SWELL AND COLLAPSE OF A PARTIALLY SATURATED EXPANSIVE CLAY

GONFLEMENT ET COMPORTEMENT D'UN PARTIELLEMENT SATRUES GONFLANT ARGILES

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SYNOPSIS

Swell tests, in which the full stress state was known, were performed on an expansive Black Cotton soil from Kenya. The results are used to explain differences observed using three standard procedures in which only the vertical stress is known. Further data is used to show how the initial condition of the sample can affect the soil behaviour during the test.

These findings are used to postulate a method of predicting the behaviour of an unsaturated soil by including quantitatively the effect of the microfabric.

INTRODUCTION

There have been many claims regarding the correct choice of laboratory test procedure for estimating the magnitude of swell that might occur in the field. It has long been recognised that use of the three 'standard' procedures leads to three different measured swell values in the laboratory. Several authors have presented data showing this, e.g. Brackley (1975) and Justo et al (1984).

In addition to the problem of procedural effects is the comparison with field data. It is often found that the laboratory test procedures overpredict the field vertical strain, but it must be noted that the changes in the stress and suction in the field are rarely known, although simple methods of suction measurement in the field are becoming available (Crilly et al 1991 and Gourley and Schreiner 1992). It is also true that the initial stress and suction are generally not known in the laboratory.

One of the fundamental requirements in engineering is the ability to relate deformation to stress change. Without this ability we can not hope to make reasonable predictions of engineering performance. In order to achieve this ability we must make measurements of both stress and deformation to formulate a relationship which can then be used in design or analysis.

STANDARD PROCEDURES

Three standard test procedures for predicting swell have been in use for many years and are summarised by Schreiner and Burland 1991). The types of test and original sources are:

Procedure 1 Swell followed by consolidation

This type of test, originally developed for studying collapse, was described by Jennings and Knight (1957). It has been widely used in expansive soil testing, and has been modified over the years, most recently by Jennings et al (1973). The changes in vertical total stress and void ratio are shown in Fig. 1a.

Procedure 2 Swell under constant vertical stress

This test type is described by Holtz and Gibbs (1956) and is illustrated in Fig. 1b.

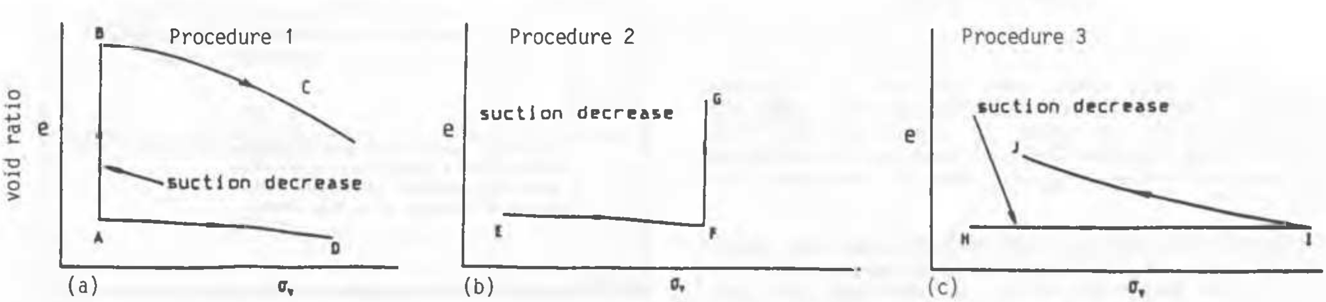

Fig. 1 Standard test procedures

Procedure 3 Swell pressure followed by rebound

This form of test was recommended by Sullivan & McClelland (1969) and is illustrated in Fig. 1c.

STRESS STATES

The possible radial stresses at the end of the swelling stages of these procedures have been discussed previously (Schreiner 1987).

After swelling one-dimensionally from A to B in Fig 1a, at constant applied stress σ_v, the radial effective stress σ_r' will tend towards the passive limit $K_p.\sigma_v'$ since the swelling process involves a reduction in stress. As loading from B to C takes place the stress ratio will tend towards K_o and the radial effective stress will approach $K_o. \sigma_{vc}'$.

In contrast, in Fig. 1b the sample is allowed to swell from F to G under σ_{vc} and the final effective stress will tend towards $K_p. \sigma_{vc}'$. Thus we see that the stress paths and final effective stress states for the two tests are very different.

TERMINOLOGY

It has been common to express the stresses in unsaturated soil laboratory tests in terms of the pore air pressure, the pore water pressure and the total stress. This has been improved for this paper by using atmospheric air pressure as the reference pore air pressure. Where, in a laboratory test, the pore air pressure is raised above atmospheric pressure by an amount Ua, the stresses used in this paper are:-

σ_{av} = the total vertical or axial stress at a pore air pressure equal to atmospheric pressure, $= (\sigma_v - U_a)$

σ_{ar} = the total radial stress at a pore air pressure equal to atmospheric pressure $= (\sigma_r - U_a)$

U_{aw} = the pore water pressure at a pore air pressure equal to atmospheric pressure. It is equal in magnitude, but of opposite sign, to the suction $= (U_w - U_a)$

$p_a = (\sigma_{av} + 2\sigma_{ar})/3$

These stresses are consistent with the total stresses and pore water pressure that would exist in the field. The total stresses, σ_{av} and σ_{ar}, are referred to as the applied stresses, as these are the stresses that would be applied to or by the soil in the field and in the test apparatus. It is equivalent to expressing saturated soil test data without including the elevated pore water pressure in any of the stresses.

APPARATUS

The standard swell tests are performed in standard oedometers. These permit determination of σ_v and the void ratio only. In order to understand the test procedures and the resulting soil behaviour properly we must know all of σ_{av}, σ_{ar}, U_{aw} and e throughout the tests.

The oedometer used for the tests described in this paper is shown in Fig. 2. It has a fine pored ceramic plate in the base to permit the use of an elevated pore air pressure of up to 1500 kPa and it has a load cell mounted in the oedometer wall to measure the radial applied stress (Schreiner 1988, Schreiner and Burland 1987, 1991). Direct measurement of the radial applied stress, σ_{ar}, is achieved by applying the elevated pore air pressure within the sample and within the load cell chamber.

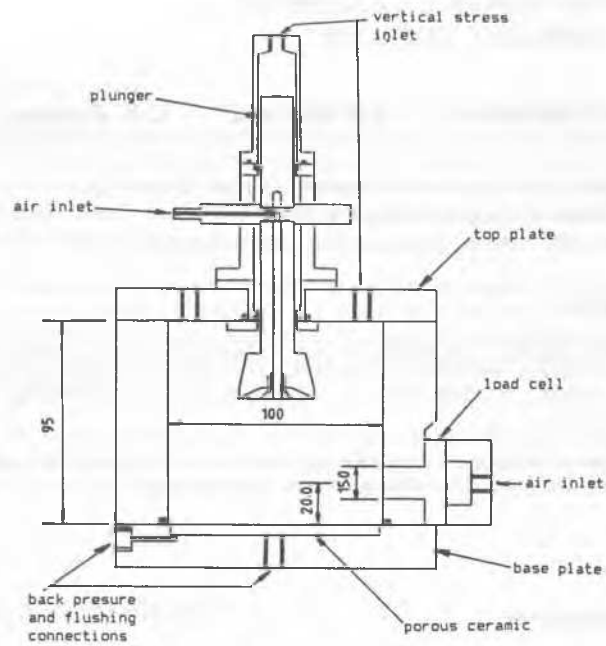

Fig. 2 Oedometer details

SOIL TYPE

The soil used in these tests is a highly expansive Black Cotton soil from the Athi River plains at km 68 on the Nairobi-Mombasa road. Soil properties are listed in Table 1 and the soil profile is shown in Fig. 3.

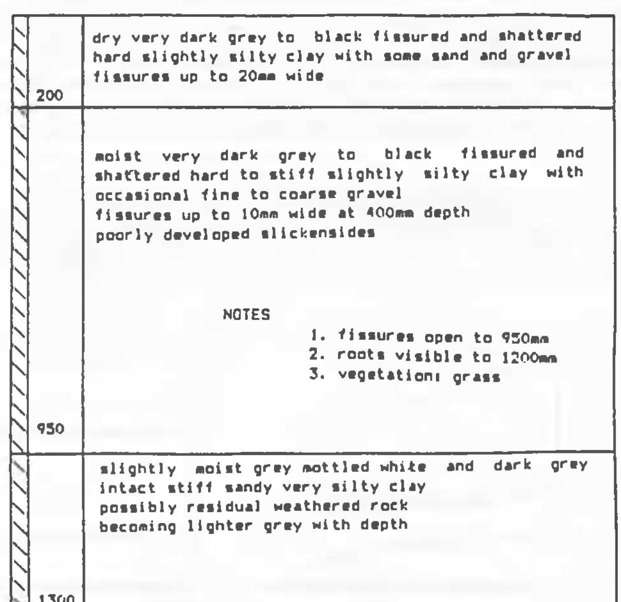

Fig. 3 Trial pit profile, km 68, Nairobi-Mombasa Road Kenya

TABLE 1: Soil Properties

W_L	W_P	W_S	%-.425 mm	%-.002 mm
whole soil values				
118	48	10	94.5	67

BS Max 2.5 kg kN/m^3	W_{opt} %	% salt by dry mass
11.8	38	0.52

The bulk sample was air dried to 6% moisture content and then pulverised. Particles larger than 2 mm were removed. Distilled water was added to obtain the required moisture content, which was checked before each test.

Compaction was by static loading in the stress path oedometer under 1 Mpa vertical stress.

EXPERIMENTAL PROGRAMME

The results of five tests are presented in this paper. Tests 1 and 3 followed procedures 1 and 3 respectively, whilst tests 2, 4 and 5 all followed procedure 2. The test procedures differed from the standard procedures described above only in that the swelling stages were performed by raising the pore water pressure, U_{aw}, (which was initially at a high negative value) in one increment per stage such that the pore water pressure was known.

The axial applied stress, σ_{av}, was controlled for all tests using mercury pots in which the elevated pore air pressure also acted in the top mercury pot so that changes in the pore air pressure did not affect the axial applied stress.

The axial displacement was controlled, for procedure 3, to within .005 mm on a 36 mm thick sample by manually adjusting the mercury pot system. This is equivalent to 0.01% strain.

All tests were designed to permit comparison of volume change at an arbitrarily chosen vertical effective stress of 50 kPa after swelling to zero suction. In addition the stress paths resulting from each of the procedures can be compared.

RESULTS

Test results are summarised in Table 2. Fig. 4 shows the results of tests 1, 2 and 3, permitting comparison of procedures 1, 2 and 3. Fig. 5 shows the results of tests 2, 4 and 5 all of which followed procedure 2.

These figures show the void ratio, e, the radial applied stress, σ_{ar} and the axial applied stress σ_{av} all plotted against the total suction. The use of total suction in preference to the pore water pressure or matric suction is explained in Schreiner and Burland (1991).

The values of U_{aw} given in Table 2 correspond to the matric suction. Measurements of total suction were made separately using the calibrated filter paper technique on a series of samples of the same soil compacted in the same way as the samples used in the swell tests. Total suction values for the samples in the swell tests have been derived from the relation between total suction and moisture content and are 3 MPa for Test 5, 4 MPa for Tests 1, 2 and 3 and 9.8 Mpa for Test 4.

A comparison was made by Schreiner and Burland (1991) of the results obtained from the three standard procedures which are shown in Fig. 4. Differences in void ratio and percent swell at 50 kPa were attributed to:-

(i) the stress paths differing from test to test
(ii) the mean normal stresses at equal values of suction differing from test to test
(iii) though initially nearly identical, the microfabrics of each sample which were altered in different ways during the tests.

Turning to Fig. 5, three test results are shown for three samples compacted at different moisture contents resulting in different void ratios, suctions and microfabrics.

Although all three followed the same sequence of application of the vertical applied stress followed by decrease of the suction to zero in stages, the results are significantly different. Only Test 5 follows the form of stress and volume change that would be expected from a similar test on a saturated sample, i.e. that the radial total stress would increase as the pore water pressure increases. In both of the other tests this type of relationship does not hold, with the radial stress first increasing and then decreasing, without approaching

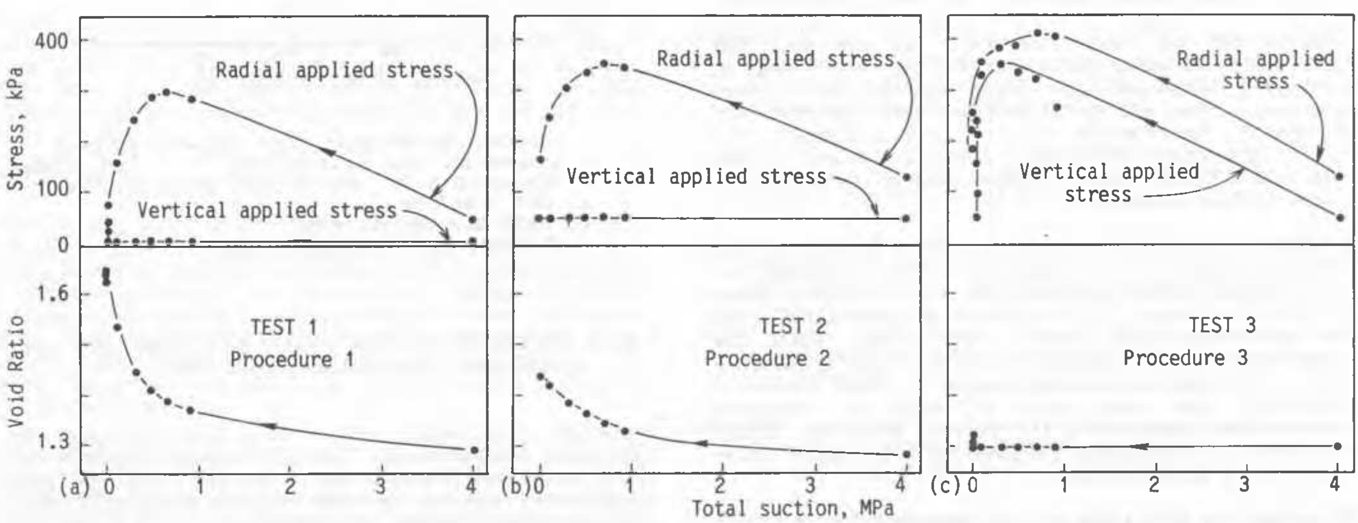

Fig. 4 Results of Tests 1, 2, & 3

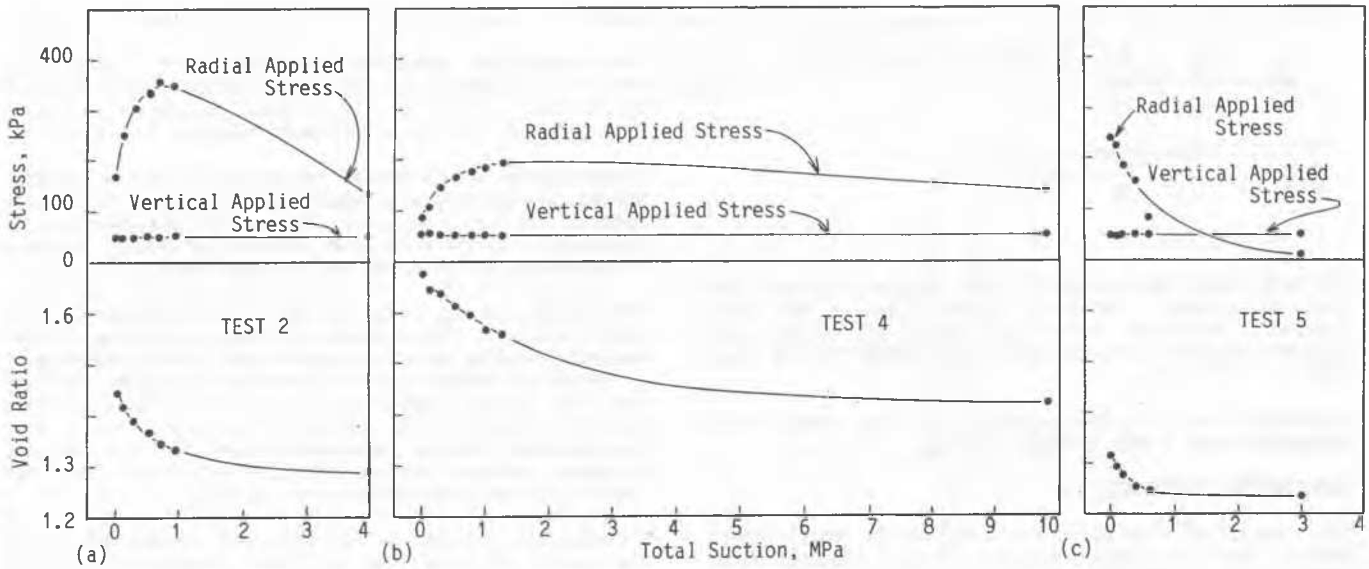

Fig. 5 Results of Tests 2, 4 & 5

the limiting conditions discussed above. Once again it is argued that this is due to the different microfabrics present in each sample which are due to the different moisture contents during compaction. Changes in the microfabrics during the tests are of the collapse type and occur at the same time as swelling takes place due to absorption of water by the clay minerals.

DISCUSSION

This type of behaviour which conflicts with effective stress concepts has been noted for the collapse settlement problem, e.g. Burland (1965). If we now accept that the changes in microfabric are an integral part of the behaviour of unsaturated soils then we need to include microfabric in our soil model. It is quite possible, even probable, that the 'chi' model (Bishop 1959) failed because it was tested on data from compacted samples in which the microfabric varied from soil to soil and from one set of initial conditions to another. Consider, for example, Fig. 6 from Jennings and Burland (1962). For any one value of Sr for one soil the microfabric is unique and different from that formed at any other value of Sr. Those samples which were compacted driest and which therefore have clustered and aggregated microfabrics will be more susceptible to fabric alteration or collapse than the samples of the same soil with microfabrics established by compaction at higher moisture contents.

Consider now the data for test 1, plotted in Fig. 7. The starting point, A, is shown at 4 MPa in terms of total suction and the end point, B, at 1.5 kPa, in terms of effective stress. CD is a saturated compression line for a reconstituted sample. Point A lies above the virgin compression line where it is not normally possible for a saturated and unbonded sample to lie. Point E represents the void ratio at which a saturated reconstituted sample would lie under an effective stress of 4 MPa. EF represents the swelling that would occur from E for a saturated soil.

If we take the void ratio at E to represent stable voids in the sense that they will not be alterable in the way that would lead to collapse, then we can consider AE to

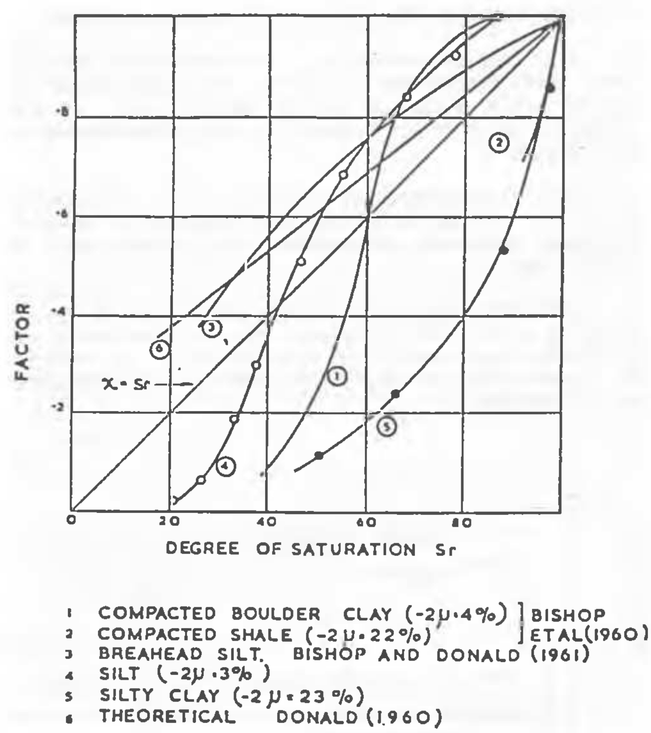

Fig. 6 Variation of "chi" with soil type and test conditions (Jennings & Burland 1962)

represent the alterable voids. If no alteration were to take place during the test then the ratio of stable voids to alterable voids would remain constant. We can thus calculate by simple proportion from the stable void ratio at F what the combined void ratio would be at G, and it falls close to B. This suggests that there has been little alteration of the microfabric during this test.

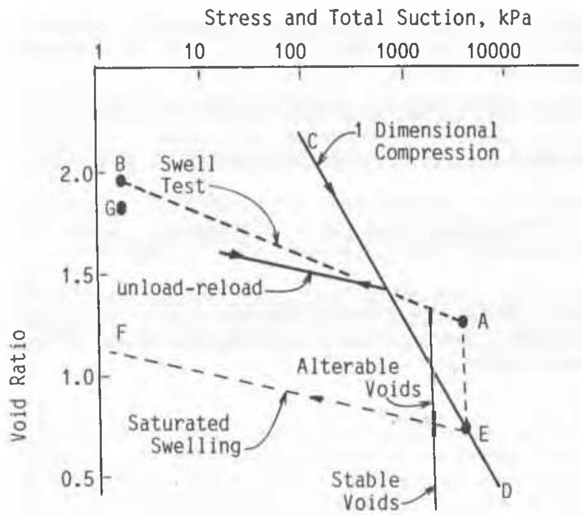

Fig. 7 Saturated and Unsaturated Swelling

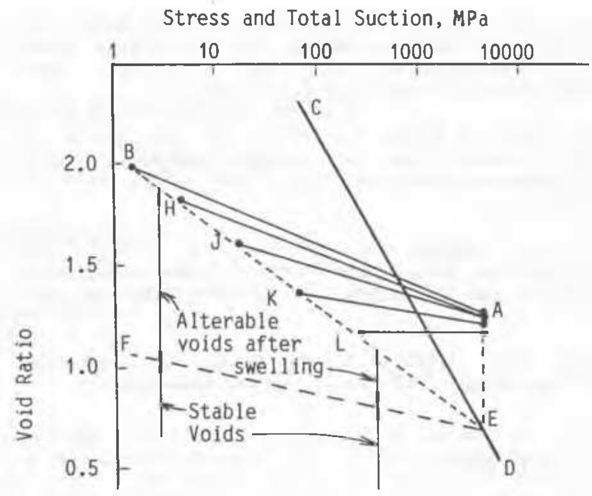

Fig. 8 Fabric changes during swelling

Fig. 8 shows the same data with a further 4 tests, which followed procedure 2 under various values of σ_{av}, added. These started at the same moisture content and suction and were compacted under the same static stress as Test 1. The final stresses were as shown in Fig. 8 at H, J, K and L. Projection of the line BHJKL through the end points of these tests conveniently meets the virgin compression line at E.

The data needed to determine how the soil will behave are thus:-

(i) The compression data for a reconstituted sample
(ii) An unload - reload loop for the reconstituted sample
(iii) The void ratio of the sample under investigation
(iv) The value of the total suction present in the sample under investigation.

Point E can be determined, giving the stable void ratio. Line AB can then be constructed to represent a constant ratio of alterable to stable voids. Joining B and E gives the line on which the end points of swell under load tests will fall. The void ratio at the end of any such test is found simply from the point on BE at the effective stress which will prevail at the end of the test.

CONCLUSIONS

For too many years we have tried to make unsaturated soil mechanics fit into the effective stress mould. It is more correct to consider saturated soils to represent one simplified section of the greater soil mechanics which includes residual soil, unsaturated soil and saturated soil. Rather than try to make unsaturated soil fit, we should seek a model for the greater soil mechanics, of which the effective stress model for saturated soils is but a simplification.

This study has shown that one of the components missing from, or simplified out of, the overall model is the microfabric of the soil. A means of including the effect of the microfabric in the model has been proposed. This is not expected to be the last word on unsaturated soil, but will hopefully lead to a more constructive approach to research into unsaturated soil. It should not be forgotten that the results presented in this paper are for compacted soils.

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TABLE 2 : Test Data

TEST 1

STAGE	1	2	3	4	5	6	7	8	9	10	11	12
σ_{av}	7	7	7	7	7	7	7	14	28	46	73	140
U_{aw}	-910	-910	-665	-470	-300	-100	0	0	0	0	0	0
σ_{ar}	50	289	298	284	245	153	80	72	81	79	91	168
q	-43	-282	-291	-277	-238	-146	-73	-58	-53	-33	-18	+28
p_a	36	193	201	192	165	104	56	53	63	68	85	121
e	1.293	1.372	1.388	1.414	1.444	1.533	1.649	1.645	1.637	1.627	1.608	1.543

TEST 2

σ_{av}	50	50	50	50	50	50	50
U_{aw}	-900	-900	-700	-500	-300	-100	0
σ_{ar}	131	349	356	334	305	250	168
q	-81	299	-306	-284	-255	-200	-118
p_a	104	249	254	239	220	183	129
e	1.285	1.331	1.345	1.365	1.387	1.417	1.438

TEST 3

σ_{av}	50	273	327	340	355	330	240	210	150	100	50
U_{aw}	-900	-900	-700	-500	-300	-100	0	0	0	0	0
σ_{ar}	128	405	416	390	387	360	253	258	251	219	187
q	-78	-132	-89	-50	-32	-30	-13	-48	-101	-119	-137
p_a	105	361	386	373	376	350	248	242	217	179	141
e	1.301	1.301	1.301	1.301	1.301	1.301	1.299	1.300	1.307	1.317	1.327

TEST 4

σ_{av}	50	50	50	50	50	50	50	50
U_{aw}	-1260	-1260	-1010	-750	-515	-310	-120	0
σ_{ar}	135	191	181	174	163	143	106	75
q	-65	-141	-131	-124	-113	-93	-66	-25
p_a	93	144	137	133	125	112	94	67
e	1.421	1.553	1.569	1.591	1.610	1.636	1.644	1.676

TEST 5

σ_{av}	50	50	50	50	50	50
U_{aw}	-585	-585	-400	-200	-100	0
σ_{ar}	10	87	154	185	221	237
q	40	-37	-104	-135	-171	-187
p_a	23	75	119	140	164	175
e	1.239	1.245	1.255	1.276	1.295	1.311