Stress-strain Paths in the Collapse-swelling-shrinkage of Soils in Situ

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SYNOPSIS

The volume change of partly saturated soils is strongly path-dependent. A comparative study of laboratory and in situ stress-strain paths allows to choose the appropriate calculation stress-paths.

At the laboratory several hundreds of tests on undisturbed and compacted samples have been carried - out.

A three-dimensional finite element method for the calculation of any type of foundation on expansive or collapsing soils has been developed. This method has been applied to the foundation of instrumental buildings using different hypotheses and calculation stress-paths: the resulting in situ stress-strain paths have been analyzed and compared with the calculation ones.

INTRODUCTION

As shown extensively by Alonso et al. (1987), - the volume change of partly saturated soils is strongly path-dependent. The analysis of stress-path in situ in a soil during and after construction has a great interest at the design level. The simulation at the laboratory of in situ stress-path allows to choose the appropriate laboratory and calculation stress-paths.

A long-run research has been undertaken with - three fields: laboratory tests, in situ measurements, and calculation with a three-dimensional finite element method (FEM) developed for expansive or collapsing soils.

At the laboratory the oedometer test has been - chosen due to its simplicity and suitability - to the design problems as will be shown later. Several types of oedometer and testing conditions have been chosen and compared (v. Justo - et al., 1984a; Delgado, 1986). Several hundreds of tests on compacted and undisturbed samples - have been carried out.

In situ, the seasonal displacement of several - buildings has been measured during 11 years. Levelling marks have been placed on the buildings and at several depths in the soil (Justo et al., 1985a & b).

A three-dimensional FEM for the calculation of any type of foundation on expansive or collapsing soil has been developed. This method has - been applied to the foundation of instrumented buildings using different hypotheses and calculation stress- paths: the resulting in situ -- stress-paths have been analyzed and compared - with the calculation ones.

THE INFLUENCE OF STRESS-PATH IN THE VOLUME CHANGE OF PARTLY SATURATED SOILS AT THE LABORATORY

A partly saturated soil may suffer volume changes due to pressure or suction changes. It has - been observed that the stress-path has an important influence in the final void ratio reached.

Tests in normal oedometers

Figure 1 shows the "natural moisture content" - curve, obtained loading the soil without changes in moisture content.
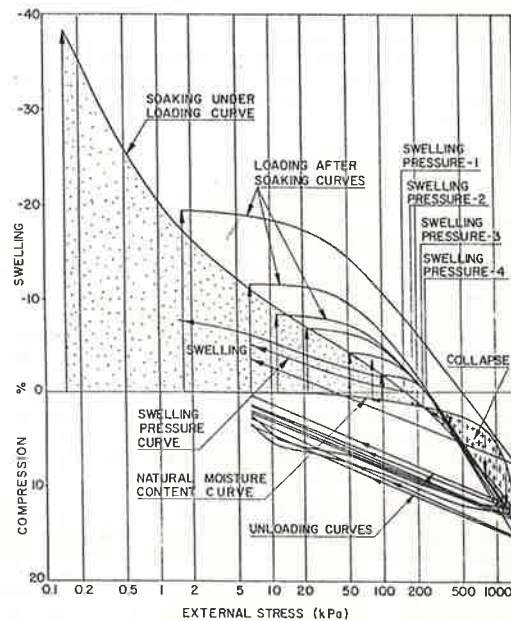

Figure 1: Soaking tests on compacted samples of El Arahal (Justo et al., 1984a)

The same figure shows the curve obtained soaking the sample under loading.

As seen in figure 1 there are many possible definitions of the "swelling pressure" (v. Justo et al., 1984a; Delgado, 1986). From a practical point of view, the most important one is the "swelling pressure-3" of figure 1, corresponding to the crossing of the natural moisture content and the "soaking under loading" curves. When we apply the "swelling pressure-3" to the sample, soaking does not produce any deformation on it. Below this pressure soaking causes swelling, and above collapse.

So, in general, a soil will swell or collapse after being flooded, depending upon whether the external pressure is smaller or larger than the swelling pressure (v. Justo and Saetersdal, 1979).

Many soils when wetted under low pressure may swell, while under high pressures may show a tendency to collapse. Low pressures may be applied by many engineering structures such as floors and foundation beams, and that must be taken into account in design. Even clays may collapse under moderate loads (v. Justo, 1986; Justo et al., 1987).

This shows that instead of distinguishing between expansive and collapsing soils, we should talk about "soils that usually behave as expansive and soils that usually behave as collapsing".

The "swelling pressure-2" is the one obtained in a swelling pressure test.

The "loading-after-soaking" curves stay above the "soaking-under-loading" curve in the swelling zone. This result has been confirmed by many authors (Justo and Saetersdal, 1979).

The difference between the different possible definitions of swelling pressure has been attributed by Delgado (1986) to stress path. Swelling pressure-1 is, by definition smaller than swelling pressure-3.

If the sample at the oedometer is left to shrink after loading, the suction of the soil will come finally to an equilibrium with the relative humidity of the air at the room in which the oedometer is placed. In the tests carried out by Delgado (1986) this relative humidity was around 50% and the final suction was so pF = 6. Under these conditions the "shrinkage under loading" curve is a straight line in a natural scale. According to several authors, volume change ceases at pF values between 5.5 and 6.0 (v. Justo, 1986).

Figure 2 shows, in a same graph, the "natural moisture content", the "shrinkage-under-loading" and two "soaking-under-loading" curves, obtained from the "natural moisture content" and the "shrinkage-under-loading" curves respectively. As indicated in the figure, the soaking-under-loading curve from the shrinked sample falls usually below the corresponding curve obtained from natural moisture. This is consequent with the theory established by Alonso et al. (1987) for partly saturated soils, which assumes plastic (irrecoverable) deformations when a certain upper suction is surpassed.
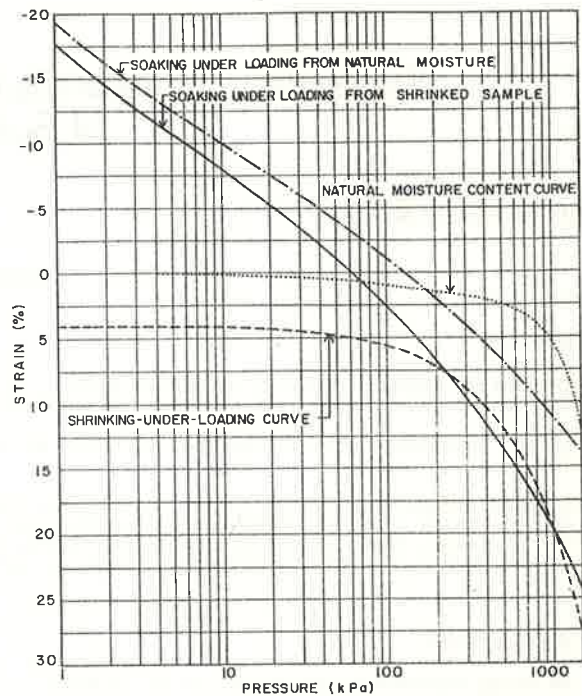
The curves of figure 2 are regression curves ob

Figure 2: Soaking-under-loading and shrinking-under-loading curves in compacted samples of El Arahal (Delgado, 1986)

$w_L = 74$ $I_P = 44$ $w = 23\%$ $\rho_d = 1430$ g/cm³
Proctor test: $w_o = 30\%$ $\rho_p = 1400$ kg/m³

tained from the individual points of test. Actually collapse from soaking after shrinkage has not been obtained as would be deduced from the figure, although swelling reaches a minimum of only 0.11% for a pressure around 400 kPa. Shrinkage deformations are largely recoverable for small pressures, but are essentially plastic for large pressures in plastic soils (v. fig.2; Escario and Sáez, 1986).

Figure 2 shows also that collapse from natural moisture content reaches a maximum for a certain pressure, that in this case is near 1000 kPa (v. also Alonso et al., 1987).

Figure 3 shows the stress-strain paths in an undisturbed clay. Qualitatively the picture is as in compacted clays, but the stress-path dependence during wetting and loading is smaller, and the plastic deformations during shrinkage are much larger.

Usually after the first soaking subsequent changes in suction produce very small deformations (v. Justo, 1982; Delgado, 1986; Alonso et al., 1987). This is a clear demonstration of the plastic behaviour of clay when the yield locus for suction decrease is reached (Alonso et al., 1987).

Exceptions to this general behaviour are shown in a companion paper (Justo et al., 1987). In the other undisturbed clays the stress-path dependency during wetting and loading is more im-
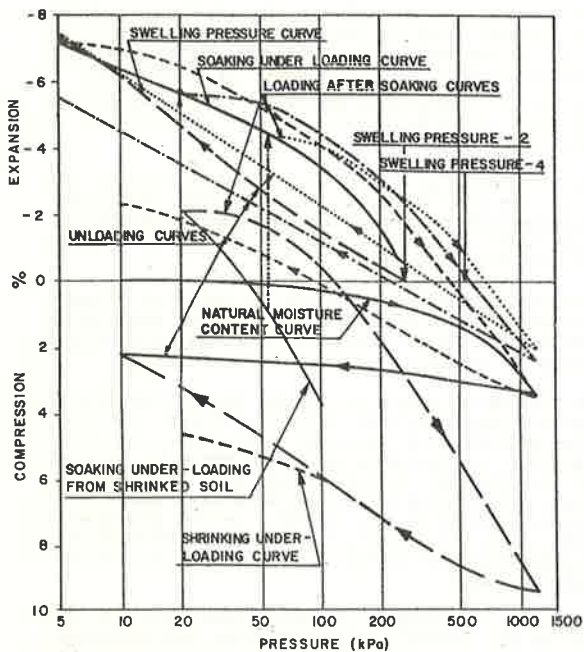
Figure 3: Stress-strain paths in undisturbed soil from Camas

$w_L=66$ $I_p=44$ $I_L=0.08$ $\rho_d=1680$ kg/m^3

Unconfined compressive strength 1.2 MPa

portant than in figure 3 (v. Justo et al.,1985b). With liquidity indexes larger than 0.2 and dry densities not very low, the "natural moisture content" and the "loading after soaking" curves become assymptotic (fig. 4) as assumed in the - double oedometer test. Notwithstanding, the expansion under loading is much than would - be deduced from this test (fig. 4). Possibly in these soils there is no collapse.
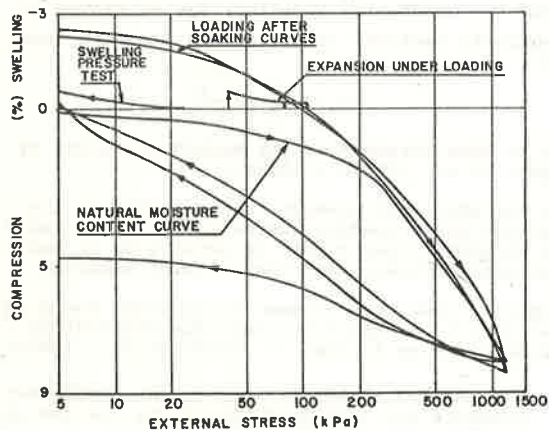

Figure 4: Swelling of undisturbed samples of - Camas

$w_L=59$ $I_p=39$ $I_L=0.22-0.24$ $\rho_d=1580$ kg/m^3

(Justo et al., 1984a)

Tests in suction-controlled oedometers

Actually, the pressure deformation relationship should be measured simulating suction changes - under field conditions.

Figure 5 shows several tests carried out in Escario's suction-controlled oedometer (Escario, 1969). The samples are first loaded and then - suction is brought up to zero step by step.
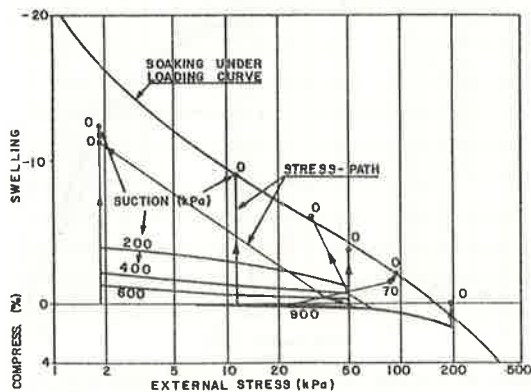

Figure 5: Suction-controlled oedometer tests on compacted samples of El Arahal (Justo et al., 1984a)

When a clay swells around a pier foundation, the pier friction induces stresses in the soil to - counteract the tendency to swelling. So, the - suction decrease is coupled with an increase in vertical normal stress above the base of the - pier. Below the base the vertical normal stress may decrease or increase depending upon the distribution of swelling in the free soil profile (v. Justo et al., 1984b; Delgado, 1986). Fortunately, when suction is decreased as the sample is loaded or unloaded, following in situ stress paths, the final swelling reached is little dependent on stress-path, as seen in figure 5, - and depends mainly on the final suction and total stress, reaching finally, in the practical range, the "soaking under loading curve". The - explanation of this experimental fact might be that the yield locus for suction decrease (v. - Justo et al., 1987; Alonso et al., 1987) corresponds to suction = 0, and below this value volume change is recoverable.

Figure 6 shows the regression curves corresponding to the suction values of figure 5. The curve for the initial suction (900 kPa) is a straight line in natural scale. The curve for zero suction is a cubic parabola in semilog scale - (as it happens with the soaking under loading - curve). It may be noticed, comparing figures 2 and 6 that collapse seems to be smaller in suction controlled tests.

CALCULATION STRESS-STRAIN PATHS

As far as we know, the first true three-dimensional FEM for the analysis of soil-structure interaction in expansive soils was proposed by Justo (1982) and Justo et al. (1983).
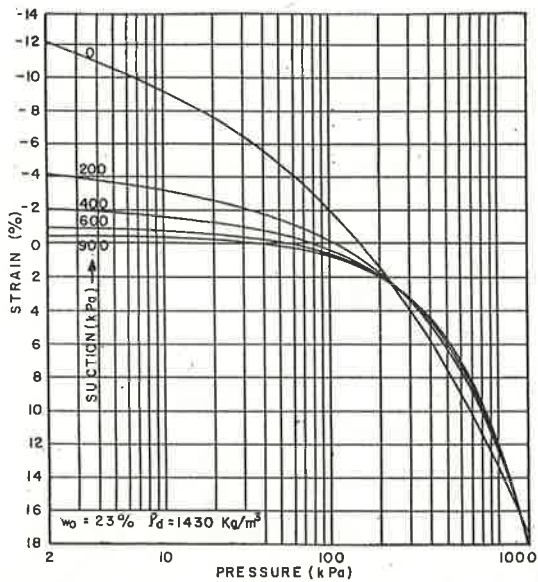
Figure 6: Curves of equal suction obtained in wetting tests under constant load in compacted samples of El Arahal (Delgado, 1986)

In its present form the method is applicable to any type of foundation on expansive-shrinking-collapsing soil.

The calculation stress-strain path followed is a consequence of the laboratory studies summarized above, and is indicated in figure 7.
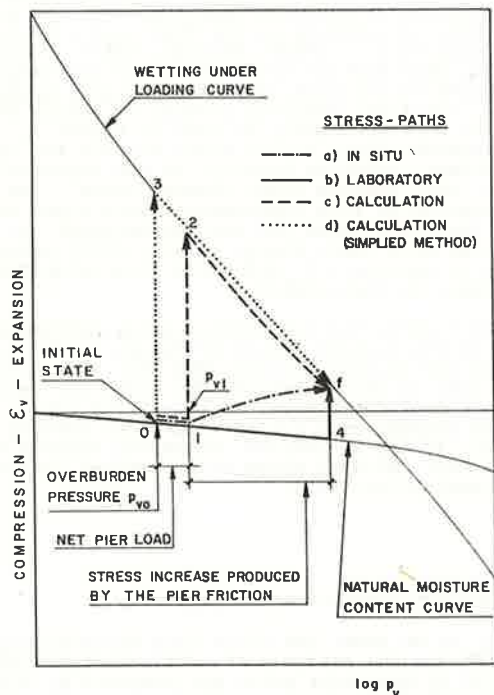

Figure 7: Stress-strain paths around a pier

We are only interested in the strains produced by the net load on the foundation and the wetting of the soil. So, the zero state of strains corresponds to the overburden pressure p_{vo} -- (point 0). From this point on, the stress-strain path in situ is 01f, and the calculation path 03f.

To this end, we apply to each soil element, within the active layer, the swelling 03, which corresponds to its overburden pressure and to the "wetting-under-loading" curve corresponding to its final suction.

The modulus of deformation of the soil element corresponds to the line 3f. As this modulus depends upon final pressure, unknown at the beginning, an iterative procedure is followed.

As far as we know, it is the first time that the stress-path has been duly considered. Most of the methods that consider soil-structure interaction in expansive soils recommend the use of the modulus corresponding to a loading process, which might lead to an overestimate of heave. Livneh et al. (1973), in its plane-strain FEM, draw the elasticity modulus from wetting-under-loading curves, but without justifying exactly why.

The soil is assumed elastic, isotropic (v. Justo et al., 1983), but non-linear and heterogeneous. It is heterogeneous as in each soil element swelling and elasticity modulus are, as a rule, different. Non-linear, because the elasticity modulus depends, generally, upon stress level. Volume change under load, in expansive soils, depends only on the octahedral normal stress, as predicted by the elastic method (v. Justo et al., 1983). During the swelling pressure-2 test, $\Delta\sigma_h \simeq \Delta\sigma_v$, which indicates isotropic conditions.

The method is based only upon oedometer tests.- This is justified by the fact that if horizontal movements are restrained, the influence in deflections is negligible for expected values of Poisson's ratio.

From the oedometric swelling, ε_v, of figure 6, isotropic swelling is calculated from the formula:

$$\varepsilon_o = \varepsilon_v \frac{1 - \nu}{1 + \nu}$$

Up to date versions of the method are given by Justo et al. (1984b & 1985a).

In the practical pressure range 5-200 kPa the wetting-under-loading curves may be assimilated to straight lines either in semilog or in natural scale without important loss of accuracy.

Figure 8 shows the stress-strain paths for a canal in cut on expansive clay. The calculation sequence is as follows (v. Justo et al., 1985c).

1. The initial state, defined by the overburden pressure and isotropic conditions, is the zero state for strains.

2. The non-excavated soil is subject to the effect of excavation, represented by a stress normal to the excavation surface and pointing outwards, of value γz_e, were z_e is the depth
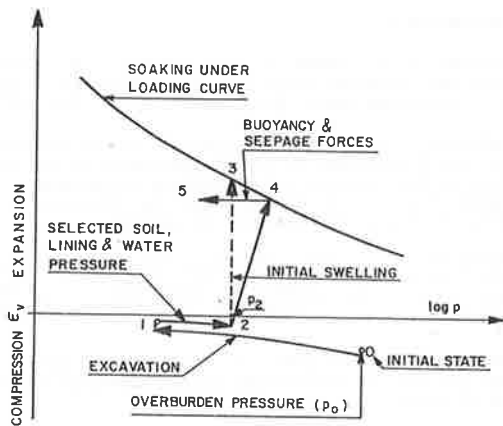
Figure 8: Stress-strain path for a canal in cut on expansive clay (Justo et al., 1985c)

of excavation at each point.

3. The weight of a possible layer of selected - soil, lining, and water load are applied.

4. It is assumed that water may reach the so-called "active layer", nullifying, in a first phase, the suction.

The isotropic volume change produced by the suction change is introduced in the FEM as a function of stress in each element (state 2 to state 3 in figure 8). This initial volume change (swelling or collapse) induces new increments of stress and strain in the soil and lining (state 3 to 4).

5. Either buoyancy in the selected material or seepage in the natural soil produce an unloading in the "active layer". In this case an unloading stress-path is followed (fig. 1 and 3; v. Justo et al., 1984b).

IN SITU STRESS-STRAIN PATHS

Figure 9 shows two seasonal displacements of - the free soil profile measured at El Arahal, - near a group of instrumental buildings with pier foundation, in two periods of time, correspon - ding respectively to shrinkage and swelling.

The displacement of the buildings has been cal- culated using the following hypotheses:

1. The shrinkage of figure 9a was used as an - input.

Figure 10 shows the laboratory natural-mois- ture-content and shrinkage-under-loading li- nes, which are both straight in natural sca- le. For each layer, the overburden pressure and measured shrinkage have been taken to - the graph (state 0). Interpolating between - both lines, we draw for each layer the state line(straight) corresponding to the final suc tion reached. This gives us the oedometric - modulus of each layer, that in this case is a constant.
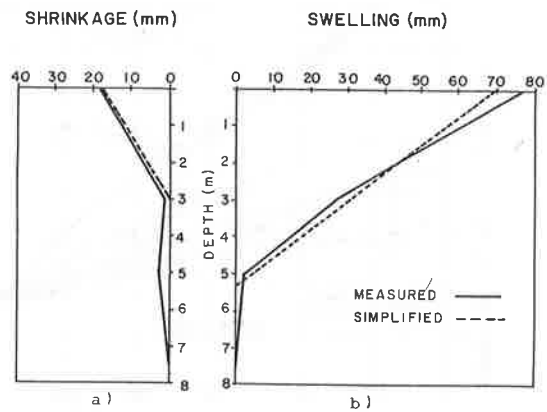

Figure 9: Measured vertical displacements, and simplified volume change considered in the foundation calculation
a) Period May 82-Nov.82. Calculation shrinkage 0.57%
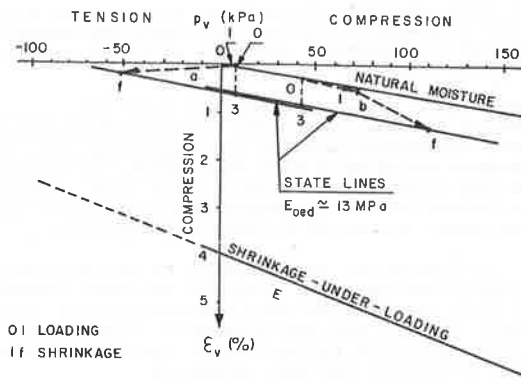b) Period Nov.81-April 84. Calculation swelling 1.3%

Figure 10: Stress-strain paths, obtained in one iteration, for two finite elements in a pier foundation,using as an input for the free soil profile the shrin- kage of figure 9a
a) Soil element in the upper layer, adyacent to the pier
b) Soil element at the base of the - pier

In the figure the states corresponding to the - loading of the building under natural moisture conditions (1), and final after shrinkage (f) - have been drawn. Both, vertical strain and -- stress haven been taken from the FE computations. The two elements, a and b, indicated in the foot of the figure, have been drawn. It is remarka - ble how the state points, obtained from a three- -dimensional calculation, fall nearly exactly - in the state lines assumed in a calculation in which the oedometric moduli (and so the moduli of elasticity) depend only upon the vertical nor mal stres, p_v.

2. The swelling of figure 9b was used as an in- put.

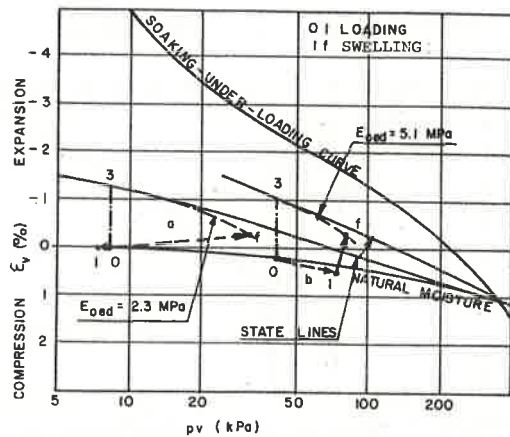The process (fig. 11) was as in paragraph 1 abo-

Figure 11: Stress-strain paths for swelling of
fig. 9b (v. caption of fig. 10)

bove, but using the soaking-under-loading curve,
and swelling instead of shrinkage.

In this case the state lines do not give a cons
tant oedometric modulus. An iterative procedure
should be followed, starting with the tangent -
modulus at point 3 (v. fig. 7 and 11).

The figure shows what happens when only one ite
ration is made. The lines corresponding to cons
tant oedometric modulus corresponding with the
tangent value at 3 have been drawn. In this ca
se the final state falls on this line, instead
of ending at the corresponding state line. In -
any case, the error is not very important.

3. In figure 12, shrinkage up to the shrinkage-
-under-loading curve (fig. 2) has been assumed.
In this unrealistic case tensions at the upper
layer of soil are non-allowable. A procedure -
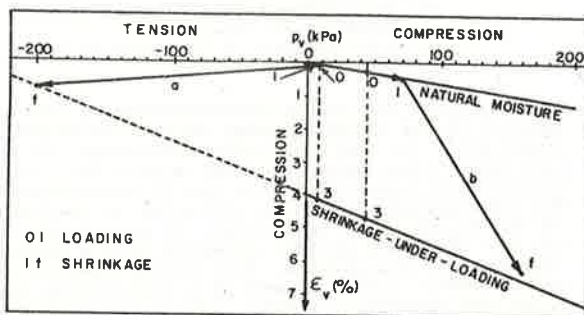which considers rupture after the tension stren
ght is reached has been prepared.

Figure 12: Stress-strain paths assuming shrinka
ge up to pF = 6 for elements a, adya
cent to upper part of pier, and b, -
at the base

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