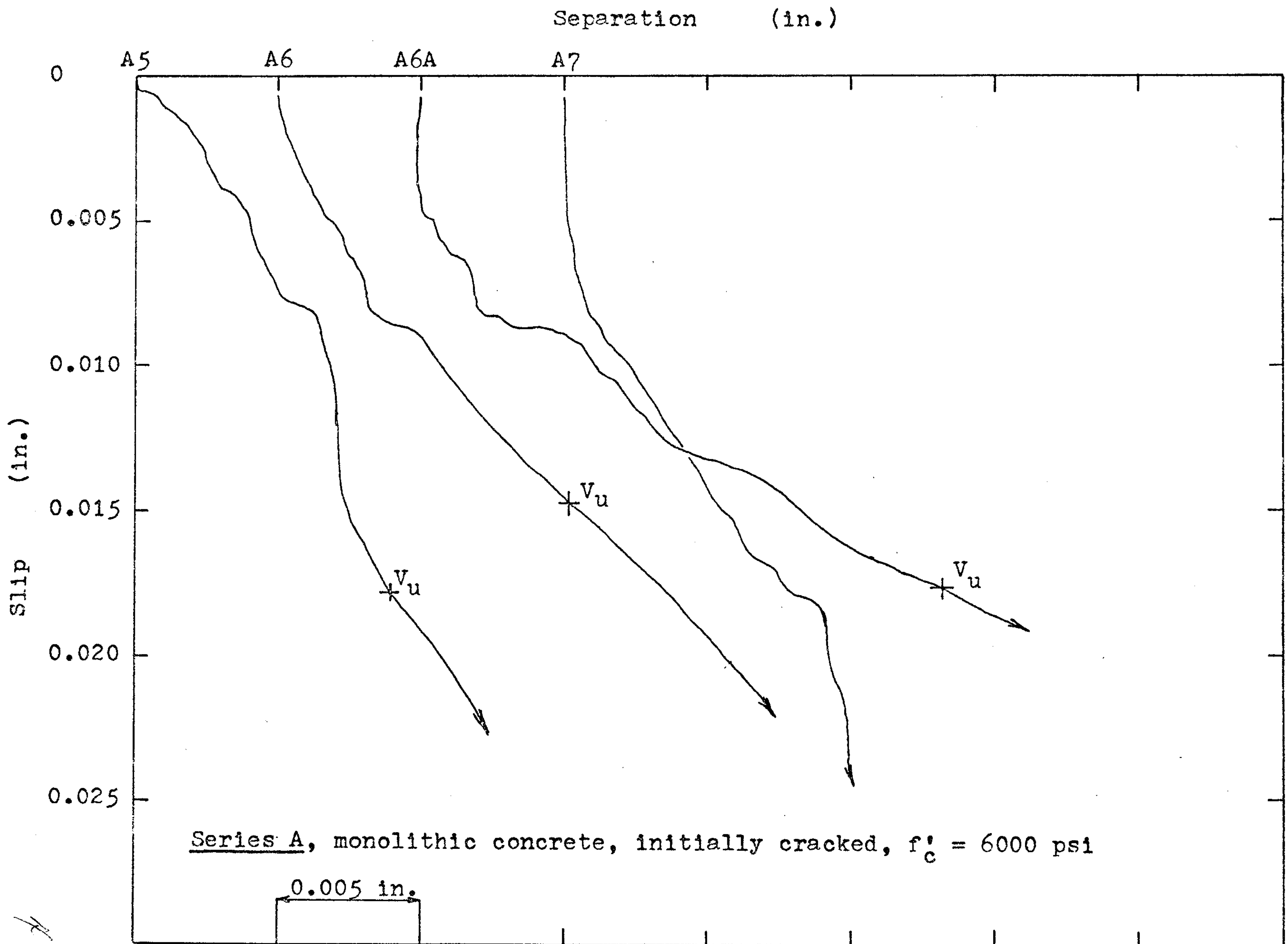


Fig. 3.9b - Slip-separation curves, Series A

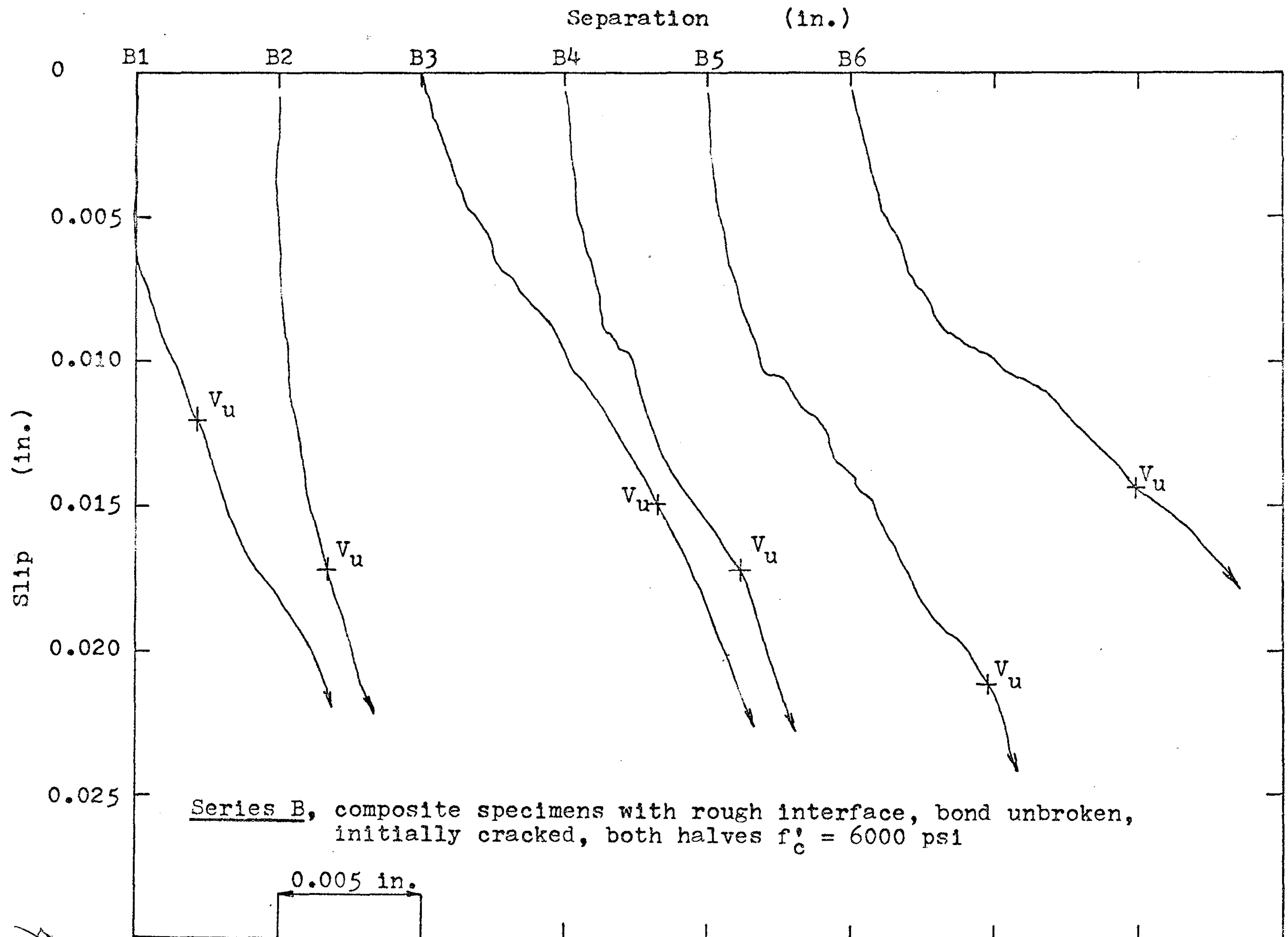


Fig. 3.10 - Slip-separation curves, Series B

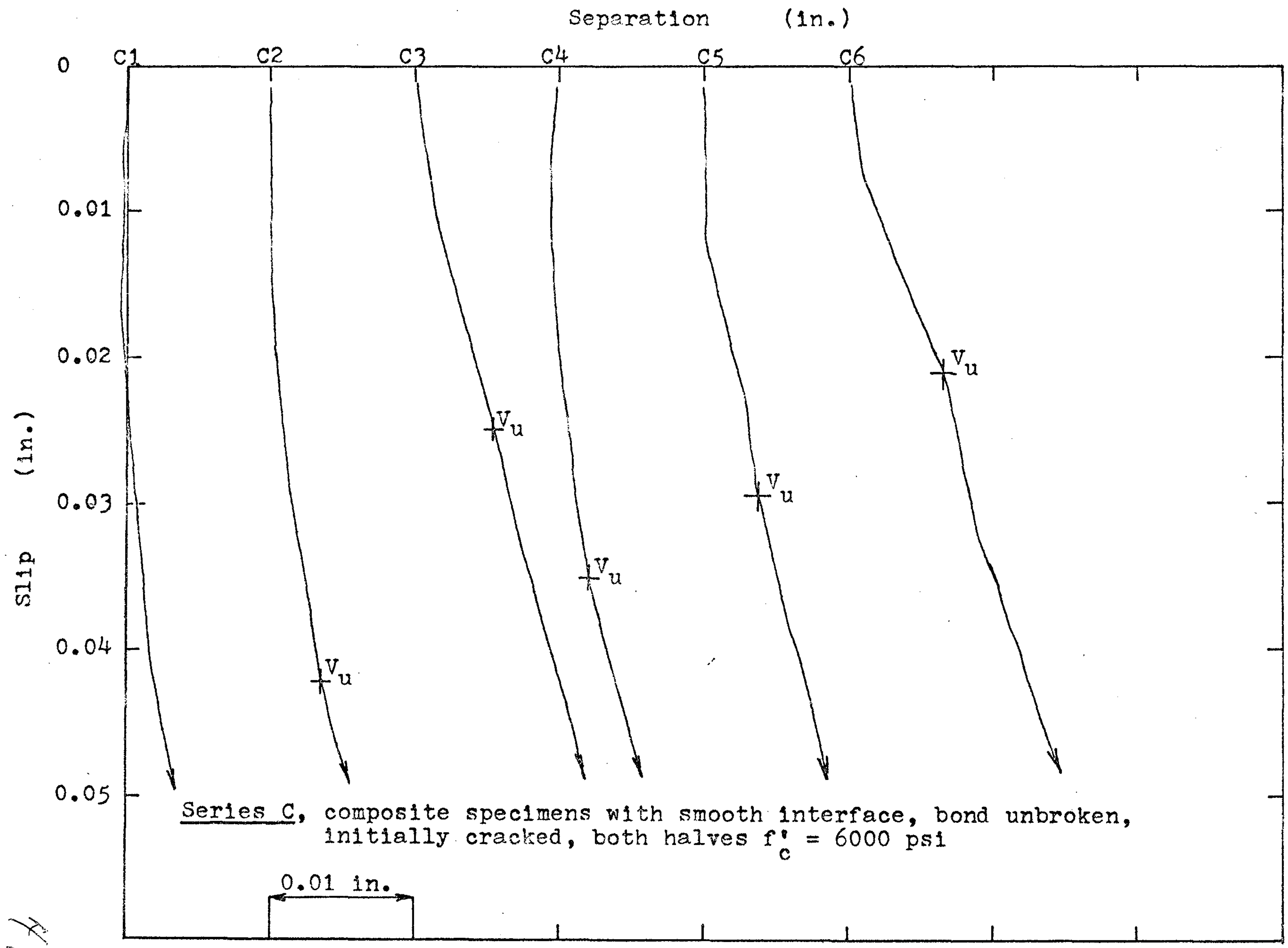


Fig. 3.11 - Slip-separation curves, Series C

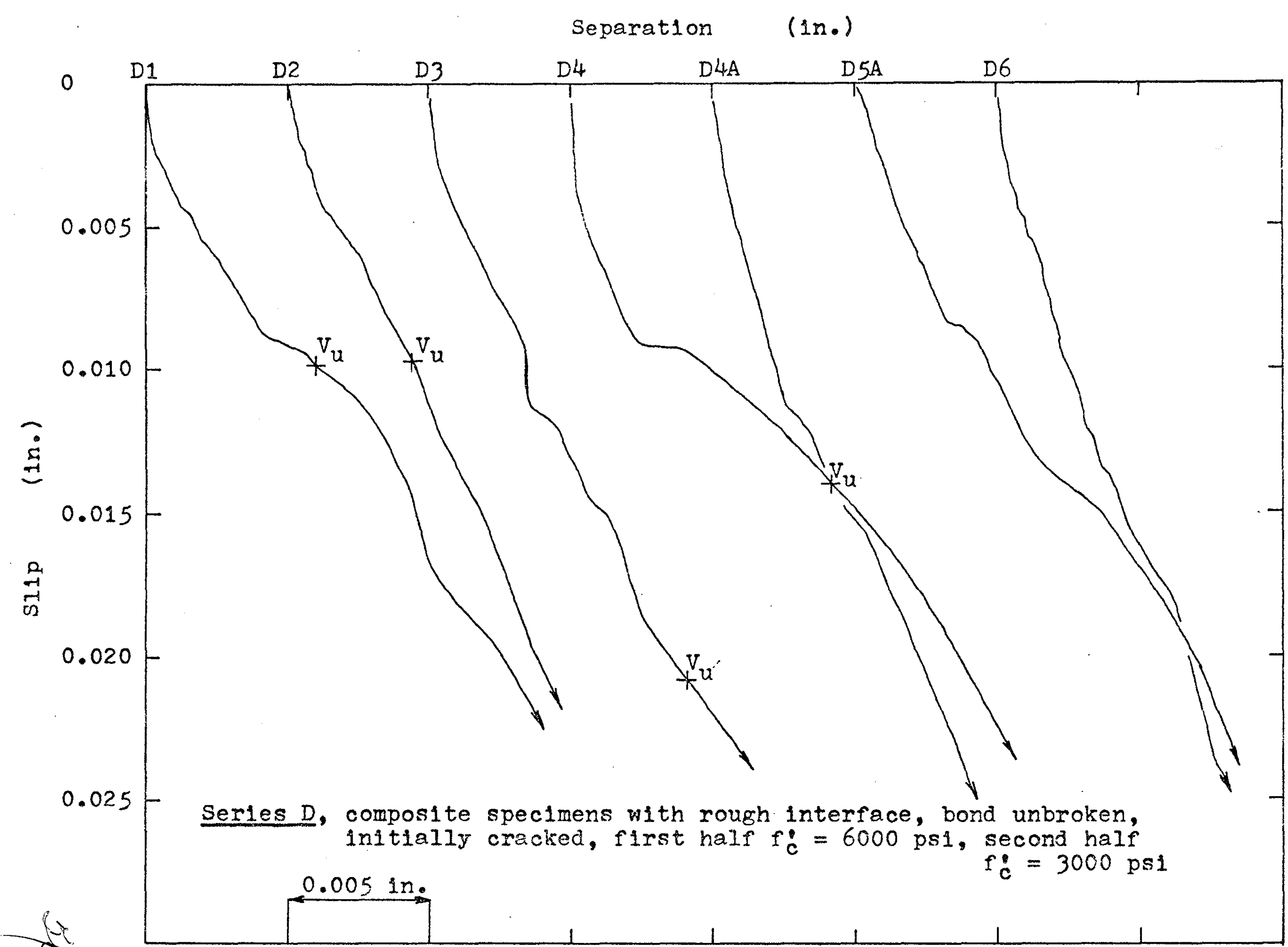


Fig. 3.12 - Slip-separation curves, Series D

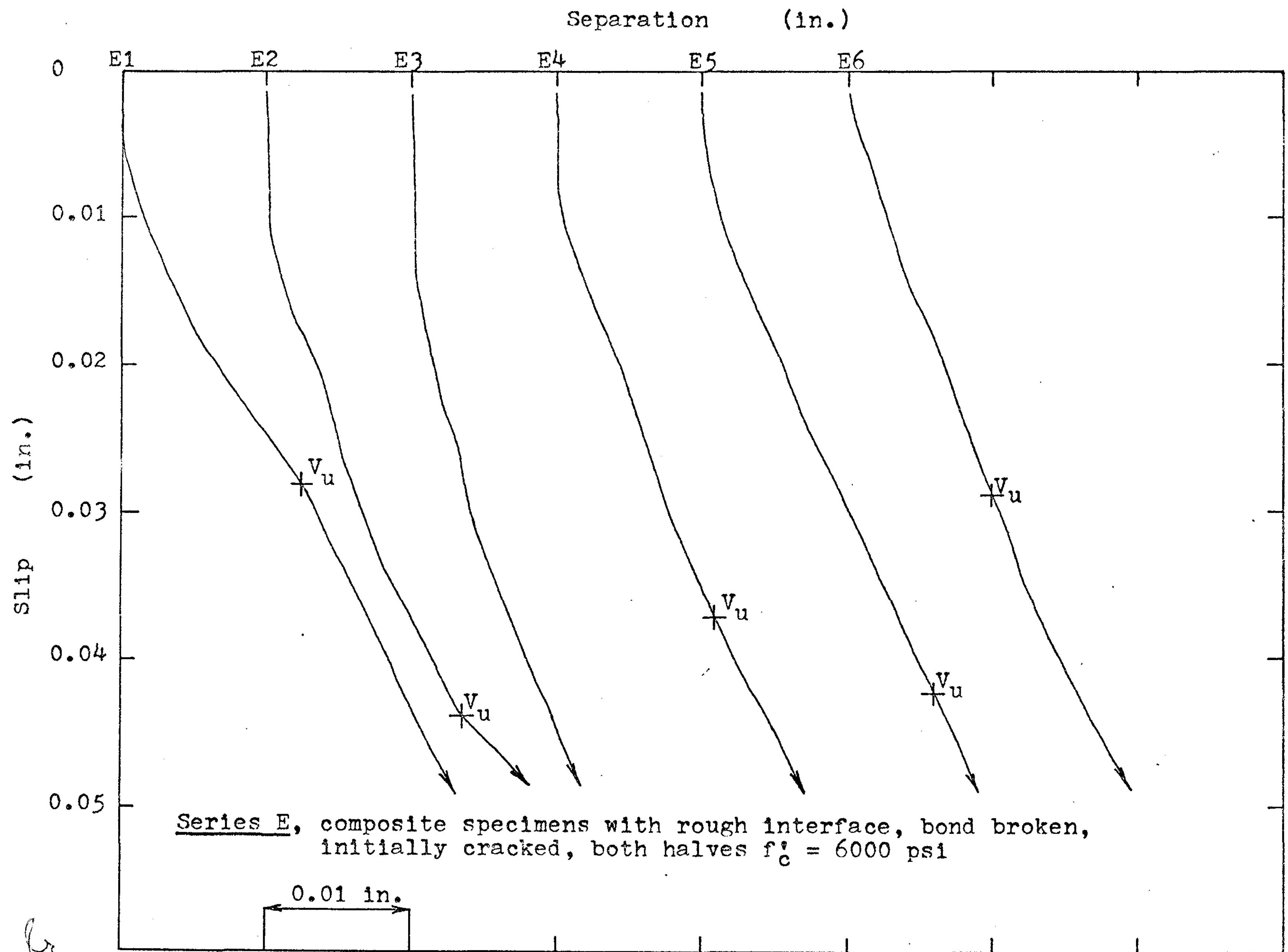


Fig. 3.13 - Slip-separation curves, Series E

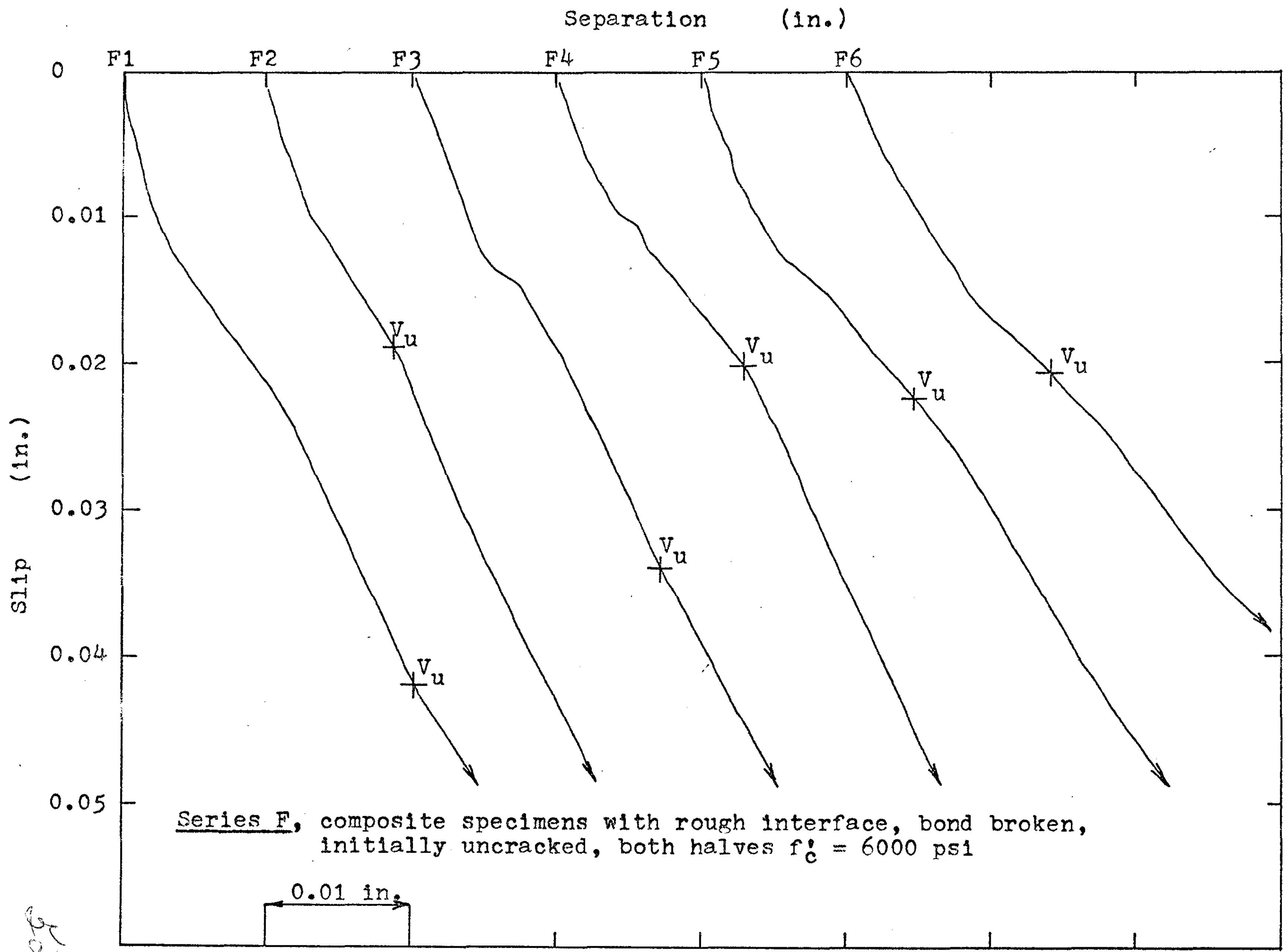


Fig. 3.14 - Slip-separation curves, Series F

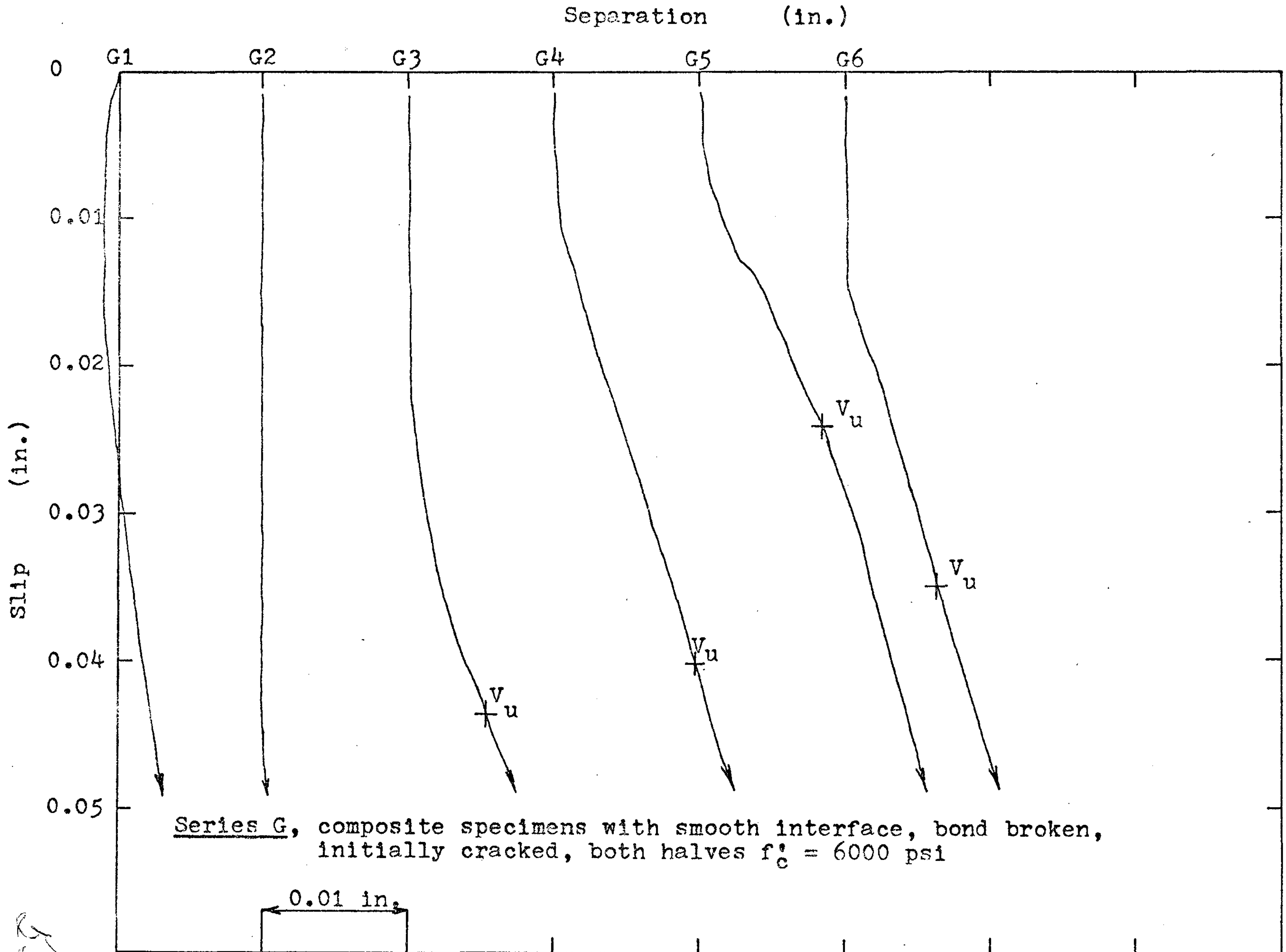


Fig. 3.15 - Slip-separation curves, Series G

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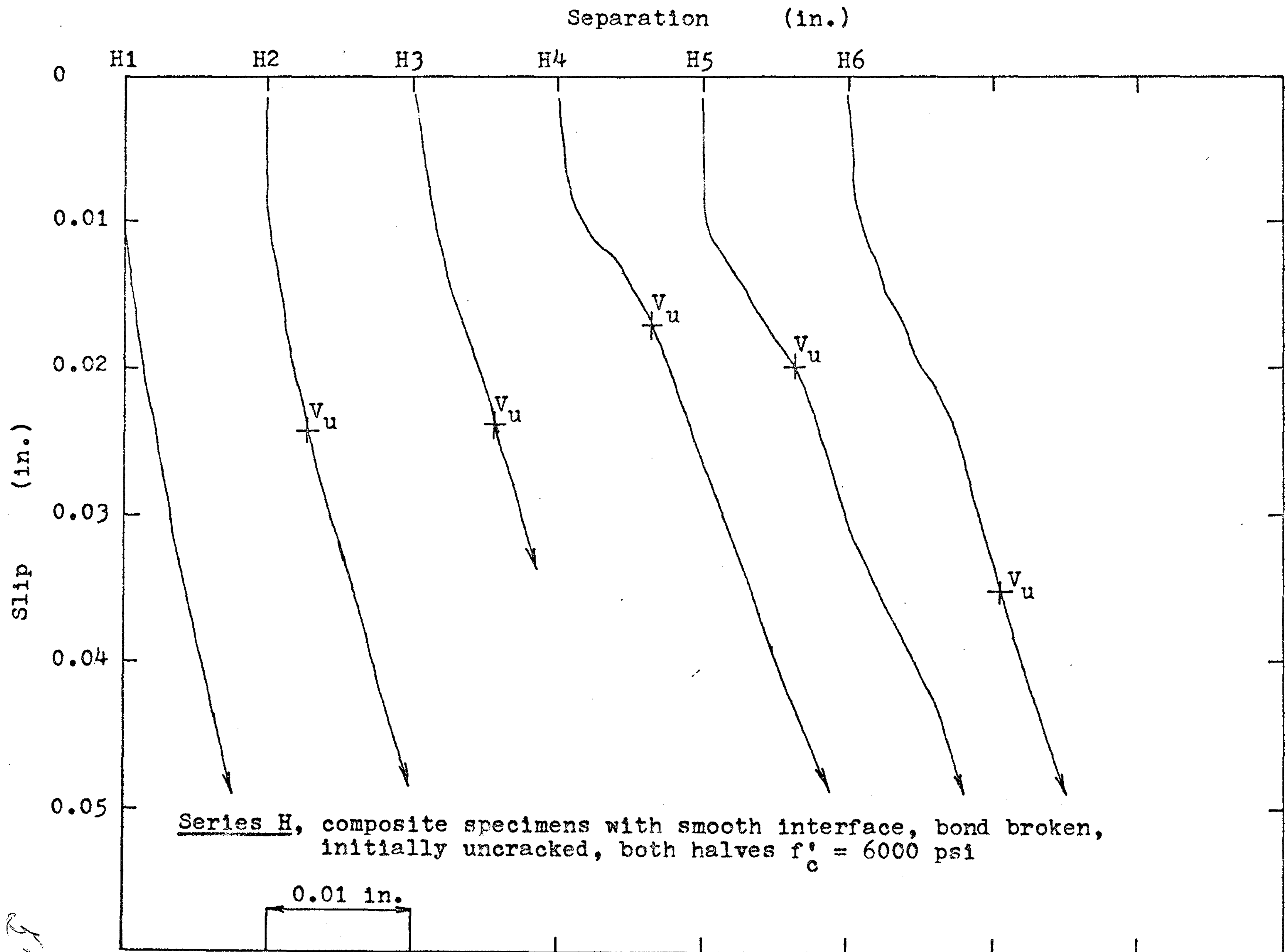

Fig. 3.16 - Slip-separation curves, Series H

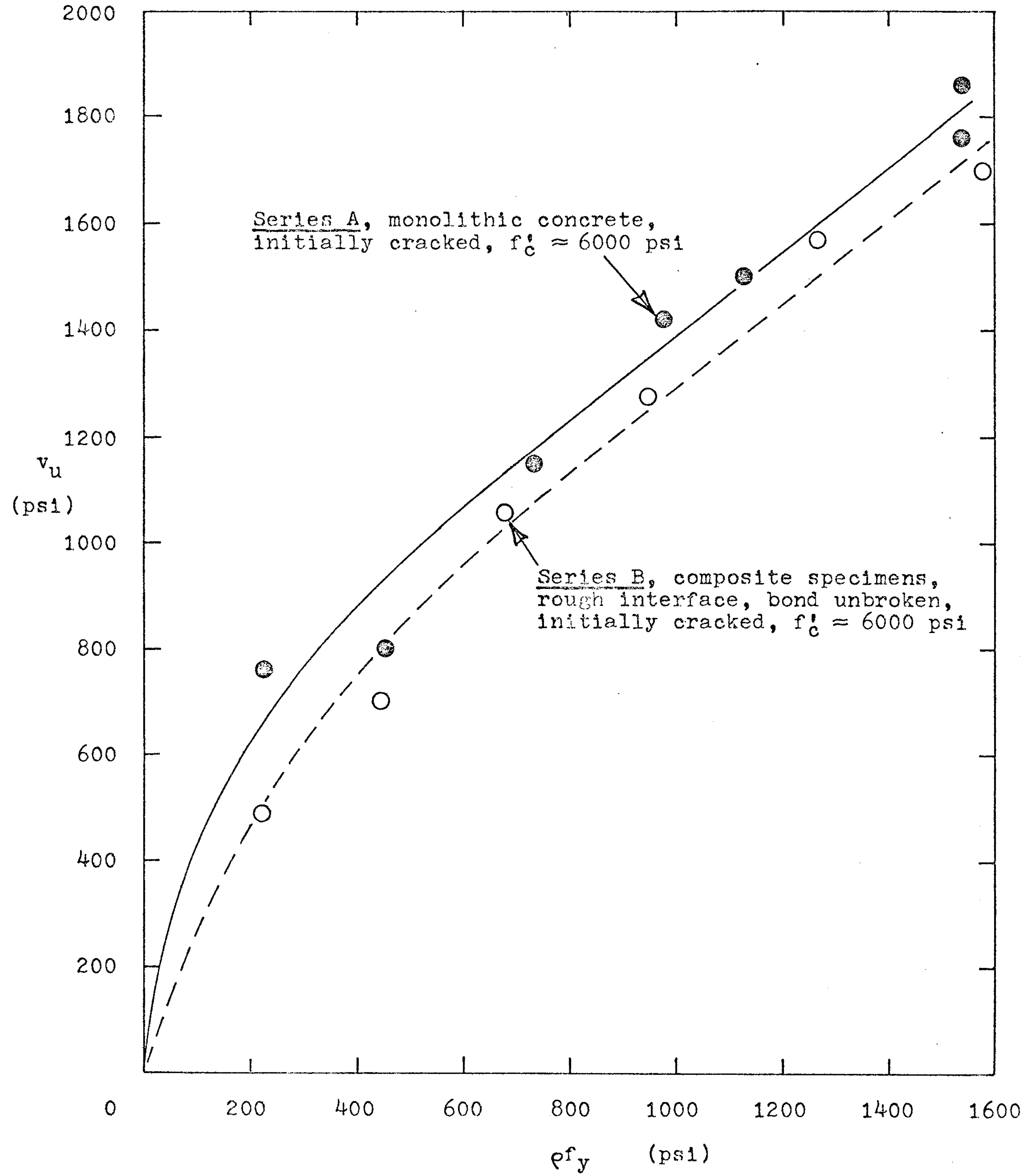

Fig. 4.1 - Comparison of the shear transfer strengths of initially cracked monolithic specimens and initially cracked composite specimens with a deliberately roughened interface.

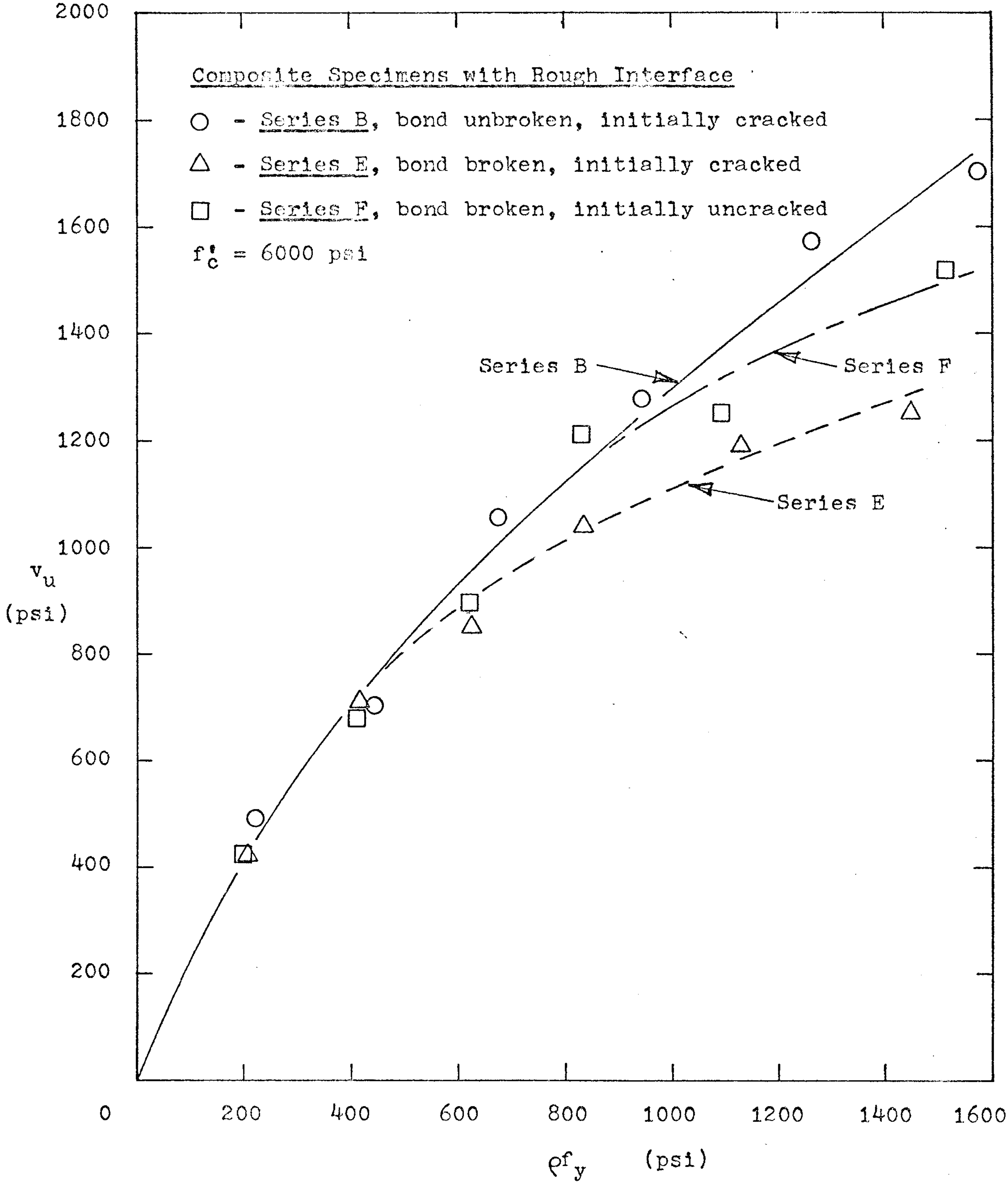

Fig. 4.2 - Influence of interface conditions on the shear transfer strength of composite specimens with a deliberately roughened interface.

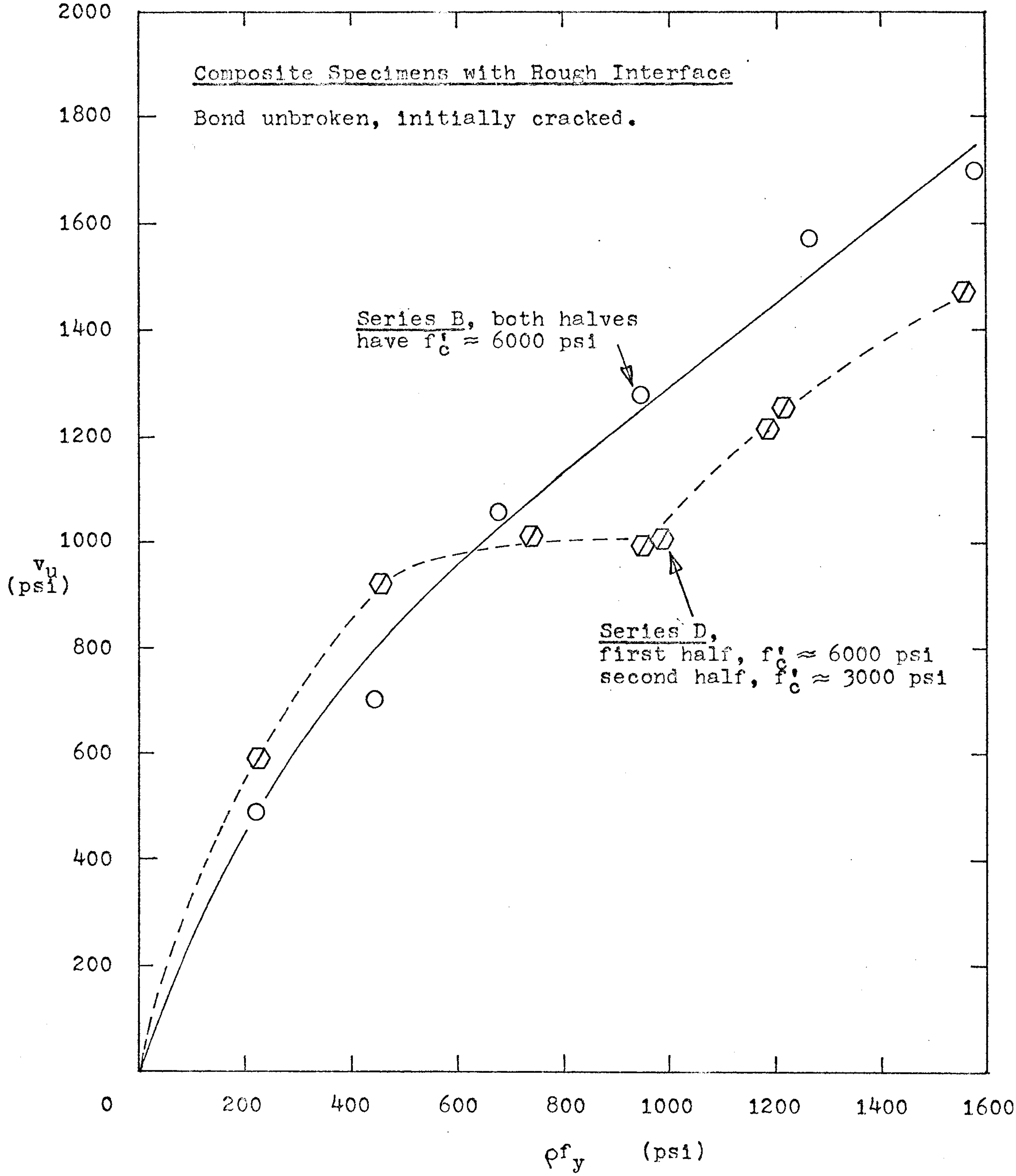

Fig. 4.3 - Comparison of the shear transfer strength of composite specimens made using the same or different strengths of concrete in the two halves of the specimen.

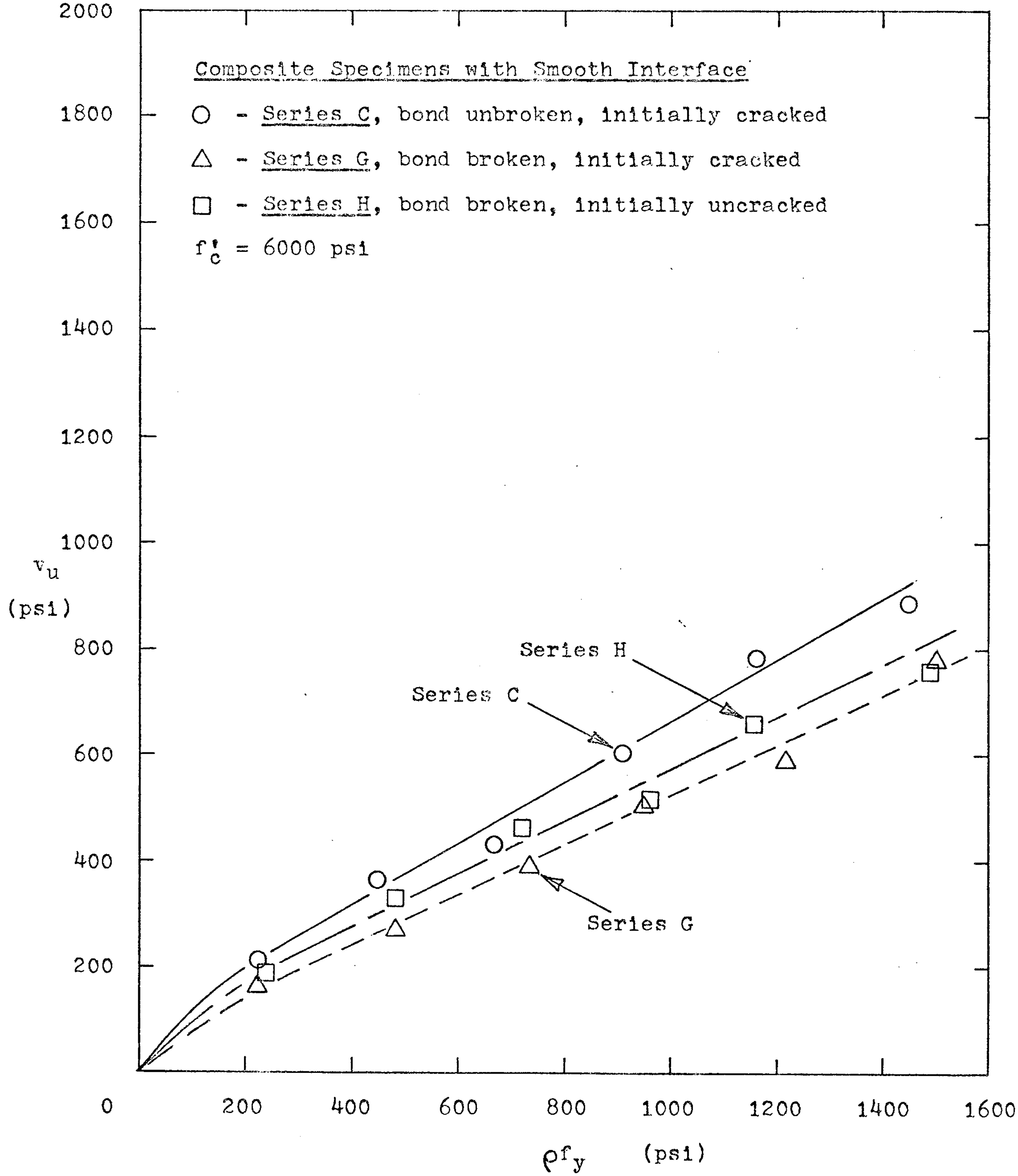

Fig. 4.4 - Influence of interface conditions on the shear transfer strength of composite specimens with a smooth interface.

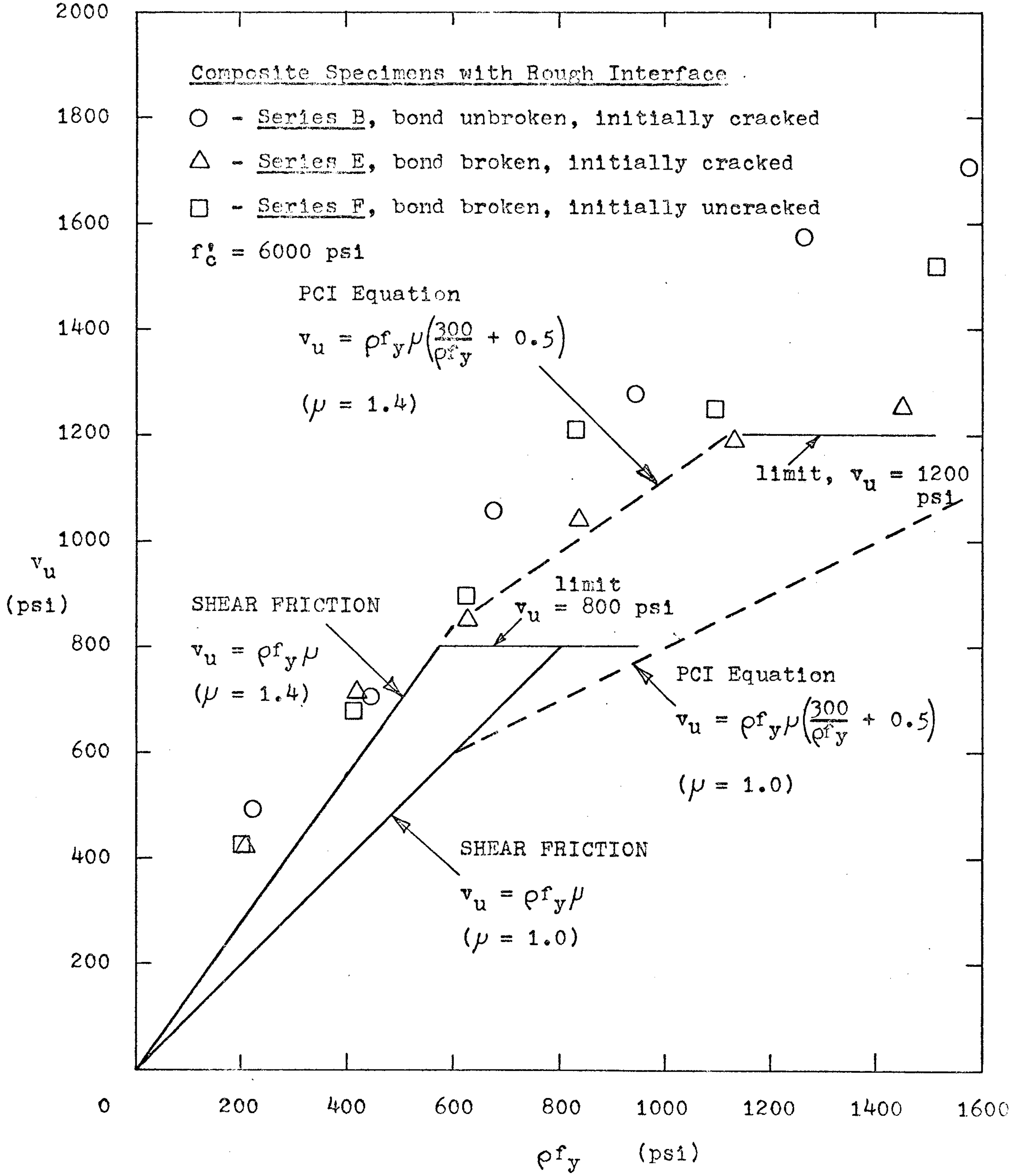

Fig. 4.5 - Comparison of the measured shear transfer strengths of composite push-off specimens having a deliberately roughened interface, with the strength calculated using the Shear Friction method of calculation.

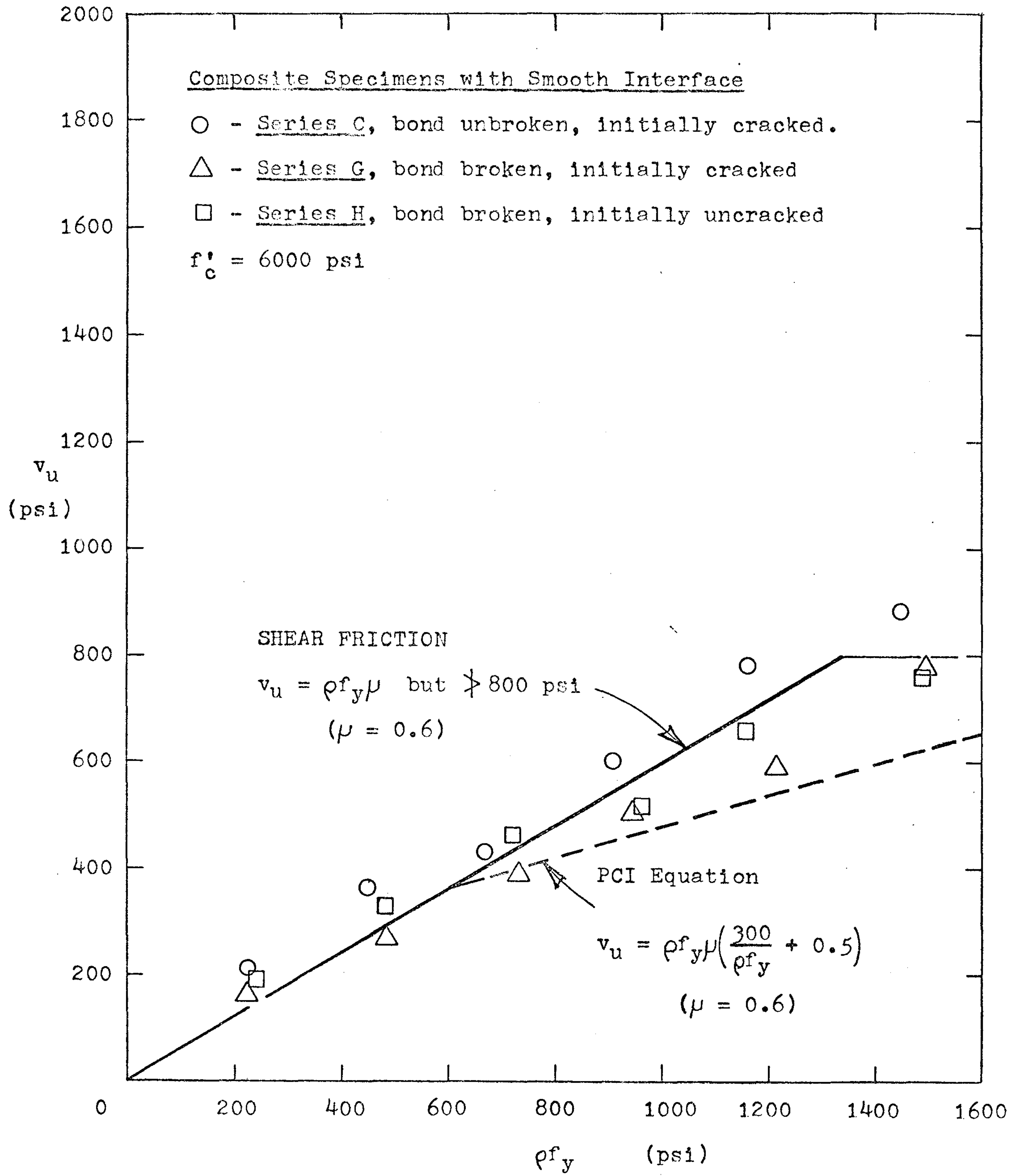

Fig. 4.6 - Comparison of the measured shear transfer strengths of composite push-off specimens having a smooth interface, with the strength calculated using the Shear Friction method of calculation.

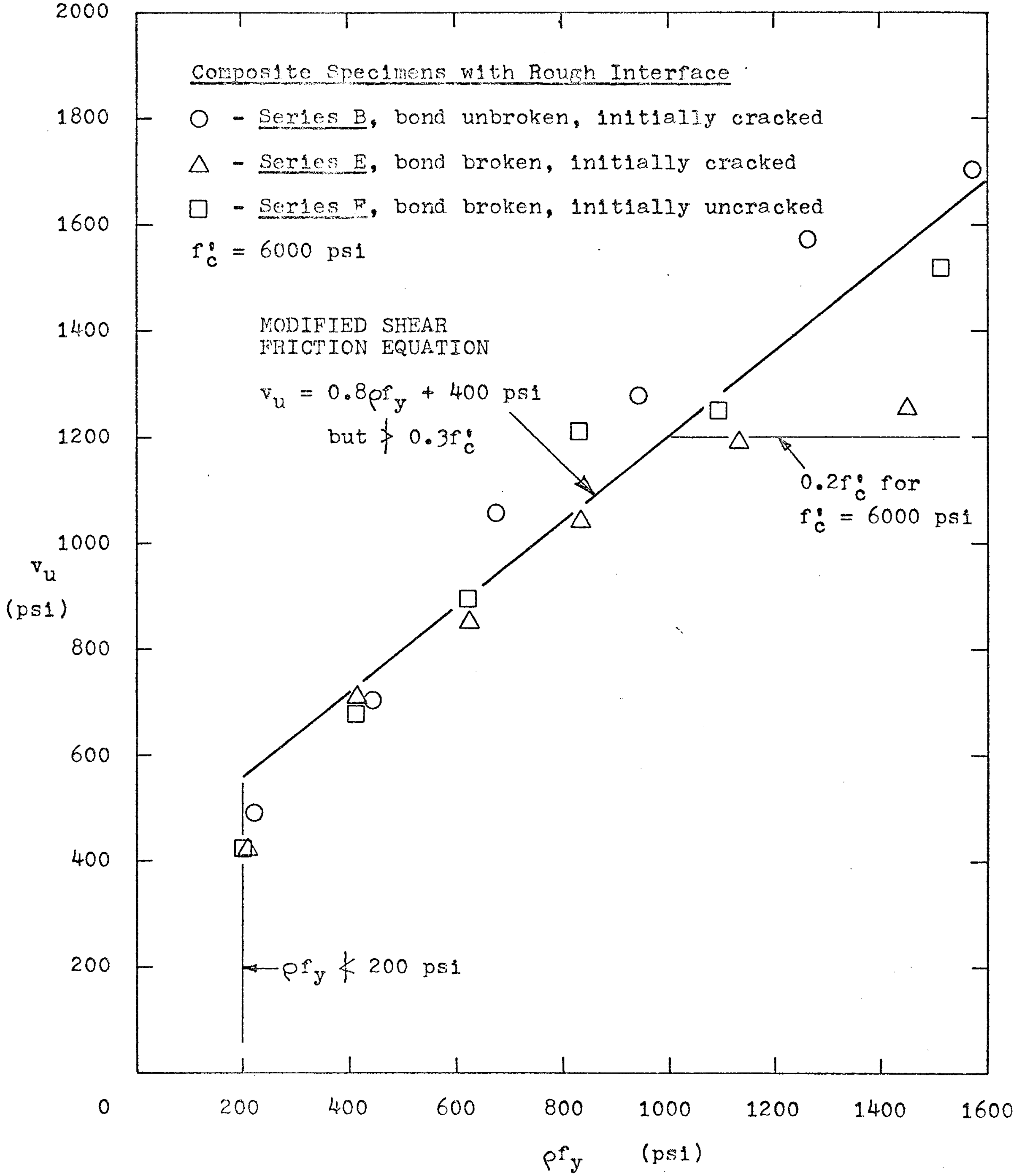

Fig. 4.7 - Comparison of the measured shear transfer strengths of composite push-off specimens having a deliberately roughened interface, with the strength calculated using the Modified Shear Friction equation.

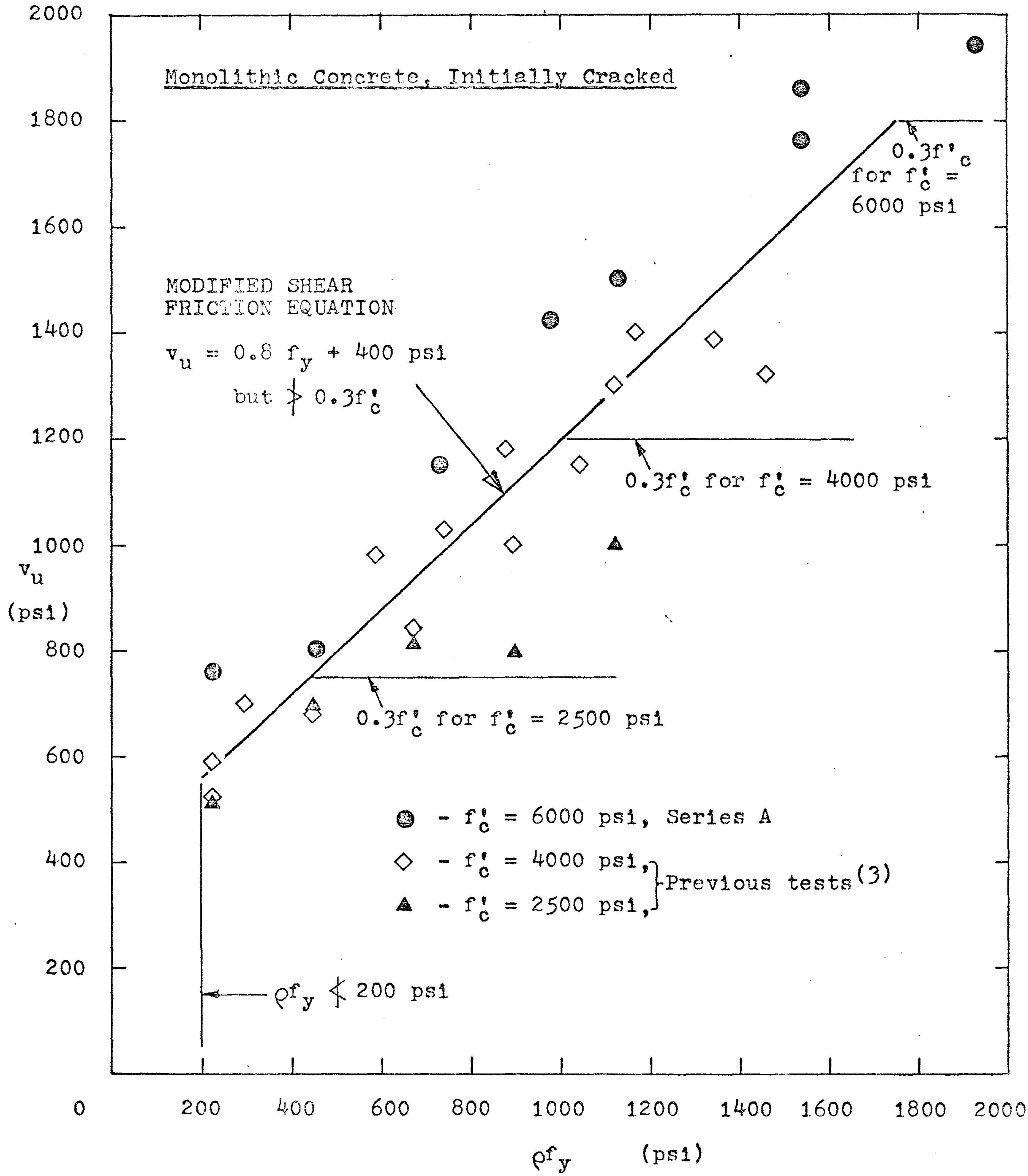

Fig. 4.8 - Comparison of the measured shear transfer strengths of initially cracked, monolithic push-off specimens, with the strength calculated using the Modified Shear Friction equation.

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