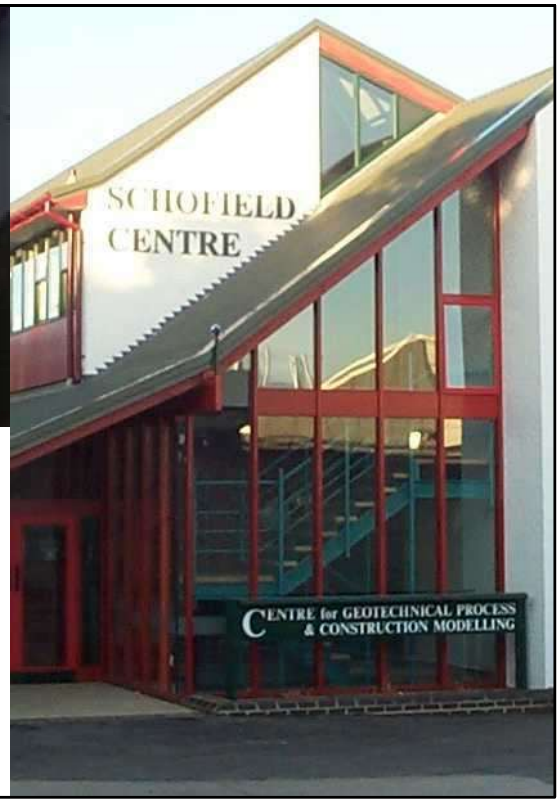
1st Schofield Lecture

Centrifuge modelling: expecting the unexpected

Malcolm Bolton



I am delighted to have been chosen as the first Schofield lecturer. Andrew Schofield recruited me in Manchester in 1969 as a student both of soil mechanics and of centrifuge testing. I am immensely grateful for that, and for his continuing interest and friendship. One goal today is to explain why I can recommend a similar vocation to the next generation.

Scope

- Models, prototypes and reality
- Some achievements of centrifuge modelling
  - Earth retention: retaining walls and slopes
  - Piled foundations
- "Into the eye of the storm"
  - Grain size effects and localisation
  - Fluid flow and granular-fluid interactions
- Expecting the unexpected
  - Science, technology and practice

My talk focuses on physical models, goes on to discuss imaginary scaled-up prototypes, and touches from time to time with reality – which is different of course.

I have picked 5 contrasting series of centrifuge tests performed by my graduate students – old and new. Applications include retaining walls, slopes and foundations.

And the behaviour seen in the models will raise again the two issues most frequently raised as objections to centrifuge testing – distortions due to grain size scaling and fluid-granular interactions.

The theme of the lecture is the occurrence of unexpected events, which I want to put in the context of the profession, with its three contrasting aspects - of science, technology and practice.

Models, prototypes and reality

- If a real soil body at 1g has a characteristic length h and density ρ, then a sample of identical shape with length scaled h/N, tested at Ng, should reproduce full scale stresses (e.g. $h\rho g$).
- If an actuator then imposes changes $\Delta\sigma$ similar to reality, or displacements $\Delta h/N$ in reduced times t/N, the mechanisms observed in the model will be representative of some full-scale prototype.
- But which? Should grains scale? What about fluid flow? We must analyse the mechanisms!

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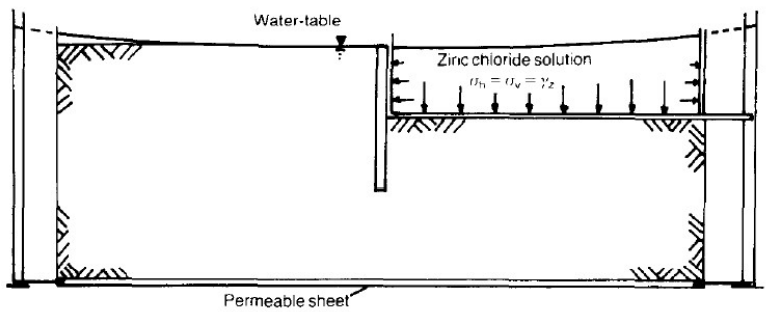
The USP of centrifuge modellers is "small scale models with full-scale stresses responding to representative actions": 1/N scale models flying at Ng reproduce stresses $h\rho g$. A key difficulty is the question of time. The act of centrifuging necessarily enhances accelerations by factor N, but displacements must be scaled down by factor N so the duration of an event should also be reduced by factor N. However, diffusive processes such as consolidation will have their own timescale of N^2, arising from pressure gradients that are scaled up by N and travel distances that are reduced by N. These issues are well-known to centrifuge modellers. I will look later at less familiar fluid interactions.

Throughout, I will be extolling the virtues of "mechanisms" that can be discerned from the behaviour of such models. A mechanism captures one aspect of observed behaviour. Having been applied successfully at model scale it can then be used at full scale. However. a real landform or foundation will only behave in a similar way if its material properties are sufficiently similar, and if the actions upon it are also similar. Hear "similar" is often discussed by invoking "scaling laws" to define an imaginary "prototype" that can be compared with "reality". We will need examples to chew over.

Diaphragm walls in stiff clay: Powrie

Bolton & Powrie (1987) The collapse of diaphragm walls retaining clay, Geotechnique, 37, No.3, 335-353

Bolton & Powrie (1988) Behaviour of diaphragm walls in clay prior to collapse, Geotechnique, 38, No.2, 167-189



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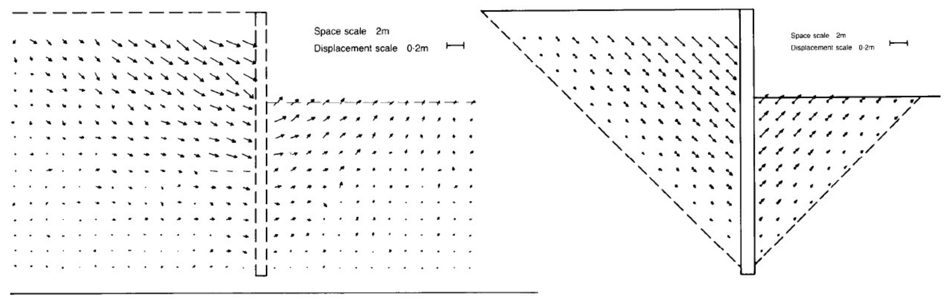
The first example concerns in situ walls in clay, first studied on the Cambridge centrifuge by William Powrie, PhD in 1986, now Dean of the Faculty of Engineering and the Environment at the University of Southampton. Two Geotechnique papers are referenced.

Aluminium model walls were trenched into stiff kaolin at 1g, a rubber bag containing heavy fluid replaced the soil in front, transducers were installed, and the models brought into equilibrium with a high water table prior to draining out the heavy fluid to simulate an excavation.

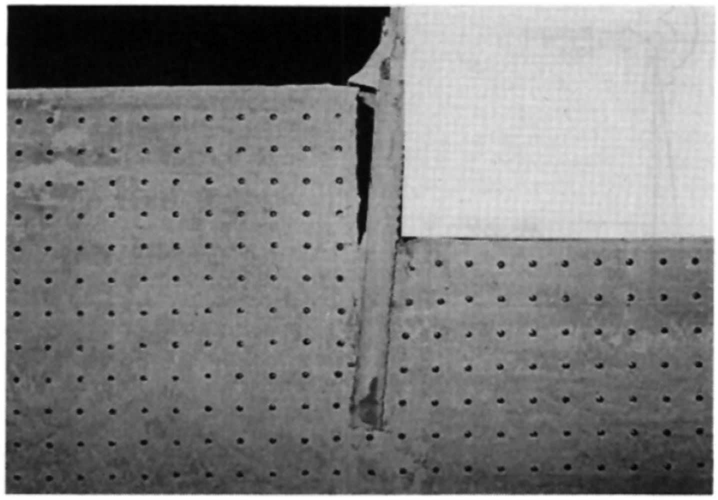
Expected deformation mechanism?



These observed and idealised deformation mechanisms, in conjunction with Rankine stress fields and plane strain soil data, validated the first Mobilizable Strength Design (MSD) predictions for soil displacements in the working range.

The photographic record of displacements is on the left – this is before the days of Particle Image Velocimetry (PIV) and digital image processing, so markers had to chased by eye. The behaviour on the right is that "expected" in a simple mechanical model, compatible with rotation of a smooth wall about a point near the toe, and with active and passive shear mechanisms and shear directions at 45 degrees to the wall. By deducing stresses in equilibrium at any stage of excavation using Mohr circles, and linking them to strains observed in a plane strain element test, and then imposing those strains on the assumed deformation mechanism, we were able to predict wall and ground movements. This is an early example of Mobilizable Strength Design (MSD) – which continues to be developed for a variety of foundation, excavation and earth retention scenarios.

An unexpected failure mechanism?



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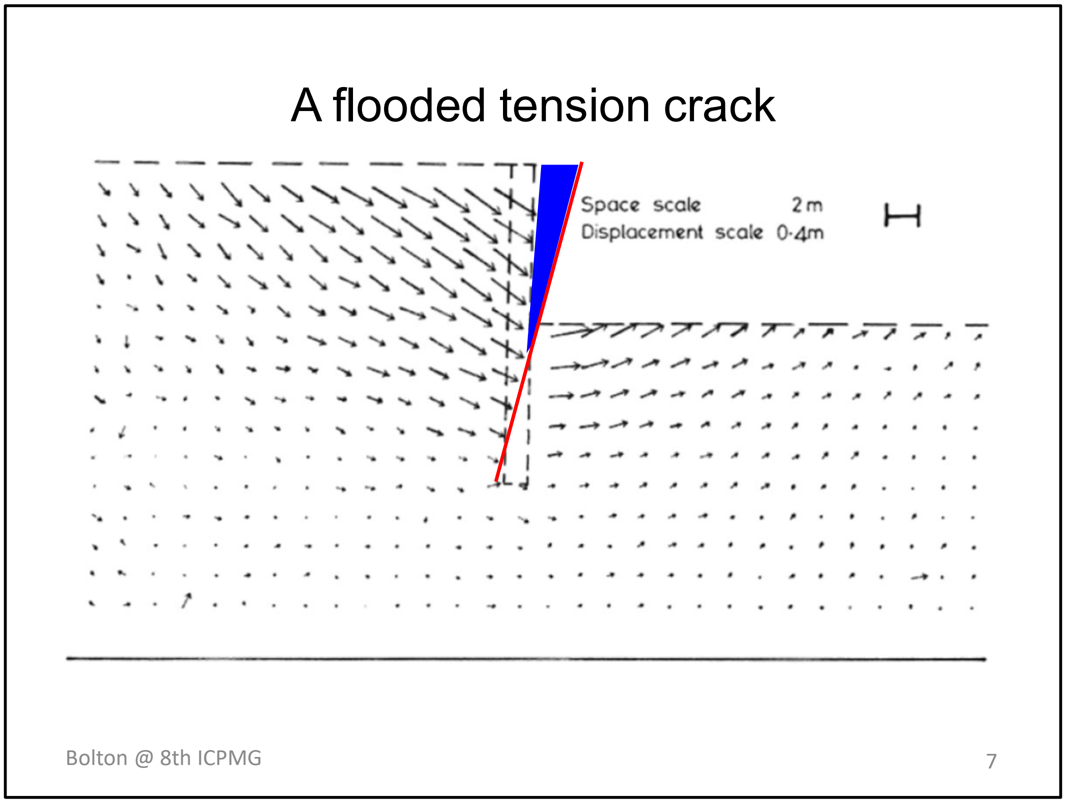
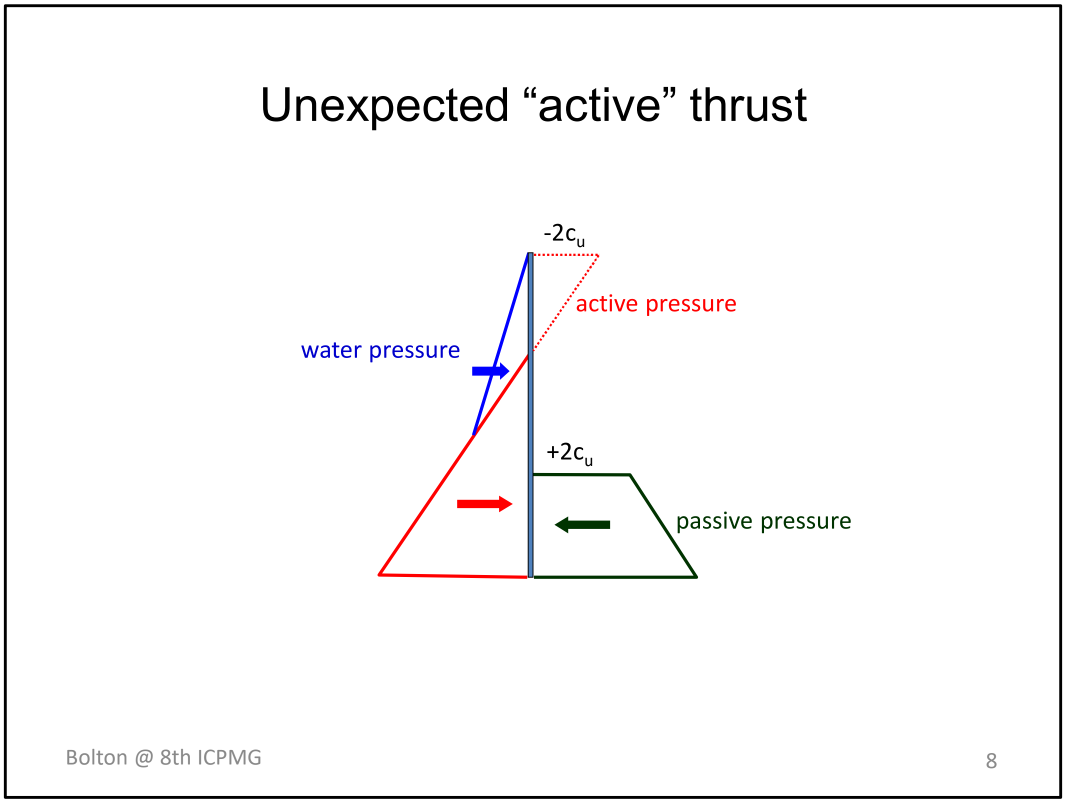
But here is a failure mechanism that was not initially expected. The wall has a relatively shallow embedment. But what happened?



Water flooded a tension crack against the wall on the active side – remember the high water table; it was a few mm above ground level prior to “excavation”.



So here is the correct active pressure diagram including the blue hydraulic thrust from the fracture. Some engineers still make the mistake of following the red line even where surface water may be available.

And this is what can happen when that mistake is made. A flood wall washed away in New Orleans in 2005 by the storm surge created by hurricane Katrina.

Scott Steedman's IPET report: March 2006

Katrina-struck levee moved before it failed, probe shows

HURRICANE STORM surges in New Orleans pushed a sheet piled levee wall out of position before it failed, flooding a part of the city, engineers said this week.

The surge exerted huge lateral forces on a whole section of sheet piled levee on the city's 17th Street canal, causing a 139m long breach on its the east side.

It occurred as Hurricane Katrina's storm surges pushed water from Lake Ponchartrain into the canal last August (*NCE* 8 September 2005).

Initial results of centrifuge modelling of the failure were released last week by the US government-appointed Interagency Performance Evaluation Task Force (IPET).

These show that as canal water levels rose, the flood wall moved landward. The movement created an underwater gap between the canal side of the floodwall and the clay embankment.

Water was then able to push down against the toe of the sheet pile, causing it to shift landwards.

Weak shear strengths in the clay layer below the sheet pile and hydrostatic forces on the pile toe pushed the sheet pile vertically along a slip plane in the clay.

Centrifuge testing of models of the 17th Street levee were led by ICE vice president Scott Steedman, who is heading up one of 10 IPET teams.

"If you have clay next to a sheet pile wall and water flooding on top it's very easy for the two to separate," said Steedman.

John McKenna

INFOPLUS

View a video of the centrifuge and storm surge models at www.nceplus.co.uk

6 NEW CIVIL ENGINEER 16 MARCH 2006

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Centrifuge tests were commissioned in the USA to try to explain why the flood walls failed early, and before they were overtopped. The designers had missed the Powrie mechanism. This was all explained by Scott Steedman who chaired an IPET investigation. I will return to his own PhD research in a moment.

Lesson #1: behaviour of in situ walls

- Replicated changes of stress cause similar strains and proportional displacements δ/H.
- Observed deformation mechanisms need to be idealised for MSD to make predictions.
- Surface water can cause devastating hydraulic fractures: there is a world of difference between a water table 1cm above ground and 1cm below.
- Both expected and unexpected outcomes are valuable.

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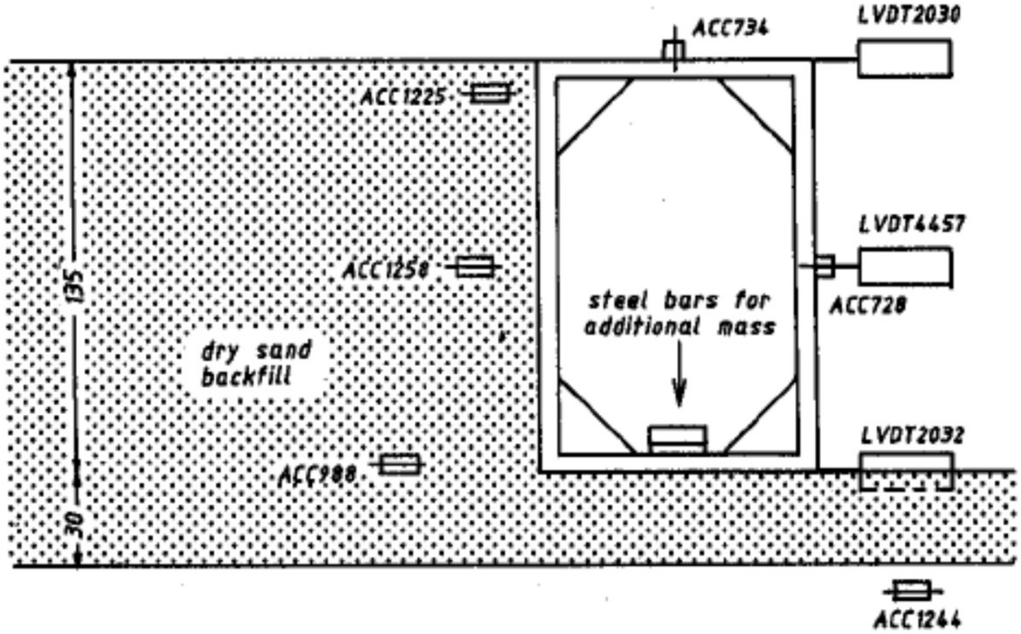
Expected and unexpected outcomes have proved significant.

MSD has been developed from these early beginnings and is now available as a straightforward and well-validated tool for serviceability assessments. One might say PIV + MSD = SLS.

But we have also been reminded of the potentially catastrophic but, in 1983 unexpected, effects of surface water filling cracks.

Earthquakes on gravity wall in sand: Steedman



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The second application concerns earthquake effects on mass gravity walls, studied by Scott Steedman who took his PhD with me in 1984. Scott is now Director of Standards at BSI.

Dry sands were poured dense at 1g behind rough block walls, centrifuged, and tested by model earthquakes produced by the Cambridge “bumpy road” mechanical actuator.

Failure mechanism

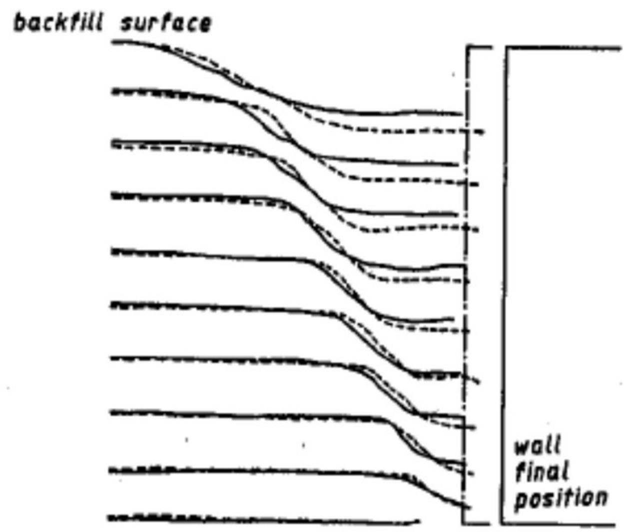

Fig.4 Test 81 Active Failure Wedge

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Here as an example is a failure wedge that pushed the wall forwards.

Analytical model

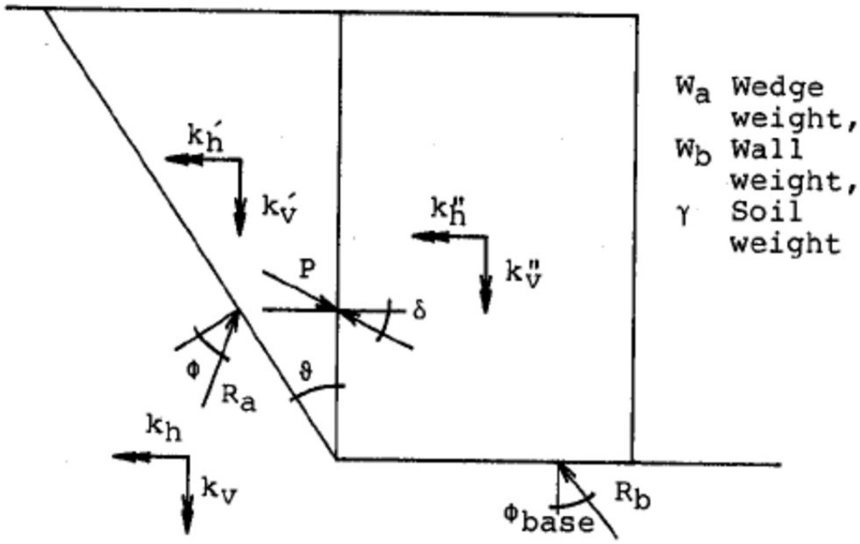

Fig. 5 Notation of Double Block Model

And here is the idealised mechanism that was used to analyse these events: a quasi-dynamic Newmark sliding analysis with consistent horizontal and vertical acceleration components both for the wedge and the wall.

Evidence of softening

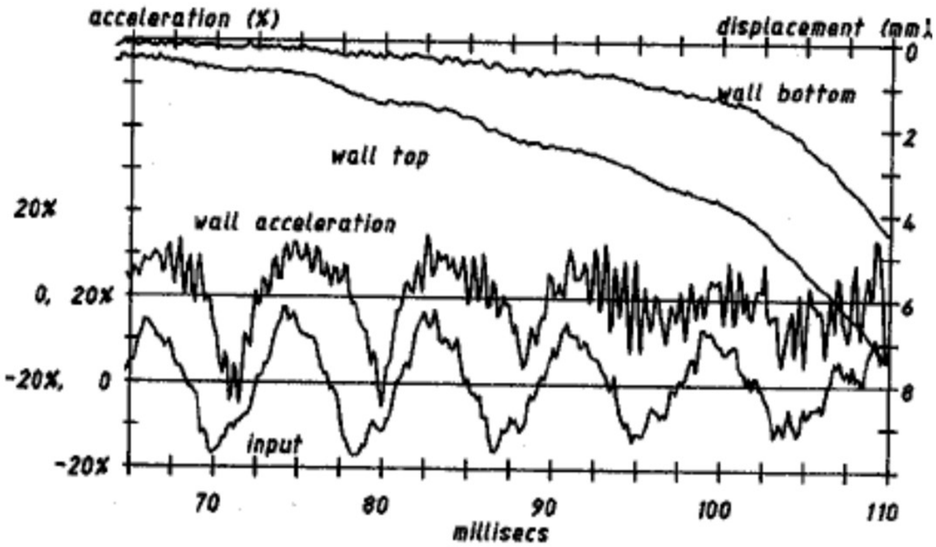

Fig.6 The Failure of Test 82

Here is a fragment of the data stream. At the bottom we see the accelerometer traces of the base input and the wall. Above, we see the displacement records. The forcing acceleration reduces after 90 milliseconds just as the LVDT displacements begin to run away! Isn't that inconsistent with the sliding block model?

Analysis show ϕ_{max} drops to ϕ_{crit}

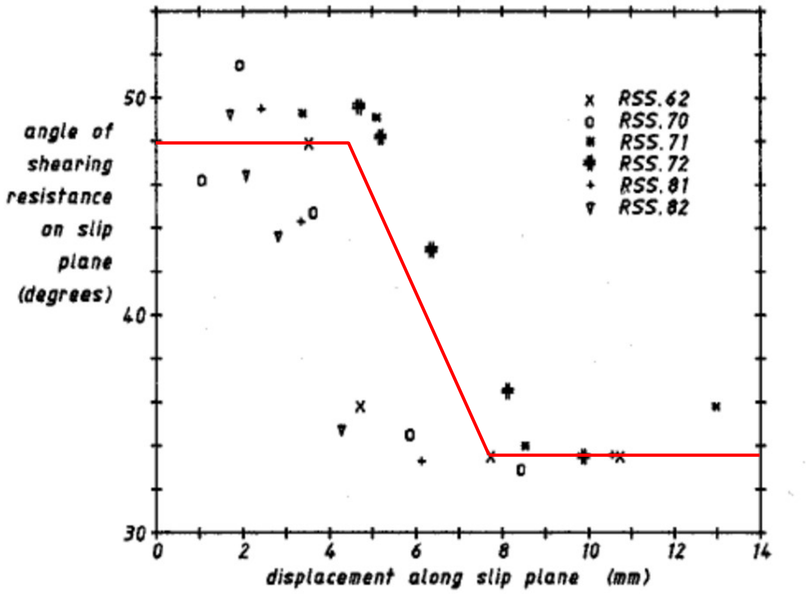

Fig.7 ϕ vs. displacement on slip plane

Not at all: the yield acceleration had decreased. MSD style sliding block analyses performed successively throughout the event shows friction angle ϕ starting at 48 degrees, but reducing to 33 degrees after about 5mm of sliding. That explains why a reduced shaking amplitude towards the end of the earthquake evoked a much larger response.

Summary on earthquake failure of dense sand

- Sliding block analysis found the soil strength ϕ and wall base friction δ consistent with observed accelerations: an early example of MSD.
- Direct shear tests gave $\phi_{max} \approx 50^\circ$, $\phi_{crit} \approx \delta \approx 33^\circ$ for soil-soil and soil-wall friction in dense sand.
- Centrifuge tests showed ϕ dropping from 50° to 33° as a rupture zone formed during about 10 particle diameters of relative sliding.
- Bolton & Steedman (1985) Modelling the seismic resistance of retaining structures, 11th ICSMFE, San Francisco, **4**, 1845-1848.

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A sliding wedge dynamic analysis comprehends the full spectrum of behaviour. A detailed prediction requires ϕ_{max} and ϕ_{crit}, boundary friction δ, and also knowledge that softening from peak strength would occur over about 10 particle diameters of shearing. This was written up for the San Francisco conference.

Note that a robust structure could be designed simply by ignoring the initial peak strength and using ϕ_{crit} , however. This is equivalent to assuming instantaneous softening, as though the grain size were infinitesimal compared to the wall, and as though failure would happen on a single rupture surface.

Lesson #2: localisation in sands

- A "scaling factor" of N on sliding displacement of a model wall at Ng would apply at full-scale only if the grain size were also scaled up by factor N.
- Sand sliding at full scale may soften faster, due to grain scale localisation (Desrues & Viggiani, 2004; Kutter et al, 1994).
- But robust seismic designs can be based on ϕ_{crit} and δ_{crit} mobilized in a Newmark analysis of sliding blocks.

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What does this tell us about grain size effects?

Where the ratio of grain size to structure size is significant, a model grain diameter should be scaled up by factor N to make an equivalent full-scale prototype.It is well known that a shear rupture band is about 10 particle diameters thick and requires at least 10 particle diameters of relative sliding to dilate fully to critical state strength. This does not scale, so for a given size of soil grain a larger structure will soften earlier, i.e. at a smaller *proportional* displacement, and its ultimate displacement will exceed N times the model displacements for a similar earthquake.

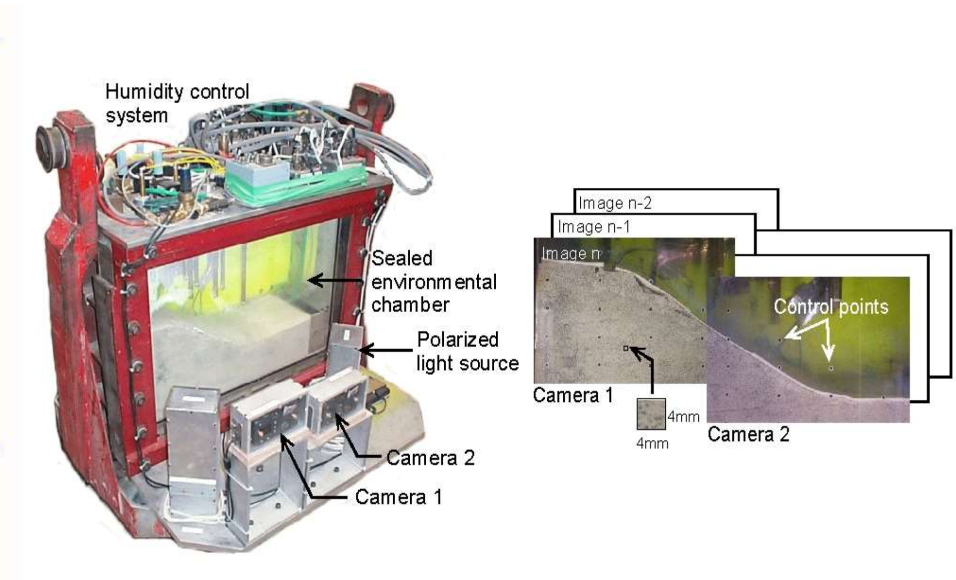
We have seen that wall translation is susceptible to localisation and softening. It is still worthwhile to investigate the possible consequences of localisation in other centrifuge model scenarios. Spread foundations under vertical loads appear to be almost immune: see Lau & Bolton (2013) in Geotechnique. And we recognise that CPTs in dense and fine sands of the same grain size produce markedly different resistances: similarly, pile end bearing is relatively immune.

First-time failures in clay slopes: Andy Take

- The clay exposed in slopes is inevitably overconsolidated, and therefore dilatant.
- Does that mean that sliding failures in clay slopes will inevitably localise on very thin rupture bands?
- If so, should clay slopes be designed using ϕ_{res}?
- Before this can be answered, clay slopes have to be seen "performing". This was achieved by Andy Take in an atmospheric chamber that could impose varied "weather" conditions.

We will now switch our focus to possible localisation and progressive failure in clay slopes. Should we use ϕ_{res} in the design against first-time failures? First, we have to provoke slopes in some way. This was investigated in Andy Take's PhD in 2003. He is now a Professor in Queen's University, Canada.

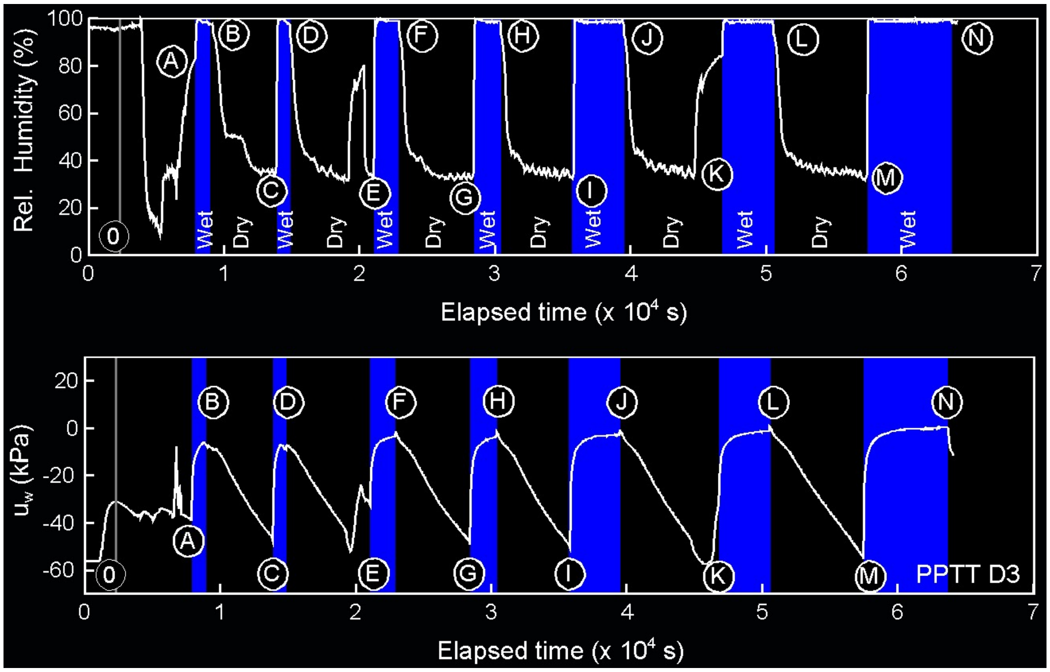
Andy Take's atmospheric chamber



8.7m embankments of stiff kaolin clay modelled at 60g

A home-made air conditioner is seen on top of a sealed chamber. Warm air can circulate to simulate a dry season; misting nozzles can simulate rainfall in a wet season. Cameras can record the displacement of "texture" placed on clay behind the window. Then PIV, developed as PhD students by Dave White and Andy Take, can chase patches of texture as they shift behind fixed photogrammetric markers painted on the rear of the window.

Seasonal humidity control



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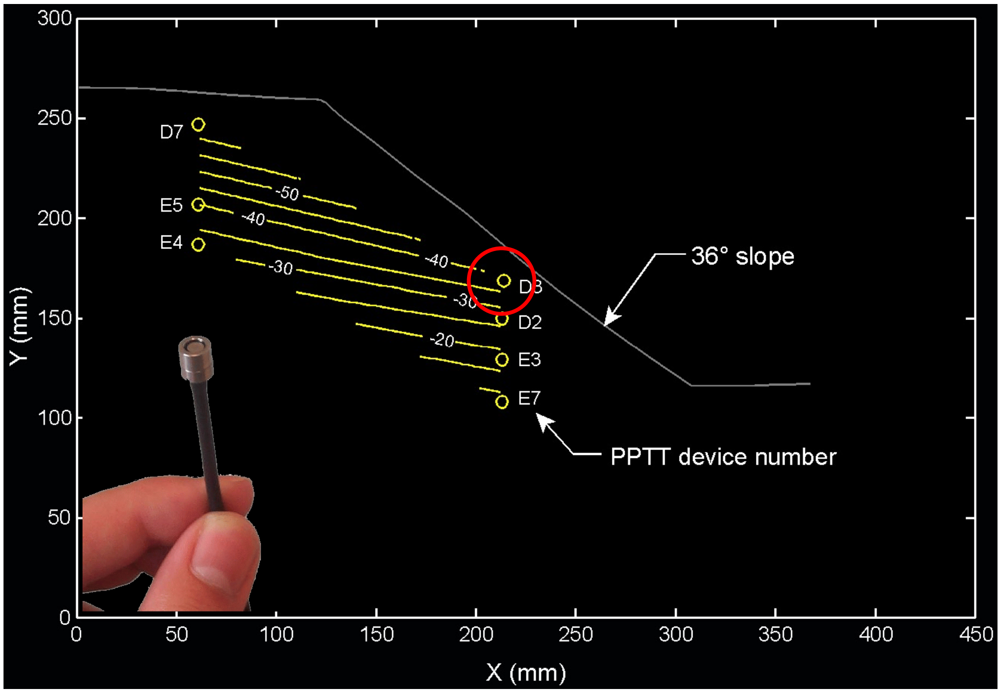
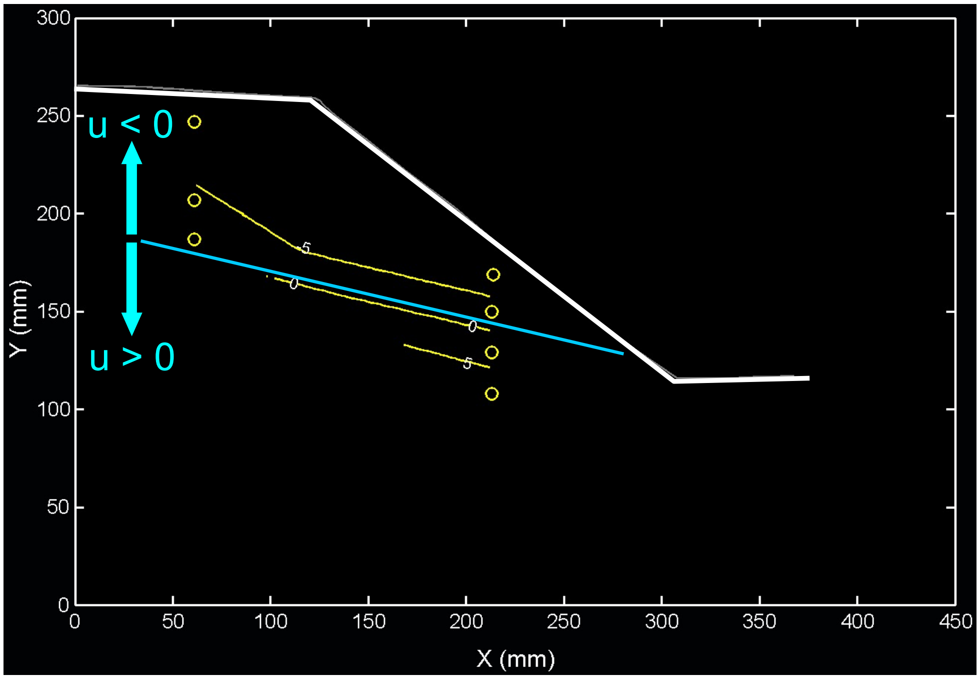
Relative humidity sensors allow efficient control of the model “weather”. Buried pore pressure transducers, which also act as tensiometers, respond to the changing atmospheric humidity. Wet seasons are marked in blue.

Pore pressure measurement



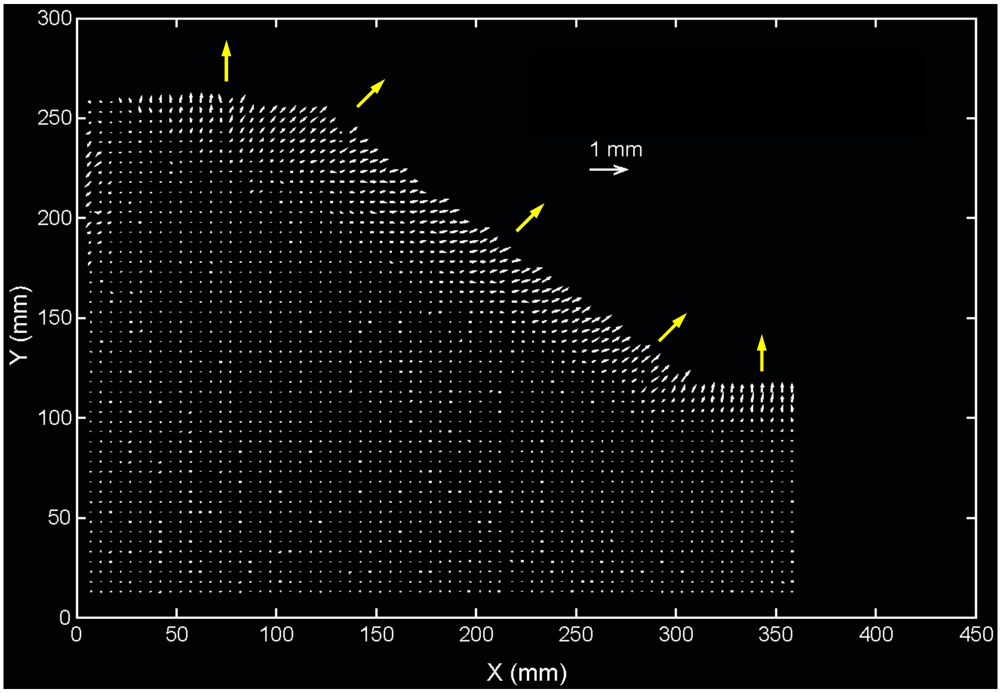
A group of PPTTs allows the pore pressure distribution to be determined.

Pore pressures after 1st short wet season



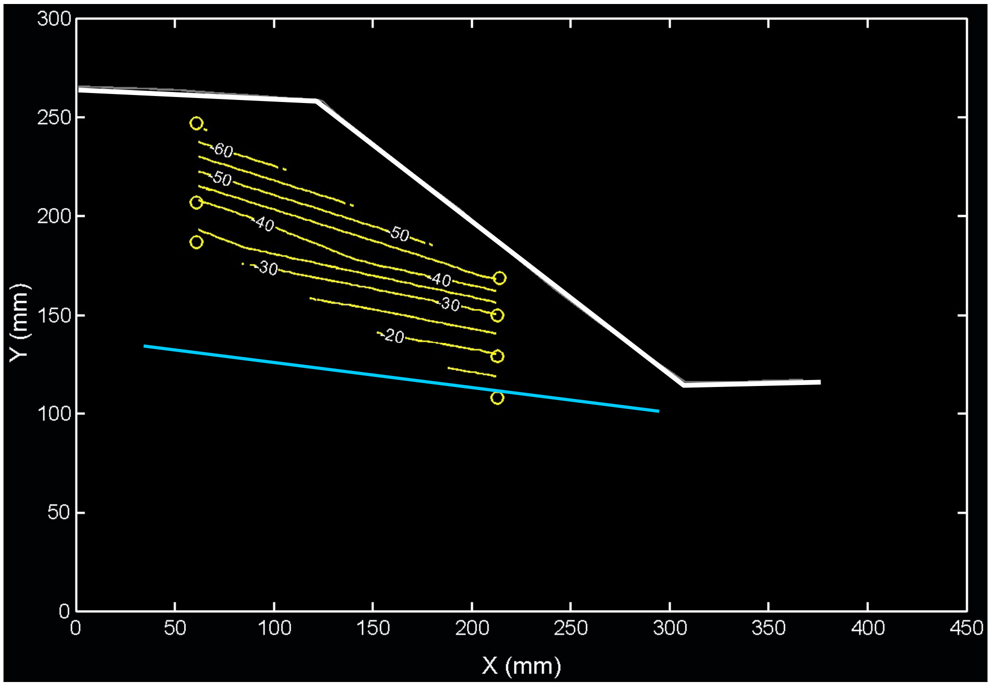
The water table is marked here in blue: with suction above, positive pressure beneath. The water table rises after the wet season.

Swelling during 1st short wet season



Swelling is seen after the wet season. Vectors are normal to the surface.

Pore pressures after 1st long dry season



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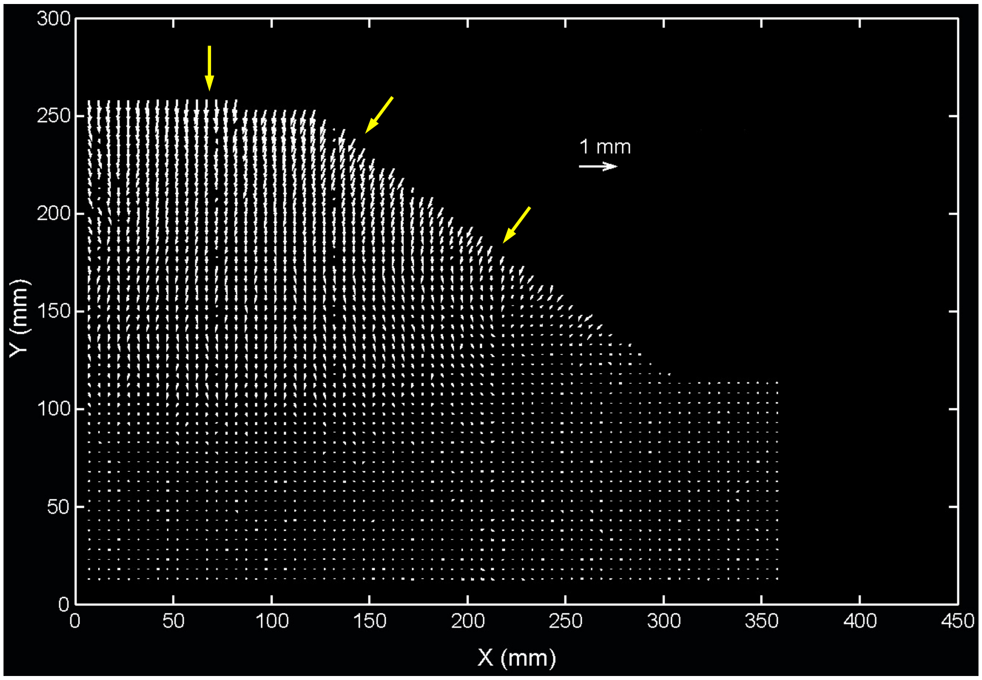
The water table drops in the dry season – but the clay above the water table remains saturated, albeit with 80 kPa negative pore pressure.

Shrinkage in 1st long dry season



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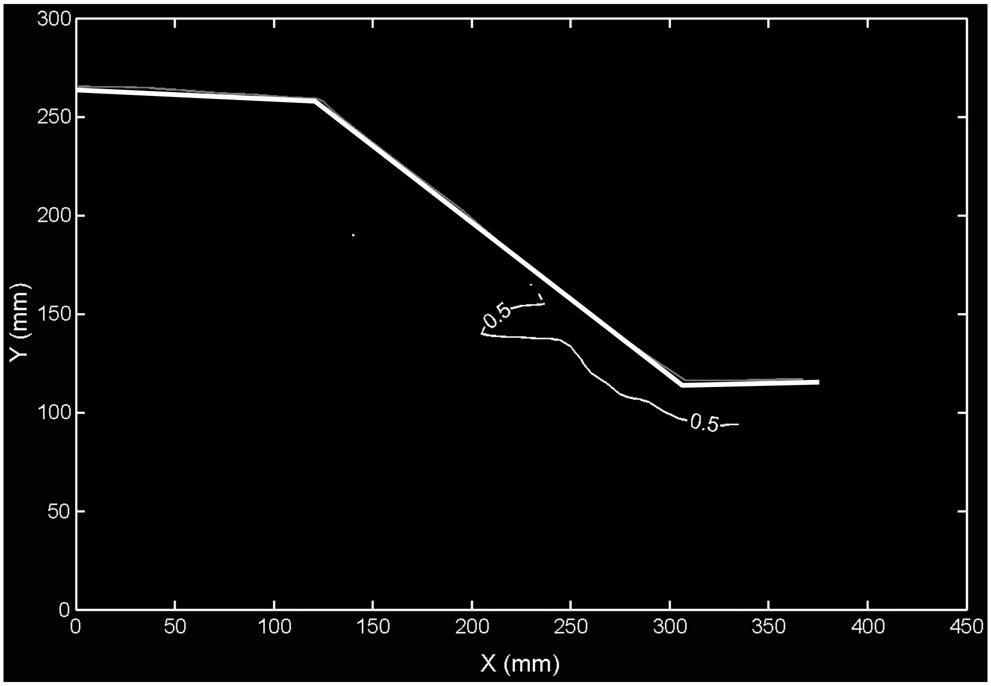
Shrinkage is seen after the dry season.

Cumulative strain γ % after 1st year



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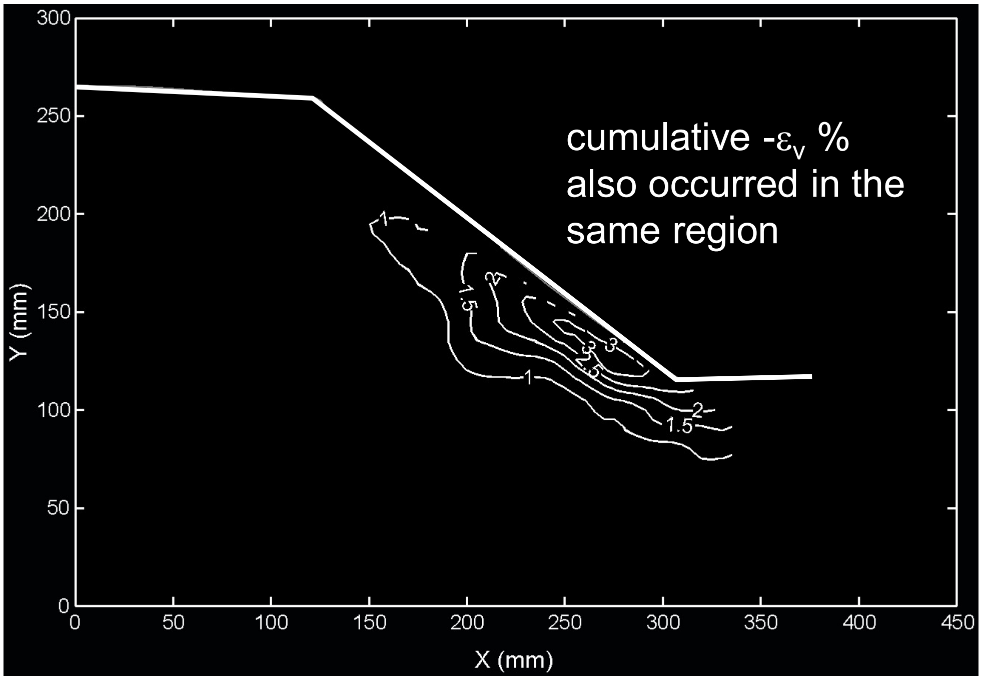
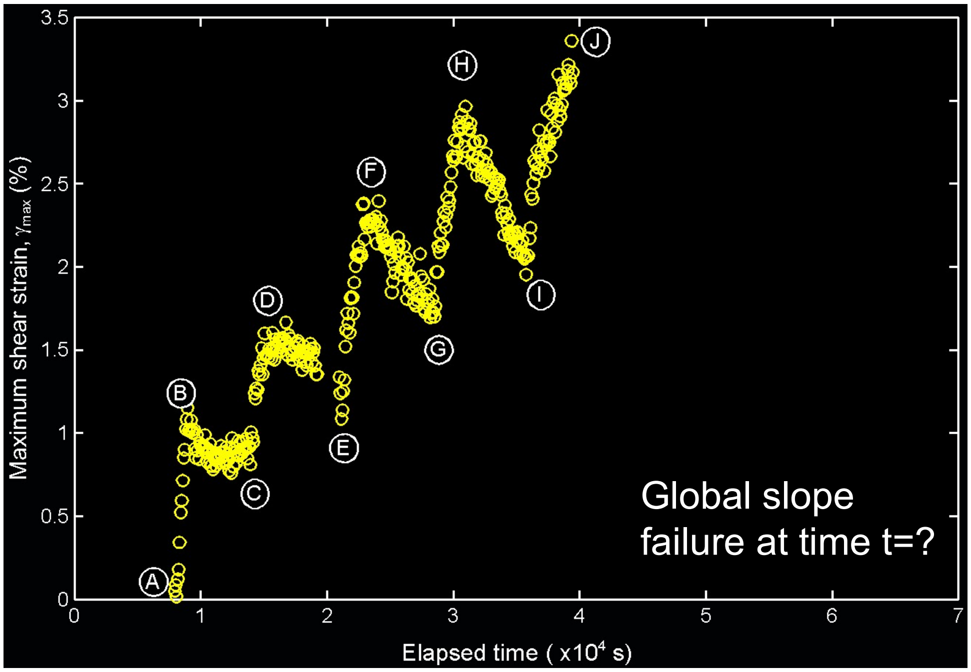
The dry season does not completely remediate the strains induced in the wet season. Residual strains build up “year” by “year”.

Cumulative γ % after 5 years



After 5 “years” of wet and dry seasons a large zone around the base of the slope has suffered 3% shear strains, and dilation was found in the same region.

Accumulating damage γ % due to softening



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The shear strains ratchet up year by year, towards failure.

Mobilisation analysis

- Make a modified slip circle analysis, to find how τ_{mob} varies with the seasons.
- Use $\tau_{max} = c' + \sigma' \tan \phi'$
- Iterate to find c' ϕ' for $FoS_{min} = 1$
- Mobilise ϕ' first, up to ϕ_{crit}
- Then as much c' as necessary.
- Gives simple average c'_{mob} required over the whole slip circle, for equilibrium.

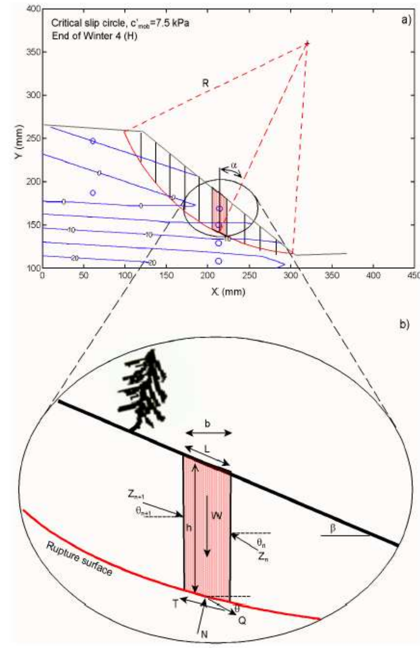
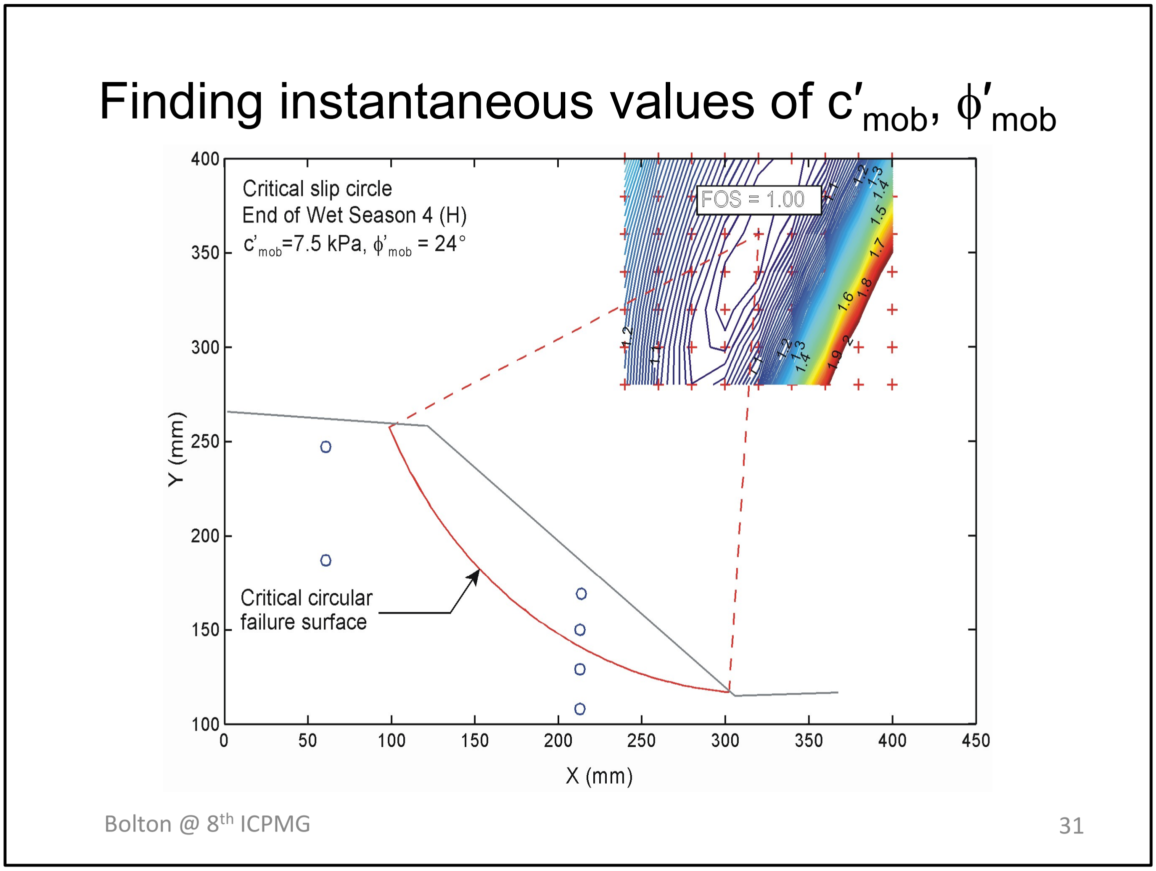


Figure 7.10. Application of Spencer's method to the model slope.

In an effort to understand the origins of the slope creep that had been observed we used slip circles to analyse the state of equilibrium instant by instant, using our own pore pressure data. We searched for mobilised strength parameters that would guarantee a factor of safety of unity at any given instant. We allowed the friction to mobilize first, up to the critical state angle, only then invoking "true cohesion" as required. This solved the well-known problem of having two unknowns, c' and ϕ', and only one equation, equilibrium.



You could all go into the office in the morning and do this class of analysis yourselves. The critical slip circle at the end of year 4 is shown, with its FoS = 1, requiring strength parameters ϕ'_{mob} = 24 degrees, c'_{mob} = 7.5 kPa.

Cyclical mobilisation of strength

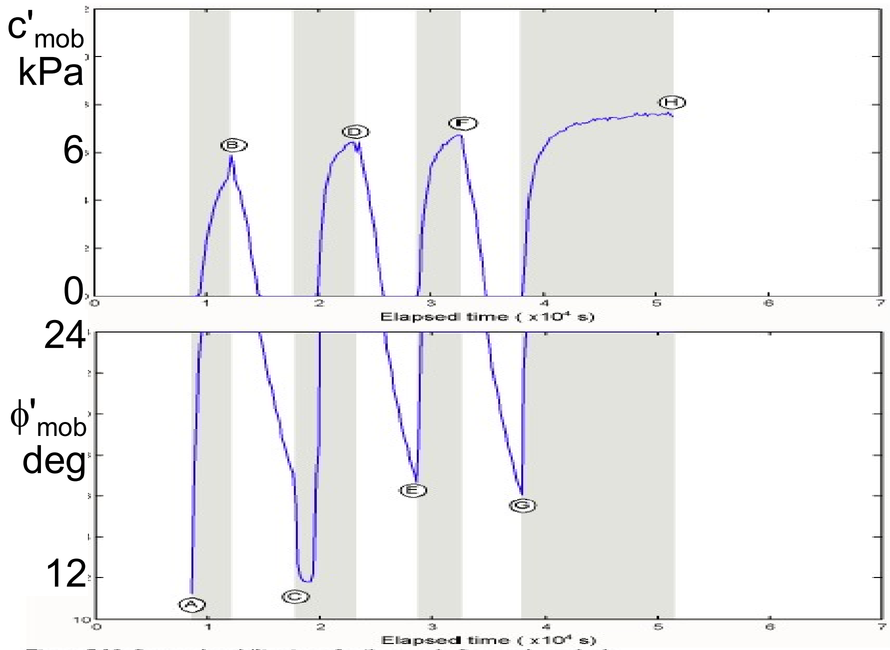
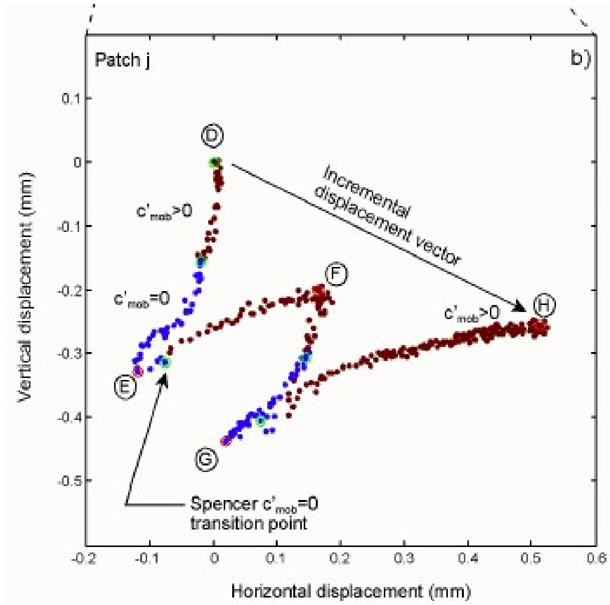


Figure 7.12. Seasonal mobilisation of soil strength, Spencer's method.

Here is the resulting time sequence of mobilised strength, season by season. Every wet season, the soil has to mobilize more than ϕ_{crit}.

Cyclical mobilisation of c' causes softening

- Swelling is recoverable for $\phi'_{mob} < \phi_{crit}$
- Swelling, softening and slope “creep” is cumulative for $c'_{mob} > 0$



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And here is the pay-off. This tracks the motion of a shallow PIV patch below the slope. Blue points correspond to the mobilization of sub-critical strength, red points to super-critical strength. The slope breathes in and out, elastically, until it has to mobilize super-critical strength. Then it dilates and shears downslope a bit. In successive wet seasons it ratchets towards failure.

Seasonal softening leads eventually to local failure at the toe

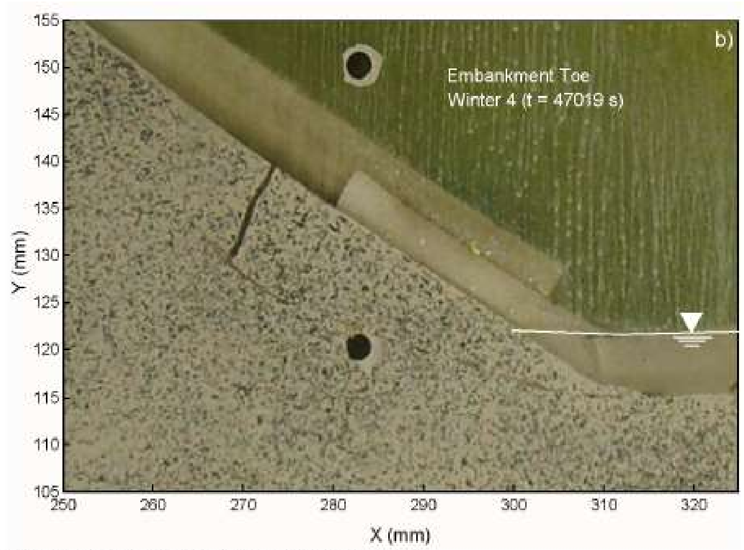

Figure 7.15. Localisation at embankment toe, model test WAT7a.

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Eventually, cracks open up and fill with rain water, a residual slip surface begins to form, and the toe of the slope drops away. But by this stage the soil had already softened, and the slope had already moved a long way.

Lesson #3: drained cyclic creep and failure of clay slopes

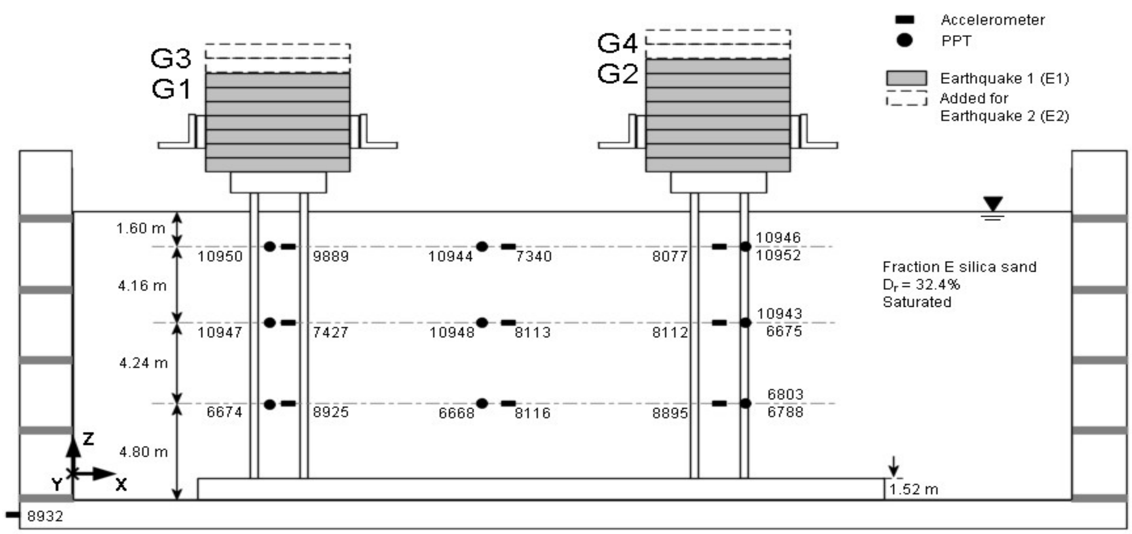
- If we use slip circles in design to mobilise $\phi \leq \phi_{crit}$ after a wet season, we should prevent seasonal slope creep, dilation, softening and failure.
- The "fatigue crack" model of progressive failure, with ϕ_{max} dropping suddenly to ϕ_{res} on a growing rupture surface, appears to be irrelevant here.
- Drained cyclic failure of sands and clays is similar.
- Take & Bolton (2011) Seasonal ratcheting and softening in clay slopes, leading to first-time failure. Geotechnique 61(9): 757-769.

Skempton was right. Design intact clay slopes to ϕ_{crit} *NOT* to their peak strength, and *NOT* to their residual strength. Now we know why – so that they should not even begin to ratchet towards failure. This should apply to any soil body which carries constant shear stresses, but within which pore pressures cycle (whether due to rainfall on top, or an aquifer or leaky sewer beneath) such that super-critical soil strength has to be mobilized.

And note that peak strength defined by c', ϕ' is irrelevant. It remains irrelevant when reduced by a specified partial factor. It is the wrong place to start to derive a design value. You need to start with ϕ_{crit}; then you do not need a safety factor! But you do need to make a projection of future pore pressures. This requires experience, judgement, and countermeasures.

The drained cyclic shearing of sands (Steedman's earthquakes) and clays (seasonal moisture movements within Take's slopes) leads to similar behaviour and much the same design recommendation: use ϕ_{crit}.

Soil liquefaction and pile buckling: Bhattacharya



We now move on to seismic liquefaction around slender end-bearing foundation piles. Subhamoy Bhattacharya conducted shaking tests on the centrifuge using a laminar box mounted on the SAM actuator. Different piles and pile groups had to transmit heavy axial loads through the saturated sand. PPTs and accelerometers were mounted in various locations.

Pile groups restrained from inertia effects

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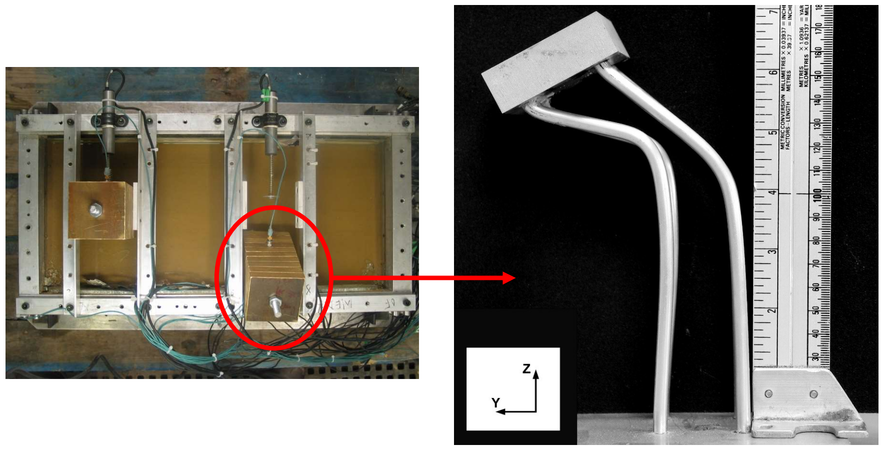
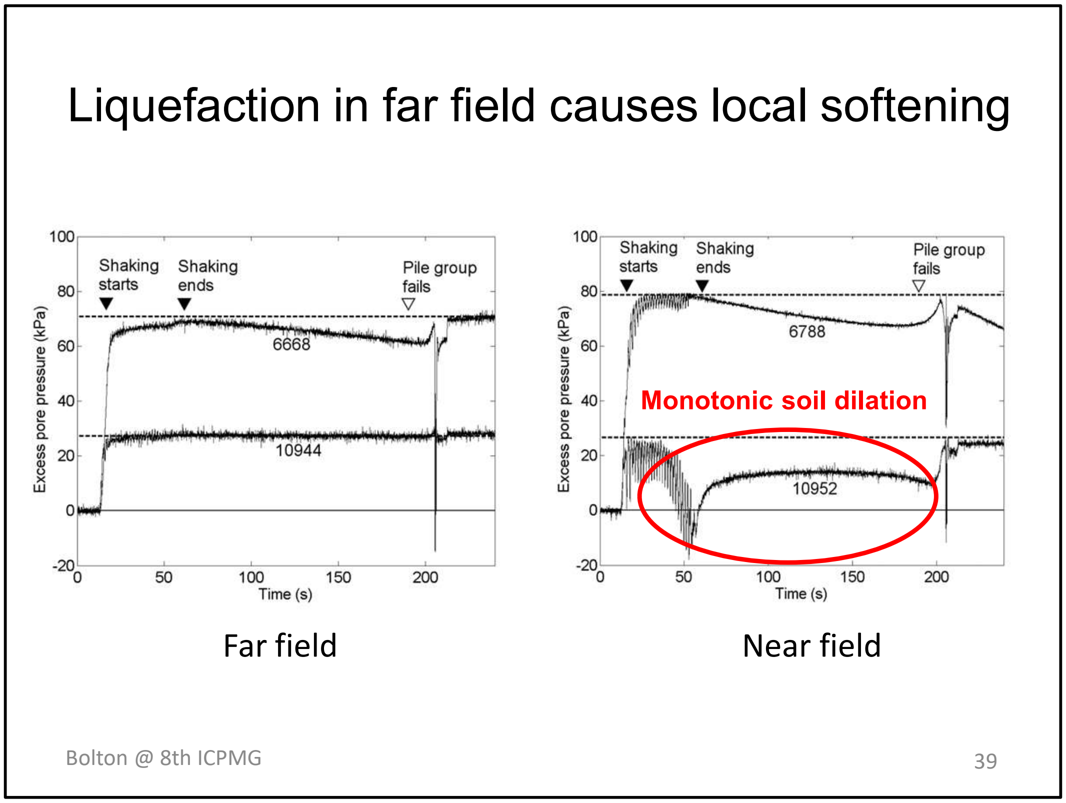
The photograph shows that the loads were restrained in the direction of shaking, but free to move laterally. This is “before”.

Earthquake liquefaction effects



Group G4

And this is “after” an earthquake. The pile group on the right failed – by buckling as you can now see.



Excess pore pressures in the free field during the earthquake reach the levels of pre-existing vertical effective stress. In other words the sand suffers “initial liquefaction” in which effective stress, and therefore stiffness, is eliminated. On the right, however, you can see that pore pressures adjacent to the pile then go temporarily into suction. That is not good news. It indicates that the piles are cutting through the sand, which temporarily tries to dilate. However, water floods in and the sand softens again as the piles continue to buckle.

Showa bridge collapse due to pile buckling

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We hypothesised that the buckling of its slender steel piles, seen in the picture, was the cause of the failure of the Showa bridge one minute after the Niigata earthquake in 1964.

Lesson #4: piles can buckle in liquefied soils

- Fast cyclic shearing of sands leads to excess pore pressures, loss of soil stiffness, and the buckling of slender piles after earthquakes.
- This mechanism surprised most experts, because it was not in the Codes, but was it "unexpected"?
- Designers should make FoS > 3 on Euler buckling.
- Bhattacharya, Madabhushi & Bolton (2004) An alternative mechanism of pile failure in liquefiable deposits during earthquakes. Geotechnique, 54 (3), 203-213.

Piles are sometimes designed to be slender, requiring lateral bracing from the soils they pass through to prevent their buckling. If the soil liquefies, buckling occurs. This should not have been surprising. But, again, reviewers found it so – probably because the seismic codes had not mentioned the possibility.

Piles need a large FoS > 3 against Euler buckling, to account for their eccentricity, and for plastic deterioration of the elastic critical load.

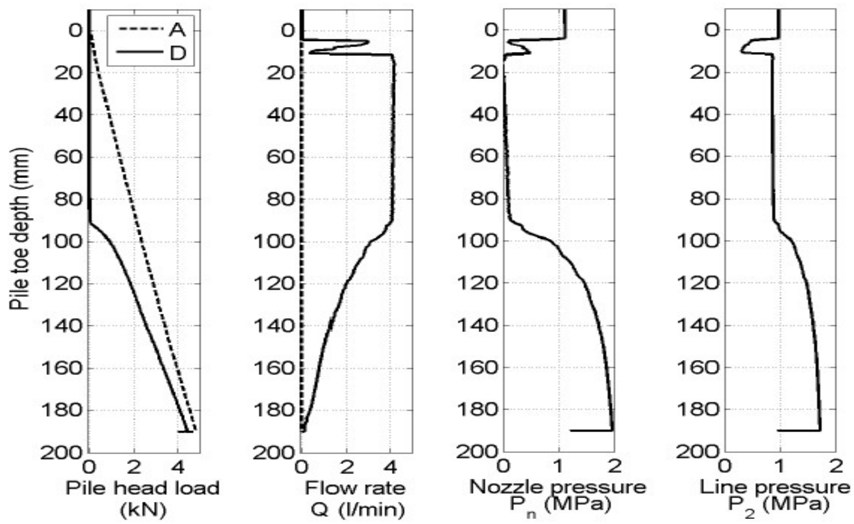
This work was published in Geotechnique, and a paper on the Showa bridge collapse was eventually accepted and published in Soils and Foundations, in 2005.

Suby Bhattacharya got his PhD in 2003 and is now a Professor in Surrey University.

Water injection to aid pile jacking: Shepley

The final application takes us from science towards technology, looking at the use of water-jetting to ease the process of pile-jacking in dense sands. On the right is one of a set of model piles tested by Paul Shepley, with a variety of nozzle arrangements in and around the pile base. On the left is the 2D actuator ready to fly on the Cambridge centrifuge. It is holding a model pile above the saturated sand, prior to a test.

Water injection works above a critical depth



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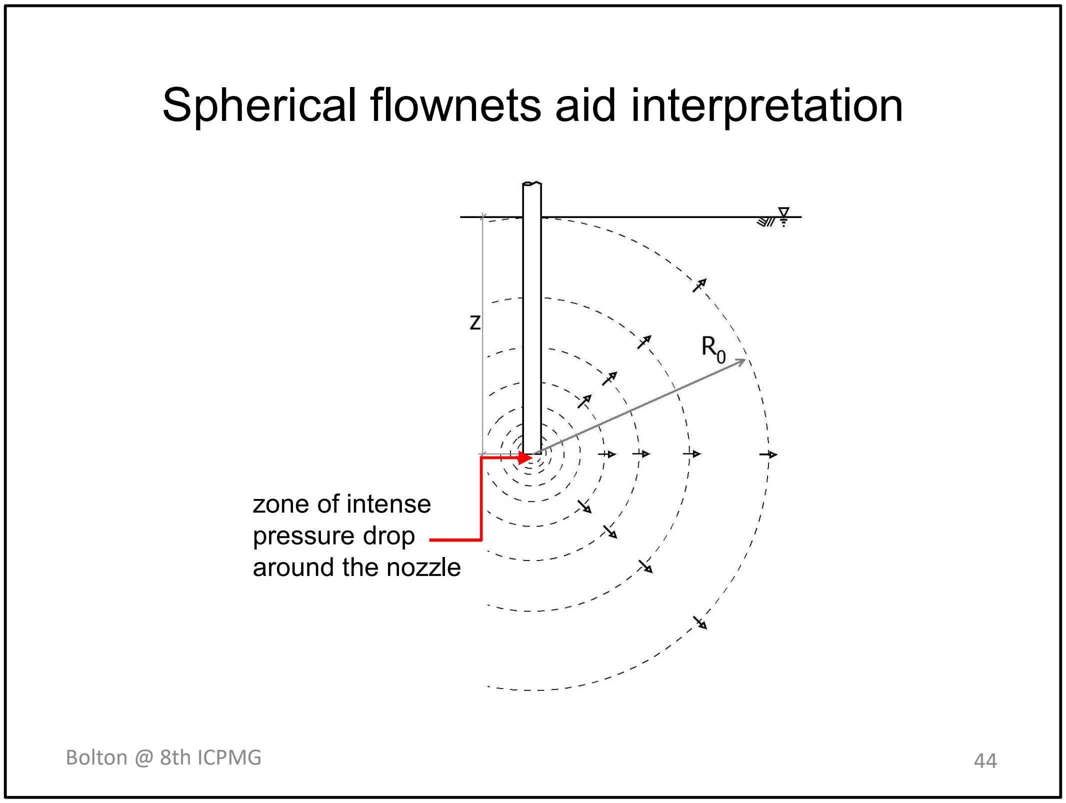
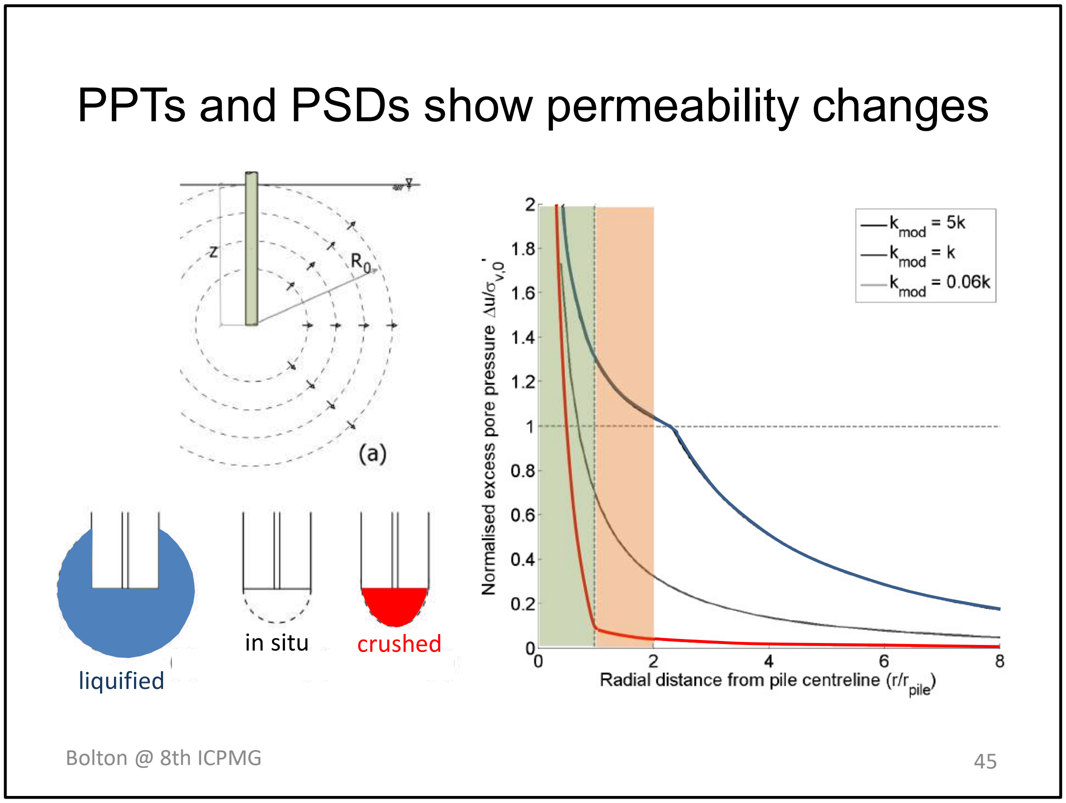
On the left we have comparative driving records with (D) and without (A) water injection. Apparently, up to some critical depth, we can eliminate installation loads. Then we see the flow rate record during installation D: flowrate suddenly starts to reduce at the same point at which base load begins to pick up – the critical depth. At the same point, Paul could infer that the nozzle pressure started to increase dramatically. This behaviour was unexpected.



The behaviour mechanism we chose to represent what was going on is a steady spherical flownet centred on the nozzle. Because of the small spacing between equipotentials of head drop immediately around the nozzle, these have been omitted. The head eventually drops to zero at a radius equal to the pile depth z. These excess water heads reduce the effective overburden pressure around the pile.



Data from PPTs, and a post-flight autopsy including samples for particle size analysis, showed that the simple spherical flownet needed modification as follows. If the injection could eliminate the effective overburden pressure one pile radius beyond the shoulder, the soil immediately around the pile would liquefy (the pink zone) and eliminate all driving stresses. In doing so, its permeability can increase by a factor of 5 to 10 according to parallel work published by Stuart Haigh. This makes it easier to transmit pore pressures outwards beneath the pile – indicated by the top curve of normalised excess pore pressure, for which unity corresponds to "liquefaction".

So if the injection is maintained as the pile penetrates, the liquefaction can be maintained. Eventually, however, the increasing overburden pressure makes it impossible to eliminate: that is the critical depth. Then the soil permeability beneath the pile reverts to its in situ value, and liquefaction abruptly ceases – as is evident on the normalised pore pressure curve. Base loads immediately increase. This causes the sand beneath to crush, reducing its d_{10} size and strongly reducing its permeability. Effectively, a hemispherical plug of relatively impermeable soil now isolates the nozzle, the flow rate drops towards zero, and the nozzle pressure rises to the pump pressure.

Pile insertion by soil liquefaction

- Water injection to set $\delta u = \sigma'_{v,0}$ one radius beyond the pile shoulder eliminates driving resistance.
- The required flow rate could be assessed using spherical flownets if soil permeability were known.
- But tests also show the crucial effect of changing soil permeability below the pile toe, e.g. x10 if sand is liquefied, ÷10 if it crushes (σ_b > 5 MPa?).
- Full scale predictions are best made by applying the flownet model directly, after considering all possible changes to the flow regime.

So the key to successful pile jacking in dense sand is the elimination of $\sigma'_{v,0}$ by water injection. And the problem for prediction is the order-of-magnitude changes of soil permeability beneath and around the pile. The spherical flownet is good enough, as far as it goes. Paul was able to report on the flow quantity required in the centrifuge models, and to correlate with the implied permeability based on its in situ value.

Behaviour mechanism is clear, but scaling?

- Flow near the nozzle may be turbulent; the nozzle travels so seepage is transient; some zones are fluidised; some suffusion occurs.
- Shepley (2013) points out that his models had Re x N, Fr x $\sqrt{N}$ compared with a prototype of the same grains, so his fluid instability was greater.
- Is this apparent inconsistency significant?
- And if so, can it be resolved?

But the question arises: to what prototype does the centrifuge model refer?

The granular-fluid interactions are complex, involving liquefaction and internal erosion. Paul Shepley points out that by applying normal centrifuge scaling, both the Reynolds and Froude numbers of the fluids in the model will be larger than they would be in a full-scale prototype made of the same sand. Does this matter?

Scaling

- Darcy's permeability relies on laminar flow for which velocity V varies as $\frac{V\mu}{d^2\nabla p}$ = f (e, U_c, fabric)
- In a centrifuge model: $g_m/g_p = n_g = N$; $x_m/x_p = n_x = N^{-1}$
- If we use water in the model $n_\rho = n_\mu = 1$, and $n_p = n_\rho n_g n_x = 1$ so $n_{\nabla p} = N$, and $n_V = N{n_d}^2$
- We can then explore the similarity of the model with a prototype of identical (e, U_c, fabric) but with scaled particle sizes $d_p = d_m/n_d$

Let us rehearse elementary scaling laws. The scale factors from the prototype to the model are given the symbol n on the slide, with the quantity being scaled recorded as a subscript.

We see that pressure is replicated, but distances scale down, so pressure gradient scales up, and flow velocity scales up by n_V, which includes the centrifuge g-factor N, but also includes the square of the grain size factor, assuming such a thing exists.

Particle scaling for fluid interactions

- Reynolds number $Re = \frac{\rho V d}{\mu}$ so $n_{Re} = N {n_d}^3$
- Froude number $Fr = \frac{V}{\sqrt{gd}}$ so $n_{Fr} = N^{0.5} {n_d}^{1.5}$
- Suffusion number $Su = \frac{\mu V}{\rho g\ {}^2} = \frac{Fr^2}{Re} = 1$
- We can achieve $n_{Re} = n_{Fr} = 1$ if $n_d = N^{-1/3}$
- So in a centrifuge with N = 64, a complex soil-fluid mechanism should be similar to that in a prototype with grains 4 times larger.

We can then determine that the Reynolds number and Froude number of pore flow, and the number controlling internal erosion (suffusion), would all be identical between model and prototype if the latter had grains scaled up by the cube root of N. For example, fine sand in the model would behave like medium sand in the imaginary prototype. This is enough to show that Paul Shepley's model observations will be broadly applicable to real sands in the field.

Lesson #5: mechanisms are more valuable than scaling laws

- Although model performance provides good evidence of the behaviour mechanism, fluid mechanics scaling is only approximate.
- Consider that fine sand models medium sand?
- Be satisfied that standard linear soil mechanics is apparently sufficient to back-analyse the more non-linear fluid-soil interaction mechanism?
- Shepley & Bolton (2014?) Using water injection to remove pile base resistance during installation, CGJ, *under review*.

The key to understanding, as with the previous applications, is a behaviour mechanism. In this case there is some theoretical doubt about applicability due to scaling issues. But we have shown that any distortion can be eliminated by referring fine sand models to medium sand prototypes.

And looking at it another way, Paul was able to verify the spherical flow model to sufficient accuracy using standard, linear soil mechanics. If the higher Re and Fr made the soil-fluid interaction strangely non-linear, then this was apparently not significant.

We have submitted a paper to CGJ, and Paul’s PhD is under examination. He has now taken up a Lectureship at Sheffield University. He also has a poster in this conference, so you can catch more of the details later!

What have we been reminded of?

- Cyclic loads create volume changes if pores drain, and excess pore pressures δu otherwise.
- Soils mobilising $\phi > \phi_{crit}$ must dilate and soften; if they do this repeatedly, they will fail.
- Excess pore pressures $\delta u>0$ lead to a loss of soil stiffness and then to fluid migration and mixing that can permanently reduce soil strength.
- *Centrifuge models show these ideas and their practical consequences in the context of realistic landforms and structures under realistic stresses.*

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In going through these 5 applications we have been reminded of a number of soil mechanics fundamentals including friction, dilation, and the induction of excess pore pressures, both in sand and clay. But we have also seen these concepts brought to life in a practical context. I have also emphasised that each of the test outcomes was initially unexpected, first by ourselves, and often by the reviewers who first saw the publications. Physical modellers can, and should, challenge conventional wisdom.

Is there a new perspective?

- Behavioural mechanisms have to be imagined, even (especially) when models provide “big data”.
- Mechanisms are transferrable technology, and can be used directly in decision-making at full scale.
- Dimensional analysis based on fundamental parameters can lead to unwarranted pessimism when scaling distortions are unavoidable.
- Key distortions relate to grain size effects, and fluid flow interactions.
- The opportunity for further centrifuge research does not conflict with practical applications.

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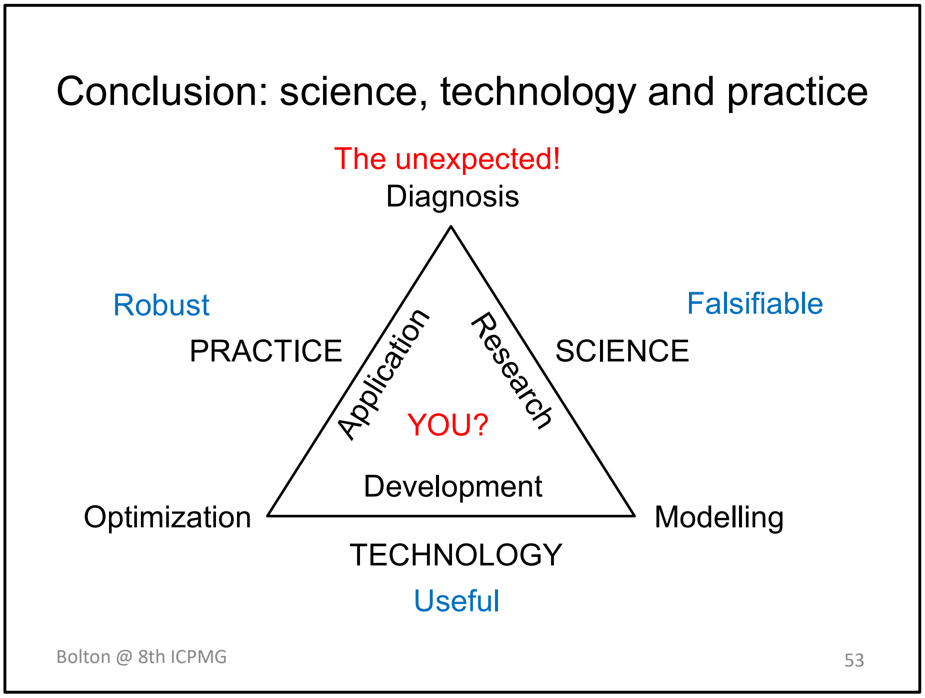
But I hope you have also seen a new perspective. Centrifuge models provide “big data” of PIV sequences and PPTs etc varying throughout a process. But it is also necessary to imagine some simplified mechanisms that provide an approximate explanation and commentary on the data.

Once validated, these mechanisms are transferrable technology. They can be applied directly at full scale.

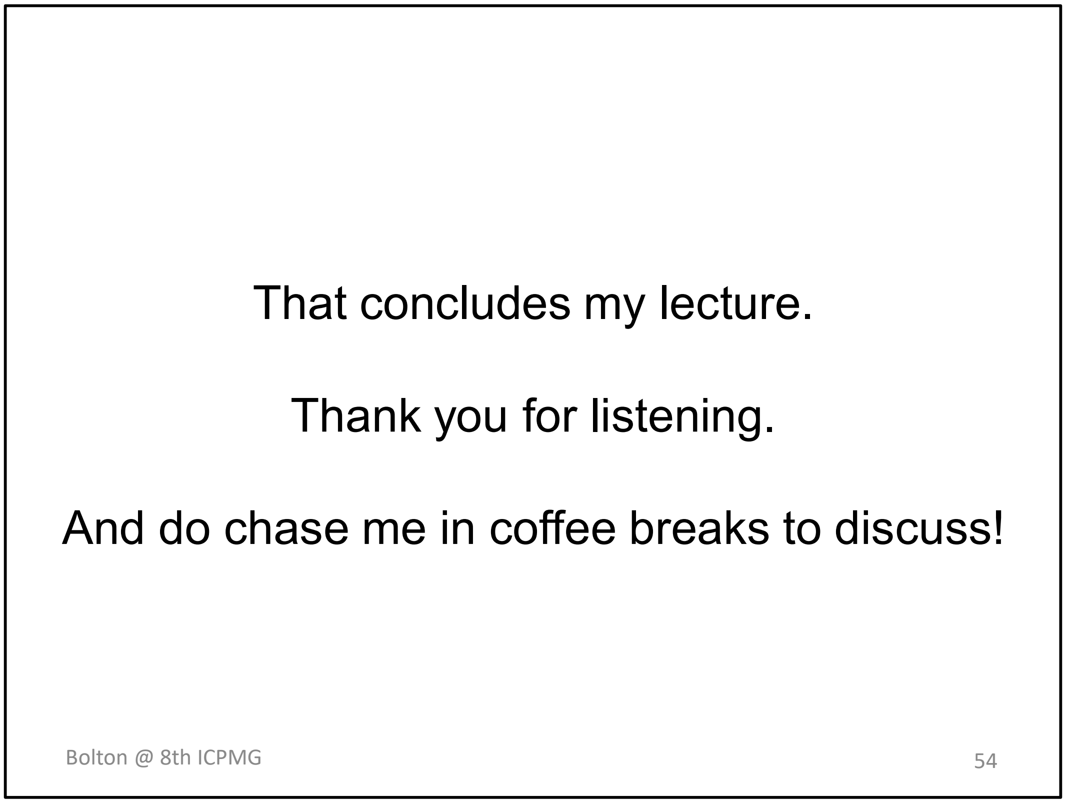
Of course, we also have to be careful that our scaling up is not naïve. But dimensional analysis often raises doubts that can be dispelled by further thought. Although further research would be useful on grain size effects, localisation, and fluid-granular interactions, there appears to be nothing standing in the way of the immediate application of the ideas I have introduced.



There are three modes of geotechnical activity relevant to this conference: science, technology and practice. Academics may pursue science through research. Specialists develop new technology in the light of new research and with a view to industry applications. And practitioners have to recognise the opportunities and overcome the difficulties presented by new projects, using whatever new technology may be available.

Science is meant to be falsifiable – observations should be capable of overturning any scientific theory, at least in theory. Technology is meant to be useful. And practice is meant to be robust in the face of uncertainty.

Between research and development is modelling, very often numerical modelling these days, but physical modelling is also available as we saw with Paul Shepley's tests where numerical modelling would struggle. Between technology and practice we find optimization, where the objectives are clear and an ideal solution has to be selected. And at the junction of practice and science there is diagnosis, encompassing the observation of unexpected events in centrifuge research and their assimilation into the discipline as new mechanisms of behaviour. But it is not only centrifuge testers who must diagnose. A matching function has to be served by practitioners – starting with pattern recognition, and moving on to the creation of a design. And increasingly, clients see the value of design-as-you-go based on construction monitoring and data processing, which is the full-scale equivalent of centrifuge testing. They will occasionally encounter the unexpected! So they should try to recruit a physical modelling PhD…



Wherever you are on my triangle, I hope you found something of interest. If so I may have succeeded in adding a fraction to the enthusiasm for centrifuge modelling that Andrew Schofield created over the last fifty years, and which was largely responsible for TC104, as it now is, being so lively and so very successful. Thank you for listening.