

# A THEORETICAL ANALYSIS OF PROGRESSIVE AND RETROGRESSIVE FAILURE IN DISCONTINUOUS BLOCK MASSES SUBJECTED TO 'IN-SITU' SHEAR

'Yerinde' Makaslama Tabi Tullulan Süreksiz Kayag Kütlelerinde İleri ve Geri Yenilme Teorik Analiz

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**ABSTRACT.** — *This paper presents a study of shear deformation which includes a theoretical approach to progressive and retrogressive failure, involving discontinuous and elasüc-elatoplastik finite element method. It has been concluded that choice of boundary conditions exerted an important control on failure mechanism. Under certain boundary conditions, the ultimate failure of the test block is a consequence of multiple fracture mode« 'In-situ\* shear tests on geological materials should be interpreted in more sophisticated terms; Le., as a conséquence of variable combined stress state, involving inhomogeneous stress field, one or several of principal stresses being tensile; 'extensive stress réorientation; and multiple crack propagation. The theoretical approach to the mechanism of shear deformation. The theoretical approach to the mechanism of shear deformation and failure characteristics of 'in-situ' shear tests, utilizing finite element method, appears to be a valid approach for prediction of certain experimental results.*

**ÖZ,** — *Bu yazıda takdim edilen makalede deformasyonu çalışması, süreksiz ve elâstic-elasioplastik sonlu elementler metodu ile, ileri ve geri yenilmenin teorik analizini kapsamaktadır. Bu çalışmada çıkarılan sonuçlara göre, kenar yükleme şartlarının seçim/İn yenilme mekanizmasının kontrolü yönünden çok önemli etkiler vardır. % belirli kriterler/r ümler şartları altında, deneme blokunun en son yenilmesi birden fazla kırılma şeklinin bir neticesidir. Jeolojik materyaller üzerinde yapılan 'yerinde\* makaslama denemeleri, daha değişik bir terminoloji ile, bir veya birkaç asal gerilme tansiyon şeklinde, olduğu, homojen olmayan bir gerilme alanı, yaygın bir gerilme reorientasyonu ve birden fazla çatlak ilerlemesi gibi değişebilen birleşik gerilme hallerinin bir neticesi olarak değerlendirilmeli ve açıklanmalıdır, 'yerinde\* makaslama denemelerindeki makaslama deformasyonunun ve yenilme mekanizmasının, sonlu elementler metodu ile yapılacak teorik analizleri ile bazı deneysel neticeleri önceden belirleyebilme mümkün*

## INTRODUCTION

Direct tests in the laboratory and in 'in-situ' field conditions are important sources of information on strength parameters for soil and rock materials, for both geologic and engineering purposes.

Despite this widespread usage, the physics of deformation involved in these tests are not well understood, and as a result important misinterpretations of the data resulting from sheartesting are possible. One purpose of this work has been, in fact, to obtain some general information on actual mechanism of shear deformation and shear failure in both laboratory and 'in-situ' shear tests, and shed light upon some of these difficulties, in particular the development of progressive failure in isotropic and anisotropic non-linear materials as a function of the method of application of boundary forces.

Under conditions in which all stresses are compressive and normal stresses on all discontinuities are high, it is realistic to treat the rock system as an elastic continuum. However, the possibility of development of the tensile stresses at the base of the block is considered to be extremely important for understanding of the mechanism of deformation and failure in 'in-situ' shear tests. Any displacement within the rock mass may change the relative position of the rock "block" and result in high localized stresses on them, which may cause individual localized failures. These failures may be of tensile, indirect tensile, or shear mode type. The stresses in the rock system may redistribute themselves in a characteristic fashion after localized failures.

Progressive type failure is very common in many soil and rock materials. Understanding of the mechanism of this type of failure is very important; yet the conditions under which it may occur are poorly understood.

## ANALYSIS TECHNIQUES

The finite element method (FEM) has been employed to analyze stresses and strains in plane-strain shear block model. The concept utilized here, as described by Wang and Voight (1969), involves the ordinary finite element partitioning of a solid model into a discrete number of two-dimensional elements with "dual nodal points" used along prescribed planes of discontinuity. A Coulomb-Navier representation with a tension cut-off has been utilized. Progressive failure in

the potential shear zone (i.e., a plane of discontinuity at the base of the shear block) has been considered.

Dahl's (1969) finite element code with suitable modifications has also been used for the elastic-elastoplastic analysis; the material is assumed elastic-perfectly plastic and analysis has been based on elastic-elastoplastic idealization. Formulation of the problem is in terms of incremental theory of plasticity where by constitutive non-linearity in the post-yield region is analyzed in a number of increments (Dahl, 1969; Dahl and Voight, 1969; Voight and Dahl, 1970). Each increment is independent; total stresses and strains from the previous increment are added to the incremental stresses and strains of the present increment in order to compute total stresses and strains of the present increment,

### MODEL FORMULATION

Two basic computer models, labeled M1 and M2 (Figures 1, 2) were developed for the theoretical analysis by finite element techniques and used during the course of this investigation.

M1: a model of an 'in-situ' shear block with three internal discontinuities, labeled D1, D2 and D3; where D1 coincides with the hypothetical shear plane at the base of the block; and D2 and D3 coincide with potential tension fractures predicted from experimental analysis. This is the model most commonly used for the elastic analysis in major part of this study,

M2: a model of an 'in-situ' shear block with no discontinuity. This model was principally employed for the elastic-elastoplastic analysis.

The two-dimensional idealization was assumed to provide an adequate approximation to prototype conditions.

The analysis has been limited to one particular rock, Berea sandstone; the mechanical behavior of this rock was well suited to the requirements of this study; data on its physical properties was unusually complete and was available to the author (Table 1). The material constants for the elastic, homogeneous and isotropic continuum were  $E = 1.1 \times 10^6$  psi., and  $\nu = 0.2$ . The material constants for the transversely isotropic continuum were as given below :

$$\begin{array}{ll} E_a = 1.1 \times 10^6 \text{ psi} & E_k = 0.6 \times 10^8 \text{ psi.} \\ \nu_n = 0.2 & \nu_t = 0.13 \\ G = 0.46 \times 10^6 \text{ psi.} & \end{array}$$

## BOUNDARY CONDITIONS

Eight different ways of loading the shear block model, in terms of various force and displacement boundary conditions, were considered (Figures 3) ; these conditions are described as follows.

- L1. Uniformly distributed load parallel to the base of the block.
- L2: Uniformly distributed load parallel to the base of the block; and uniformly distributed load perpendicular to the base of the block.
- L3: Uniform displacement of the left-hand side of the block parallel to its base.
- L4.: Uniform displacement of the left-hand side of the block parallel to its base ; and uniformly distributed load perpendicular to the base of the block.
- L5: Uniformly distributed load parallel to the base of the block as concentrated at the lower third of the block.
- L6: Uniformly distributed load parallel to the base of the block as concentrated at the lower third of the block; and uniformly distributed load perpendicular to the base of the block.
- L7: Uniformly distributed load inclined to the base of the block at an angle of 20°.
- L8: Uniformly distributed load inclined to the base of the block at an angle of 20° ; and uniformly distributed load perpendicular to the base of the block.

TABLE 1

Nominal Value of the Physical Properties of Rerea Saradstone  
(After Khair, 1971)

Physical Property	Value
Unconfined Compressive Strength	9.000 psi.
Unconfined Tensile Strength	300 psi.
Shear	1.400 psi.
Young's Modulus in Compression	$1.15 \times 10^6$ psi.
Young's Modulus in Tension	$0.58 \times 10^6$ psi.
Poisson's Ratio in Compression	0.2
Poisson's Ratio in Tension	0.1
Independent Shear Modulus	$0.46 \times 10^8$ psi.

## RESULTS AND DISCUSSION

The results for the elastic continuum, discontinuum, and elastic-elastoplastic solutions are grouped and considered in three separate sections.

### 1. Elastic Continuum Solutions

Results of plane strain FEM analyses using model M1 and assuming an elastic, homogeneous and «either isotropic or transversely isotropic continuum are presented herein, for various boundary conditions in terms of:

*Principal stress distribution:* All analytical solutions, in terms of direction and magnitude of major and minor principal stresses, were plotted by computerized (CalComp) plotter at the centroid of each triangular element. A typical isotropic solution data for the boundary condition L-4 is presented in Figure 4. Direction and magnitude of the major and minor principal stresses are inhomogeneously distributed throughout the rock mass, with significant variations across the hypothetical shear plane at the base of the block and significant stress concentrations at the corners. The zone of principal stresses in tension is most extensive for purely edge loaded models (VI» 2,3,5,7) being somewhat more suppressed for the displacement boundary condition (L-3) and for concentrated loading (L-5). There are significant changes in the principal stress directions as function of boundary conditions; in all cases the steepness of the principal compression trajectory is enhanced by application of normal force to the shear block; the most extreme examples are perhaps I»3, and Jb-4 where application of normal force was sufficient to alter a predominantly sub-horizontal compression axis to a predominantly sub-vertical orientation.

*Distribution of strains and stresses at the base of the block:* Both strains and stresses at the base of the block are distributed nonuniformly for all boundary conditions. In general, both normal and tangential strains, as well as the normal and tangential stresses have tensile (reckoned positive) values in the left-hand part of the base of the block for D-1, 3, 5, 7; ie., for conditions without vertical confinement. For the boundary conditions L-2 and L-8 the extensile strains and tensile stresses remain, and even then only for the tangential components. Figures 5 and 6 show the distribution of

strain and stresses at the base of the block for the boundary conditions L-3 and L-4. For the boundary conditions L-4 and L-6, all the strains and stresses are compressive, i.e., no extensile strains or tensile stresses occur at the base of the block. This situation is considered important because this is the only situation in which failure of the block could initiate wholly as a consequence of shearing. However, even for these models, tensile regions exist may be mechanical significance in direct shear testing. For example, Figure 7 shows the distribution of the strains and stresses along the inclined discontinuity D3 (see Figure 1) for the L-4 boundary condition at the incipient yield. Initial yielding occurs in the tension mode at the free boundary of discontinuity D3 as a consequence of high tensile stress concentration at the corner.

*Dependence of tensile zone of normal force:* Figure 8 shows the progressive development of a tensile zone in the block with respect to various ratios of applied normal force to applied tangential (edge) force). The area of the tensile zone increases as the ratio of normal to tangential force ( $R$ ) decreases. There is no tensile zone developed at the base of the block for  $R = \infty$  (i.e.,  $F_y \neq 0$ ); conversely the tensile zone is the largest for  $R = 0$  (i.e.,  $F_x = 0$ ).

*Maximum shear stress contours:* Contours of the maximum shear stresses developed in the shear block under various boundary conditions were plotted in the neighbourhood of the block base. For most boundary conditions, concentration of the maximum shear stresses occurs near the corners of the base of the block, but not necessarily the plane of the rock block base. This fact, also reported by Ruiz et al, 1968; possibly explains the occurrence of shear failure surfaces out of the plane of the block base on a number of tests reported by various investigators (e.g., Evdokimov and Sapegin, 1970). Concentration of the maximum shear stress contours around the right-hand corner of the block base also points out the effect of rotational deformation of the shear block on failure mechanism. The boundary conditions L-3 and L-4 are the only conditions under which this effect is minimized due to minimized rotation of the block. Figure 9 shows the maximum shear stress contours in the neighbourhood of the block base for the boundary condition L-4.

*Displacement field and distortion of the block:* The total displacements of the nodal points were also plotted on a CalComp plotter. Figure 10 and 11 show the elastic displacement fields for the boun-

dary conditions L-3 and L-4. Distortion characteristics of the block corresponding to the boundary displacement field for L-3 and L-4 are shown in Figure 12. These results show that type of deformation in the block basically depends upon the nature of the applied boundary conditions.

## 2. Discontinuum Solutions

The results presented in the previous section represent loading conditions associated with incipient initial failure. Any further increase in that critical tangential load causes (at least local) failure along any one of three discontinuities D1, D2, D3 (see Figure 1), in either a slip or separation mode-

*Mode of deformation:* The theoretical results are of interest, clearly showing the dependence of failure mode on the boundary conditions. All boundary conditions, except for L-4, resulted in initial yielding of the block at the first node of the discontinuity D1, in the form of dual node separation; failure propagated along that discontinuity in the separation mode until complete failure occurred in the form of separation of all dual nodes on D1. For boundary condition L-4, however, yielding of the block initiated in the separation mode at the top the discontinuity D3; failure propagated diagonally along D3 until the last dual nodes on D3 are separated (Figure 13). This is considered to be the termination of the "first stage" of failure. Further loading after the complete opening of D3, produced the initiation of the "second stage" of failure, this time at the base of the block at the first dual nodes of D1 in the "slip" mode (i.e., shear). This second stage of failure progressed towards the center of the block in the "slip" mode between subsequent dual nodes. At the time the second stage of failure reached to about one-sixth of the total length of D1 from the left-hand corner, a "third stage" of failure of the block was initiated on D1 at the opposite corner, and retrogressed towards the center of the block in the "slip" mode. Complete rupture at the base of the block occurred when the progressive and retrogressive failure surfaces met on D1 in the middle of the block base (Figure 14). The progressive and retrogressive failure series with associated displacement fields and principal stress distributions are summarized in Figures 15, 16 and 17, respectively\*.

*Mechanism of failure:* In most cases concentration of tensile stresses occurred around the left-hand corner of the shear block. Initial failure thus occurred at this corner in form of a tensile crack which opened and propagated diagonally along the discontinuity D3- Separation along D3 releases tensile stresses originally developed and thus causes a redistribution of strains and stresses along the base of the block (Figure 18) ; this redistribution is responsible for subsequent failure along D1, predominantly in the "slip" (shear) mode. A temporary cessation of crack propagation, and the inception of a "third stage" of failure associated with retrogressive slip at the opposite corner, can also be explained in terms of subsequent stress redistribution, which finally results in critically high shear stress concentration at the lower lefthand corner of the shear Mock.

*Peak and residual strength and progressive failure:* Two series of experiments were conducted; in the first series, constant strength parameters (i.e., experimentally determined peak values for CF, SS and TS) (Table 2) were assigned to D1 and were maintained as the edge displacements were applied in successive increments. The total edge displacement required in order to cause "slip" (shear failure) at each successive nodal point along D1 were determined (Figure 18). In the second series, peak strength parameters were initially prescribed, but not necessarily maintained. A new set of strength parameter (i.e., experimentally determined residual value for CF; Zero for SS, and for TS) were subsequently assigned to the each point on D1 if a minimum horizontal displacement between dual nodes reached to a specified critical value (i.e.,  $u_y = 1.0 \times 10^{-5}$  in.) in the previous loading increment; the original peak strength values are retained for points which had either not yet failed in the "slip" mode, or which had not undergone sufficient "slip". The applied edge displacements required to cause shear failure were determined for each successive point. Results obtained for this series are also summarized in Figure 19. The shear strength failure envelopes predicted from these two series of computer experiments are plotted together with the "intrinsic" peak and residual strength failure envelopes, drawn on the basis of the fundamental values of SS and CF (Figure 20). The results are of interest, clearly showing the significant differences between the fundamental values of SS and CF (Figure 20). The results are of interest, clearly showing the significant differences between the fundamental values of SS and



CF, and the predicted values SS', and CF; and the effect of residual values of TS<sup>K</sup>, SS<sup>R</sup> and CF<sup>R</sup> on the shear strength characteristics of the rock.

**TABLE 2**  
**Fundamental Values of CF, SS. and TS Assumed Along the Discontinuities D1, D2, D3.**

Discontinuity	CF	SS	TS
D1	0.65*	1200* psi	300 psi
D2	0.975	1800 psi	450 psi
D8	0.975	1800 psi	450 psi
	1.30**	2400** psi	583 psi** psi

- \* Values determined by experiments in the direction parallel to the bedding.
- \*• Values determined by experiments in the direction normal to the bedding.

### 3. Elastic-Elastoplastic Solutions

Elastic-elastoplastic analyses of the direct shear problem were limited to the boundary conditions L-2, U-3 and L-4; model M2 (see Figure 2) was employed. Distribution of the major and minor principal stresses in the plastic state of the block, corresponding plastic displacement fields and the progressive yield zones for the above boundary conditions were considered; as were assumption of both linear (Colomb) and non-linear (Torre) yield criteria. The elastic and strength properties used for the elastic-elastoplastic solutions are given in Table 3.

*Distribution of principal stresses:* Nature of principal stresses (L e., either being tensile or compressive), in the plastic state of the block, were found to depend primarily upon the boundary conditions. Neither transverse isotropy nor non-linearity of the yield functions employed showed any significant effect on the direction and the nature of the principal stresses when compared with the isotropic and linear analysis. Distribution of the principal stresses in the plastic state of the block, under the boundary condition LH£, is shown in Figure 21. Location of the principal stresses in tension, and their direction around the lower left-hand corner of the block support the previous discussion on the mode of initial failure of the shear block.

*Displacement fields:* For the similar boundary conditions, neither\* non-linearity of the failure criteria used, nor the transverse

isotropy assumed showed any significant effect on the nature of the displacements. The differences between the boundary conditions is primarily one of variations in slope of the displacement vectors (Figures 22 and 23).

*Development of plastic zones:* The location of the zone of plastic elements appears to be affected in large measure by the development of tensile stresses; it is not extensively developed in regions of large compressive stress. Both the point of initiation and the direction of propagation of the plastic elements appears to be in good agreement with the point of initiation and the direction of propagation of (extensional) yielding. When applied edge displacement is further increased, the progression of the plastic zone continues along the base of the shear block still in the form of (extensional) yielding (Figures 24 and 25). Plastic yielding occurs along the upper boundary of the shear block seems to be the result of tensile stresses which develop in that region due to the nature of the applied boundary condition, L-4. These tensile stresses appear to be perpendicular to the left-hand boundary of the shear block where edge displacements are applied to the block. This is, however, physically and unrealistic situation; this type of failure probably will not occur under actual test conditions.

TABLE 3

The Elastic and Strength Properties Used For Elastic-Plastic Solutions.

Elastic properties:		Isotropic Case	Transversely Iso-tropic Case
Ei		1.1X10 <sup>6</sup> psi	1.1X10 <sup>8</sup> .psi
Ea	1.1	X10 <sup>6</sup> psi	0.58X10 <sup>6</sup> psi
$\nu_1$		0.2	0.2
$\nu_2$		0.2	0.1
G		0.46X10 <sup>6</sup> psi	0.46X10 <sup>6</sup> psi
Strength Properties:			
CSÜ		9000 psi	10000 psi
CSt		9000 psi	7000 psi
TSn		300 psi	500 psi
TSt		300 psi	200 psi
SS		1400 psi	1400 psi

## SUMMARY

This paper presents a study of progressive shear deformation which involved a computer experimentation employing elastic continuum, discontinuum and elastic-elastoplastic finite element method. Two basic computer shear block models were developed for this purpose. Two-dimensional plane-strain idealization was assumed. Eight different methods of loading the shear block model were considered. A discussion of failure mechanism, in terms of progressive and multiple modes of failure, was introduced.

## CONCLUSIONS -

Results of the preceding analyses led to the following general conclusions:

Choice of boundary conditions exerts an important control on failure mechanism.

Tensile zones always developed within test block in response to applied shear force are of mechanical importance: Local failure which occurs\* in separation mode, in these tensile zones leads to progressive failure. Under certain boundary conditions/ the ultimate failure of the test block is a consequence of multiple fracture modes.

Mechanical behavior and strength of an 'in-situ' shear block can not be adequately explained solely in terms of some fundamental shear strength parameters only. Importance of rock mass tensile strength which is generally very low, and pre-existing discontinuities, which offer little tensile resistance, should not be overlooked in 'in-situ' shear experiments, inasmuch as they exert important control on force-displacement relationship measured by such tests.

'In-situ' shear tests on geological materials should be interpreted in more sophisticated terms, i.e., as a consequence of variable stress states, involving inhomogeneous stress field, one or several of principal stresses being tensile; extensive stress reorientation; and multiple crack propagation.

Uniform edge-displacement boundary condition produces the most consistent theoretical results; hence may be suggested as a standard method of application of shear force to the test block both in 'in-situ' and laboratory shear experiments.

Discontinuum solutions, utilizing discontinuous model; allow prediction of localized failures and analysis of progressive nature

of failure mechanism. Elastic-elastoplastic solutions, on the other hand, appear to be more suitable for analysis of progressive development of yield zones in areas around the points of high stress concentrations.

## ÖZET

'Yerinde' makaslama denemelerinde deformasyon ve yenilme mekanizmasının saptanması amacı ile yapılan bu çalışmada, sorun önce teorik yönden ele alınmış ve tipik bir 'yerinde' makaslama bloğunun detaylı Mr kompüter (matematiksel) modeli geliştirilmiştir. Bu model üzerinde, düzlem-deformasyon şartları varsayılarak, sonlu elementler metodu ile, değişik kenar yükleme şartları altında gerilim ve deformasyon analizleri yapılmıştır.

Teorik analiz sonuçları, deformasyon modunun kenar yükleme şartlarına bağlı olduğunu açık bir şekilde ortaya koymuş olması bakımından ilginçtir. L-4 kenar yükleme hali dışında, diğer bütün yükleme şartları altında, makaslama bloğunun ilk yenilmesi Dİ süreksizliği üzerinde, sol uçtaki ilk noktada, bloğun tabandan ayrılması (tansiyon yenilmesi) şeklinde oluşmaktadır. Uygulanan makaslama yükünün sürekli olarak artırılması halinde, bloğun ilk yenilmesi ile oluşan tansiyon çatlağı blok tabanı (Dİ süreksizliği) boyunca sağa doğru ilerlemekte; bloğun son yenilmesi ise, Dİ süreksizliği boyunca makaslama bloğunun tabandan tamamen ayrılması şeklinde oluşmaktadır. D-4 kenar yükleme şartı altında, makaslama bloğunun ilk yenilmesi bu defa D3 süreksizliğinin üst uç kısmında, yine 'ayrılma' (tansiyon yenilmesi) şeklinde oluşmakta ve meydana gelen tansiyon çatlağı D3 boyunca ilerlemektedir. Makaslama bloğunun D3 süreksizliği boyunca tamamen ayrılmış hali ve o andaki dış deformasyonu Şekil 13 de gösterilmiştir. Yenilmenin 'ilk evre' si olarak tanımlanan bu durumdan sonra, uygulanan makaslama yükünün sürekli olarak artırılması halinde, yenilmenin 'ikinci evre'si blok tabanı boyunca, Dİ süreksizliğinin sol ucunda 'kayma' (makaslama yenilmesi) şeklinde oluşmakta ve sağ uca doğru ilerlemektedir. Bu şekilde oluşan makaslama çatlağı, Dİ uzunluğunun henüz altında biri kadar ilerlemiş iken, yenilmenin 'üçüncü evre' si makaslama bloğunun sağ alt köşesinde, Dİ üzerinde yine bir 'kayma' (makaslama yenilmesi) şeklinde oluşmaktadır. Sağ köşeden geriye (sola) doğru ilerleyen bu makaslama çatlağı, blok tabanının orta kısmında, sol köşeden sağa doğru ilerlemekte olan 'ikinci evre' çatlağı ile birleşerek deneme bloğunun 'tüm yenilme' sini oluşturmaktadır.

Gerilim analizlerinden elde edilen sonuçlara göre, çoğunlukla, tansiyon gerilimi yoğunlaşması makaslama blokunun sol alt köşesinde oluşmaktadır. Yenilmenin 'ilk evre' si sırasında D3 boyunca oluşan tansiyon çatlağının açılması, makaslama blokunun sol alt köşesinde yoğunlaşmış olan tansiyon gerilimlerinin boşalmasına sebep olmaktadır. Bunun sonucu olarak da, blok içerisinde yeniden bir gerilim dağılımı oluşmakta ve bu da, Dİ süreksizliği boyunca, yenilmenin ikinci ve üçüncü evreleri sırasında oluşan 'kayma' şeklindeki yenilmelere sebep olmaktadır. İkinci evre sırasında makaslama blokunun sol alt köşesinde oluşan makaslama çatlağının Dİ boyunca ilerlemesinin geçici olarak durması veya yavaşlaması; ve karşıt köşede, yenilmenin üçüncü evresinin 'kayma' (makaslama yenilmesi) şeklinde oluşması da yine, birinci ve ikinci evreler sonunda, her defasında yeniden oluşan gerilim dağılımının makaslama blokunun sağ alt köşesinde sebep olduğu büyük makaslama gerilimi yoğunlaşmasının bir sonucu olarak açıklanabilir,

Bu analizler sırasında, deneme bloku içerisinde ve süreksizlik düzlemleri boyunca, tansiyon çatlakları şeklinde oluşan lokal yenilmelerin deneme blokunun son yenilmesi üzerinde, mekanik yönden çok önemli bir rol oynadıkları saptanmıştır. Bu nedenle, gerek 'yerinde' gerek laboratuarda yapılacak makaslama denemelerinde, kayaçların çok düşük olduğu bilinen çekme (tansiyon) dayanımlarının ve kayaç yapısında bulunabilecek süreksizlik düzlemlerinin özellikle dikkate alınması gerekir.

#### REFERENCES

- Dahi, D. and Voight, B., 1969 "Isotropic and anisotropic plastic yield associated with cylindrical underground excavations" — Proc. Int. Symp. on Large Permanent Underground Openings, Oslo.
- Evdokimov, P. D. and Sapegin, D. D., 1970 "A large scale field shear test on rock" — Proc. 2nd. Conf. Int. Soc. Rock Mech., Belgrade, v. 2 pap 3-17
- Khair, A. W. 1971 "A study of mechanical properties of Berea sandstone for use in the A.G.A. large model studies!" — The Pennsylvania State University Internal Report RML-IR/71-20.
- Ruiz, M. IX, Camargo, F. P. and Nieble, Ç.M., 1068 "Some considerations regarding the shear strength of rock masses!" — Int. Symp., on Rock JMDch, Madrid, 159-161,
- Voight, B., 1969 "Numerical continuum approaches to analysis of non-linear rock deformation" — Conf. on Research in Tectonics, Hamilton, Ontario.
- Wang\* Y. J. and Voight, B., 1969 "A discrete element stress analysis model for discontinuous materials" — Proc. Int. Symp. on Large Permanent Underground Openings, Oslo.

## NOMENCLATÜBE

The following defines the major symbols used in this text.

E	Young's modulus
$E_{in}$	Young's modulus an the direction normal to the discontinuity D1
$E_t$	Young's modulus in the direction parallel to the discontinuity D1
$\nu$	Poisson's ratio
$\nu_n$	Poisson's ratio in the direction normal to the discontinuity D1
$\nu_t$	Poisson's ratio in the direction parallel to the discontinuity D1
G	Independent elastic shear modulus
CF	Coefficient of friction
CS	Compressive strength
$CS_t$	Compressive strength in the direction normal to the discontinuity D1
$CS_t$	Compressive strength in the direction parallel to the discontinuity D1
TS	Tensile strength
$TS_h$	Tensile strength in the direction normal to the discontinuity D1
$TS_t$	Tensile strength in the direction parallel to the discontinuity D1
SS	Shear strength
$F_x$	Total boundary force applied in x-direction
$F_y$	Total boundary force applied in y-direction
$F_{x,y}$	Total boundary force applied in x-y plane
$U_x$	Total edge displacement in x-direection
$U_y$	Total edge displacement in y-direction
$\sigma$	Stress
$\sigma_x$	Stress in x-direction
$\sigma_y$	Stress in y-direction
$\tau_{x,y}$	Shear stress
$\tau_{max}$	Maximum shear stress
$\epsilon$	Strain
$\epsilon_x$	Strain in x-direction
$\epsilon_y$	Strain in y-direction
$\epsilon_{xy}$	Shear strain

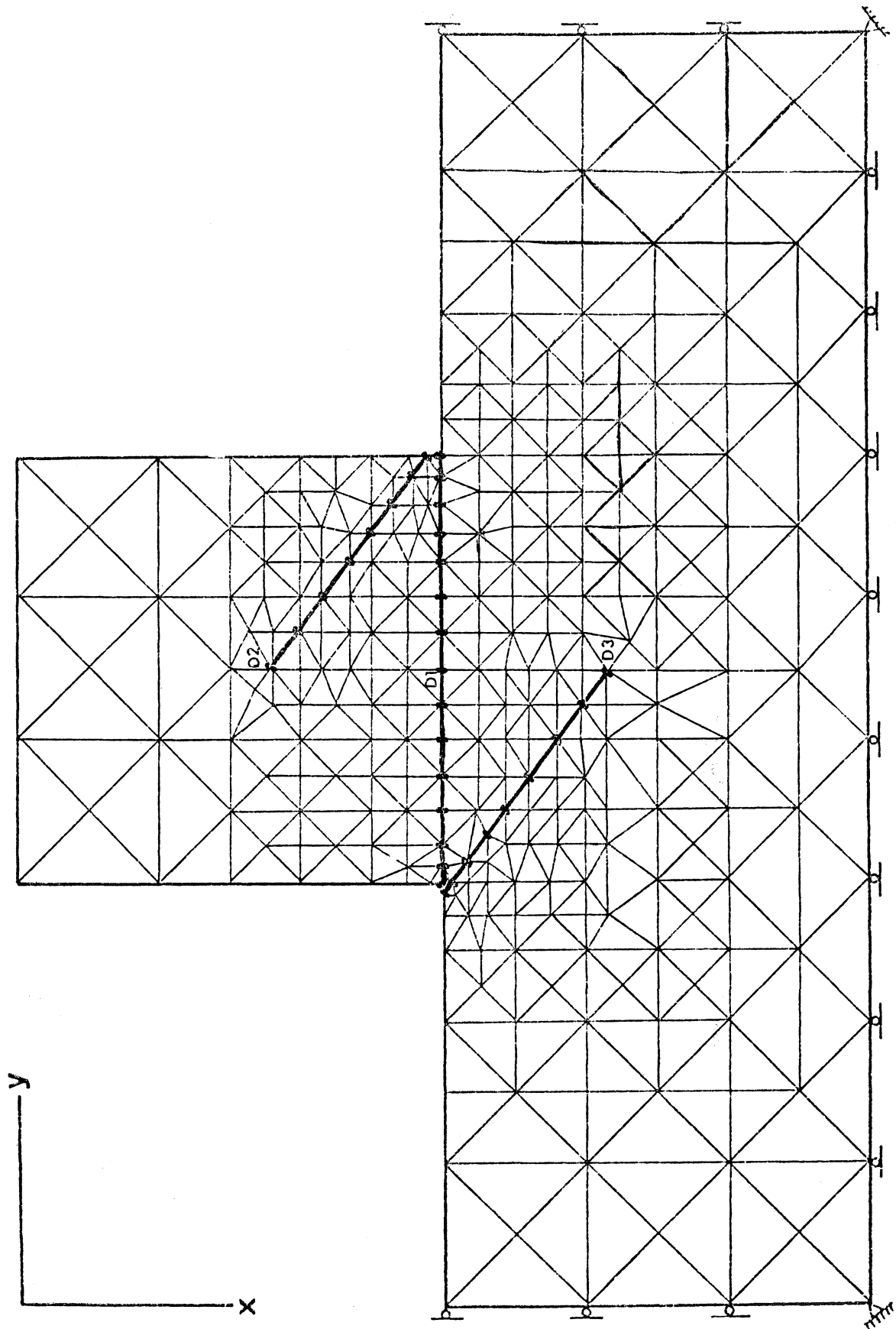


Figure 1. Finite element idealization of 'in-situ' shear block model M, 1

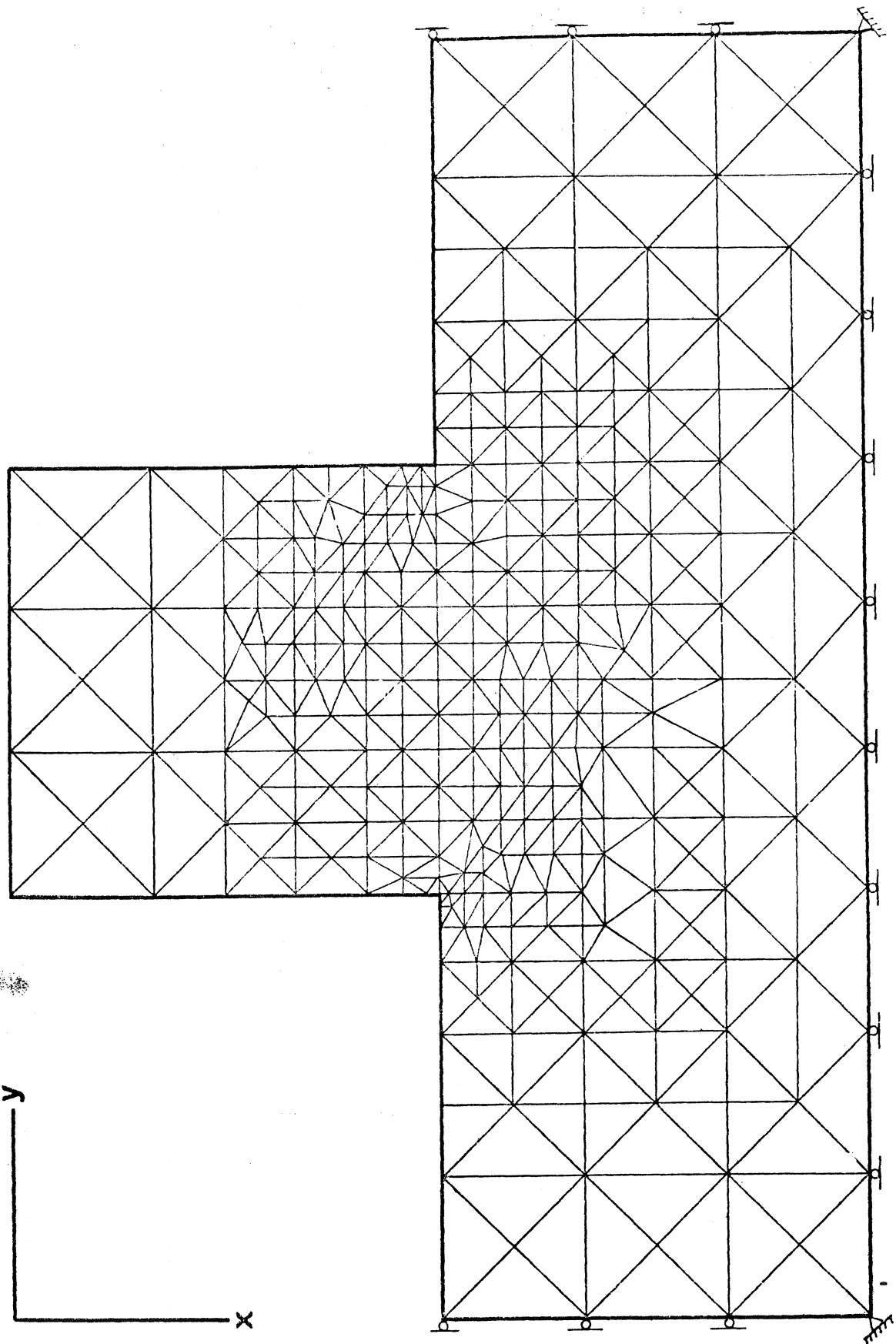


Figure 2. Finite element idealization of 'in-situ' shear block model M2.



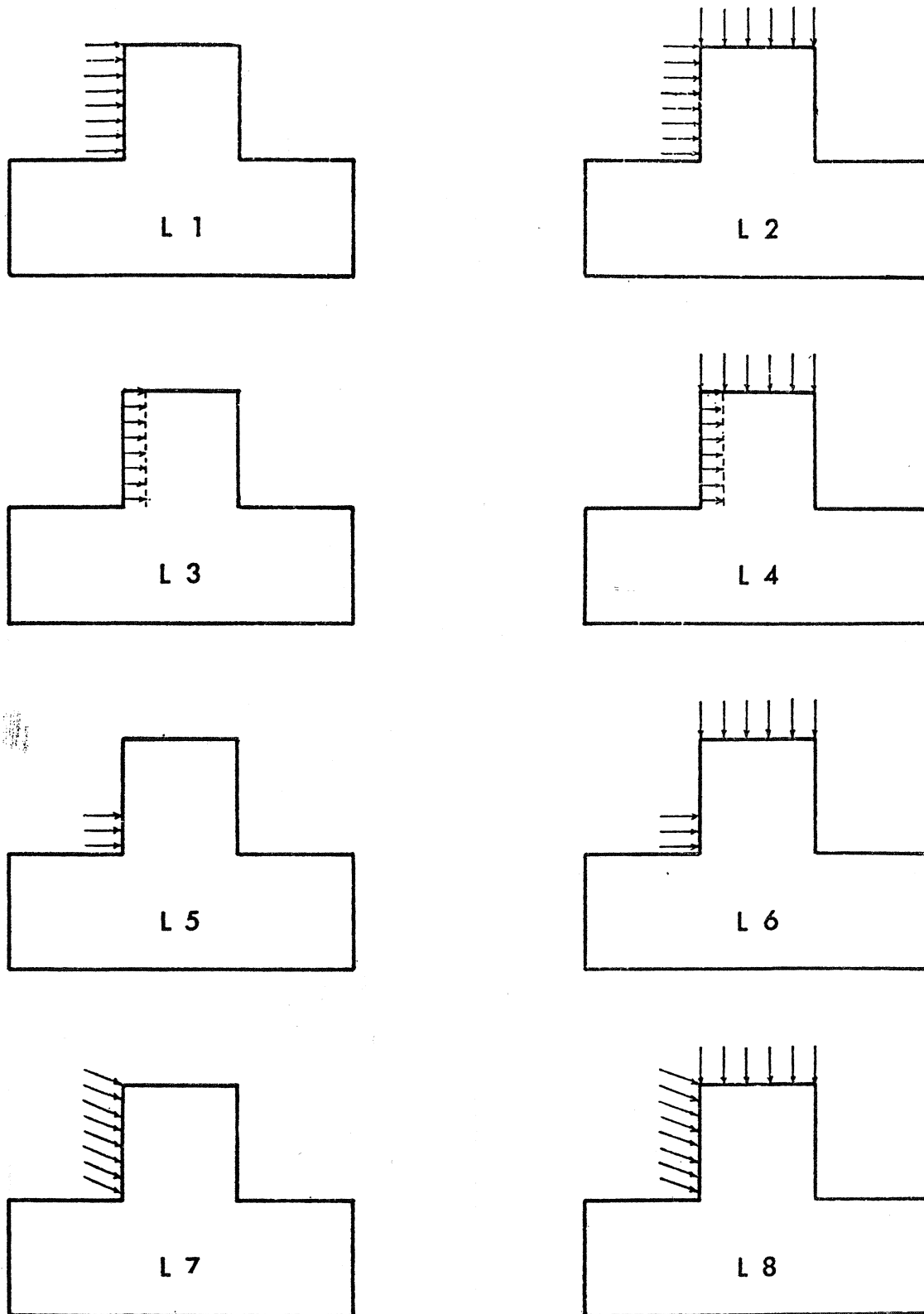


Figure 3. Boundary conditions.

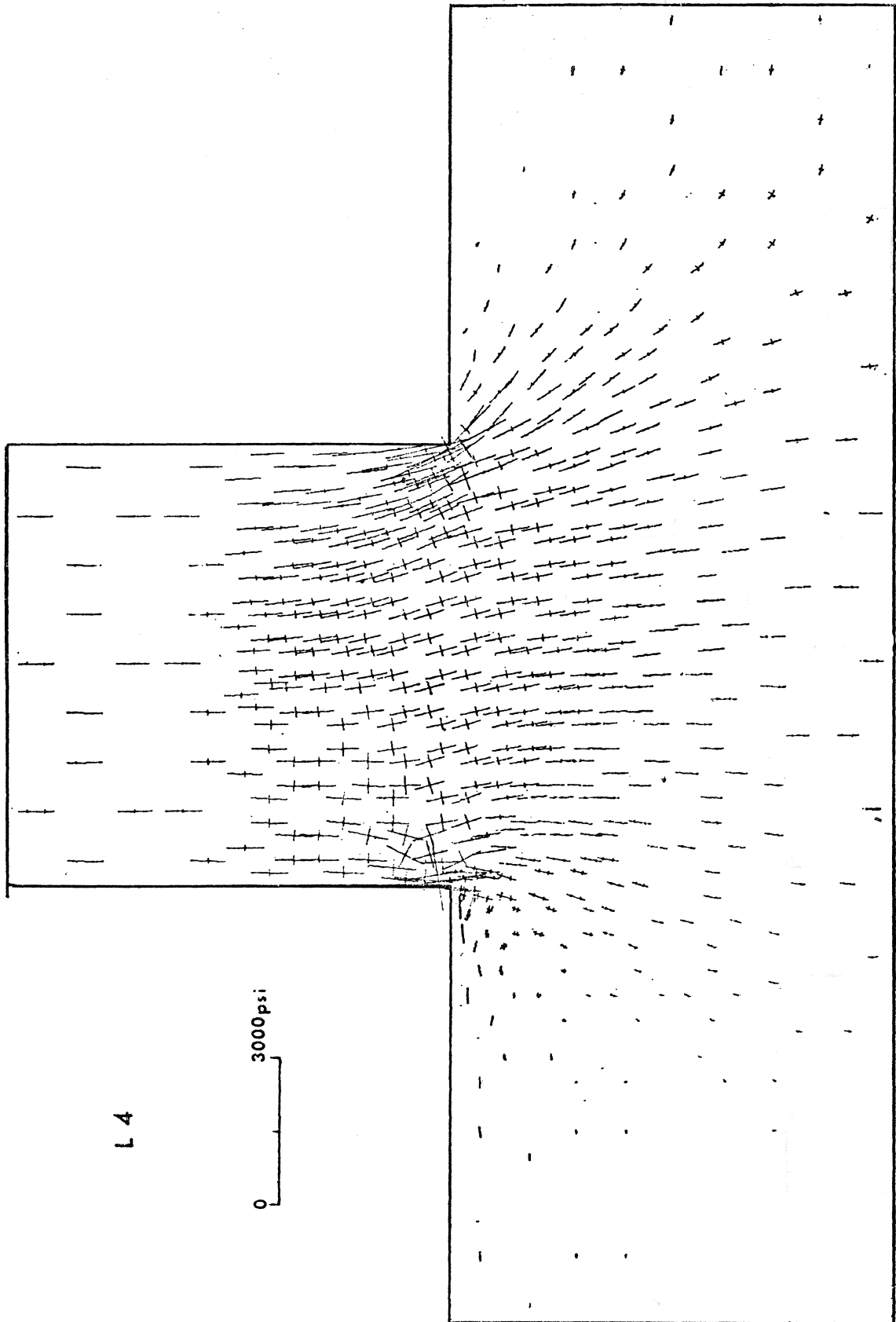


Figure 4. Principal stress distribution in the shear block model M1 at the point of incipient yield.

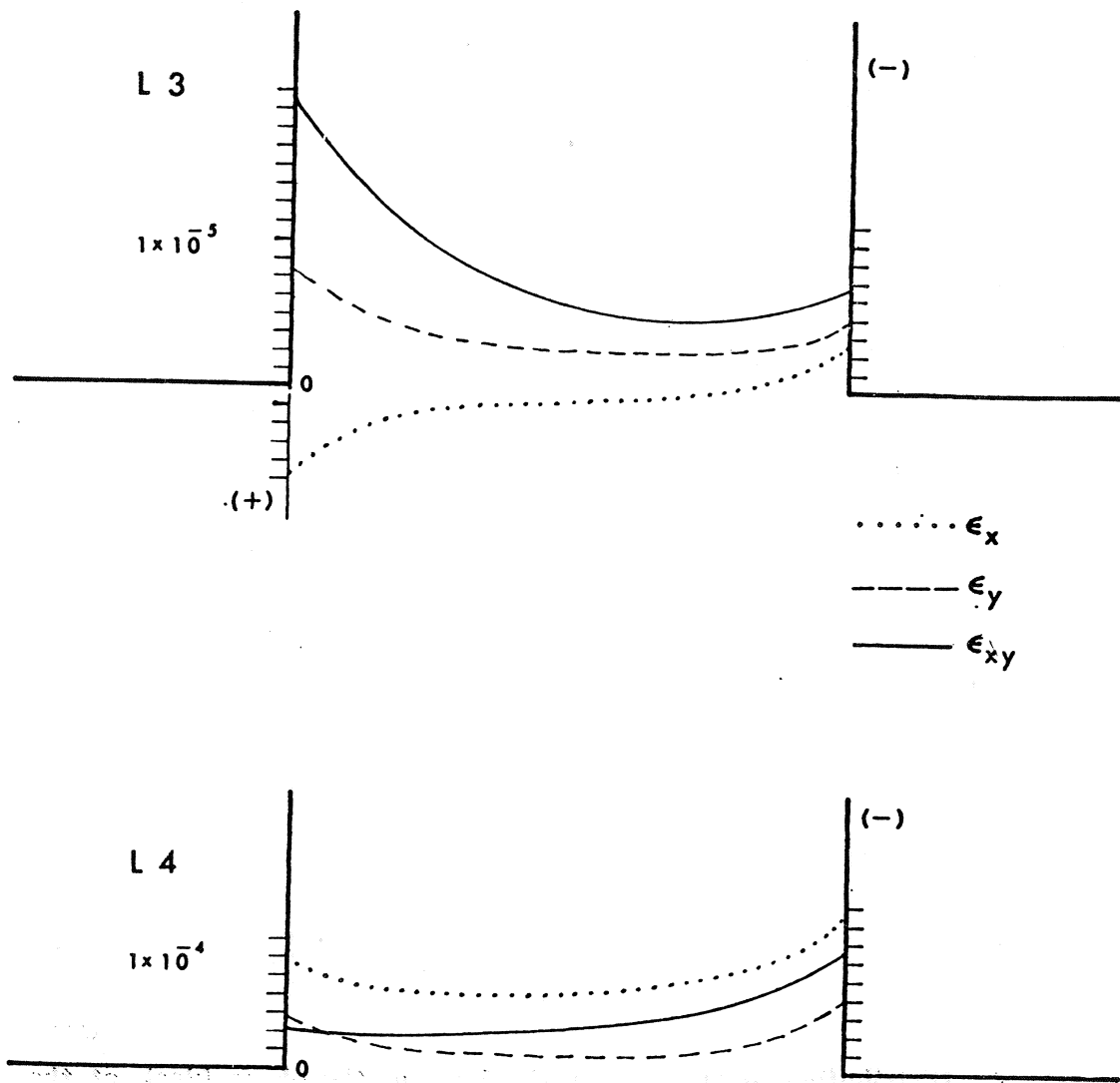


Figure 5. Distribution of strains at the base of the shear block at the incipient yield.

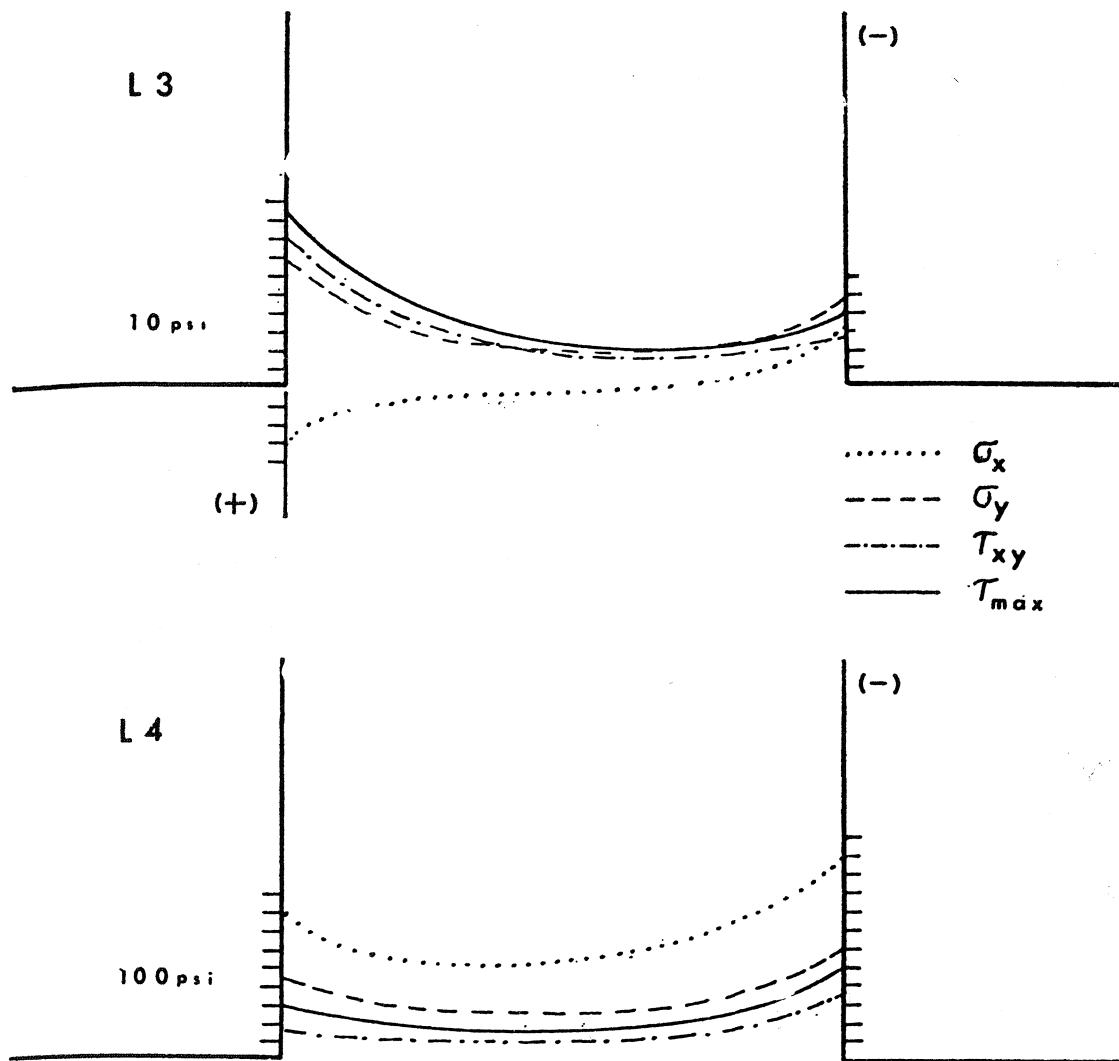


Figure 6. Distribution of stresses at the base of the shear block at the incipient yield.

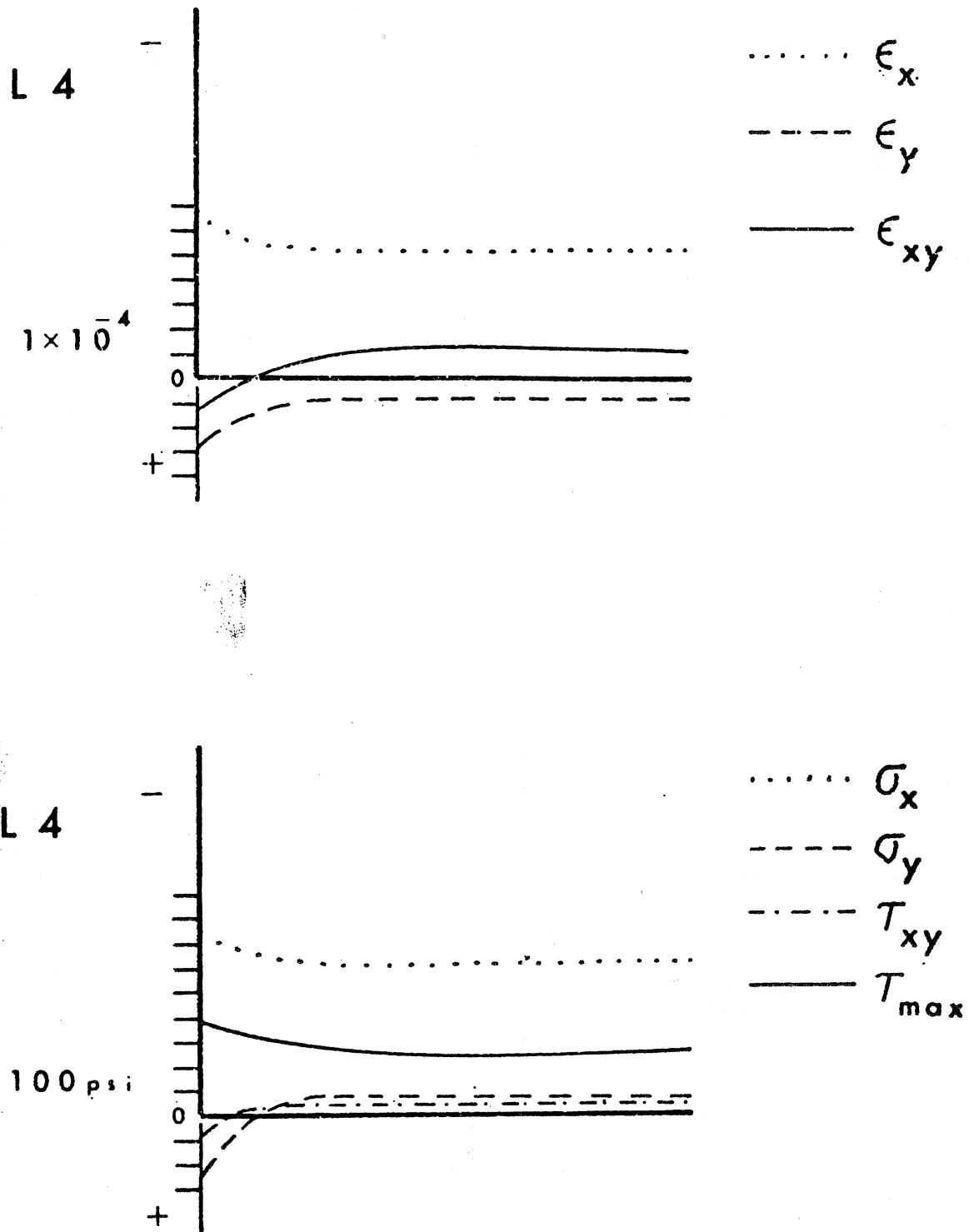


Figure 7. Distribution of strains and stresses along discontinuity D3 at the incipient yield.

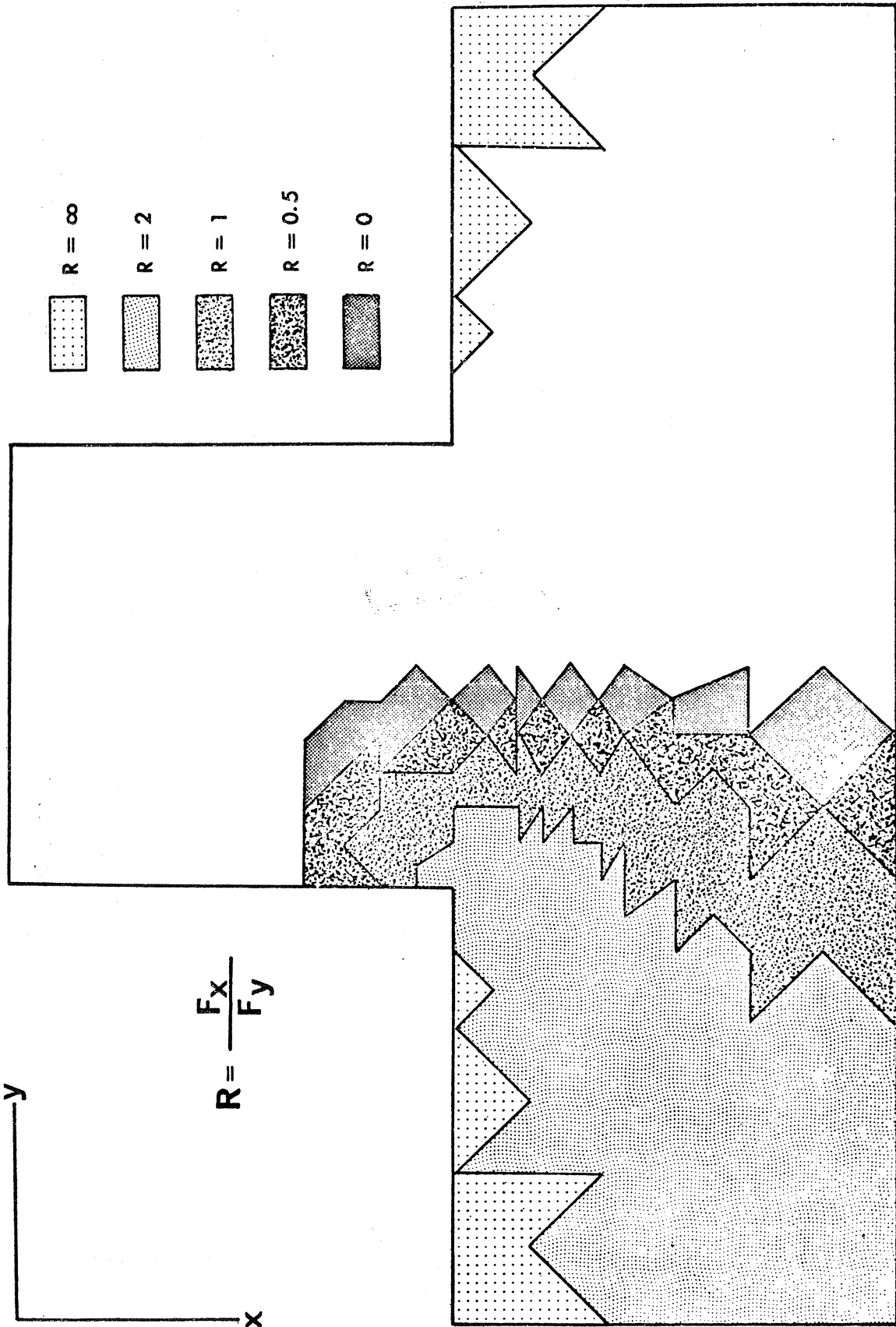


Figure 8. Progressive development of tensile zone in the shear block.

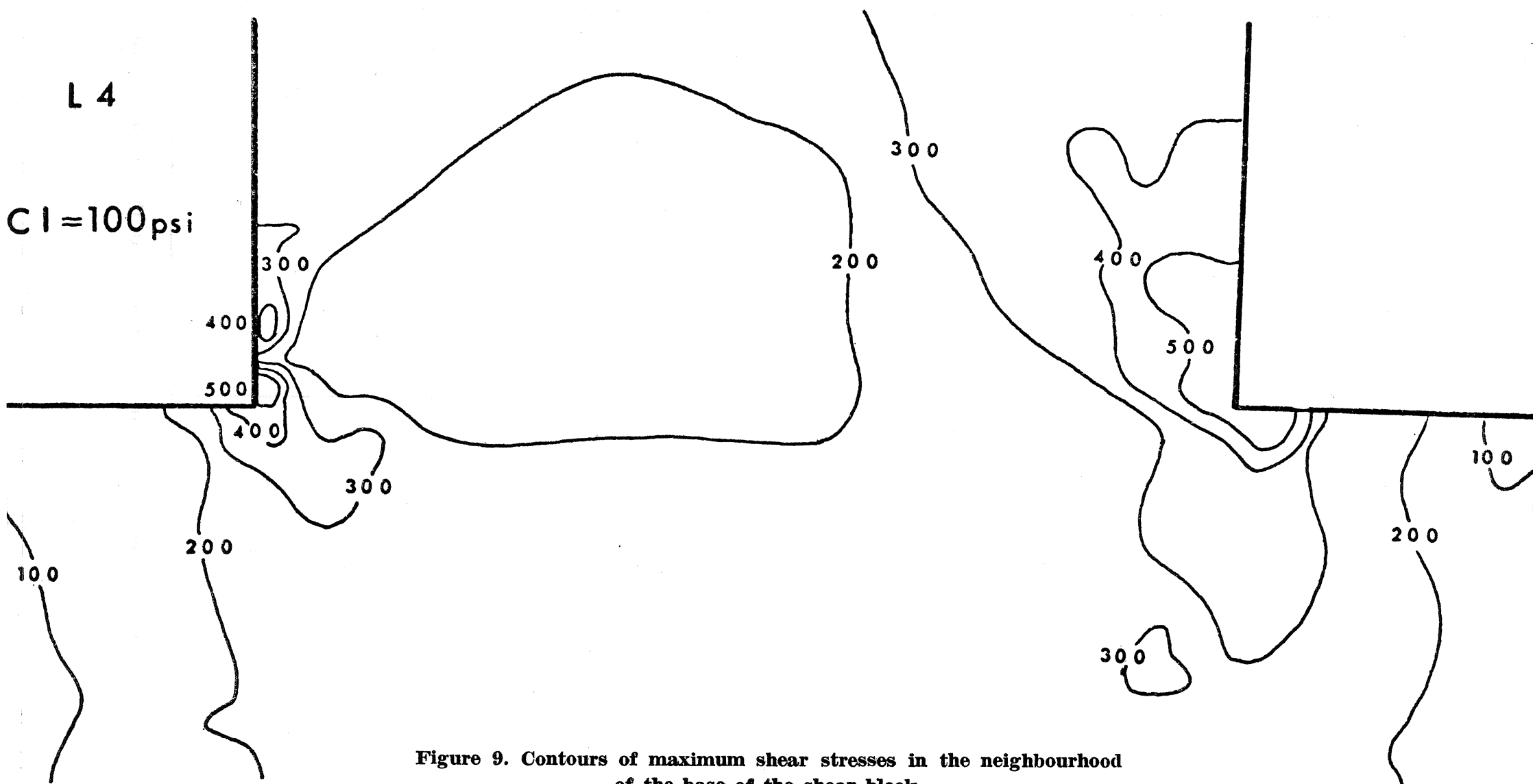


Figure 9. Contours of maximum shear stresses in the neighbourhood of the base of the shear block.

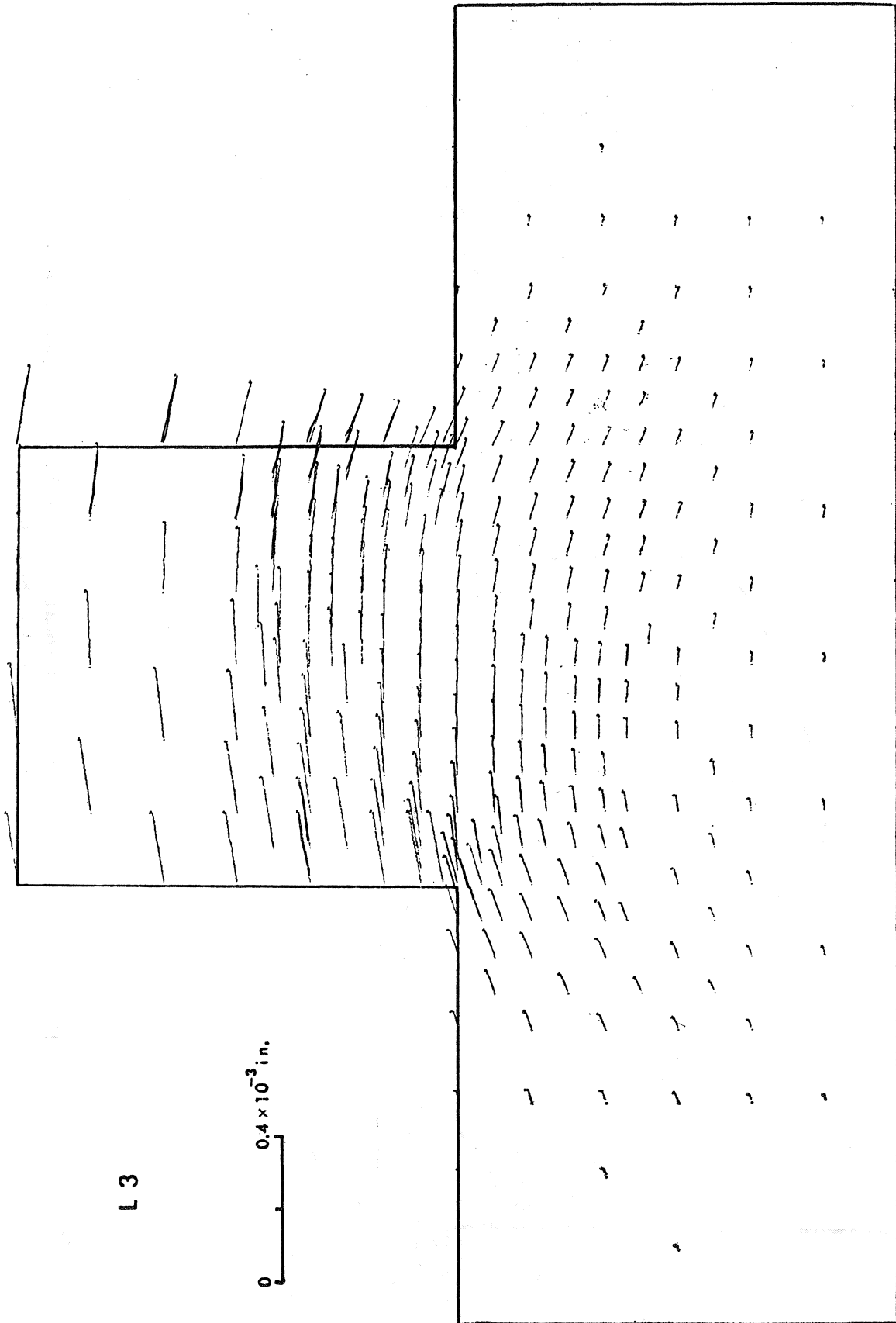


Figure 10. Displacement field in the shear block at the incipient yield.



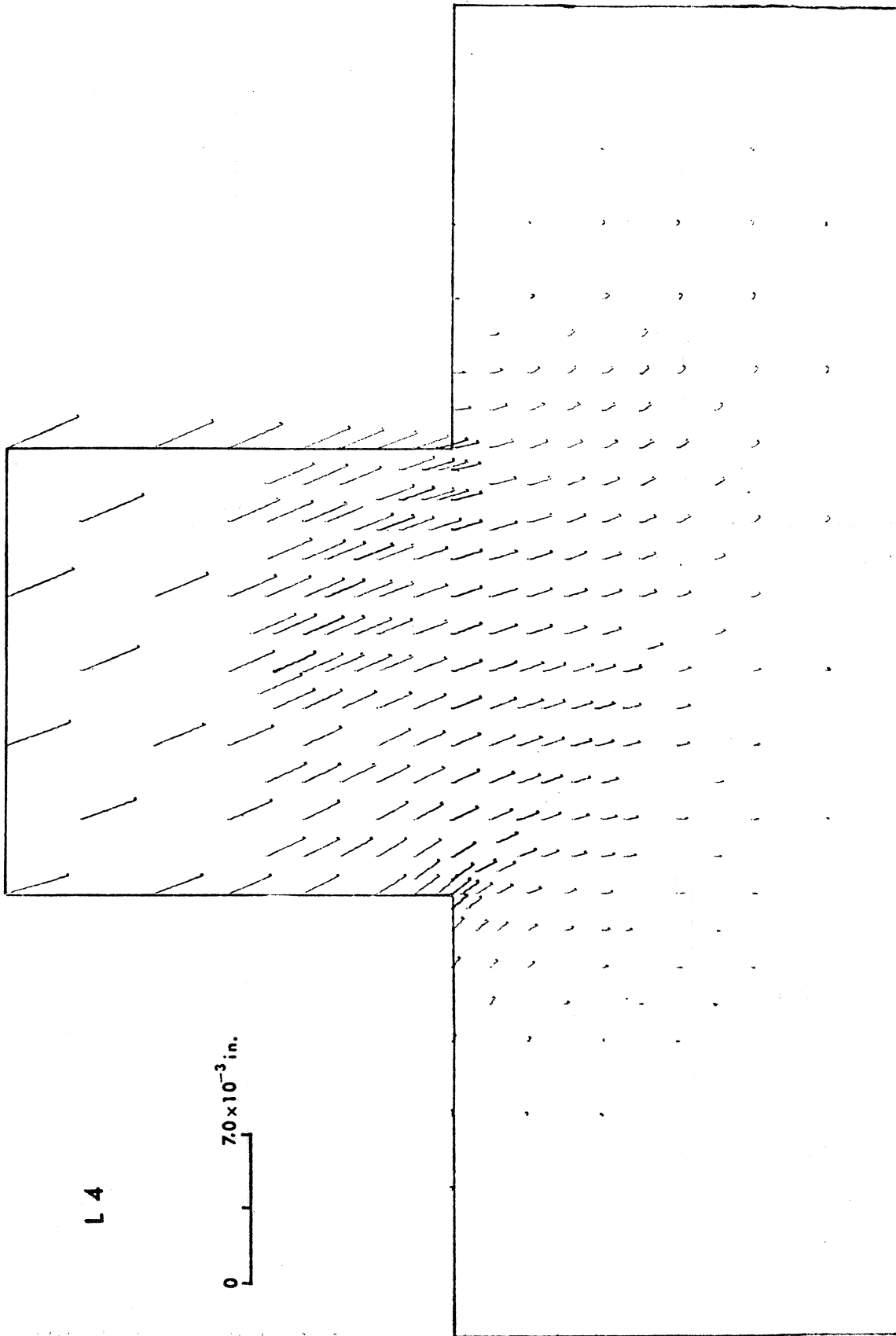


Figure 11. Displacement field in the shear block at the incipient yield.

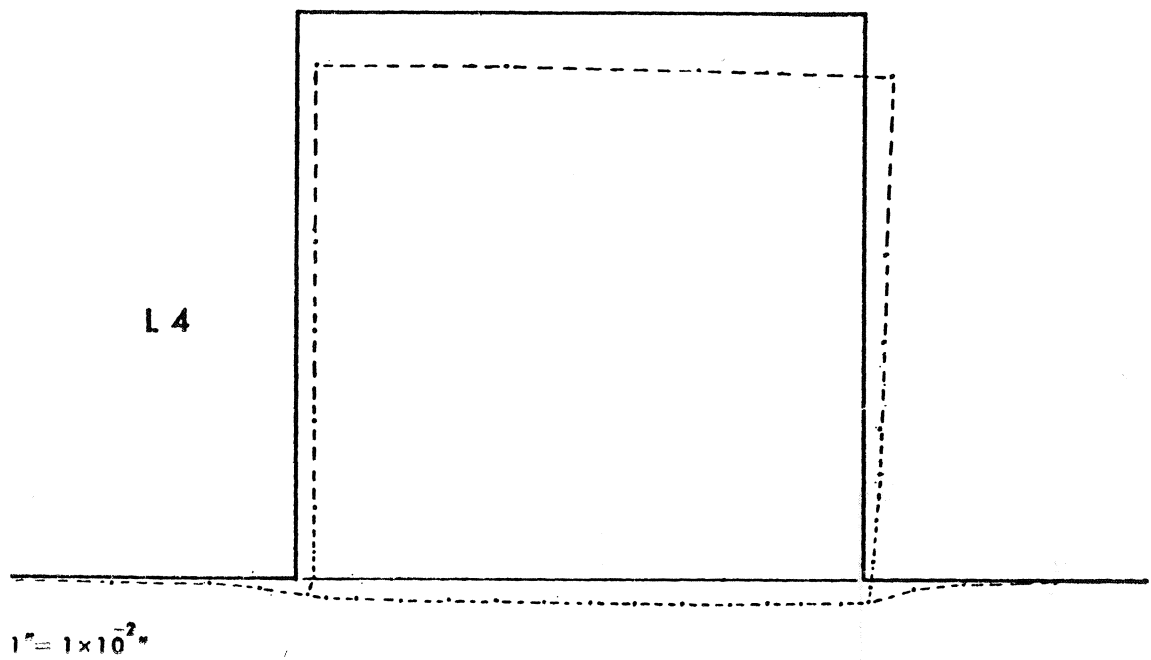
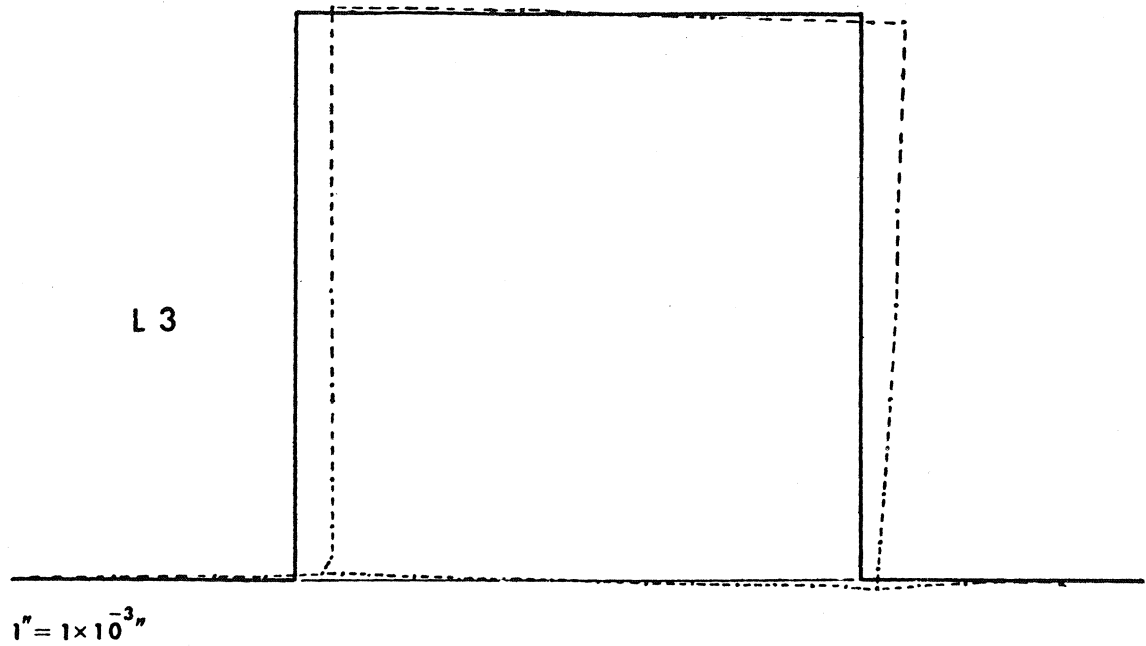


Figure 12. External distortion of the shear block at the incipient yield.

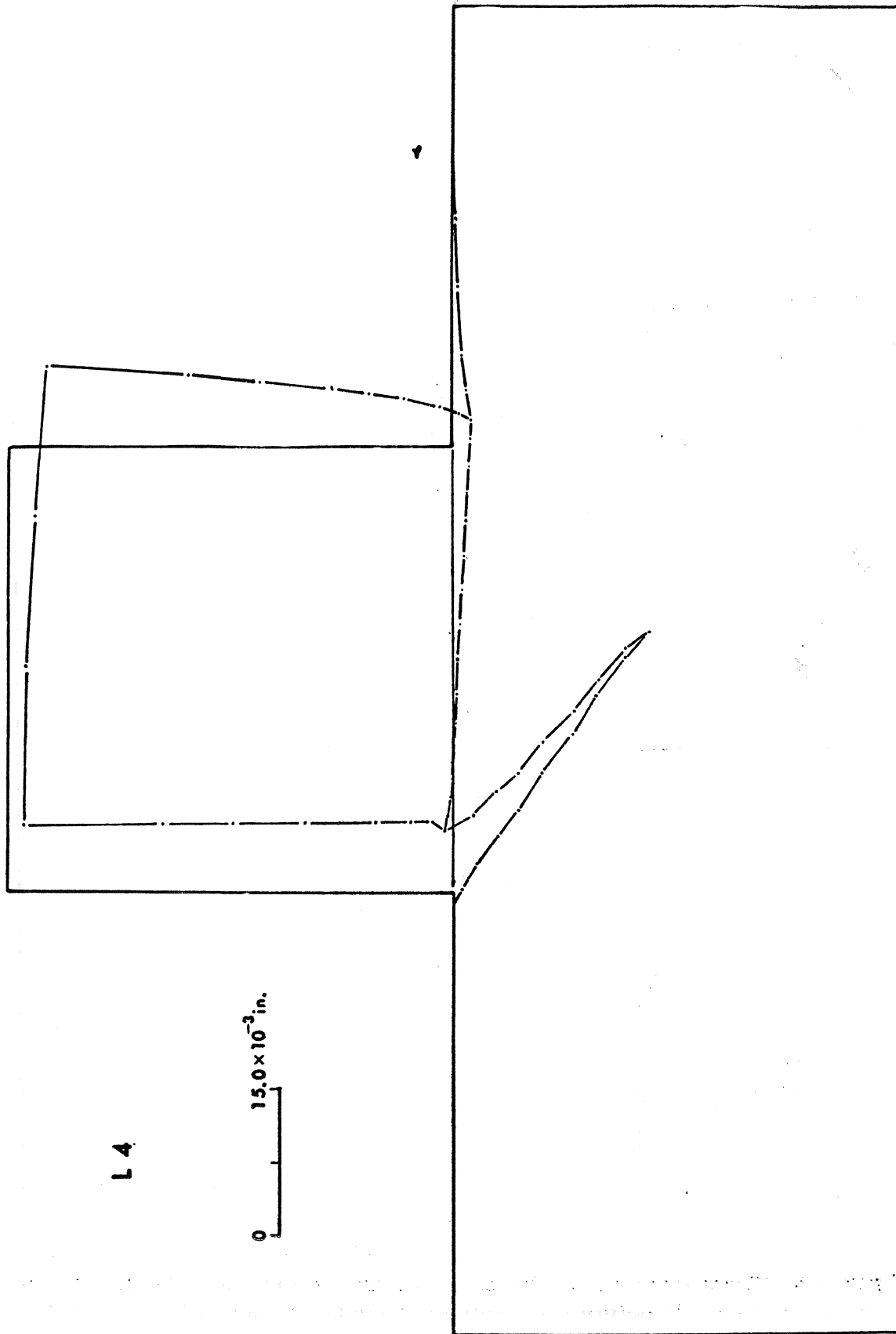


Figure 13. Complete opening of D3 and external distortion of the shear block.

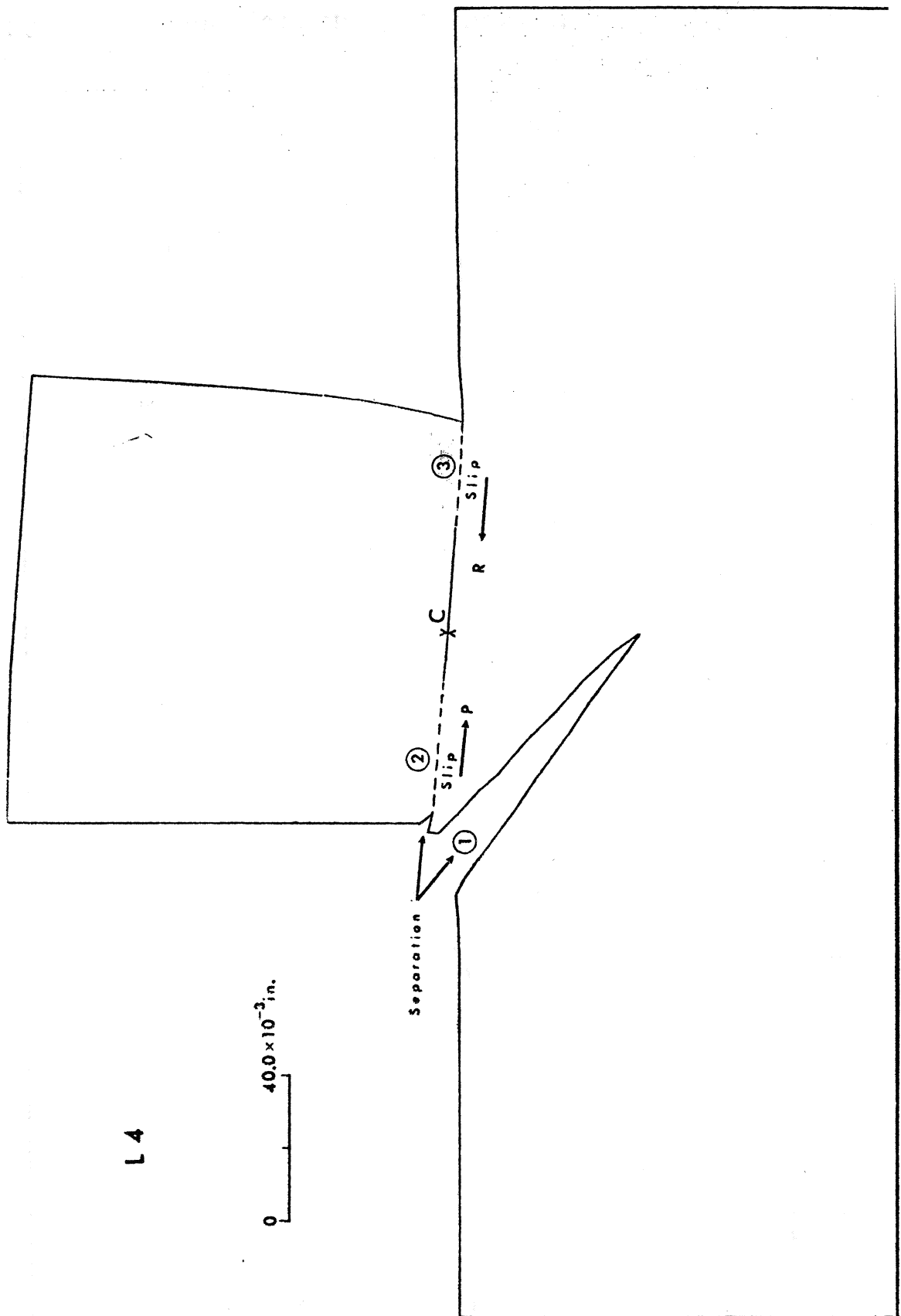


Figure 14. Three consecutive stages of failure of the shear block (Arrows refer to direction of fracture propagation; P implies "progressive failure"; R implies "retrogressive failure"; C is the point at which progressive and retrogressive failure surfaces meet.

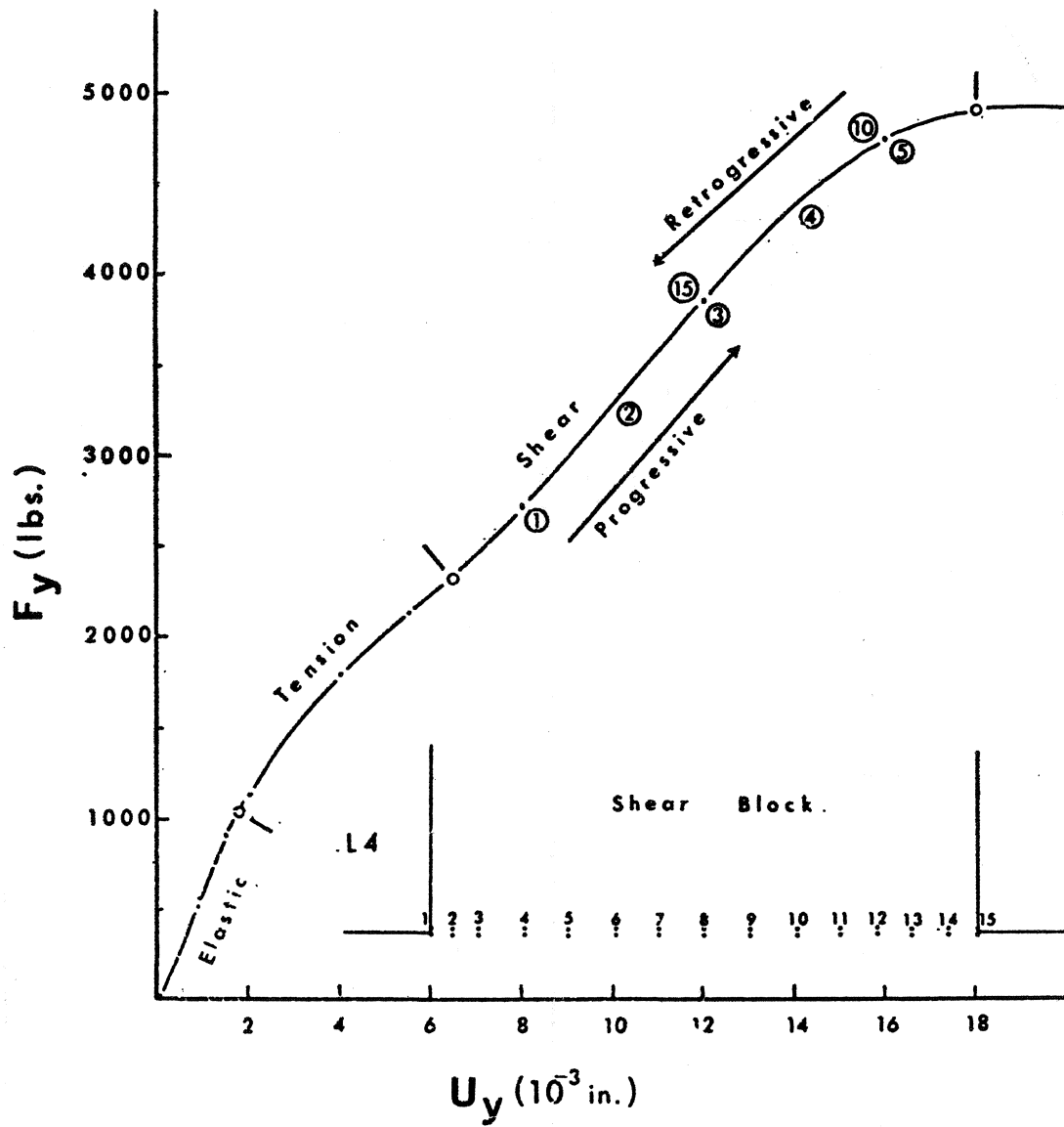
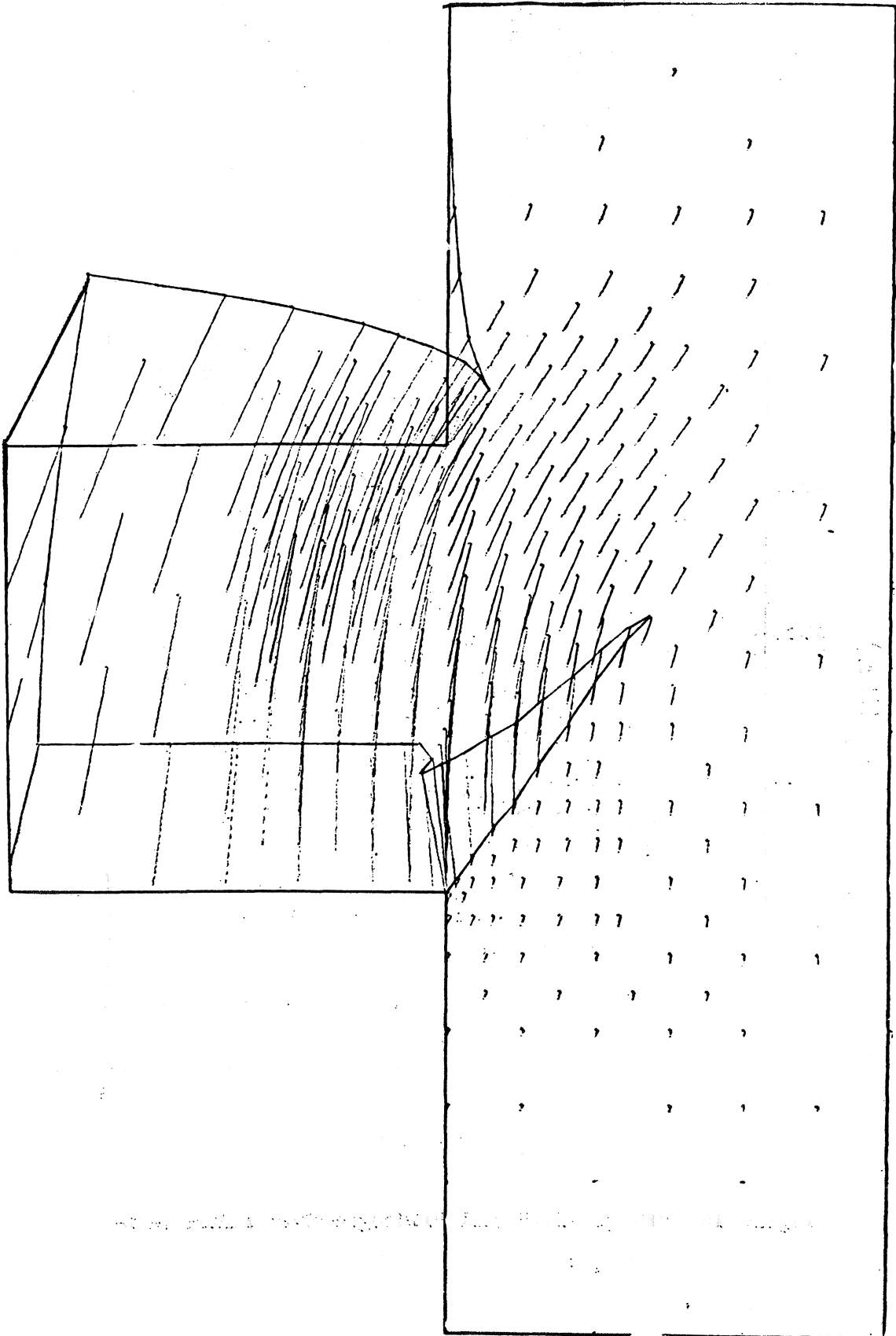


Figure 15. "Progressive" and "Retrogressive" failure series



**Figure 16. Displacement field associated with "Progressive" and "Retrospective" failure series (Complete opening of D3)**

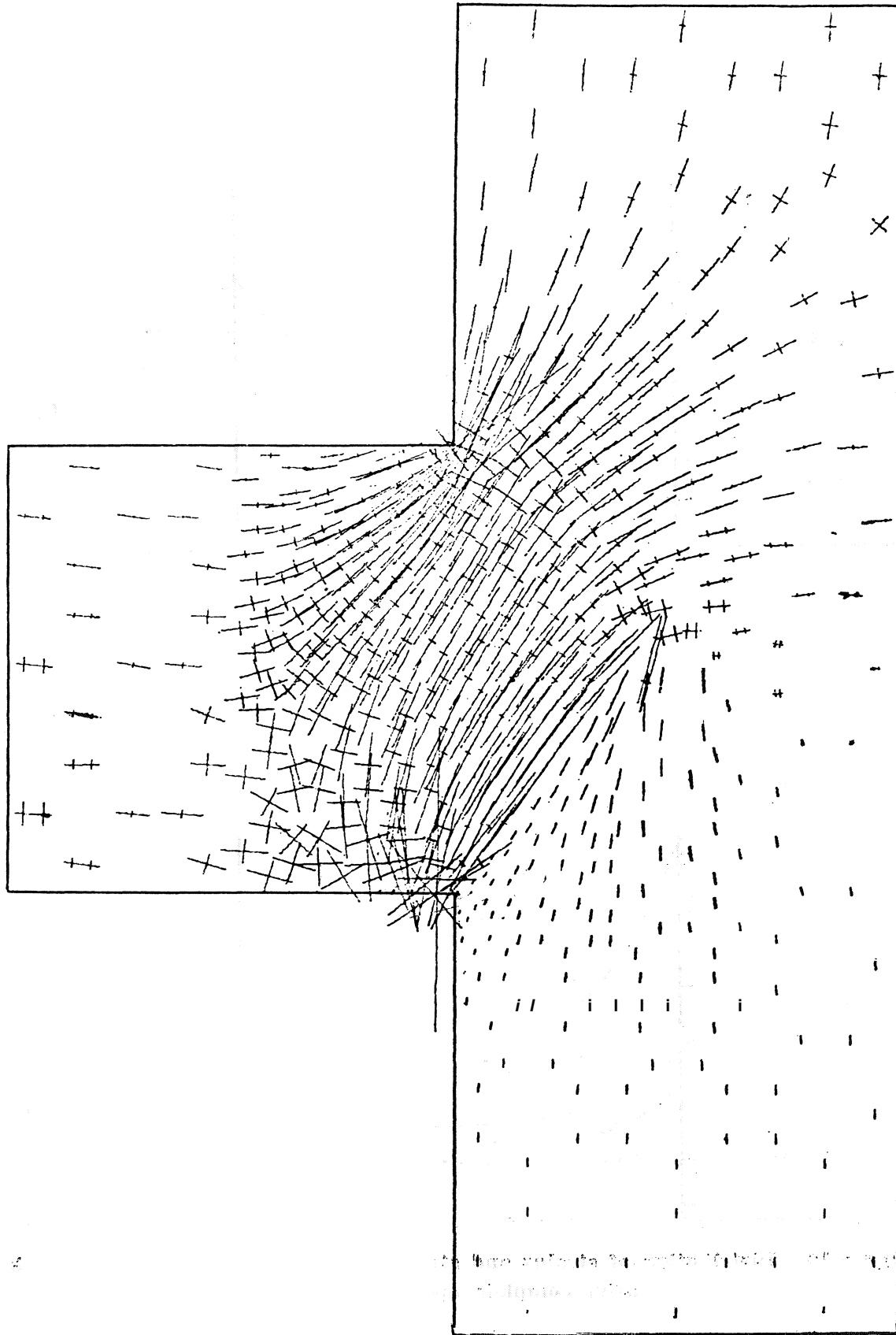
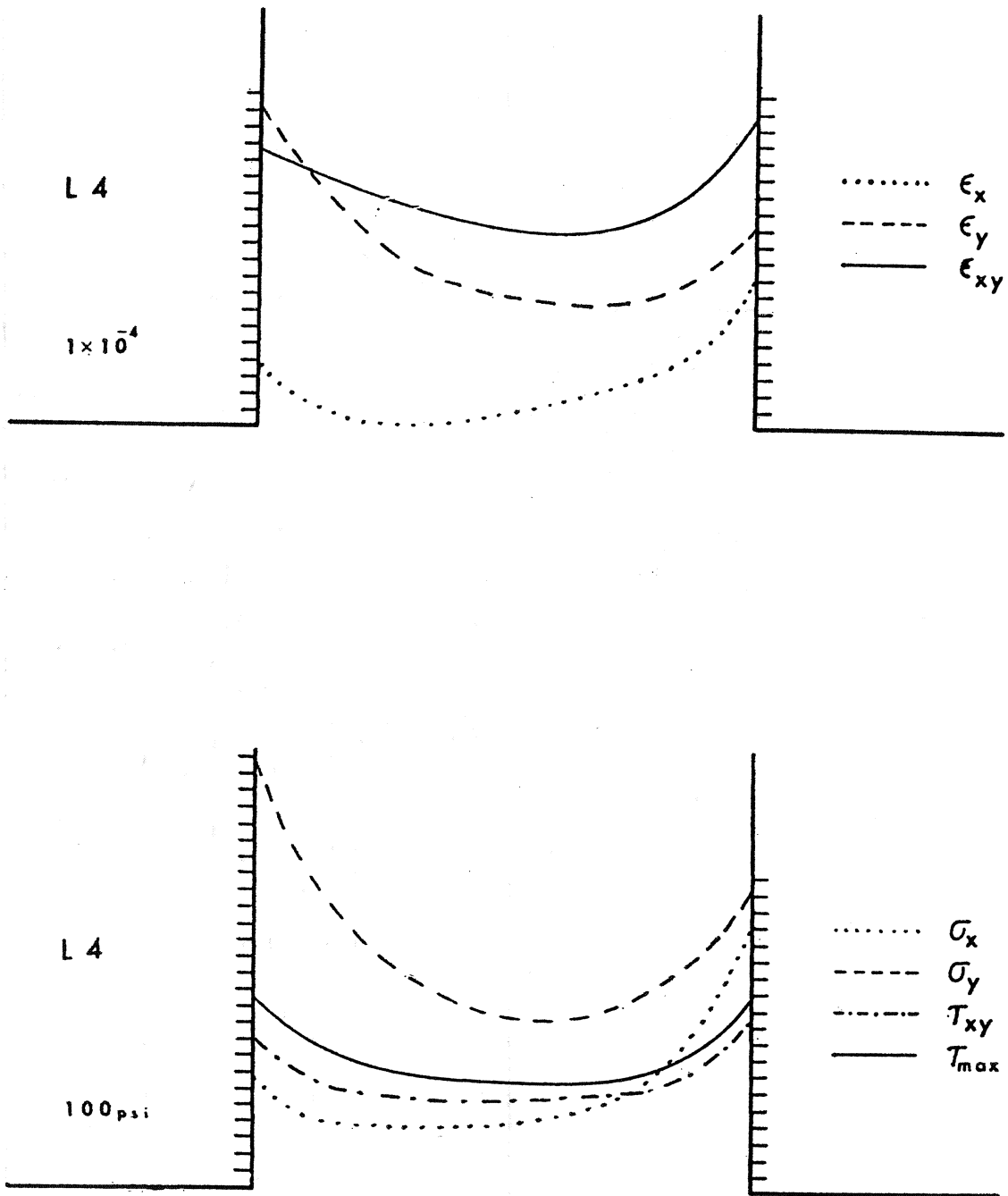


figure 17. Principal stress distribution associated with ^Progressive and "Retrogressive"\* failure series (Complete opening of D3»



**Figure 18. Distribution of strains and stresses at the base of the shear block after complete opening of D3.**



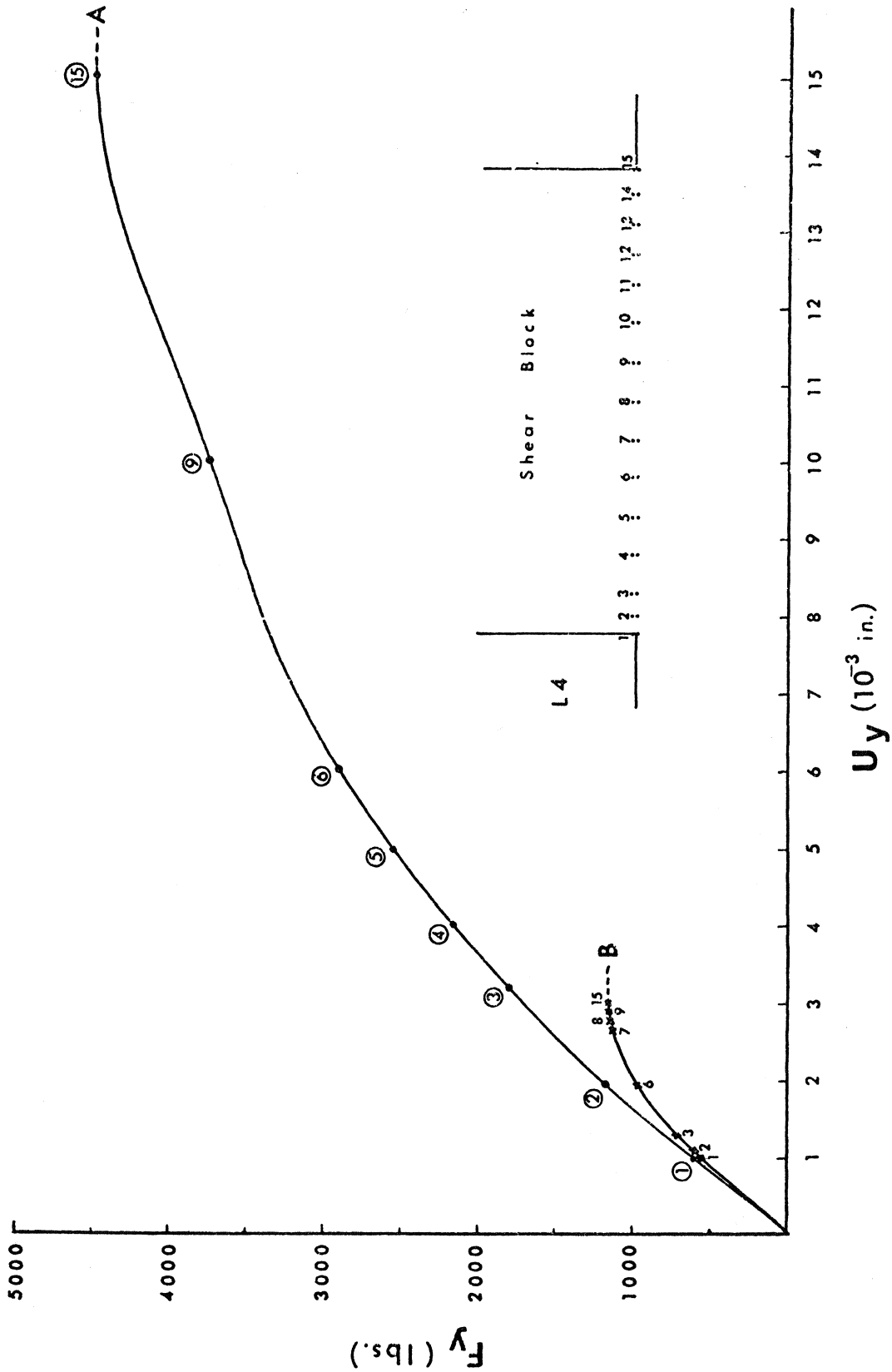


Figure 19. Force-Displacement relationships for progressive failure: A, First series; B, Second series of computer experiments.

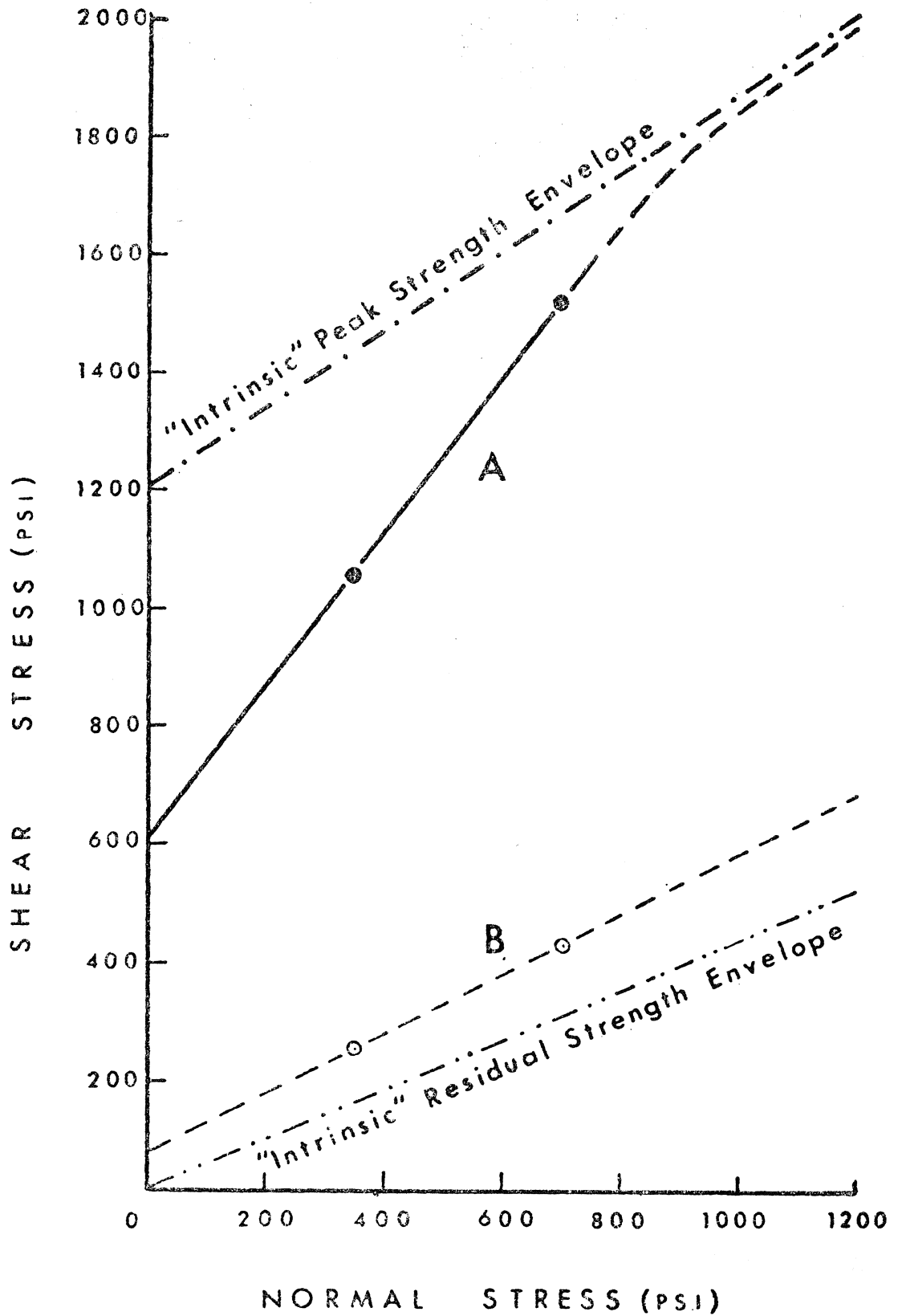
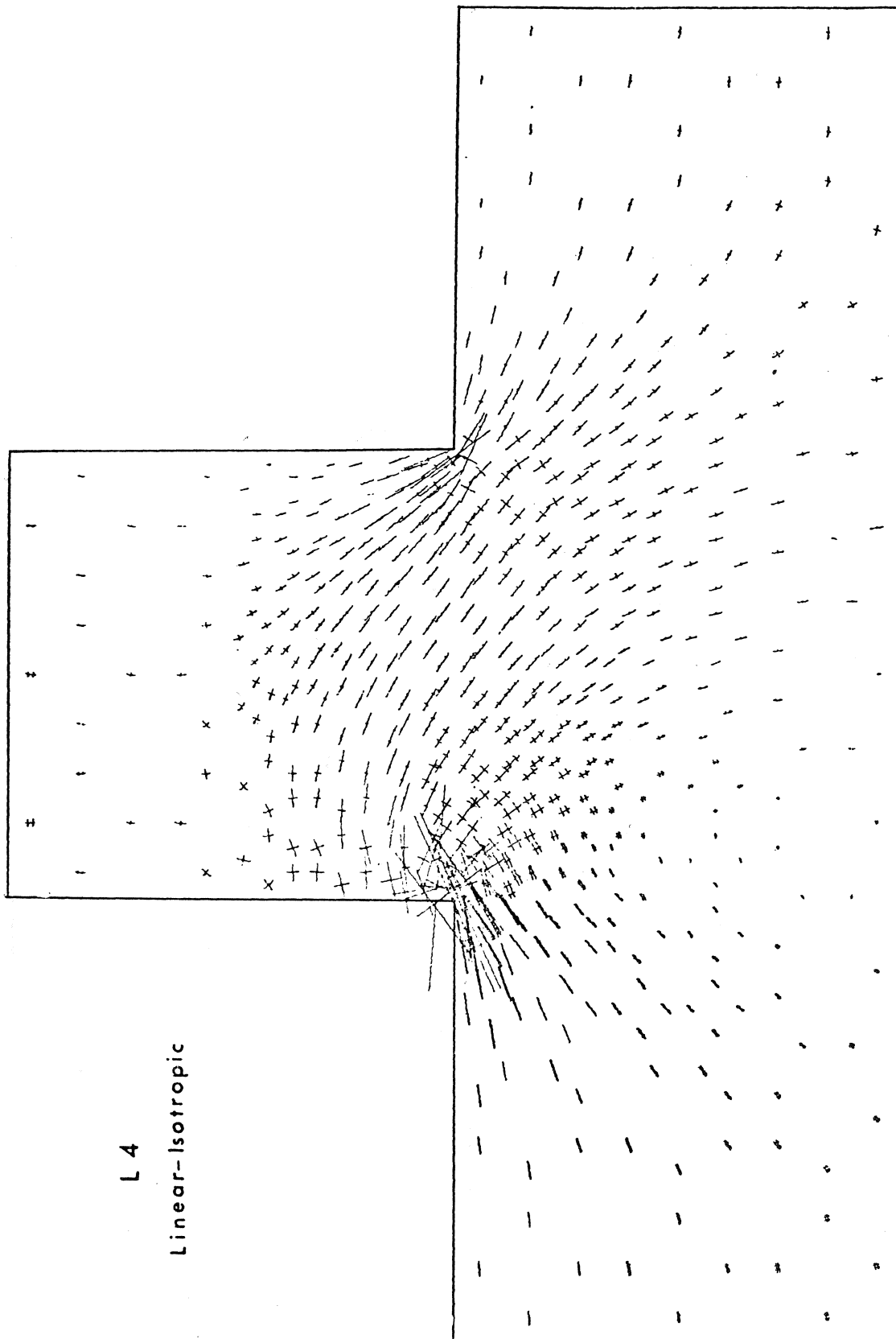


Figure 20. "Predicted" and "Intrinsic" shear strength failure envelopes: A, predicted on the basis of peak strength retainment in "failed" region; B, predicted on the basis of residual strengths for "failed" region.



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Figure 21. Distribution of principal stress in the plastic stage of the shear block.

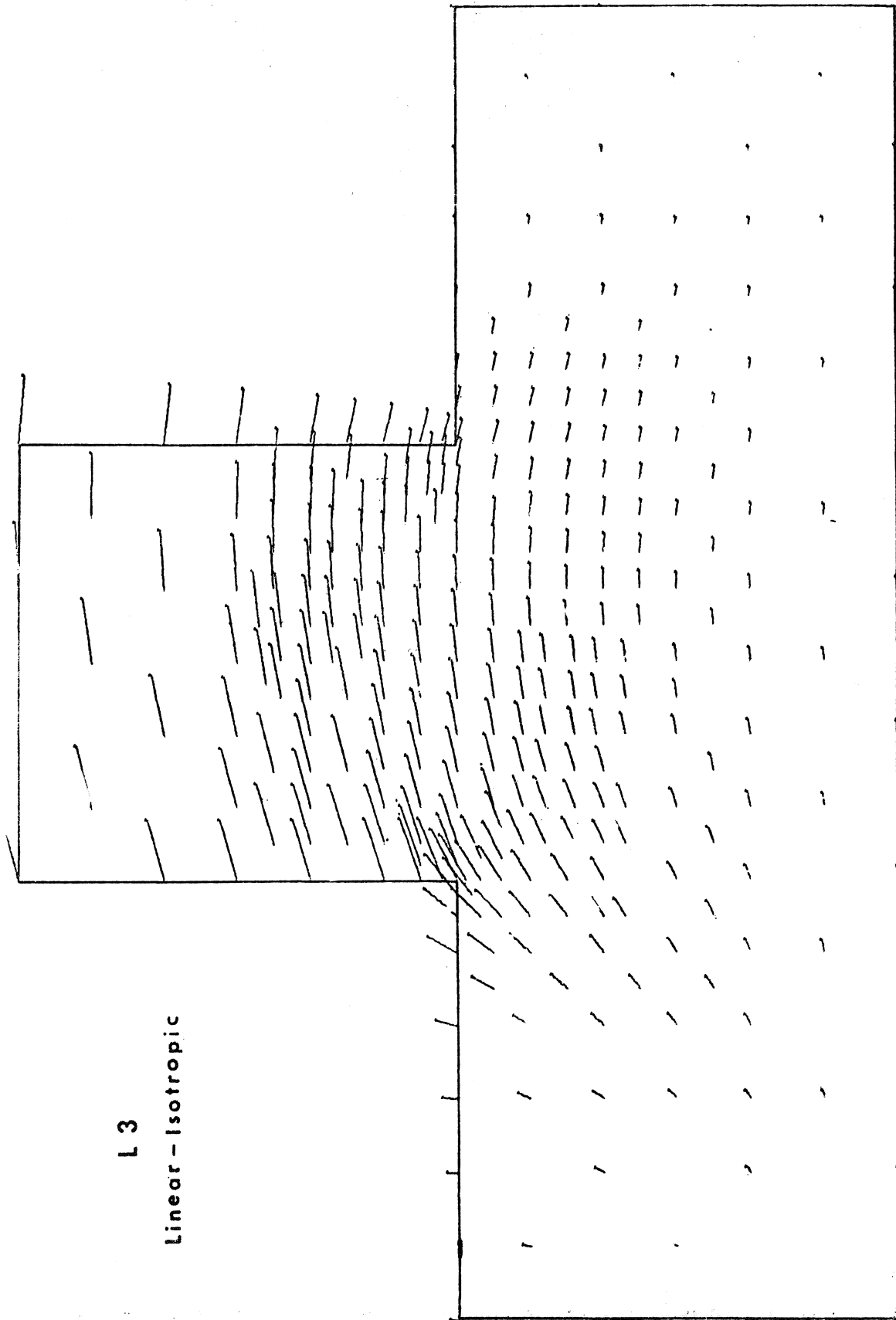


Figure 22. Plastic displacement field in the shear block.

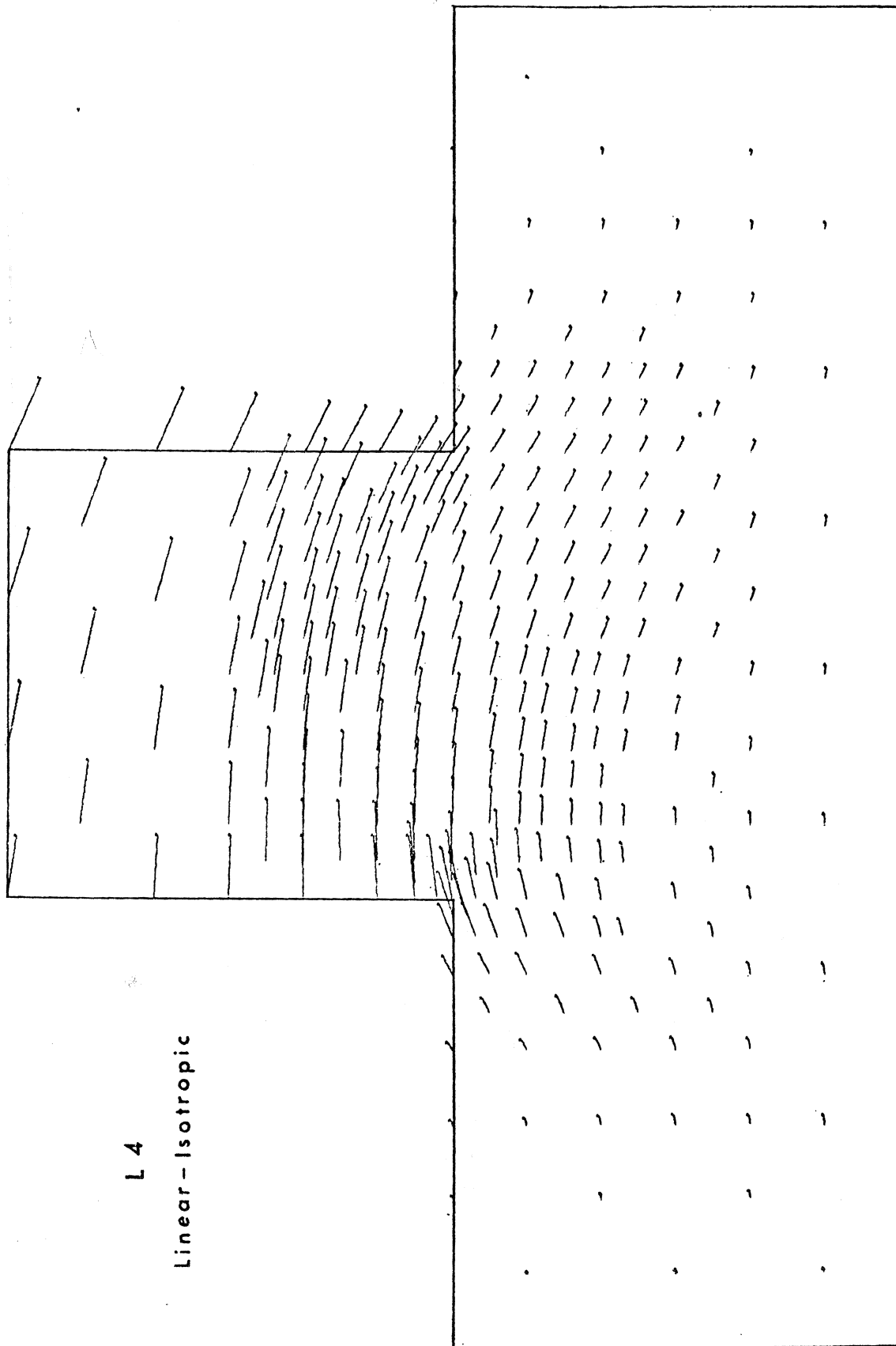


Figure 23. Plastic displacement field in the shear block.

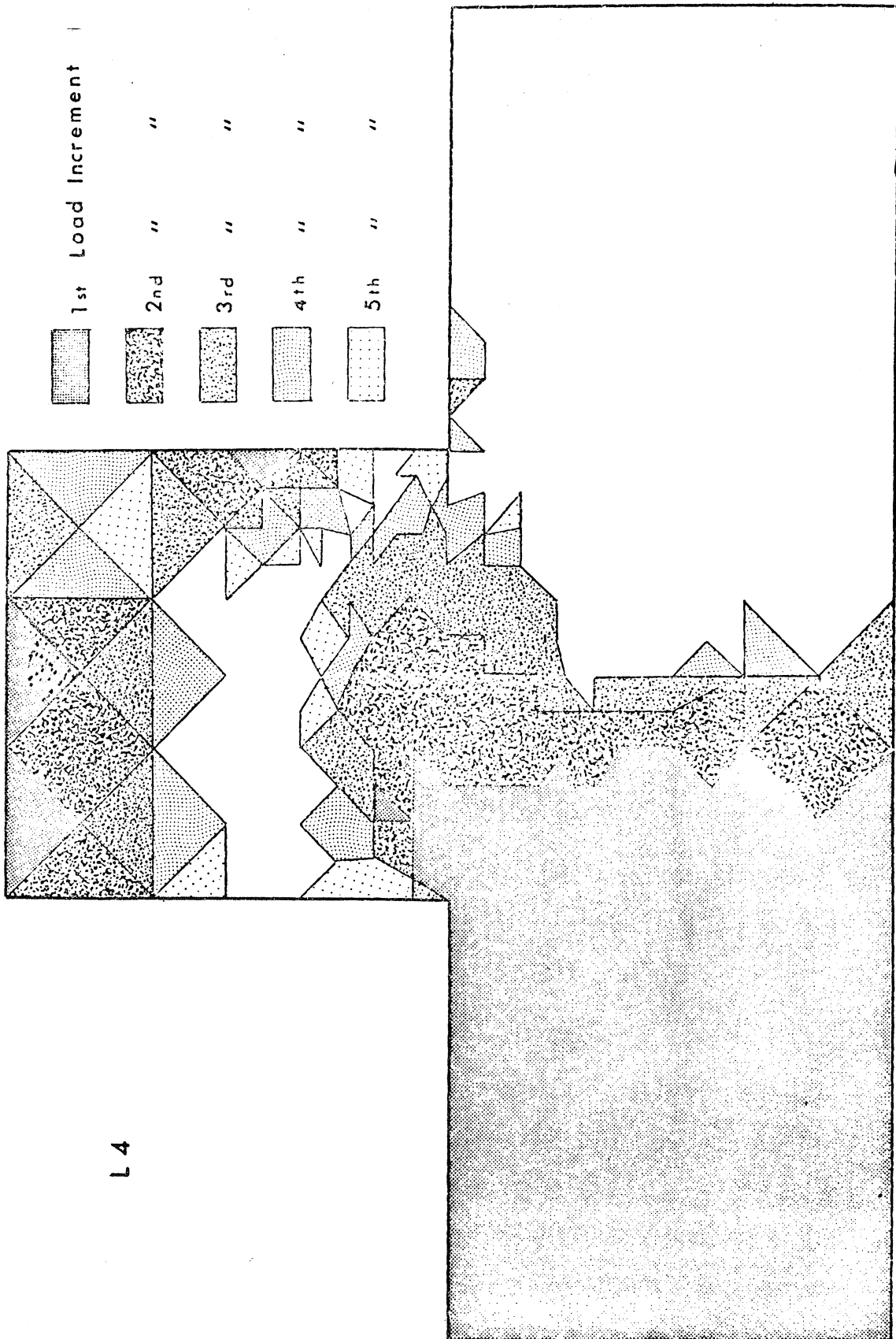


Figure 24. Progression of the plastic yield zones in the shear block.