1.2.5 Mechanisms of Rock Failure

Jennings (1970) in his work on the stability of slopes, uses a limit equilibrium approach. He bases his work on the four geological propositions laid out in section 1.2.3. He illustrates the concept of a mean plane of potential two-dimensional failure, formed as a result of the composite interaction of two sets of joints, defined as the α-joints and the ψ-joints, Figure 1.2.5.1 (a). Although only one of each of these two joints is indicated, the plane represented by line

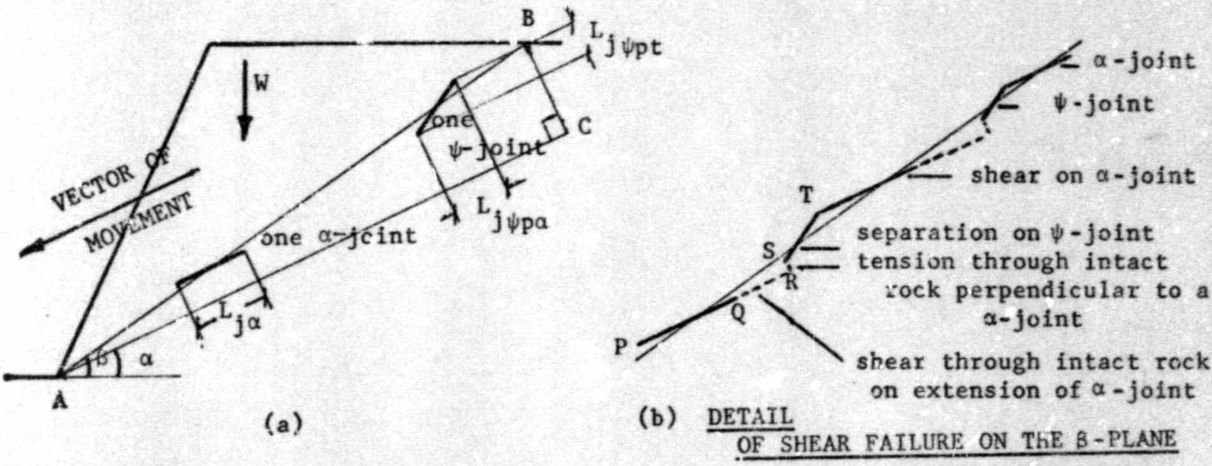


FIGURE 1.2.5.1 POTENTIAL FAILURE ALONG A SURFACE AB WHICH INCLUDES PRE-EXISTING JOINTS

AB, and defined by the angle β, intersects definable numbers of each of these joints. The mechanics of failure are shown in Figure 1.2.5.1 (b). The movement vector takes place in the direction defined by the angle α, with joint shear taking place over the length PQ of the α-joint and shear through intact rock on the extension of the α-joint, QR. At a certain point, R, on this extension the mode of failure changes to tension on a surface RS, at right angles to α and this tension surface extends until it intersects the lower end, S, of a ψ-joint, ST, which, for purposes of this model starts at the bottom end of the next α-joint. As movement takes place in the α-direction, simple separation occurs on the ψ-joint. The process is then repeated for the next pair of α- and ψ-joints. The whole mechanism has been called the step-joint mechanism and has been observed in the field by, for instance, Müller at Vajont. It has also been demonstrated (in a slightly different form) in the laboratory by, for instance, Kawamoto (1970).

The mechanism involves the following assumptions:

1. Both α- and ψ-joints occur in a pattern which is spatially homogeneous in a statistical sense and this pattern may be represented by joints having average lengths with average spacings. It is probably not unreasonable to use mean conditions providing the β- line intersects a sufficiently large number of each joint type.
2. The ψ-joint, as illustrated by TS, starts systematically at the lower end of an α-joint and is of length TS. This obviously does not happen in practice. α-joints will intersect at least some ψ-joints away from their ends and thus reduce the length along which separation can occur.
3. Failure through intact material takes place by simple shearing along QR, the extension of the α-joint, and simple tension on SR, normal to the α-direction. Other possibilities of failure can be visualized and, therefore, this assumption requires further investigation. The research presented in this thesis is an investigation into this assumption.

Jennings states that, "The factor which is of the greatest importance in any calculations is the failure through intact rock, determined largely by the lengths QR for shear and SR for tension". From these two lengths, two coefficients of continuity of the jointing, one with respect to shear in the α-direction and the other with respect to tension in the direction (α + 90^0) can be developed. Both depend on β; the shear continuity depends also upon the lengths of the α-joints and the tension continuity depends only on the lengths of the ψ-joints. The continuity coefficients are defined as follows:

$k_{\alpha\psi\beta}$ = coefficient of continuity of jointing with respect to shear (along the mean β-plane with vector of movement in the α-direction).

$k_{\psi\beta t}$ = coefficient of continuity of the jointing with respect to tension.

These coefficients of continuity can be considered in terms of the projections of the joints into the α and $(\alpha + 90^0)$ lines AC and BC, as defined in Figure 1.2.5.1(a), giving:

$$k_{\alpha\psi\beta} = \frac{\Sigma L_{j\alpha}}{AC} \quad (\leq 1)$$

and

$$k_{\psi\beta t} = \frac{\Sigma L_{j\psi pt}}{BC} \quad (\leq 1)$$

The length involved in shearing on joints is (see Figure 1.2.5.1 (a))

$$\Sigma L_{j\alpha} = k_{\alpha\psi\beta}.AC = k_{\alpha\psi\beta}.AB.\cos(\beta-\alpha)$$

The length involved in shearing of intact material is

$$AC - \Sigma L_{j\alpha} - \Sigma L_{j\psi p\alpha} = AC.(1- k_{\alpha\psi i}) = (1-k_{\alpha\psi i}).AB\cos(\beta-\alpha)$$

where $k_{\alpha\psi i}$ is defined as

$$\frac{\Sigma L_{j\alpha} + L_{j\psi p\alpha}}{AC} \quad (\leq 1)$$

The length of intact material involved in tension is

$$BC - \Sigma L_{j\psi pt} = (1 - \frac{\Sigma L_{j\psi pt})}{BC} \cdot BC = (1 - k_{\psi\beta t}).AB.\sin(\beta-\alpha),$$

The quantities which are of importance to the calculation of the margin of safety of the slope are the disturbing force (DF) and the maximum possible resisting force $(RF)_{max}$, both of which should be considered in the vector direction of movement, α.DF is a simple quantity being the component of the gravity force down the α-direction, i.e. DF = $W\sin\alpha$ where W is the weight of the material which slides out. (For problems involving lateral accelarations, the form of W should be changed). $(RF)_{max}$ is more difficult since it is composed of the ultimate or failure components of shear on the α-joints, shear through intact rock in the α-direction and tension normal to the α-direction; bearing in mind that these components attain their maximum strength values at different displacements.

Referring to Figure 1.2.5.1; after movement has taken place in the α-direction and separation has taken place along the ψ-joints, the component of W acting normal to the α-direction (W cos α) is transmitted across the β-plane along a length $AC - \Sigma L_{j\psi p\alpha}$, making the normal stress on those surfaces across which movement is taking place

$$\sigma_{n1} = \frac{W\cos\alpha}{AC - \Sigma L_{j\psi p\alpha}}$$

For the special case where there are no ψ-joints, this expression simplifies to

$$\sigma_n = \frac{W\cos\alpha}{AC}$$

Taking C_m, ϕ_m, C_j and ϕ_j as the Coulomb parameters applying to the intact rock and to the α-joints, t_m as the tensile strength of the intact rock in the α-direction and L_β = AB, the length of the β-plane, we get

$$(RF)_{max} = AC.(1-k_{\alpha\psi i})(C_m + \sigma_{ni}\tan\phi_m) + AC.k_{\alpha\psi\beta}(C_j + \sigma_{ni}\tan\phi_j) + BC.(1 - k_{\psi\beta t}) .t_m$$

$$= AC\ (\ (1-k_{\alpha\psi i})(C_m + \sigma_{ni}\tan\phi_m) + k_{\alpha\psi\beta}.C_j) + AC.k_{\alpha\psi\beta}.\sigma_{ni}\tan\phi_j + BC.(1-k_{\psi\beta t})t_m$$

$$= L_\beta\ (\ (1-k_{\alpha\psi i})(C_m + \sigma_{ni}\tan\phi_m).\cos(\beta-\alpha) + k_{\alpha\psi\beta}.C_j.\cos(\beta-\alpha) + (1-k_{\psi\beta t})\ t_m.\sin(\beta-\alpha)\ ) + L_\beta\ .\ \cos(\beta-\alpha).k_{\alpha\psi\beta}.\sigma_{ni}.\ \tan\phi_j.$$

This equation assumes that no tension is mobilized across the ψ-joints.

In using a limit equilibrium approach, it is assumed that the maximum values of all the shear strength components are reached simultaneously. This disregards the different stress-strain characteristics of the individual strength components. It cannot be expected, without supporting evidence that, for example, the maximum tensile strength of the intact material will be developed at the same strain as the maximum joint shear strength. In fact, limit equilibrium theory applies to ductile material in which all points on the failure surface are at a similar condition of yield simultaneously. Now the failure characteristics of rock fall between the two extremes which are commonly described as ductile on one hand and brittle on the other, so that it is realized that the limit equilibrium approach does not accurately describe rock behaviour. It is necessary to be mindful of this point since it could be of considerable importance.

Next the question of continuity arises. Jennings' method is based on a measured joint system and this necessarily relies on data accumulated in advance of each succeeding phase of excavation. From this data, continuities are calculated. Any answer obtained using these calculations depends on the continuity values used. Continuity values may change as excavation proceeds due to crack propagation, loss of cohesion and other time dependent effects. Thus, one should always consider the possibility that progressive loss of strength (fracturing of interlocking sections of rock) may occur.

However, limit equilibrium theory has been used for a long time and with much success on soil slopes. It is relatively easy to apply and has given acceptable results for rock slopes. Several facts become clear. The theory proposed should be viewed as a preliminary theory. Certainly, much more careful thought must be given to all factors, particularly continuity of jointing. This thesis attempts to throw some light on the failure of the rock between the joints.

CHAPTER 2

THE EXPERIMENTAL WORK

Chapter 2

2.1 Aim of the Experimental Work

The aim of the experimental work was to investigate the mode of failure through the intact material in a mass of jointed rock subjected to load. As shown in Chapter 1, Jennings, working from the basis of the Griffith crack, has assumed that shear takes place on the extension of the α-joint and that tension occurs at right angles to this joint, the tension surface picking up the lower end of the next ψ-joint. The object of this work is to investigate the reasonableness of this mechanism.

Various cases were foreseen.

1. The stepped-joint failure on the mean β-plane for one set of joints, defined by the apparent dip, α:

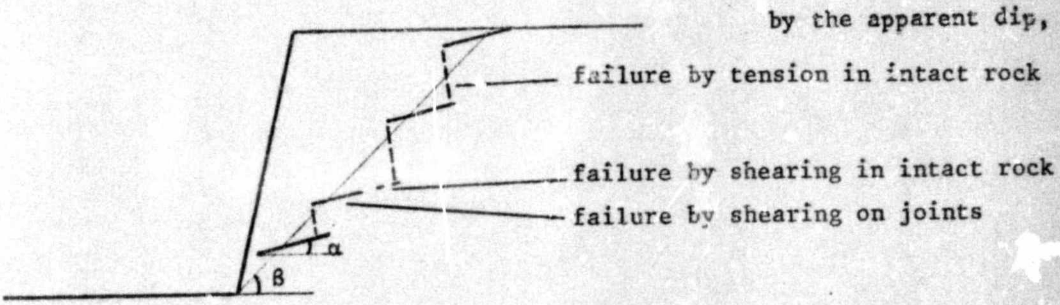


FIGURE 2.1.1 STEPPED-JOINT FAILURE FOR α-JOINTS ONLY

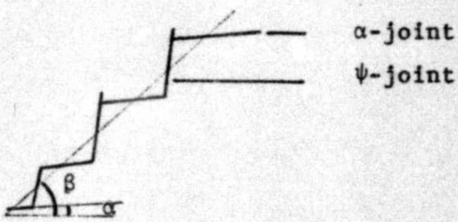
It is unlikely that stepping down will occur from the bottom end of an upper joint and will always encounter the higher end of a lower joint (as shown in Fig. 2.1.2(a)) and it is more probable that the step-downs will sometimes fall inside the next joint and sometimes fall outside the joint (Fig. 2.1.2 (b).

(a)  (b)

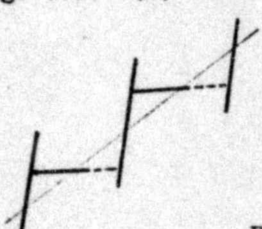
FIGURE 2.1.2

2. Plane failure involving two joint sets:

FIGURE 2.1.3



Above we have the unlikely situation that ψ-joints always occur at the ends of the α-joints. The ψ-joints, however, are more likely to cut the α-joints at points away from their extremities, e.g. long ψ-joints (Fig. 2.1.4 (a) or short ψ-joints (Fig. 2.1.4 (b)).

(a) 

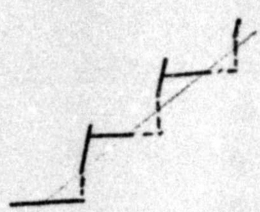
 (b)

FIGURE 2.1.4

Peak loads were recorded for the specimens that were failed so that these values could be compared with failure loads calculated from various theories.

2.2 The Test Material

Several difficulties arise if an experimental program such as this one is to be carried out using natural rock. It is well recognised that as well as often being anisotropic, the mechanical properties of a given rock may vary from block to block. This presents a major problem to the investigator who wishes to carry out a protracted investigation of some fundemental question in rock mechanics. A material that is readily available and fairly constant in its properties has obvious advantages in this regard.

The second major problem encountered in carrying out this type of experimental investigation using natural rock is the difficulty of preparing the samples for testing. Certain joint configurations were required for testing and the finding, collecting and preparing of many consistent specimens of each configuration would have been an unenviable if not impossible task. In this regard the usefulness of a material that can be moulded to any required shape is obvious.

In a number of recent rock mechanics investigations gypsum plasters have been used as experimental materials. Plasters have also found widespread use in the model testing of arch dams. It was decided to use plaster of paris in the experimental work reported herein.

The various advantages of the use of plaster as the sample material in this context may be summarized as follows:

1. Plaster is inexpensive and readily available.
2. With due care properties are reproducible from one mix to the next.
3. Samples may be readily moulded into any required shape.
4. Generally samples are free from anisotropies and defects present in many rocks.
5. The stress-strain characteristics and modes of failure of plaster have been found to be similar to those of rocks in a variety of tests.

Depending on the number and size of samples to be made, a quantity of water was measured out in a measuring cylinder. To this water, 1% by weight of borax powder was added and mixed in. For every one millilitre of water measured out, two grams of plaster of paris powder were weighed out into a large mixing bowl. The water was then added slowly while mixing started. The mixture was then thoroughly mixed into a smooth paste, before being put into the appropriate moulds. It was found that air bubbles tended to become trapped in the mixture, and so the moulds were vibrated on a small vibrator while and immediately after being filled to help bring excess air bubbles to the surface. After this the samples were screeded off with a steel spatula and left to stand until the next day when they could be stripped without any fear of damaging the sample.

Three methods were investigated for simulating jointed rock in plaster of paris. These were:

a) Spread a thin film of soap onto a hardened plaster of paris surface and then cast a second batch of plaster onto this surface.
b) Cast plaster of paris onto a hardened plaster surface and then steam the interface.
c) Spread a film of vaseline on a hardened plaster of paris surface and then cast a second batch of plaster onto this, prepared, surface.

The intention of these procedures was to create two matching and separable surfaces. After some testing of the above methods it was found that a more satisfactory method for simulating joints in rock was to cast one flat plaster of paris surface, cover it with a thin film of vaseline, and then cast onto this flat coated surface another batch of plaster of paris.

When the second batch of plaster of paris had hardened, the whole was immersed in hot water. The two surfaces were then separable. In fact, if a small portion were cast onto a flat coated surface, it would fall off when put into boiling water.

On this basis, a method of modelling jointed rock, which involved soaking the samples in boiling water for 40 minutes was devised.

For this reason, all samples, having been stripped from the moulds, were placed into boiling water for 40 minutes and then allowed to cool, before being tested.

With casting taking place one day and testing the next, it appeared that the samples might vary in age from 18 to 30 hours. Tests were carried out to see what affect this had on the strength. When the unconfined compressive strength of the material was plotted against time, there was no definite trend and it appeared that, in this time range, there were other factors which affected the strength more than time.

In all, five types of samples were produced. These were

a) 3" x 1½" cylinders of the plaster of paris.

b) "Dogbone" specimens for direct tension tests.

c) Specimens for the small shear box apparatus.

d) 4" cubes of the plaster of paris.

e) 12" x 9" x 2" blocks of jointed plaster of paris.

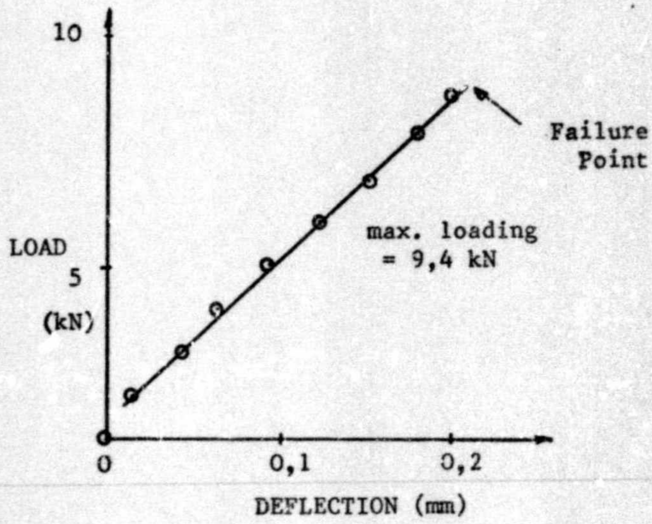
2.2.1 The Unconfined Compressive Strength

The unconfined compressive strength of the material was found using 3" x 1½" cylinders of the plaster of paris. This shaped specimen was chosen as it is more applicable to a rock core than a cube.

The plaster of paris was cast in the mould and stripped the following day. So that the specimen would be subjected to the same conditions as the jointed model, the specimen was put into boiling water before being tested in uniaxial compression in the Macklow-Smith. The bottom platen had a

FIGURE 2.2.1.1

TYPICAL LOAD-DEFLECTION CURVE



spherical seat. The load was increased until failure began. This stage could be recognised by the fact that the sample would not take an increased loading. For some of the tests, vertical deflection of the specimen was measured.

After the initial bedding down effect, most of the load - deflection curves were almost linear, e.g. Figure 2.2.1.1. The failed specimens showed internal shear failures at angles less than 45° to the direction of compression or the major principal applied stress. (An estimate put this angle at about 30° as indicated in Figure 2.2.1.2).

Six tests gave an average unconfined compressive strength of 8,3 MPa with a standard deviation of 0,7 MPa.

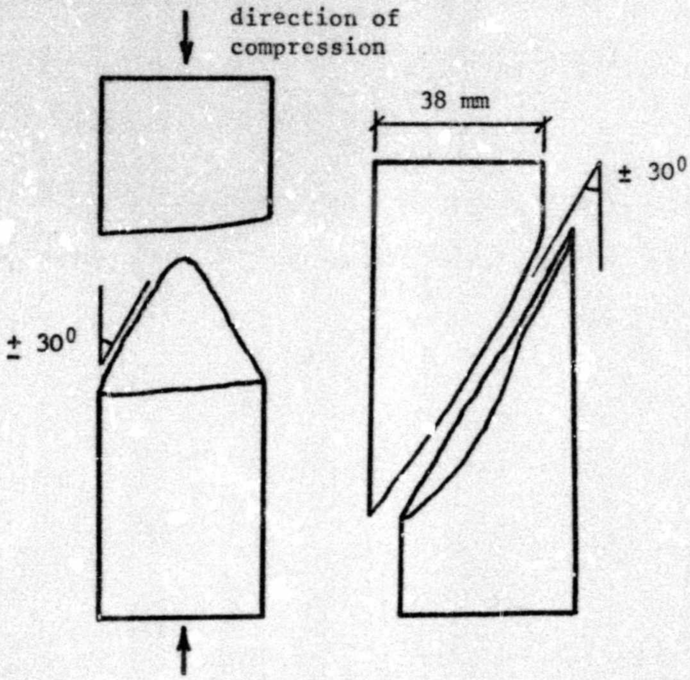


FIGURE 2.2.1.2

DIAGRAM SHOWING TYPICAL FAILURE OF PLASTER CYLINDERS. INTERNAL FAILURES OCCURRED AT $\pm$ 30^0 TO COMPRESSION DIRECTION

2.2.2 The Tensile Strength

The tensile strength was measured from dog-bone shaped samples to which uniaxial tension was applied. The smallest area of the waist of the sample is one square inch. The loading in pounds thus gives the strength in pounds per square inch.

Again the samples were cast one day and stripped the next. They were then also put into hot water before being tested. The average of 34 tests gave the tensile strength as 1,49 MPa, the standard deviation of these values being 0,26 MPa.

2.2.3 Joint Properties

The object of these tests was to find some relation between the normal stress on a joint surface and the shear stress at failure.

The apparatus used was the small shear box in the Soils and Materials Laboratory of the Department of Civil Engineering at the University of the Witwatersrand, Johannesburg. This apparatus allows varied normal loadings and a constant rate of strain to be applied across the shear surface.

The procedure followed was:

1. The bottom joint surface was formed by pouring plaster of paris onto the same perspex surface as was used for the model joints and into the detachable part of the bottom or moving half of the shear box.
2. When the plaster of paris had hardened, the detachable part was screwed into the bottom half of the shear box.
3. The top of this bottom joint surface was then coated with a thin film of vaseline.
4. The top half of the shear box was then secured in place on top of the bottom half, being separated from it by a strip of thick vaselined paper.
5. The top half is then filled with plaster of paris to form the top joint surface and the whole is left overnight.
6. The next day, the joint was put into hot water, care being taken to ensure that no movement took place on the joint interface.
7. The joint was then put into the shear box machine and tested under a known normal loading.

As the constant rate of movement (0,81 mm per minute) was applied, readings were taken on the deflection gauge attached to the proving ring (at specified time intervals). These readings represented the shear force being transmitted by the joint surface at a known displacement. Load-deflection (load-time or shear stress-deflection) curves could then be drawn. It was found (Fig. 2,2,3,2.) that the load increased to a peak and then decreased to a "residual" value. All tests were continued until the "residual" load value appeared to have been reached. There seem to be a

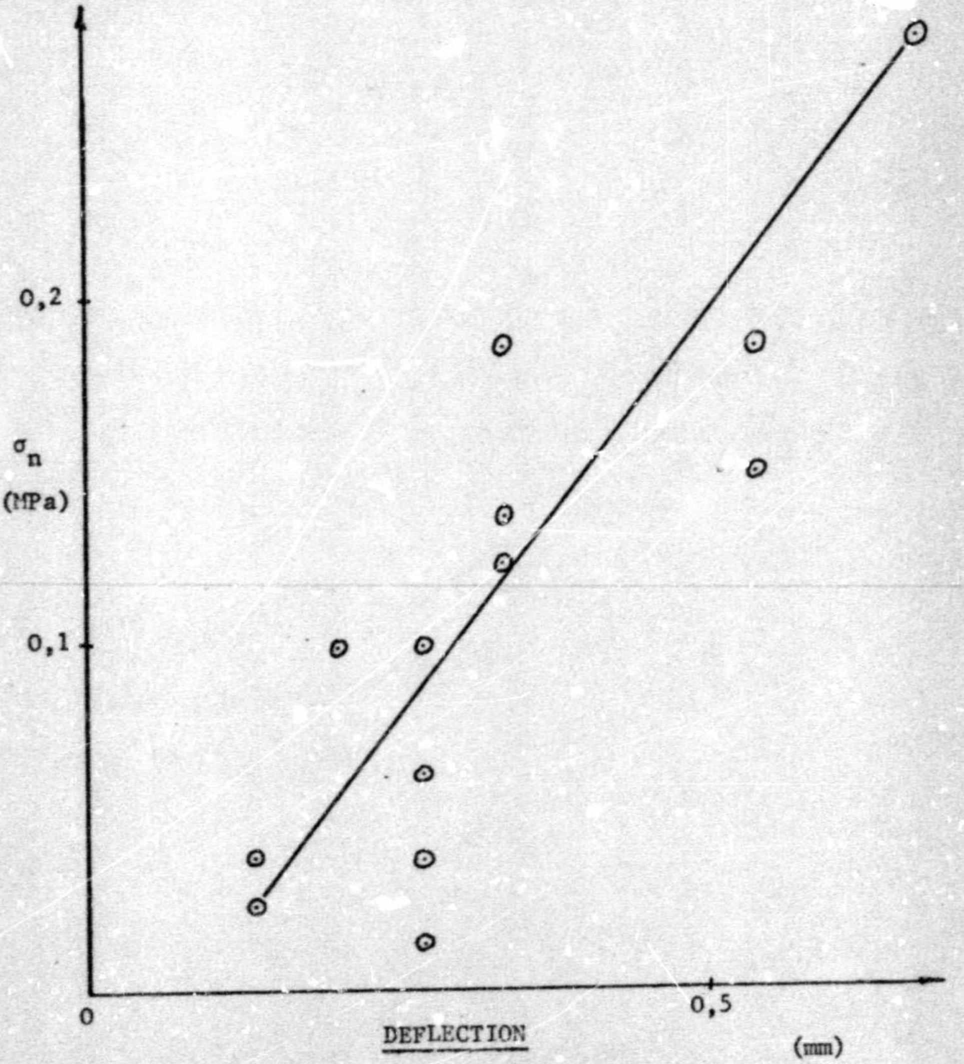
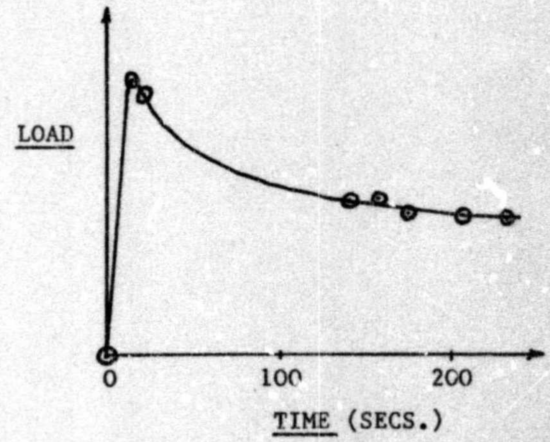


FIGURE 2.2.3.1

DEFLECTION AT PEAK LOAD FOR DIFFERING NORMAL STRESSES tendency for the peak load to be reached at a later time (and consequently a bigger displacement) for higher normal stresses (see Figure 2.2.3.1).

FIGURE 2.2.3.2

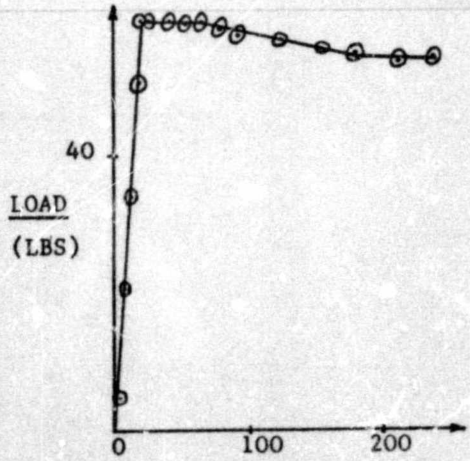
TYPICAL LOAD-TIME CURVE FOR JOINTS TESTED IN THE SHEAR BOX



When the test was complete, the two halves of the shear box were then sometimes put back into their original relative positions and the test was repeated. The load-time curves derived from these, repeated, tests showed a much less pronounced peak (Figure 2.2.3.3).

FIGURE 2.2.3.3

TYPICAL LOAD-DEFLECTION CURVE OF USED SURFACE



It is seen therefore, that for the joints being used for the first time at least, there are two distinct strenghts i.e. a peak strength and a residual strength. If peak strength is plotted against the normal stress on the joint surface the graph shown in Figure 2.2.3.4. is obtained.

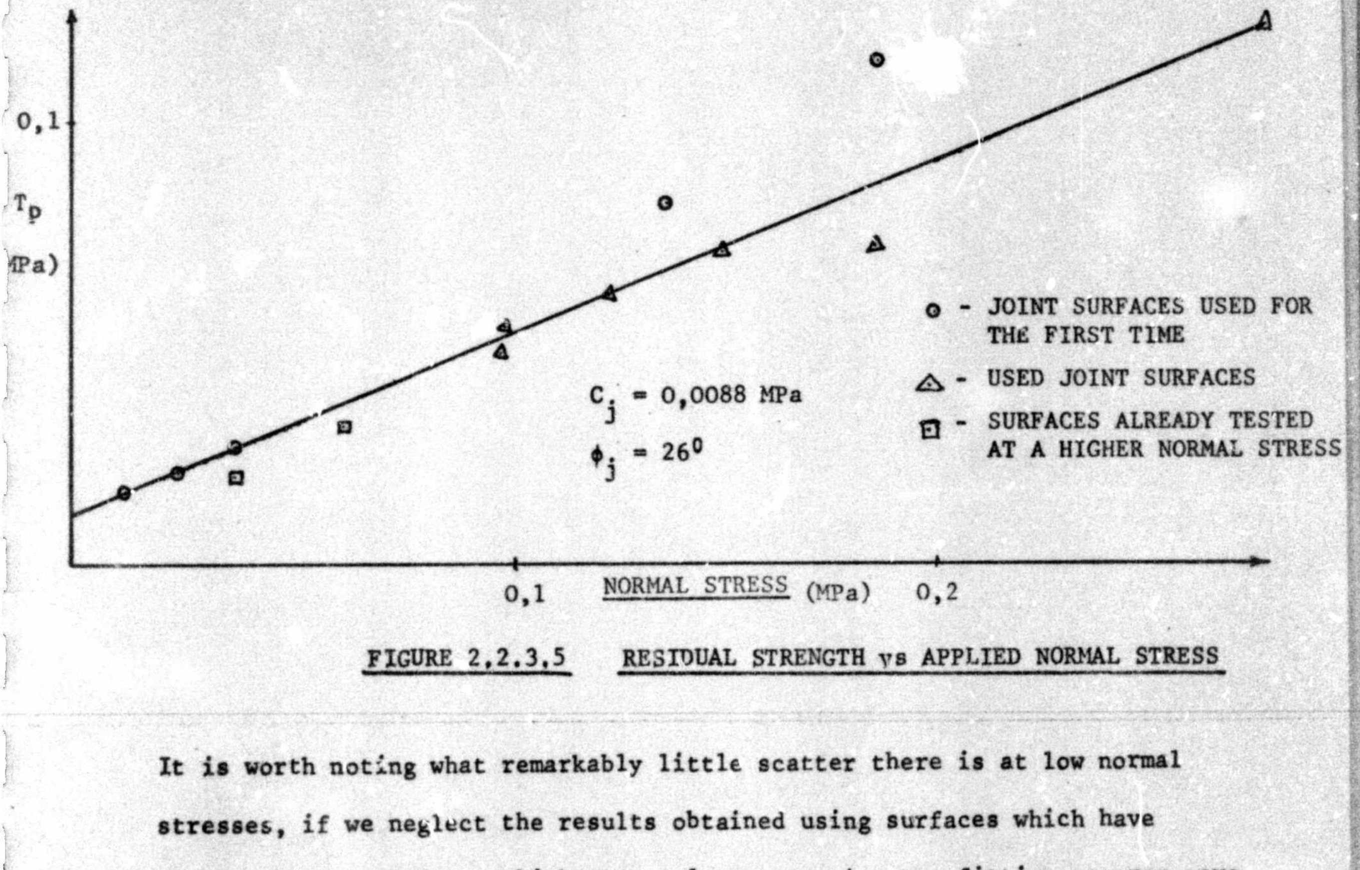


FIGURE 2.2.3.5 RESIDUAL STRENGTH vs APPLIED NORMAL STRESS

It is worth noting what remarkably little scatter there is at low normal stresses, if we neglect the results obtained using surfaces which have already been tested at a higher normal stress. A curve fitting program gave the shear parameters as C_j = 0,0088 MPa and $\phi_j = 26^0$ when asked to fit a straight line to all 13 points.

Two facts noted above, i.e. (a) the lower peaks of the joints not being used for the first time, and (b) the low strength of joints which have already been used at a higher normal stress, lead one to believe that shearing of asperities takes place on the joint surfaces during testing and that, not unexpectedly, the higher the normal stress the more severe this shearing is.