52nd Rankine Lecture: Australian version

Performance-based design in geotechnical engineering

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Performance

• Dictionary definition
  ➢ observable behaviour, success when measured against a standard, achievement

• Here, performance will be explored in 3 aspects
  ➢ performance in soil tests
  ➢ performance of soil structures
  ➢ performance of geotechnical designers
Performance in soil shear tests

- Triaxial, Simple Shear, Torsion, etc.
- Shear stress \( \tau_{13} = 0.5(\sigma_1 - \sigma_3) \)
- Shear strain \( \gamma_{13} = \varepsilon_1 - \varepsilon_3 \)
- e.g. undrained triaxial compression \( \tau_{\text{mob}} = 0.5q; \gamma_{\text{mob}} = 1.5\varepsilon_1 \)
- e.g. simple shear
- Volume change and excess pore pressure
- Failure: ductile or brittle
Ranges of strain: small, moderate and large

Undrained triaxial tests on kaolin sheared from various overconsolidation ratios (OCR)
Friction, dilatancy and “true cohesion”

Taylor / Schofield energy balance:
\[
\tan \phi_{\text{max}} = \tan \phi_{\text{crit}} + \tan \psi_{\text{max}}
\]

Bolton (1986) empirical equivalent:
\[
\phi_{\text{max}} = \phi_{\text{crit}} + 0.8 \psi_{\text{max}} \\
= \phi_{\text{crit}} + 3^\circ [l_D \ln (\sigma_{\text{crush}}/p') - 1]
\]
Friction, dilatancy and “true cohesion”

Taylor / Schofield energy balance:
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Application #1: Performance of clay slopes

- First example of performance of a “soil structure”.
- Seasonal slope movements and shallow failures.
- Effects of “live loading” from varying suction.
- Andy Take: modelling the weather in a centrifuge
- Shows how to avoid large ground movements.
Shallow slope failures after ~ 5 years

Typical motorway cutting with steep side slopes in clay
Clay embankment & track maintenance!

Old rail embankment of compacted clay
- courtesy Network Rail
Andy Take’s atmospheric chamber

8.7m embankments of stiff kaolin clay modelled at 60g
Humidity control

Cyclic period of 8000 seconds ≈ 1 year prototype
Pore pressure response

Air entry suction 125 kPa, so clay stays saturated
Pore pressure measurement

Seasonal boundary conditions

Rel. Humidity (%)

Elapsed time (x 10^4 s)

u_w (kPa)

Elapsed time (x 10^4 s)

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Take & Bolton (2011) Geotechnique
Pore pressures after 1st short wet season

Take & Bolton (2011) Geotechnique
Swelling during 1\textsuperscript{st} short wet season

Take & Bolton (2011) Geotechnique
Pore pressures after 1\textsuperscript{st} long dry season

Take & Bolton (2011) Geotechnique
Shrinkage in 1st long dry season

Take & Bolton (2011) Geotechnique
Cumulative strain $\gamma$ % after 1$^{st}$ year

Take & Bolton (2011) Geotechnique
Cumulative $\gamma$ % after 2$^\text{nd}$ short wet season

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Take & Bolton (2011) Geotechnique
Cumulative $\gamma$ % after 5 years

cumulative $-\varepsilon_v$ % also occurred in the same region

Take & Bolton (2011) Geotechnique
Accumulating damage $\gamma \%$ due to softening

Global slope failure at time $t=?$
Mobilisation analysis

- Make a modified slip circle analysis, to find how $\tau_{mob}$ varies with the seasons.
- Use $\tau_{max} = c' + \sigma' \tan \phi'$
- Iterate to find $c' \phi'$ for $FoS_{min} = 1$
- Mobilise $\phi'$ first, up to $\phi_{crit}$
- 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.
Finding instantaneous values of $c'_\text{mob}$, $\phi'_\text{mob}$
Cyclical mobilisation of strength

Figure 7.12. Seasonal mobilisation of soil strength, Spencer's method.
Cyclical mobilisation of \( c' \) causes softening

- Swelling is recoverable for \( \phi_{\text{mob}}' < \phi_{\text{crit}} \)
- Swelling, softening and slope “creep” is cumulative for \( c'_{\text{mob}} > 0 \)
Dilation, softening and “creep” leads eventually to local failure at the toe.
Analytical insight

• Recall the equation of work and dissipation:

\[ \tan \phi_{\text{mob}} = \tan \phi_{\text{crit}} - \frac{\delta \varepsilon^p_v}{\delta \gamma^p} \]

• If we mobilise supercritical strength \( \phi_{\text{mob}} > \phi_{\text{crit}} \), and if any plastic shear \( \delta \gamma^p \) occurs, some irrecoverable dilation \( \delta \varepsilon^p < 0 \) must also occur.

• We have seen that holding shear stress constant beneath a slope, while cycling pore pressures induce \( \phi_{\text{mob}} > \phi_{\text{crit}} \), does indeed lead to creep, softening and the eventual failure of clay slopes.
Application #1: Summary on clay slopes

• In order to avoid first-time failures following...
  ➢ slope creep due to seasonal wetting and drying,
  ➢ progressive softening towards critical states,
  ➢ and cracking leading to water ingress...

• Find the critical state friction angle of the soil, $\phi_{\text{crit}}$
  ➢ design to mobilise no more than $\phi_{\text{crit}}$ after a wet season
  ➢ use membranes, vegetation, and drains to ensure that suction remains high enough to provide $\phi_{\text{mob}} \leq \phi_{\text{crit}}$ in the wettest foreseeable event (or use flatter slopes)

• In which case neither $\phi_{\text{max}}$ nor $\phi_{\text{res}}$ are relevant.
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• In which case neither \( \phi_{\text{max}} \) nor \( \phi_{\text{res}} \) are relevant.
  ➢ so it is equally irrelevant to apply any partial factors

Skempton was right! Use the “fully softened strength” i.e. Schofield’s “Critical State” Now we know why!
Clays performing at moderate strains

• Paul Vardanega’s database of 19 clays
• Undrained strength mobilization $\tau_{\text{mob}}/c_u$
• Mobilization strain $\gamma_M$ at $\tau_{\text{mob}}/c_u = 1/M$
• Shapes of stress-strain curves
• Predictions of shear strain for $1.25 < M < 5$
• Reliability of shear strain predictions
• Long term volume changes and creep need to be added – see the written version…
Undrained shearing of clays: simple model

\[ \tau_{mob} \approx 0.5 \left( \frac{\gamma}{\gamma_{M=2}} \right)^b \]

\( \tau_{mob} \approx 0.5 \) \( \frac{\gamma}{\gamma_{M=2}} \)
Moderate mobilizations: $1.25 < M < 5$

115 tests on 19 clays

$\tau_{mob}/c_u$ vs $\gamma/\gamma_{M=2}$
Moderate mobilizations: power curves

\[ \tau_{mob}/C_u \]

regression: \( b = 0.6 \)

\[ \gamma/\gamma_{M=2} \]
Kaolin: $b \approx 0.35$ to $0.6$, increasing with OCR

\[ b = 0.011(OCR) + 0.371 \]
Kaolin: $\gamma_{M=2}$ increasing with OCR

So obtain $\gamma_{M=2}$ for the top and bottom of the key stratum and join the dots on a log-log plot versus depth.

$log_{10}(\gamma_{M=2}) = 0.680 log_{10}(OCR) - 2.395$
Deformability of London clay

mobilised shear strength (t mob) MPa

shear strain, $\gamma$ (%)
London clay: simple power-law model

\[
\text{OCR} \approx \frac{D_{eroded} + d}{d}
\]

\[
\gamma_{M=2} (\%) = -0.28 \ln(d) + 1.54
\]

Note: OCR approximation
Summary: clays at moderate strains

- Our log-log elastic / perfectly plastic model works well for high quality cores tested in compression.
- Also for rebound loops in PMTs with the same b-values, but different $\gamma_{M=2}$ values: anisotropy.
- So allow for anisotropy by selecting test modes.
- If a datum for zero strain is established, the only parameters are $c_u$ increasing with depth, and $\gamma_{M=2}$ reducing: so pick $\tau_{mob}$ to achieve a permissible $\gamma$.
- Volumetric yielding must also be avoided (e.g. by not exceeding the pre-compression).
Performance of soil structures

• Mobilizable Strength Design (MSD) in clays
• Assume an undrained geo-structural mechanism
  ➢ take maximum displacement $\delta_{\text{max}}$ as the key unknown
  ➢ invoke a compatible displacement field
  ➢ differentiate to get shear strains as a function of $\delta_{\text{max}}$
  ➢ invoke a representative power curve of stress v. strain
  ➢ deduce shear stresses as a function of $\delta_{\text{max}}$
  ➢ balance work and energy to solve for $\delta_{\text{max}}$

• Factor to account for creep and consolidation
Application #2: Braced excavations

• Much previous attention has been given to prop loads, but even well-propped excavations promote ground movements and wall bulging.
• Sidney Lam’s centrifuge model tests
• Tom O’Rourke’s wall bulging mechanism
• Deformation mechanism
• MSD energy balance
• Sidney Lam’s field database
• Dimensionless groups and design criteria
Nicoll Highway collapse, Singapore, 20-04-04

soft clay, bulging wall, 4 dead, tunnel diversion
Xianghu subway site, Hangzhou, 15-11-08

soft clay, bulging wall, 21 dead
Mechanisms observed in centrifuge tests at 60g

(a) Cantilever

(b) Prop at crest

(c) Multi-propped
Shear strains inside the bulging mechanism

\[ \gamma_{\text{average}} = \frac{w_{\text{max}}}{\lambda / 2} = \frac{2w_{\text{max}}}{\lambda} \]

\[ \gamma_{\text{average}} \approx \frac{2.3w_{\text{max}}}{\lambda} \]

\( \lambda \) reduces stage by stage

Assume sinusoidal bulge: O’Rourke (1993)
MSD of bulging wall by energy conservation

- Every excavation stage creates some bulge $\delta w_{\text{max}}$.
- Subsidence leads to a loss of potential energy $\delta P$.
- Soil deformation absorbs work $\delta W_{\text{soil}}$.
- The flexure of the wall absorbs work $\delta W_{\text{wall}}$.
- Conservation of energy demands:
  \[ \delta P = \delta W_{\text{soil}} + \delta W_{\text{wall}} \]
- Since $\delta P$, $\delta W_{\text{soil}}$ and $\delta W_{\text{wall}}$ can each be expressed in terms of maximum displacement $\delta w_{\text{max}}$, the energy conservation equation will calculate it by iteration.
Unit work calculations

Work done per unit volume of soil

\[ \delta W_{soil} \]

\[ \gamma_{average} \]

\[ \tau \]

\[ c_u \]

\[ c_{mob} \]

POWER CURVE

Work done per unit length of wall

\[ \delta W_{wall} \]

\[ M \]

FLEXURE

\[ M = EI\kappa \]

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Sidney Lam’s database of field case studies

• Nine authors reported on a total of 110 deep excavations in soft clay under 9 famous cities:
  
  - Bangkok (2 sites) Moh et al (1969)
  - Boston (5 sites) Whittle (1993)
  - Chicago (10 sites) Finno & Chung (1990)
  - Oslo (9 sites) Bjerrum & Landva (1966)
  - Shanghai (67 sites) Ma (2009)

• They provided stress-strain data...
Soft clays beneath 9 cities: fitting parabolas

Shear Strain, $\gamma$ (%)

Mobilization of undrained shear strength

$C_{mob}$

$C_u$

$\gamma_{M=2} = 0.25\%$  $\gamma_{M=2} = 0.75\%$  $\gamma_{M=2} = 1.25\%$

Mexico City Clay (Diaz-Rodriguez et al., 1992)
Bangkok Clay (MOH et al., 1969)
Oslo Clay (Bjerrum and Landva, 1966)
Boston Blue Clay (Whittle, 1993)
San Francisco Bay Mud (Hunt et al., 2002)
Chicago Glacial clay (Finno and Chung, 1990)
Shanghai Clay (X.F.Ma, Personal communication, Feb 2009)
Taipei Silty Clay (Lin and Wang, 1998)
Singapore marine clay (Wong and Brom, 1989)

$M=2$
MSD “predictions” using $c_u$ profile and $\gamma_{M=2}$

- MSD calculations follow construction sequence
- At each stage a spread-sheet iterates to balance energy.
- Variation is due to
  - authors’ $c_u$ profile
  - our estimate of $\gamma_{M=2}$
  - our selection of $\lambda$
  - workmanship

$R^2 = 0.91$
$COV = 0.25$

$w_{max,m}/w_{max,p} = 1$
$w_{max,m}/w_{max,p} = 0.7$
$w_{max,m}/w_{max,p} = 1.4$

error factor $< 1.4$
Turning MSD into dimensionless charts

• The MSD deformation mechanism with the power equation for mobilization offers algebra, non-dimensional groups and design charts.

• Define a normalized displacement factor:

\[
\psi = \frac{\gamma_{\text{average}}}{\gamma_{M=2}} = \frac{2 w_{\text{max}}}{\lambda_{\text{average}} \gamma_{M=2}} \approx \left( \frac{2}{M} \right)^2
\]

• If \( \gamma_{M=2} \) is known, field data of \( w_{\text{max}} \) and \( \lambda_{\text{average}} \) allow \( \psi \) to be calculated and therefore \( M \) to be estimated continuously during construction.
Do I have a problem with this D-wall?

Suppose it is known that \( \gamma_{M=2} = 0.75\% \)

\[
\left( \frac{2}{M} \right)^2 = \frac{2 \ w_{\text{max}}}{\lambda_{\text{ave}}} = \frac{1.5\%}{\gamma_{M=2}} = 2
\]

Then \( M = 1.4 \) and all is well just now.

But if the soil has \( \gamma_{M=2} = 0.5\% \)

\[
\left( \frac{2}{M} \right)^2 = \frac{2 \ w_{\text{max}}}{\lambda_{\text{ave}}} = \frac{1.5\%}{\gamma_{M=2}} = 3
\]

Then \( M = 1.15 \) and danger looms!
Non-dimensional chart of wall bulging

\[ \psi = \frac{\gamma_{\text{average}}}{\gamma_{M=2}} = \frac{2 w_{\text{max}}}{\lambda_{\text{average}} \gamma_{M=2}} \approx \left( \frac{2}{M} \right)^2 \]

- Database, 0<H/C<0.33
- Database, 0.33<H/C<0.66
- Database, 0.66<H/C<1.0
- MSD prediction, \( \gamma_{M=2} = 0.75\% \)
- \( H/C = 0.74 \)
- \( M = 1.25 \)
- \( H/C = 1.00 \)
- \( M = 1 \)

MSD prediction, \( u = 5\% \)
MSD prediction, \( u = 3\% \)
MSD prediction, \( u = 1\% \)

System Stiffness, \( \eta = \frac{E l}{\gamma w h^4} \)
Limiting $w_{\text{max}}$: the wall

- The maximum bending strain induced in a wall of thickness $d$ bulging $w_{\text{max}}$ over sinusoidal wavelength $\lambda$ is:

\[ \varepsilon_{\text{max}} = \pi^2 \frac{w_{\text{max}} d}{\lambda^2} \]

- So a 0.8 m thick diaphragm wall bulging 0.3 m over a 20 m wavelength gives $\varepsilon_{\text{max}} \approx 3 \times 10^{-3}$. Tensile cracking begins in concrete at $\varepsilon \approx 10^{-4}$, steel yields at $\varepsilon \approx 1.5 \times 10^{-3}$ and concrete crushes at $\varepsilon \approx 4 \times 10^{-3}$: Park and Gamble (2000).

- So the Nicoll Highway wall was approaching failure at the same time as the soil it was supposed to be supporting.

- The structural engineer must limit $w_{\text{max}}$ to maintain integrity.
Application #2: Summary on excavations

- MSD “predicted” the wall bulge in 110 braced excavations, in 9 cities, within a factor of 1.4.
- New dimensionless group $\psi$ organises field data, offering instant comparisons and warnings.
- Ground movements are proportional to $\gamma_M$, to the size of the mechanism and roughly to $1/M^2$.
- Demanding $M \geq 1.25$ limits the safe depth of excavation in nc clays: then use soil stabilization.
- MSD calibrated through databases of deformation offers a genuine basis for reliability.
Performance of geotechnical engineers

- What calculations do engineers actually make?
  - Limit equilibrium with a “safety factor”
  - Deformation / serviceability is an afterthought
- And how are safety factors decided?
  - Precedent, not evidence
  - Sometimes risky? Often wasteful?
- Can we set performance requirements instead?
  - MSD with a specified limit on deformation
- A case study on performance-based design…
Application #3: Bored piles in London clay

- Design against ULS using Eurocode 7
- MSD of straight-shafted bored piles
- Selecting appropriate value of $b$, $\gamma_{M=2}$
- Formula for pile head settlement
- Comparison with Dinesh Patel’s 1992 database
- So what should we design for: SLS, ULS?
“Code” design of bored piles

• Vardanega et al (2012) show that Codes typically require a lumped safety factor of 2.5.

• Design to EC7-UKNA: DA1-2 requires:

\[ E_d = G + 1.3V \leq \left( \frac{Q_s}{1.6} + \frac{Q_b}{2.0} \right) \frac{1}{1.4} \]

G, V are estimated permanent and variable loads; Q_s, Q_b are estimated shaft and base capacities; 1.3 is a load factor; 1.6 and 2.0 are material factors; and 1.4 is a model factor to “take account of the range of uncertainty in the results of the method of analysis”.

• Where do these factors come from?
Shaft resistance $Q_s : \alpha$-method

- Maximum shaft resistance $\tau_s = \alpha c_u$ where $c_u$ relates to a mean design line through scattered data of 100mm diameter triaxial tests or equivalent SPT correlation.
- Main reason for $\alpha$: brittle fall to critical state strength.
- Following Patel (1992) for London clay:
  - in CRP tests @ ~ 60 mm/h $\alpha \approx 0.60$;
  - in ML tests @ ~ $10^{-1}$ mm/h $\alpha \approx 0.45$;
- Main reason for reduced strength in slower tests:
  - creep/relaxation ~ 12% per x10 on strain rate
- If so, for 1mm/year ~ $10^{-4}$mm/h, $\alpha \approx 0.3$ which falls below the conventional value of 0.5 by factor 1.6.
Shaft resistance $Q_s$ : $\beta$-method

- Shaft resistance $\tau_s = \beta \sigma'_v$ where $\beta = K_s \tan \delta$.
  - Although $\sigma'_v$ is reliable if soil density and WT are known, there is some uncertainty over $K_s$ and $\delta$.
  - Although the soil may start at $K_0$, casting the concrete should send the lateral total contact stress to $\gamma_{conc} z$.
  - Although driving a pile in clay reduces $\delta$ to $\phi_{res}$, there is evidence for bored piles that $\delta \approx \phi_{crit}$.

- So with a water table at $z_w$ below ground surface, we can estimate $\tau_s$ at depth $z$:
  $$\tau_s = [\gamma_{conc} z - \gamma_w (z - z_w)] \tan \phi_{crit}$$
Q_s : \alpha versus \beta for London clay

- Take a typical London clay profile with:
  - $c_u = 50 + 7.5 z$ kPa (following Patel, 1992)
  - $z_w = 3$ m, $\gamma_{conc} = 23.5$ kN/m$^3$, $\phi_{crit} = 21^\circ$

- Calculate $\tau_s$ at $z = 10$ m (e.g. mid-depth of a pile)
  - Using $\alpha = 0.5$, $\tau_s = 0.5 \times 125 = 63$ kPa
  - Using $\tan \phi_{crit} = 0.38$, $\tau_s = 0.38 \times 166 = 63$ kPa

- Apparently, there need be little uncertainty in $\tau_s$!

- Does the partial safety factor of 1.6 (to control settlements?) really need bolstering by a further model factor of 1.4? See Vardanega et al (2012).
Analysis of shearing around a long pile shaft

For a rigid pile, take soil at mid-point, find \( \tau_0 \), deduce \( \tau \) at \( r \), find \( \gamma \) at \( r \) and then integrate to get pile settlement \( w \).

If shaft safety factor is \( F \), \( \tau_0 = \frac{\alpha c_u}{F} \)

For vertical equilibrium of concentric cylinders

\[
\tau = \tau_0 \frac{r_0}{r}
\]

But clay stress-strain satisfies:

\[
\frac{\gamma}{\gamma_{M=2}} = \left(\frac{2\tau}{c_u}\right)^{1/b} = \left(\frac{2}{M}\right)^{1/b}
\]

Vardanega, Williamson & Bolton (2012) Geotechnical Engineering
Settlement of bored piles in London clay

• Typical soil properties:
  - \( c_u = 50 + 7.5 \, z \) kPa (following Patel, 1992)
  - \( \gamma_{M=2} = 1.54 - 0.65 \, \log_{10}z \) %
  - \( b = 0.6 \)

• Pile settlement (using properties at \( z = 0.5L \)):
  - \( \frac{w_{pile/soil}}{D} \approx \frac{2.4 \, \gamma_{M=2}}{M^{1.67}} \)
  - \( \frac{\Delta w_{pile}}{D} \approx \frac{2}{M} \, \frac{c_u}{E_{pile}} \left( \frac{L}{D} \right)^2 \)
  - \( \frac{w_{pile\,head}}{D} \approx \frac{w_{pile/soil}}{D} + \frac{\Delta w_{pile}}{D} \)
Comparing MSD with pile test database

Patel (1992) Piling Europe

Patel

1/1.6

1/(1.6 \times 1.4)

MSD

<10 \text{ mm} / 1 \text{ m} \text{ so OK!}

Vardanega, Williamson & Bolton (2012)
Geotechnical Engineering
Application #3: Summary on bored piles

• MSD of bored piles in London clay seems to work, both for safety and settlement. A partial factor of 1.6 on shaft resistance should suffice, justified by apparent rate effects.

• Let us dispense with additional “model factor” of 1.4, saving 40% of piles and carbon emissions.

• Normally consolidated clays may require a further factor to allow for radial stress relief due to consolidation: use an appropriate database!
Professional practice: 3 rhetorical questions

• Should we rely on engineers who simply follow the old design methods and safety factors – rooted in the 1960s?
  ➢ Or do we prefer engineers who embrace science?
• Do we need a “Rumsfeld” factor of ignorance?
  ➢ Or should we just pay appropriate insurance premiums?
• Can we identify best-practice in risk-management?
  ➢ If we do, will the industry be motivated to implement it, with the safety blanket of existing Codes in place? And will the client notice if we don’t bother?
Conclusions: Ground deformations rule!

- Avoidance of large strains typically requires $\sigma'_\text{mob} \leq \sigma_{\text{max}}$ and $\phi_{\text{mob}} \leq \phi_{\text{crit}}$ (i.e. no $c'$ !)
- Prediction of ground deformations then requires the measurement of a soil stiffness parameter: for moderate strains in clay find $\gamma_{M=2}$.
- Deformation mechanisms have been verified for simple slopes, retaining walls, and foundations.
- These mechanisms plus power curves for clays offer design formulae and dimensionless charts.
- MSD allows designers to limit deformations.
Conclusions: Rethink Limit State Design!

• Discard vague and contradictory definitions of ULS and SLS; base decisions on deformations.

• At Working Limit States, check for acceptable structural distortion; e.g. set \( (\Delta/L)_{WLS} \approx 1/2000 \).

• At Extreme Limit States, assure safety through structural continuity; e.g. set \( (\Delta/L)_{ELS} \approx 1/400 \).

• Specify a reliability level for \( \Delta/L \leq (\Delta/L)_{\text{limit}} \); e.g. \( P_{\text{exceedence}} = 5\% \) at +1.65 St. Dev.

• This approach would be objective and verifiable: Performance Based Design (PBD)
Conclusions: Discard wasteful safety factors!

• Lecturers should not introduce any arbitrary FoS!
• We might be able to save 40% of materials and carbon emissions at a stroke (e.g. bored piles).
• Time Magazine of Monday August 3rd 1959:
  
  Hard-pressed U.S. railroads figure their featherbedding bill at $500 million a year. Each diesel engine must carry a fireman as a holdover from the days of steam locomotives—though he does almost nothing.

• Abolish the fireman safety factor!