Discussion of Session 3b

Dr. P. S. Ringrose,
Dames & Moore.

I'd like to show some slides that might 'beef up' the argument that we've presented in our paper. We have tried to draw together an impression, from our field investigations, of what was happening, in tectonic terms, during late Quaternary time in Scotland. [A selection of these slides are reproduced here.]

Plate 1 shows an aerial view of a fault which has marked surface expression over a 14km length and evidence for 160m of lateral displacement during the Quaternary. The manner in which this fault cuts drainage and topographical features is reminiscent of faults in the Western United States, but it is in fact the Kinloch Hourn fault, in the north-west Highlands of Scotland. Plate 2 shows how the distinct, notch-like surface expression of this fault continues straight across major mountainsides. It is an ancient, Caledonian basement fault, trending north-north-west; however, despite its age, reactivation during the Quaternary Period is evident in the systematic offset of drainage paths along its length (as described in our paper).

A selection of other slides (not reproduced here) show phenomena in the Glen Roy area, including the evidence for offset drainage, interruption of micro-topography, displacement of the lake shorelines and fresh, soft, fault-gouge material infilling fractures within the fault plane.

I'd like to conclude with a slide (Plate 3) which shows tentative, earthquake-magnitude reconstructions for the field examples discussed in the paper, based on empirical relationships between Magnitude and fault displacement, fault length and the extent of earthquake-induced liquefaction. We have some idea of the likely dates of the fault rupture events, based on relative stratigraphy and radio-carbon dating. These estimates, although crude, suggest several events of Magnitude 6 to 7 immediately after the most recent deglaciation (in the period 13,000 to 9,000 years BP) and, more recently, some events of around Magnitude 6.

Professor H. Sutherland, University of Glasgow, Chairman, Session 3b.

Thank you very much Dr. Ringrose. Before we go on to the next item for discussion, would anyone from the floor like to contribute to the discussion of tectonics and Dr. Ringrose's work?

Professor P. Fookes

I'd like Dr. Ringrose to tell us a little more about these movements: whether they're all jerky or all continuous or a bit of each.

Dr. P. Ringrose

The largest single movement that we can infer is at Glen Roy where the shorelines are offset (vertically) by up to two metres and where about half a metre of lateral displacement is also apparent. Arguments for displacement of stream morphology involve lateral displacements of tens of metres (160m at Kinloch Hourn), but this must represent cumulative displacement over some period during the Quaternary. So it is difficult to say, but I think that individual movements of at least half a metre must have occurred.

Dr. A. B. Hawkins,
Bristol University.

Can I ask Dr. Ringrose about event D on Fig.3, which was covering 2,000 years or so. This obviously concerns me, but I can appreciate the difficulty. Can you tie that down any more because, whereas there are various beach levels at that position it is difficult to see what would have caused such movement. I've always looked at Sissons' work in this area with a certain amount of concern, and I would just ask you to comment on this.
Reproduced from an Ordnance Survey aerial photograph with the permission of the Controller of Her Majesty's Stationery Office. Crown copyright reserved.

Plate 1
Dr. P. Ringrose

Yes, I would share some of that concern. (I was discussing it with Professor Peacock a couple of hours ago.) The reason that the spread of ages is so large (Event D, Fig.3) is that it concerns displacement of various buried shorelines reported by Sissons. There is no easy way of validating his data because they are buried features. So, event D is not our work but an inference from Sissons' work. I am not fully convinced about it, but I think there is enough evidence to suggest that a fault displacement did occur in the Upper Forth Valley, at about that time (9,000 to 7,000 years BP). Apart from this, all the evidence that I have found suggests that the larger postglacial displacements occurred earlier (13,000 to 9,000 years BP), as you might expect, with some smaller displacements in more recent times (the last few thousand years).

Professor H. Sutherland

I found reading your paper and listening to you most interesting when you mentioned the parallel roads of Glen Roy. Only yesterday I was reading of James Thomson who was Lord Kelvin's brother, and who succeeded Rankine as Professor of Civil Engineering at Glasgow, and one of his many papers on a variety of subjects, including the parallel roads of Glen Roy.

Dr. M.A. Paul
Heriot-Watt University

Mr. Nash has given an excellent introduction to Bothkennar, and to the problems he faced in identifying an area suitable for the SERC Soft Clay Research Site. I would now like to outline the work currently being undertaken at Heriot-Watt University to establish the detailed engineering geology at the site. The work is being carried out under an SERC grant held by myself and Dr. Little. I should pay tribute to the work of our research assistant Ms Beverley Wood, and the contribution of Professor Douglas Peacock.

We have the following objectives: to examine the chemistry and mineralogy of the soil solids, and the chemistry of the pore fluid; to study the sedimentary fabric by optical and scanning electron microscopy, and to use the results to erect an engineering geological model of the deposits, which can be related to their geotechnical properties. Finally, to explain this engineering geological model in the context of Late Quaternary environmental change, which, as you will know, has been well studied in the Forth estuary.

Let me consider these aspects one by one. X-ray diffraction studies have shown that the clay mineralogy is essentially similar throughout the top twenty metres of the deposit, i.e. down to the buried gravel layer. The assemblage contains rock flour, principally quartz, felspar and various micas. The clay minerals are mainly kaolinites, with some illites. There seems to be little smectite. This assemblage is not surprising when one considers the likely source materials.

We have studied the distribution of sodium, potassium, magnesium and calcium as extractable cations and chloride and sulphate anions. These have been obtained by chemical extraction rather than by direct sampling of pore water. Our results so far suggest that the sodium chlorids concentration is similar to that of the adjacent Forth estuary, averaged over the tidal cycle. This is about one half the concentration in standard sea water. The other cations show a more complex pattern of variation, and at this stage we are not sure if this is due to genuinely different concentrations, or is due to differential retention by the clay minerals during extraction. We have, however, noted that both potassium and calcium seem more abundant than expected, and this may be due to some input from agrochemicals.

The fabric studies have been commenced only recently. They indicate that a facies approach to the sediment will be helpful, but I
do not wish to say more at this stage. We have carried out an extensive programme of geophysical measurements: these show that the seismic (p-wave) velocity is close to that of seawater near the surface, and rises slowly with depth. There is evidence of limited anisotropy. This is similar to my findings from similar sediments offshore. Electrical impedance is generally low, and primarily reflects the ionic content. The bulk magnetic susceptibility is also low, which is normal for clay sediments, except in the laminated parts of the core, where it is raised, possibly as a result of an increased magnetite content.

I will comment only briefly on the depositional environment. The regional context is well known from the work of Sissons and his associates. This work has generated a sea level curve for the Forth Estuary, and based on this we have prepared our own curve which we believe to be more specific to Bothkennar. From this curve we can obtain an indication of the water depths in which the sediments were deposited. Our main conclusion is that the water was probably several metres deep throughout the depositional period, and may at times have been in excess of ten metres. These depths are somewhat greater than is commonly supposed. Comparison with published descriptions of the deposits elsewhere in the upper Forth valley suggest that the bulk of the succession above the buried gravel is the equivalent of the Claret Beds.

My concluding remark is that the deposits are remarkably uniform down to about twenty metres depth, and that in consequence this has been a good choice of site from the viewpoint of the SERC, for which we should thank David Nash and his colleagues.

Professor H. Sutherland

Professor J.H. Atkinson, City University.

I would like to consider cohesion, the soil property that was touched on in an earlier session. Firstly however it is important to draw a very clear distinction between undrained shear strength, which is often called cohesion, and proper effective stress cohesive strength.

The undrained shear strength is \( c_u \); nowadays it is usually denoted \( s_u \) which is rather better. It depends only on the specific volume or voids ratio and is independent of the total stress. This means that \( \phi_u \) must be zero and if \( \phi_u \) is found to be non-zero it means that the samples were not tested at constant volume (i.e. they were not undrained). Any soil, coarse grained or fine grained, will have an undrained shear strength provided that samples can be tested without change of volume. In an unconfined compression test in which radial total stresses are zero the undrained strength arises from positive effective stresses due to pore water suctions which are the origin of the undrained strength of a sandcastle.

Turning now to effective stress cohesion \( c' \) this would normally be measured in a set of three triaxial compression tests in which the samples were consolidated to different effective stresses probably in the range 100 to 400 kPa. The tests might be drained or undrained with pore pressure measurements but the Mohr's circles at failure would be plotted in effective stresses. Fig.1 shows typical Mohr's circles obtained from tests of this kind.

The conventional interpretation of these data would be to draw a linear Mohr-Coulomb failure envelope given by

\[
\tau' = c' + c_n \tan \phi'
\]

where the cohesion \( c' \) is the intercept of the envelope on the shear stress axis. We should notice that the cohesion has no physical meaning and it is only the intersect of the linear envelope on
the shear stress axis but we often go much further and say that the cohesion $c'$ is the strength available at zero effective stress.

In order to examine the nature of effective stress cohesion we should conduct triaxial tests in which samples fail at very small effective stresses. These tests are very difficult to do but we have managed to measure the strength of samples of plastic clays at effective normal stresses in the range 2 to 20 kPa. The results of these tests, shown in Fig.2, demonstrate that the strengths at small effective stresses are considerably smaller than those given by a linear Mohr-Coulomb failure envelope, shown by the broken line, obtained from a conventional sequence of tests carried out at higher effective stresses.

These results were obtained from tests on London Clay: tests on several other plastic clays from South East England gave similar results. The strengths at small effective stresses may be approximated by a curved failure envelope. It is not clear from these data whether there is a cohesion intercept but, if there is, it represents the true strength at zero effective stress. The failure envelope could be represented by an equation of the form

$$\tau' = c' + Ao^{1b}$$

I am sure that many of us are aware of curved failure envelopes; what we may not have appreciated is the very dramatic reduction in strength below the Mohr-Coulomb line at low effective stress.

For simple or direct shear tests there is another factor to be considered which arises due to the difficulty in interpreting the results of shear tests in which only the shear and normal stress on one plane are measured. The point is discussed in detail by Atkinson, Lau and Powell (1990) who show that the conventional interpretation of shear tests will lead to a false value of cohesion, often close to $c' = 20$ kPa, even in clearly cohesionless soils. This false cohesion is due to misinterpretation of the test data and it is an artefact of the test.

If we define cohesion as the strength at zero effective stress it can arise either from relatively weak interparticle forces or from cementing where some other material is deposited at the grain contacts. Assuming that cementing will be destroyed by severe straining we may examine cohesion by conducting tests on remoulded or, better still, reconstituted samples. But to do so we should examine samples in which the effective stresses are zero. This can be done by placing a saturated sample into a beaker of water and observing its behaviour. In a coarse grained soil when there is no cohesion the sample will simply form a cone at the angle or repose. If there is some positive true cohesion the sample will retain its original shape even after considerable time when both pore pressures and effective stresses are essentially zero. But if the water surrounding the sample becomes dirty the implication is that the interparticle forces are repulsive so the true cohesion must be negative.

Tests of this kind have recently been reported by Atkinson, Charles and Mbache (1990). These demonstrate that reconstituted soils may exhibit positive, zero or negative cohesion and, for a given soil, the cohesion characteristics are governed by the chemistry of the pore water. Where the cohesion is non-zero it is likely to be relatively small; too small to measure reliably in routine field and laboratory tests and, for engineering purposes, most reconstituted soils can be assumed to have zero strength at zero effective stress. Many intact soils are, however, slightly cemented and this gives rise to a cohesive strength at zero effective stress but this strength is only available for relatively small straining before the cement bonding breaks.
Fig 1 Linear Mohr-Coulomb Failure Envelope for Conventional Triaxial Tests.

Fig 2 Mohr’s Circles for Failure in Triaxial Tests at Low Effective Stresses
Atkinson, J.H., Charles, J.A. and Mhach, H.K. (1990). Examination of Erosion and Resistance of Clays in Embankment Dams. Quarterly Journal of Engineering Geology. Vol.23, pp 103-108.

Atkinson, J.H., Lau, W.J.W. and Powell, J.J.M. (1990). Measurement of Soil Strength in Simple Shear Tests. Canadian Geotechnical Journal. (in the press)

Professor J. Hutchinson, Imperial College.

Could I ask Mr. Nash if he has managed to make measurements of the vertical pore pressures at the SERC soft clay site, in particular to see if there's any upward flow of fresh water from the rock beneath the clay which could leach ions from the lower layers?

Dr. D. Nash, Bristol University.

The answer is yes, a number of piezometers were put in at the site at the first stage of investigation, and these showed that essentially the groundwater conditions are static, there's a little bit of seasonal variation, up to half a metre or so. But the underlying gravels, there's a small tidal influence we believe, although I personally haven't carried out comprehensive measurements on the pore pressure fluctuations on them, but essentially the groundwater conditions are hydrostatic and I think there's little upward seepage causing leaching.

Dr. M. Barton, Southampton University.

I wanted to come back to Professor Atkinson's point about his experiments to determine the amount of cohesion. In the case of one of the sand samples, the procedure of dunking it in a bucket it seems to me is not going to work as a test for cohesion because if you put a sand in a bucket then you will induce slaking which is the build-up of a pore air pressure as the water is pulled into the sample and then the sample will be subject to collapse due to these forces thus inducing something else as a result of the test.

Professor J. Atkinson

Thank you very much. If I didn't say so in the contribution I ought to have done. Those samples which are being put into the water must be saturated, in this way this test differs from the older crumb test. In the old crumb test the sample is not saturated and there is no guarantee that the pore pressures will equalise and the effective stresses become zero. In this test it is essential that the sample is saturated before it is immersed.

Dr. R. Chandler, Imperial College.

Referring to Professor Atkinson's contribution one ought to be rather careful about using the term cohesive soils, and it's probably much better therefore to refer to them as clay soils.

Professor J. Atkinson

It's certainly inappropriate, I believe, to distinguish soils as frictional and cohesive. I personally would tend to call all soils, soils, and not distinguish them any further other than to think about the grain size which controls the permeability or the equalisation of pore pressures. So it's probably better to think of well draining or free draining soils as opposed to poorly draining soils. But I think it's wrong, incorrect, unhelpful, to think of clays as a whole family and sands and gravels as a quite different family.

Dr. R. Jones, University of Nottingham.

Continuing with the terminology from John Atkinson's last contribution. Those of us who've been concerned with roads and with free draining materials are not too happy about the term free draining as a material. Free draining is a hydraulic condition.

Professor J. Atkinson

Yes, I'm quite happy to stand corrected.
Mr. P. Stevens,  
Engineering Geology Ltd.

Is there any information on rate of sedimentation at Bothkennar?

Professor H. Sutherland

Dr. Hawkins would you like to reply to that?

Dr. A. B. Hawkins,  
Bristol University.

Effectively you've got 20, in one place 22, metres of sediment deposited within a space of about 2,000 years. It wasn't a totally constant rise. We have to bear in mind that for a long time, back in 1940, following work in 1938, it was suggested in southern England that sea level rose at one point 5 metres per 100 year during part of the Flandrian rise, and so the fact that we're talking about a metre per 100 years is incidental, it was not even, but it was almost all deposited by certainly 6,500 years ago.

Mr. I. D. Clarke, Electrowatt Engineering Services Ltd

I should like to show a few photographs and two slides which I think will help the reading of the paper on the San Jacinto project in Bolivia. The first shows the layout of the project. You can see the Rio Tolomosa flowing through its new gorge; its original course was further to the north. A tributary of the Tolomosa, the Quebrada Seca, flows southwards to join the river at the entrance to the gorge, and between the Quebrada Seca and the wide valley for the Rio Guadalquivir there is this unusual, pronounced ridge known as the Tablada, which is the subject of the paper.

The next picture is an aerial photograph, on which the crest of the ridge is indicated by a blue broken line. You can also see the Devonian ridge of sandstones and slates through which the river has formed its present course. This is the site of the arch dam, indicated in blue. You can see the Quebrada Seca meandering down to join the Tolomosa at the entrance to the gorge. The solid blue lines indicate the positions of three deep cuts which were excavated as preconstruction diversion works to release the area between the crest of the ridge and the bed of the Quebrada Seca for the construction of the proposed protection dam against the Tablada. These were constructed in 1980 and in places are as much as 10 to 15 metres deep.

The next photograph is taken from the west looking towards the ridge from the area of the future reservoir. You can see the gorge through the Devonian ridge in which the arch dam is being constructed, and on the left the upstream face of the Tablada ridge.

The first slide shows the eastern face of the Tablada Ridge. One can see the upper level of the ridge, with the gravel cap, overlying the sand, silt and silty clay deposits, in various formations which are dispersive in nature. These very unusual erosion features are associated with 'bad land' scenery and dispersive clays, and seen along the eastern face of this ridge over a considerable distance. The arch dam site is off the right, and the future reservoir will be impounded against the opposite side of this ridge.

The second slide taken from the crest of the ridge, looking down into the valley of the Rio Guadalquivir; the city of Tarija is off to the left, and the river is flowing from left to right. The deep eroded gullies in this ridge are clearly seen, and against the opposite face of this ridge the reservoir would be impounded to within about 50 metres of the crest of the ridge in certain places.

The next photograph is a view upstream through the gorge in which the arch dam is now constructed; you can just see the concrete cofferdam at the upstream end of the gorge. The following photograph is taken high on this right abutment looking northwards; it is possible to see the crest of the Tablada Ridge, and you can also
discern the Quebrada Seca. Also visible are the three deep cuts which were excavated in 1980, so that the proposed Tablada dam could be constructed between the Quebrada and the ridge. The next photograph shows the stable face of one of these cuts, about 10 metres high, and was taken about three years after excavation.

The final photograph shows two samples from the pinhole tests which were done in Zurich in 1983 to examine the dispersive nature of material taken from the Tablada ridge. In this sample, the original 1mm diameter hole increased to a diameter of 4.5mm in 10 minutes under a head of about 5 cm of water.

Professor H. Sutherland

In bringing the Session to a close, I, as Chairman, wish to comment on some of the topics suggested for discussion by the General Reporter. These topics are a comparison between soft clays in the U.K. and elsewhere, and the loads than can be safely applied to these soils.

Gregory and Bell in their paper to this Conference on the Geotechnical Properties of Quaternary Deposits in the Belfast Area presented the results of quick undrained triaxial tests they carried out on samples of clay cut at various inclinations to the vertical, and Fig.7 of their paper shows the anisotropic nature of these clays. The authors go on to state that the use of undrained shear strengths in slope stability analysis can lead to unreliable results and they advocate the use of effective stress shear strength results to obtain more accurate estimates of stability.

I should like to describe my experience of the sites in Canada on which I was directly involved with both total and effective stress methods of analyses of slope stability.

The first was at Steep Rock Tron Mine in North Western Ontario where I spent the summer of 1948, at a time when effective stress methods of analysis were in their infancy. Over 230 million m of soil were moved from the bed of Steep Rock before the iron ore beneath the lake bed was accessible for open pit mining. The natural bed of the lake consisted of varved clays of high sensitivity with a liquidity index between 1 and 1.5. The lake was dammed and drained and the clay covering the ore bodies was removed by an extensive dredging operation. The surrounding slopes had to be finished off in a stable condition and the most economical side slopes had to be determined as there was no profit in excavating clay unnecessarily, the iron ore being the target material. The slopes were trimmed by high pressure water jets and in order to assess the shear strength of the clay, vertical faces were cut in the clay by the jets until failure occurred. The height to which a vertical face in clay can stand unsupported in a clay can be expressed in terms of the undrained shear strength and the unit weight, and the shear strength of the mass of clay in situ was derived from a number of these excavations to failure. A simple unconfined compression apparatus was constructed in the mine workshop to obtain the undrained shear strength from field samples which were also investigated for anisotropy as in the Gregory and Bell tests. From these simple laboratory tests and from the field failure tests, a value of undrained shear strength was obtained which was used in the design of the slopes. A description of the complete geotechnical investigations and operations at Steep Rock has been given by R.F. Legget (1).

The second case which I wish to mention is in the Winnipeg area of Canada where the Lake Agassiz clays are similar to those in the Forth Valley. The well known failure of the Transcona Elevator took place in these clays, and, in a subsequent investigation, the back analysis of the shear strength results allowed the theory of the bearing capacity of clays to be verified by Peck and Bryant (2) in a similar manner to the investigation by Skempton (3) of the bearing capacity of the Forth
Valley clays. My particular involvement was with the stability of river bank slopes in these clays where there had been a history of instability.

The stability of slopes in the Winnipeg area had been studied by previous investigators, and in a number of cases stability analyses using laboratory determined shear strength data had indicated factors of safety considerably greater than unity for slopes which had actually failed. A particularly detailed study had been made of trial slopes excavated to failure during the construction of the Floodway around the city of Winnipeg. Porewater pressure measurements were made in the field and the factors of safety of the failed slopes were calculated using both total and effective stress methods of analysis. Circular arc failure surfaces were assumed in the analyses and the shear strength parameters used were obtained from laboratory tests on vertically oriented samples. The factors of safety obtained from the different methods ranged from 1.28 to 2.08 for slopes which had actually failed.

In an attempt to resolve the discrepancies between the results of slope stability computations and field performance, a study of the Winnipeg clays was undertaken at the University of Glasgow as reported by Freeman (4). Undisturbed block samples were sent from Winnipeg to the University. Four types of clay were identified as was a layered structure in the clays. This suggested possible strength anisotropy with the shear strength along directions of layers being lower than in other directions and so leading to a slip surface following these directions. Detailed laboratory investigations include the measurement of the effective stress shear strength parameters both along and across the layers and comparison was then made of the results of stability analyses for circular and non-circular failure surfaces in which the effects of anisotropy were also considered.

In a typical case, there was a reduction of 0.42 in the calculated factor of safety from that for a conventional circular arc analysis using an isotropic shear strength to that for a non-circular failure surface using the shear strength along the layers. The reduced factor of safety so obtained went a long way towards explaining why a factor of safety greater than unity had been obtained in the previous analyses of slopes. Details of the investigation are given in the paper by Freeman and Sutherland (5). This experience in Winnipeg should be borne in mind when the stability of slopes in layered soils are being analysed.

With regard to the loads that can be safely applied to soft clays, I shall restrict my comments to the loading that can be safely applied to soft clays which have a harder desiccated surface layer. The Winnipeg and Forth Valley clays come into this category. Local experience often shows that shallow relatively narrow isolated footings founded in the hard crust can be satisfactorily loaded to a certain intensity. No detrimental settlements were experienced since excessive stresses were not transmitted to the underlying soft clay. Problems can arise when that same intensity of loading is applied to larger foundations and it was this situation that led to the failure of the Transcona Elevator founded on the Lake Aggasiz clays. Small plate loading tests were satisfactorily carried out on the hard crust and the same intensity of loading was applied to the large raft foundation for the Elevator. Large shear stresses were transmitted to the soft clay at depth, the ultimate bearing capacity of the clay was exceeded, and the foundations failed by soil rupture. Details of the failure and the remedial work are given in the paper by Peck and Bryant (2). The same sort of situation has been observed in foundations on the Forth Valley clays and has led to unacceptable settlements. A further cause of detrimental differential settlements in the Forth Valley clays has been the placing of relatively small heights...
of fill material on these clays over relatively large areas. The underlying soft clay has been overstressed, with detrimental effects on overlying foundations.

The Terzaghi theory of consolidation shows that in general terms the settlement of a foundation depends on the net load transmitted to the soil. This concept of the "floating foundation" can be used to limit the settlement of structures founded on soft clays. This solution was adopted in the Forth Valley clays in the design of cellular foundations for the power station and other plant installations at the Grangemouth Refinery. The subsoil conditions consisted of soft alluvial silts and clays extending to depths varying from 25m to 75m. As far as possible, the cellular rafts were designed so that the weight of the displaced soil balanced the combined weight of the rafts and their superstructures. The rafts were sunk to the required foundation depth by grabbing as described by Pike and Saurin (6). The settlements which were recorded on five of the structures founded on the cellular rafts varied from 13mm to 5mm over a period of just under two years after construction.

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