CE394M: Finite Element Analysis in Geotechnical Engineering

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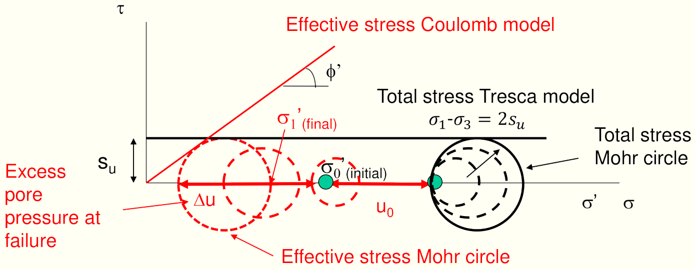
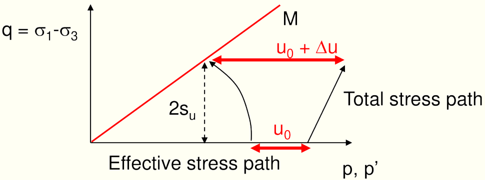
Overview

1. Total vs Effective stress
2. Nicoll Highway Collapse, Singapore
   - The Collapse
   - Post collapse investigation
3. Project on FEA of an excavation

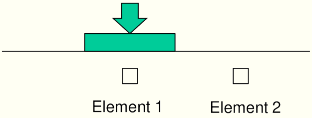
TSP v ESP footing: Tresca failure



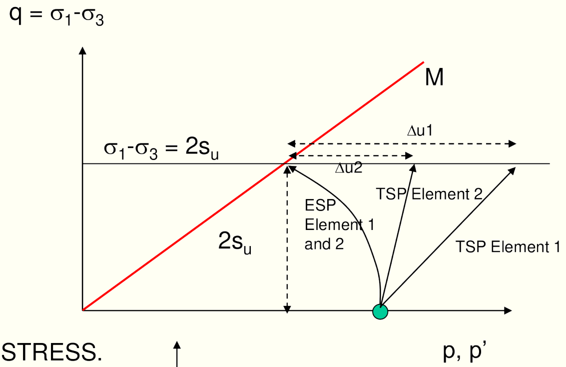


Tresca criteria: $\sigma_1 - \sigma_3 = 2s_u$

TSP v ESP footing: Tresca failure



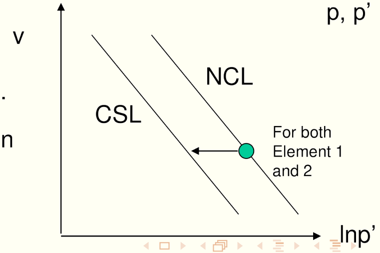
Let's assume $K_0 = 1$
But the same argument can be made for $K_0 \neq 1$



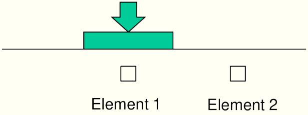
Stress equilibrium is in TOTAL STRESS.

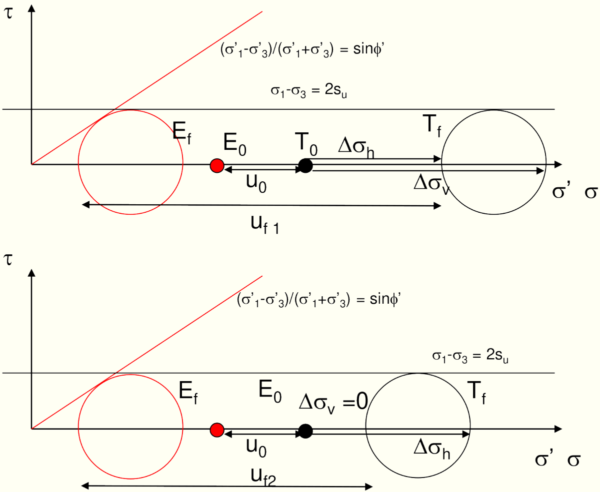
1. Total stress – Both elements fail when $\sigma_1 - \sigma_3 = 2s_u$. Use this for stress equilibrium.

2. Effective stress – Both element fail when q=Mp' but excess pore pressures need to be assigned separately for each element for stress equilibrium.



TSP v ESP footing: Tresca failure





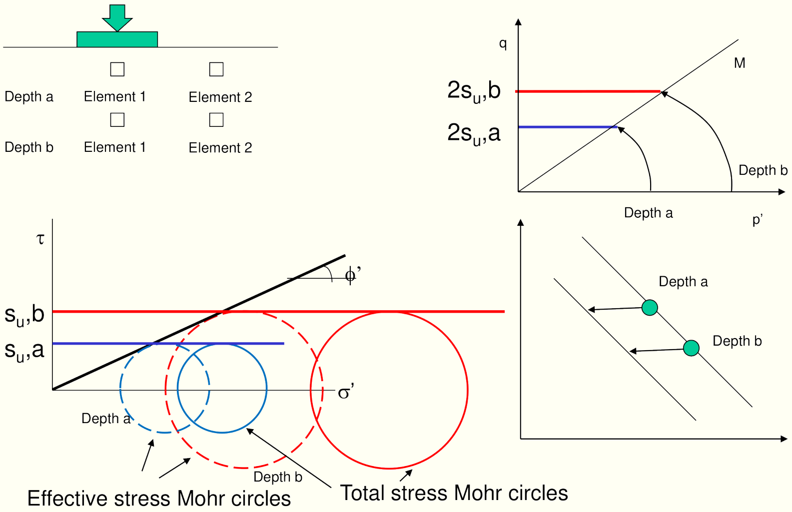
Both elements use the same total stress or effective stress failure criterion.

But for effective stress analysis, excess pore pressure ($u_{f1} \neq u_{f2}$) need to be computed separately.

No need to do this for the total stress analysis.

16

TSP v ESP footing: Tresca failure

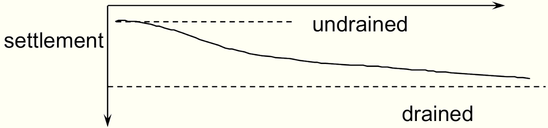


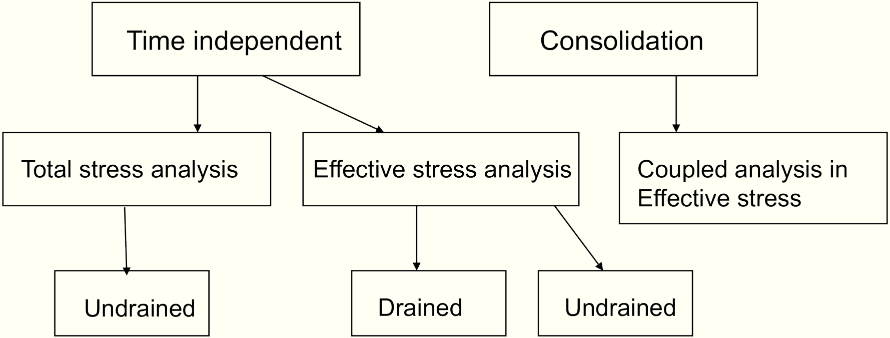
However, s_u needs to be assigned at different depths even for the same soil.

TSP v ESP footing: Tresca failure

- Better! Conduct Total Stress Analysis: $s_u = \sigma_1 - \sigma_3$
  - But s_u needs to be defined at different locations even if it is the same soil.
  - We can do quick undrained (UU) tests to obtain spatial variation of s_u. Hence, practical.
  - No information on pore pressure, so cannot assess the long-term consolidation deformation behavior.
- Effective analysis $= (\sigma_1' - \sigma_3')/(\sigma_1' + \sigma_3') = \sin \phi'$:
  - If it is the same soil, can use the same soil properties (such as ϕ') and need to conduct CU tests.
  - But need to compute the excess pore pressure at different locations.
  - This is difficult: only a good effective stress constitutive model that can predict the excess pore pressure correctly can do this.
  - If pore pressure profile can be computed, then it can be used to evaluate the subsequent consolidation process.

Pore-pressure analysis in geotechnical engineering





Pore-pressure analysis in geotechnical engineering

1. **Drained analysis**
   - No excess pore pressure - high permeable soils
   - All the loads will be transferred to the soil skeleton
   - Long-term condition - mostly interested in displacements
2. **Undrained analysis** - low permeable soil
   - Loads will be carried by both soil skeleton and pore pressure
   - No volume change
   - Short-term condition - mostly interested in stresses- undrained failure
3. **Consolidation analysis**
   - Transition from undrained condition to drained condition.
   - Check the movement of the system with time.
   - Time consuming - but correct stress paths and this can be important when the soil behaves plastically (stress path dependent)
   - Undrained analysis can be performed by making the time step small.

Drained analysis - Effective stress

1. Need to assign initial effective stresses before the analysis.
2. Can use any effective stress model: Elastic, Mohr-Coulomb/Drucker Prager and Cam-Clay models.
3. If plasticity models are used, need to update the effective stresses at each increment:

$$\sigma'(new) = \sigma'(old) + D(\text{soil skeleton})d\varepsilon$$

4. Very common.

Undrained analysis - Total stress

- Excess pore pressure cannot be calculated.
- Effective stress state of the soil cannot be examined.
- Elastic model is commonly used for deformation
  1. Use undrained stiffness E_u and strength parameters s_u
  2. Poisson's ratio close to 0.5 with a drained simulation
  3. Properties can vary with depth. (K_0 varies)
  4. Consolidation analysis has no effect and should not be performed.
- Von-Mises model is used for modeling undrained shear strength of clays. (C_u or s_u and undrained friction $\phi_u = 0$)
- Can assign different stiffness and strength at different depths explicity by assigning different model parameters at different depths.

Undrained analysis - Effective vs Total stress

- Effective stress: E' and ν'.
- Total stress: E_u and ν_u
- No volume change:

$$\nu_u = 0.5; (K_u = E_u/(1 - 2\nu_u)/3 = E_u/0 = \infty)$$

- Pore fluid cannot sustain shear stresses. Soil skeleton carries the shear stresses $\tau(orq)$.

$$G' = G_u; \quad G' = E'/(1+\nu')/2 \quad G_u = E_u/(1+\nu_u)/2$$
$$E'/(1+\nu')/2 = E_u/(1+0.5)/2$$
$$E_u = 1.5E'/(1+\nu')$$

- In finite element analysis, $\nu_u = 0.5$ cannot be used. Use $\nu_u = 0.49$ or 0.495. But be careful with *mesh locking* problem.

Undrained analysis - Effective stress

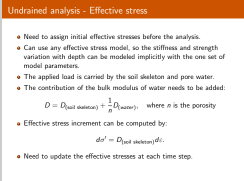
- Need to assign initial effective stresses before the analysis.
- Can use any effective stress model, so the stiffness and strength variation with depth can be modeled implicitly with the one set of model parameters.
- The applied load is carried by the soil skeleton and pore water.
- The contribution of the bulk modulus of water needs to be added:

$$D = D_{(\text{soil skeleton})} + \frac{1}{n} D_{(water)}, \quad \text{where } n \text{ is the porosity}$$

- Effective stress increment can be computed by:

$$d\sigma' = D_{(\text{soil skeleton})} d\varepsilon.$$

- Need to update the effective stresses at each time step.



$$u = K_w \cdot \varepsilon_{v,water}$$

Where, K_w is the Bulk modulus of water $\approx 2 \times 10^6 kN/m^2$.

$$\begin{aligned}
\varepsilon_{v,water} &= \Delta V_w / V_w \\
\varepsilon_{v,soil} &= \Delta V_T / V_T = \Delta V_w / V_T \quad \text{(assuming solid is incompressible)} \\
\frac{\varepsilon_{v,soil}}{\varepsilon_{v,water}} &= \frac{\Delta V_w / V_T}{\Delta V_w / V_w} = \frac{V_w}{V_T} = n \quad \text{(porosity for s = 100\%)}
\end{aligned}$$

Therefore,

$$u = K_w \cdot \frac{\varepsilon_{v,soil}}{n} = (K_w / n) \cdot \varepsilon_{v,soil}$$

Undrained analysis - Effective stress

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- Effective stress increment can be computed by:

$$d\sigma' = D_{(\text{soil skeleton})} d\varepsilon.$$

- Need to update the effective stresses at each time step.

Consider equivalent (K_w/n) in soil model. Separate total and effective stresses:

$$
\begin{aligned}
\sigma_m &= K_u \cdot \varepsilon_{v,soil} \quad K_u \text{undrained bulk modulus of soil} \\
\sigma'_m &= K' \cdot \varepsilon_v \\
u &= \sigma_m - \sigma' = (K_u - K') \cdot \varepsilon_v \\
K_w/n &= K_u - K'
\end{aligned}
$$

Undrained analysis - Effective stress

- Need to assign initial effective stresses before the analysis.
- Can use any effective stress model, so the stiffness and strength variation with depth can be modeled implicitly with the one set of model parameters.
- The applied load is carried by the soil skeleton and pore water.
- The contribution of the bulk modulus of water needs to be added:

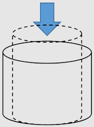
$$D = D_{(\text{soil skeleton})} + \frac{1}{n} D_{(water)}, \quad \text{where } n \text{ is the porosity}$$

- Effective stress increment can be computed by:

$$d\sigma' = D_{(\text{soil skeleton})} d\varepsilon.$$

- Need to update the effective stresses at each time step.

- $\Delta\sigma = D_{Total}\Delta\varepsilon = (D'_{Effective} + D_{water}/n)\ \Delta\varepsilon = \Delta\sigma' + \Delta u_{excess}$
- Elastic Analysis
- **Effective stress point of view**
  - $D'^{Effective}$ – (E' and ν') or (K' and G')
  - D^{water} – Large bulk modulus K_{water} and G_{water} =0 or ν_{water} ≈0.5.
  - Compressibility is dominated by K_{water} (>>K'), whereas shear behavior is dominated by G(>>G_{water} =0).
- **Total stress point of view**
  - Combination of $D'_{Effective}$ and D_{water}
  - E_u and ν_u = 0.5 (constant volume)



(E_u and $\nu_u \approx 0.5$) Mixture	=	(E' and ν') or (K' and G) Soil	+ $\frac{1}{n}$	(E_{water} and ν_{water}≈0.5) or (K_{water} and G_{water}=0) Water

Effective stress approach or undrained (A)

- Effective stiffness and effective strength parameters are used.
- *Pore pressures are generated*, but may be **inaccurate** depending on the model.
- Undrained shear strength is *not* an input parameter but an outcome of the constitutive model. The resulting shear strength must be checked against known data!
- Consolidation analysis can be performed after the undrained calculation, which *affects the shear strength*!

Equivalent effective stress approach or undrained (B)

- Effective stiffness parameters and *undrained strength parameters* are used.
- *Pore pressures are generated*, but may be hightly **inaccurate**.
- Undrained shear strength is an input parameter.
- Consolidation analysis should not be performed after the undrained calculation, s_u must be updated, if consolidation is performed anyway!

Methods of undrained analysis for Mohr-Coulomb clay

undrained analysis	material type	deformation parameters	strength parameters	initial conditions
Total stress	Non-porous / drained	E_u, ν_u	$c_u, \phi_u = 0$	$K_{0,u}$
Effective stress	Undrained (triaxial parameters)	E', ν'	c', ϕ'	K_0
Equivalent Effective stress	Undrained (strength profile)	E', ν'	c', ϕ'	K_0

Consolidation analysis - Effective stress

- Use Biot's 3D consolidation theory
- Pore pressure and displacement are computed at each time step.
- Need to use effective stress model
- Need permeability
- Lots of computational time
- More realistic. Undrained, partially drained, drained depending on the loading condition, drainage condition, permeability of soil.
- Stress path followed is correct, which should provide a good strain estimate when plasticity models are used.

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FE Disclaimer

IMPORTANT WARNING AND DISCLAIMER

PLAXIS is a finite element program for geotechnical applications in which soil models are used to simulate the soil behaviour. The PLAXIS code and its soil models have been developed with great care. Although a lot of testing and validation have been performed, it cannot be guaranteed that the PLAXIS code is free of errors. Moreover, the simulation of geotechnical problems by means of the finite element method implicitly involves some inevitable numerical and modelling errors. The accuracy at which reality is approximated depends highly on the expertise of the user regarding the modelling of the problem, the understanding of the soil models and their limitations, the selection of model parameters, and the ability to judge the reliability of the computational results. Hence, PLAXIS may only be used by professionals that possess the aforementioned expertise. The user must be aware of his/her responsibility when he/she uses the computational results for geotechnical design purposes. The PLAXIS organization cannot be held responsible or liable for design errors that are based on the output of PLAXIS calculations.

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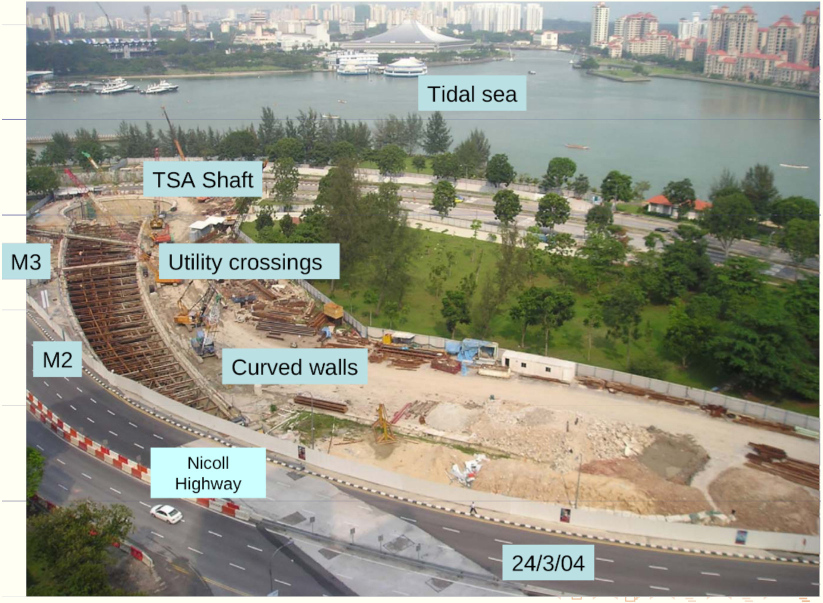
Nicoll Highway, Singapore



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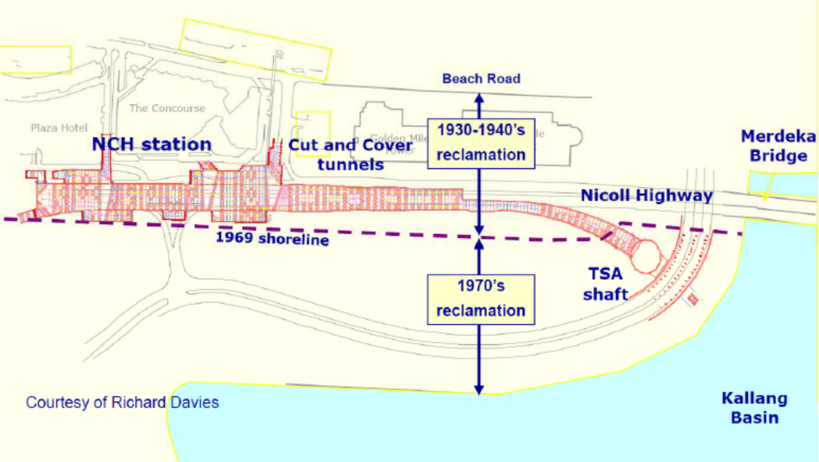
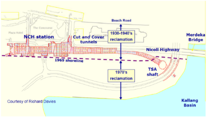
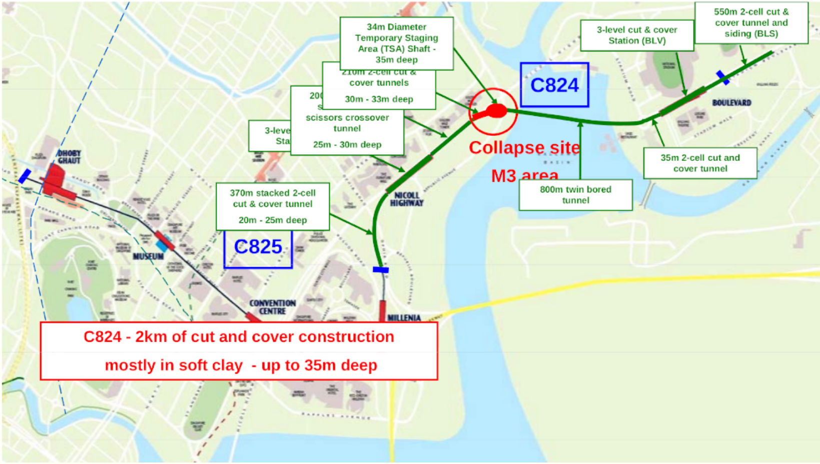


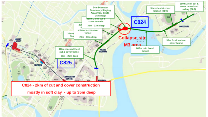
Nicoll Highway, Singapore



The tunnels were constructed on reclaimed lands. Reclamation was done in two stages. Northern part of the site was reclaimed over 50 years ago and the southern part was reclaimed about 20 years ago. Therefore the top soil of the collapse site comprised of fill material. Underneath the top layer there were layers of soft clays (marine clay and estuarine clay deposits) to a depth of 37-40 meters. Beneath these layers were layers of sands, silts, clays and an old alluvium formation. Ground water table was generally about 2 meters below the surface.

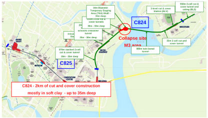
Nicoll Highway, Singapore



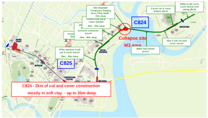


The Mass Rapid Transit (MRT) rail network system is an integral part of Singapore's public transport system. The MRT network involved construction of bored and cut and cover type tunnels. On the 20th of April 2004 at around 3:30 pm the temporary lateral support system of the cut and cover tunnel near the Nicoll Highway collapsed, resulting in the formation of a 100 ft. deep cave, which spread across six lanes of the Nicoll Highway. The collapse killed four people and injured three. The incident resulted in disruption of the gas, water and electric lines, which affected nearly 15000 people in the area. Two spans of a nearby bridge had to be demolished and reconstructed due to the damage in soil conditions the collapse had done in the nearby areas. One of the chief reasons for the failure of the temporary lateral support system was due to the overestimation of the undrained shear strength capacity of the soil.

Nicoll Highway, Singapore

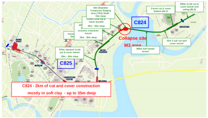


Construction of cut and cover tunnel involves building the tunnel structure inside an excavation. After the completion of the tunnel structure the excavation is covered with backfill material. Cut and cover type tunnels are typically constructed for depths of up to 30-40 feet. There are two approaches to the construction of a cut and cover tunnel (FHA Handbook 2009, pp 5-1-5-12). a) The Bottom-Up Method. b) The Top-Down Method.



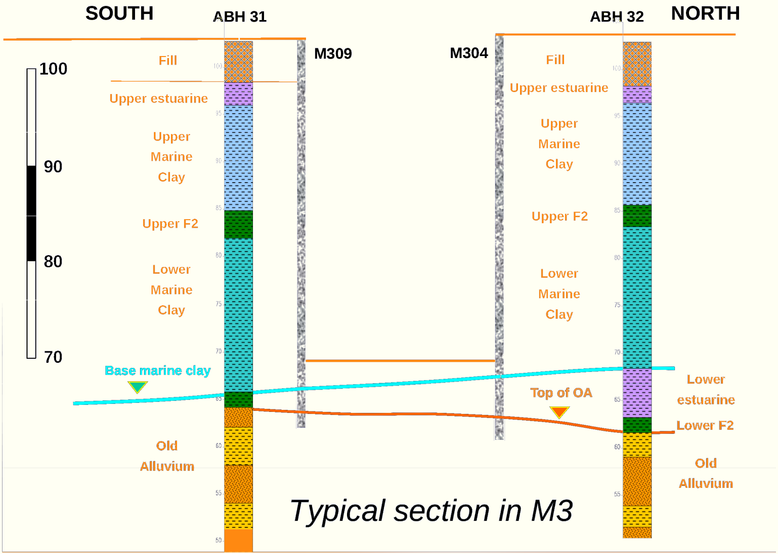
Bottom-Up method Bottom-up method involves excavation of a trench from the surface. The sides of the excavation can be sloped or vertical. The tunnel is then constructed in the trench and after the completion of the tunnel the trench is backfilled. Generally in urban areas due to limited availability of space the tunnel is often constructed with a vertical excavation supported by excavation support system. Excavation support system can be temporary or permanent. Some of the temporary excavation systems are sheet pile walls with multi levels of bracing, soldier piles with lagging walls, tieback support systems. Permanent excavation support systems form the part of the final tunnel structure. Some of the permanent excavation support systems are slurry walls, tangent pile wall support and soldier pile tremie concrete.

Nicoll Highway, Singapore

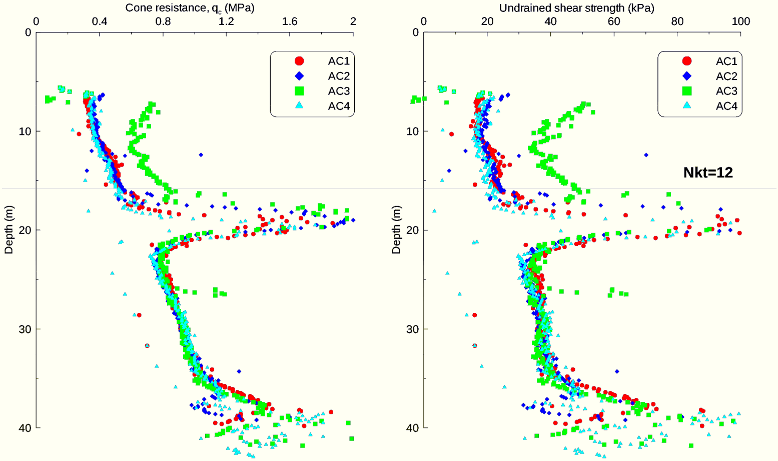


Top-Down method Top-down method involves the construction of the tunnel walls, which are made using slurry walls, secant pile walls. After the construction of the tunnel walls the roof slab is constructed on the ground and connected with the tunnel walls. The roof slab is then covered and the surface of the ground can be used. Rest of the tunnel is excavated below the protection of the roof slab and the tunnel walls. After the excavation the rest of the tunnel can be finished and the floor slab can be constructed. This method is challenging to construct but the ground surface above the tunnel can be restored earlier. Bottom-Up method was used to construct the part of the cut and cover tunnel near Nicoll highway. Diaphragm walls supported the sides of the excavation and multi-levels of bracing were constructed as the excavation progressed.

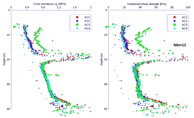
Soil profile



Soil profile

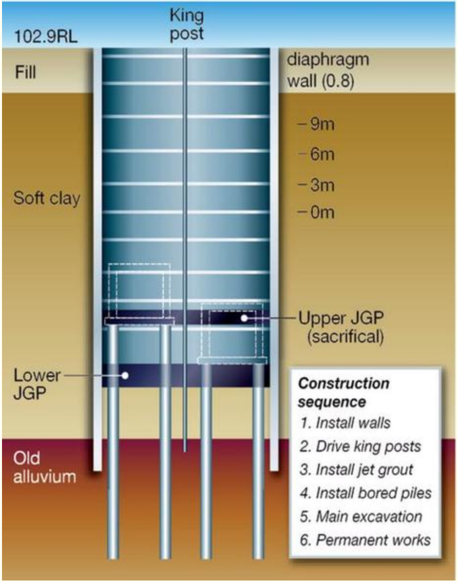


Soil profile

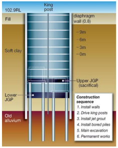


The investigation included 14 boreholes and cone penetration tests along with collection of soil samples and testing them in laboratories. After the contract was awarded an extensive testing was conducted which included 72 boreholes and cone penetration tests to determine the alignment of the tunnel and get data for the purpose of design (COI 2005, pp 16-35).

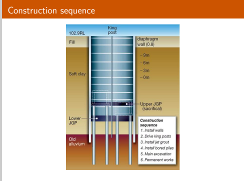
Construction sequence



Construction sequence



For design purposes diaphragm walls were divided into 40 different wall sections. Wall sections were designed based on the worst soil parameters obtained from the nearest borehole. The temporary retaining wall system consisting of diaphragm walls and struts was modeled, analyzed and designed using Plaxis, which is a widely used geotechnical modeling software based on finite element method. Soil layers were modeled using Mohr-Coulomb soil model with effective stress strength parameters. **A load factor of 1.2** was considered for obtaining the design forces on the diaphragm. **Diaphragm wall designs were optimized based on the wall movement criteria, with maximum allowable movement being 200 mm at any depth of the wall and 40 mm at the diaphragm wall toe.**



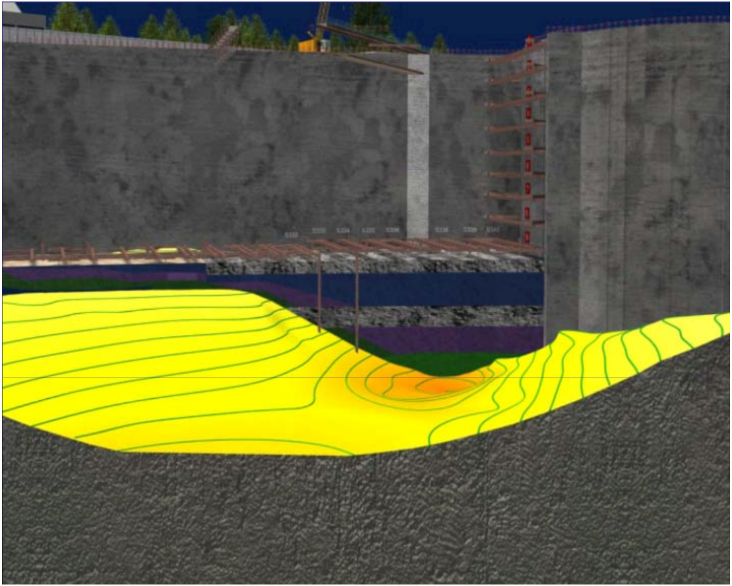
There were two layers of interlocking jet grout piles. The upper layer of the jet grout pile was 1.5 m thick and was temporary and the lower layer of the jet grout pile was 2.5 m thick and formed the base of the tunnel. Jet grout pile layers were built to minimize the deflection of the walls while the tunnel was being excavated. Bored piles were constructed to support the rail boxes. Excavation was supported by a system of steel king post and 10 levels of struts placed at 4m center to center. As the excavation progressed the struts were constructed and before the construction of the 10th level of strut the temporary layer of the jet gout pile was removed.

The entire process of tunnel construction was monitored by thousands of geotechnical instruments including settlement markers, inclinometers to monitor the soil and wall deflections, vibrating wire piezometers, strain gauges and load cells. The instruments installed in the failed part of the tunnel section provided data, which helped to understand the reasons for the collapse.

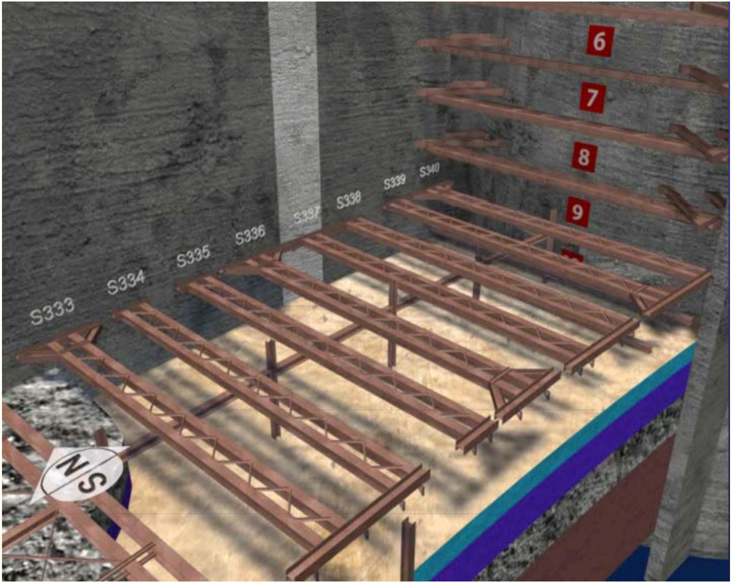
Nicoll Highway: Excavation

South side 13 March 2004

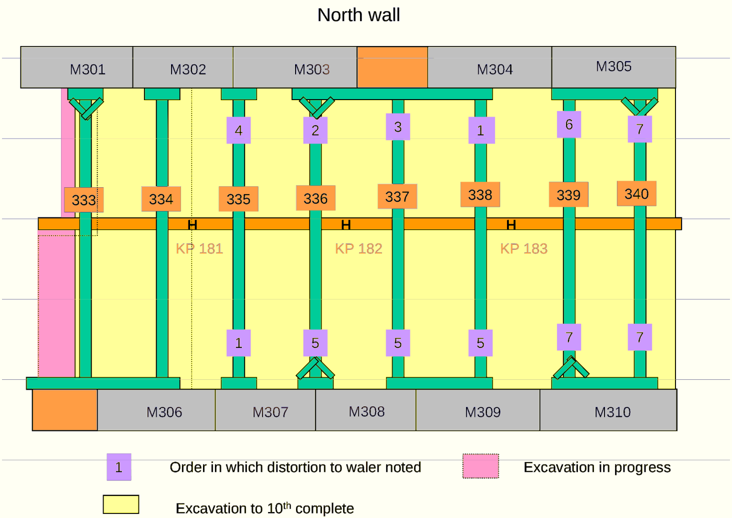
Excavating 10th level of struts



On the morning of collapse



On the morning of collapse



Location of walers, splays and Dwall gaps

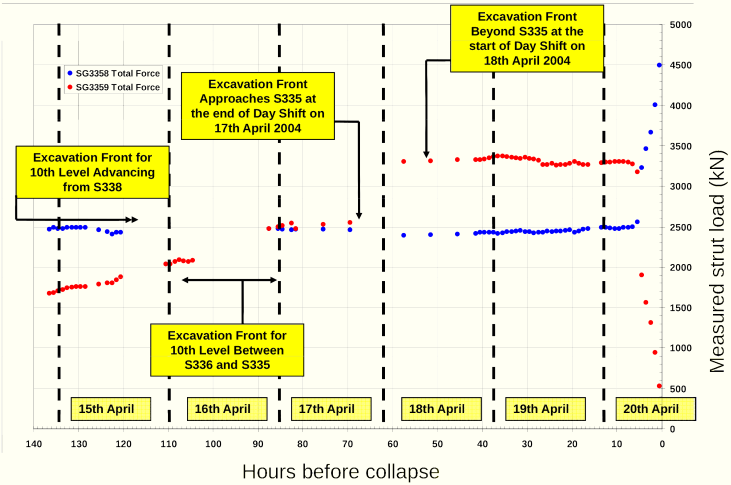
On the morning of collapse

Strut 338 North side

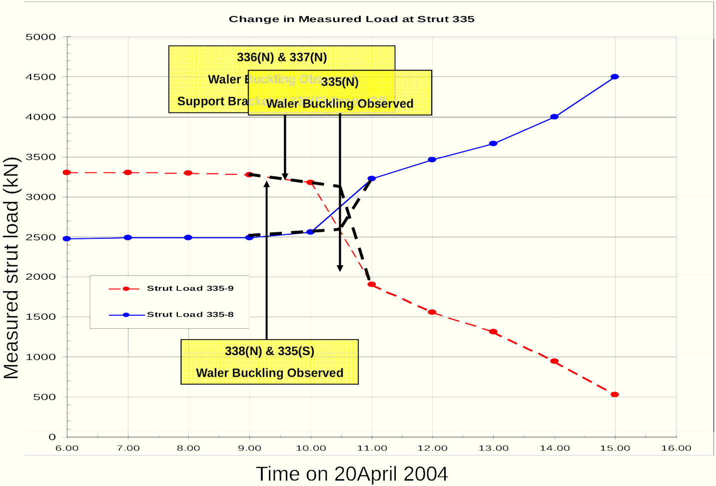
On the morning of collapse

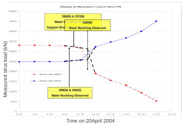
Strut 335 Sorth side

Hours before collapse



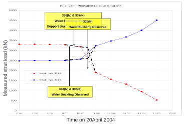
Hours before collapse





In the meanwhile the excavation progressed to about the 10th level of struts. The temporary layer of jet grout piles below the 10th level which was constructed to add to the stability of the excavation was now removed to finish the excavation of the tunnel. Two more temporary struts were added to stabilize the excavation. By 19th April the excavation of the tunnel was completed and the last bit of the tunnel section connecting it with the other section of the tunnel was cut out and the tunnels were connected. On the day of the collapse the 20th of April at around 8:00 am the workers at the site heard sounds from the multi-level strut system. The strange sounds were investigated by the senior engineers of the contractor and they found that a lot of the waler-strut connections had yielded. They immediately instructed everyone to leave the worksite.

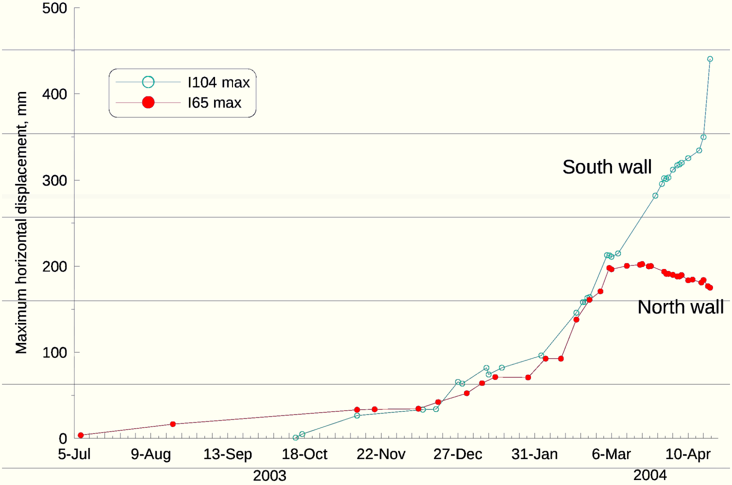
Hours before collapse



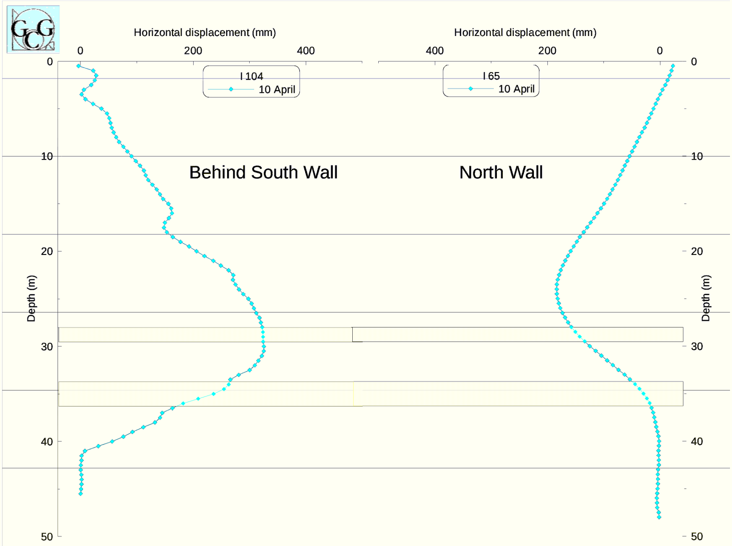
By noon engineers from the owner's side were also present at the worksite and they along with the contractor's engineers decided to pour concrete at the 9th level of the strut to stabilize the excavation and prevent it from caving in. By 3:30 pm the temporary system gave away and the tunnel had caved in.

The trends were consistent with there being yielding of the 0th level strut-waler connection when the exvavation passed beneath but with no further significant changes in load in eiter the 9th or 8th level struts until the collapse was initiated.

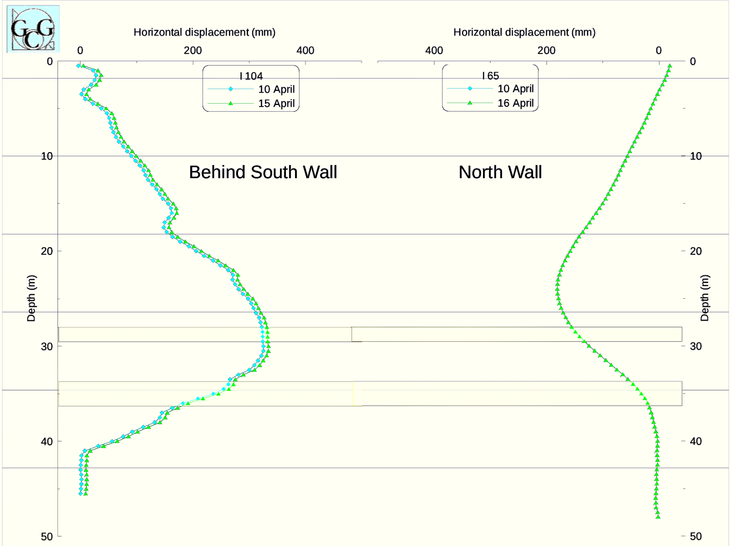
Leading up to the collapse: Inclinometer



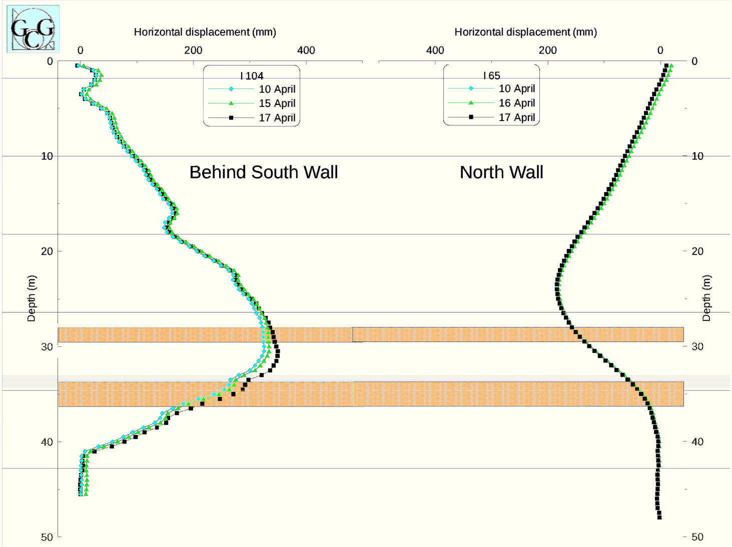
Leading up to the collapse: Wall displacements



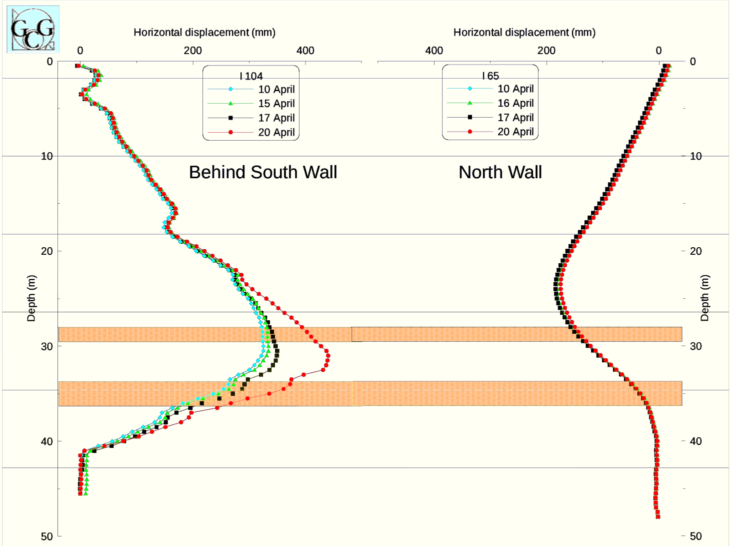
Leading up to the collapse: Wall displacements



Leading up to the collapse: Wall displacements



Leading up to the collapse: Wall displacements



The collapse

The collapse

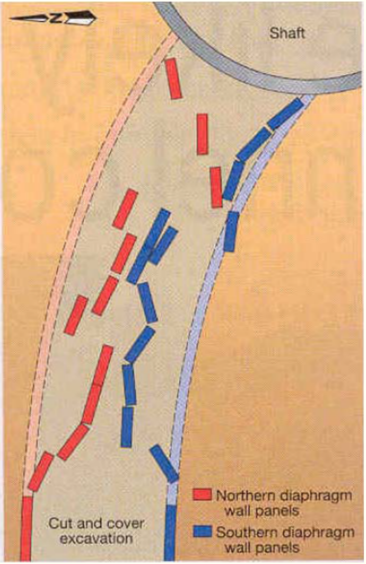
The collapse

The collapse

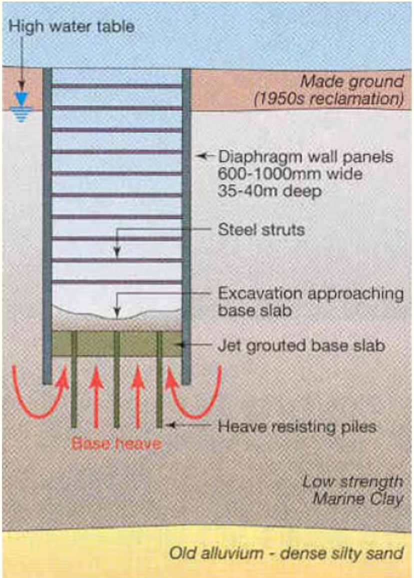
The collapse

Post collapse

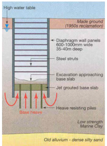
Post collapse



Reasons for collapse



Reasons for collapse

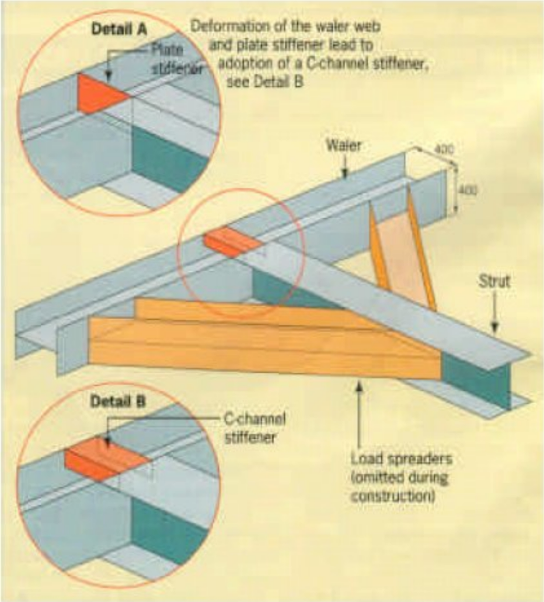


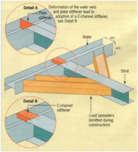
1. Problem with jet grouting at the base of the slab.
2. Struts design - connectors
3. Use of effective stress parameters to do an undrained excavation.

Strut design: Replacing plate-stiffener with C-channel

Strut design: Waler connection

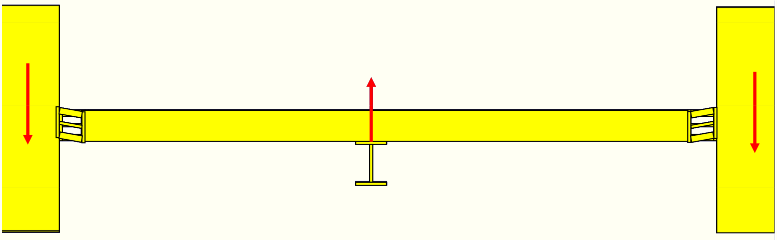
Strut design: Waler connection





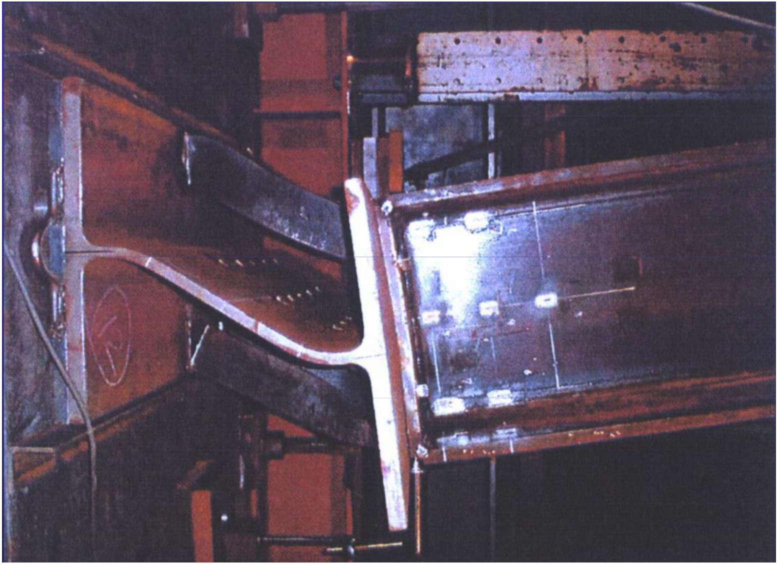
Another chief reason for the failure was the under design of the waler-strut connection. Designers misinterpreted the **stiff bearing length for the C-channels as per BS 5950 to 400 mm instead of using 65 mm**. Waler connections with C-channels were designed with the effective length factor of 0.7, where the end conditions were unrestrained and a factor of 1.2 should have been used. This resulted in axial design capacity that was about 70% of the assumed design load for the connection (COI 2005, pp 1-16). Also in some locations the splays in the waler strut connection had been omitted during construction (NCE, 2005). These design and construction errors resulted in the failure of the 9th level strut-waler system. The under designed diaphragm wall could not resist the redistributed loads as the 9th level strut-waler system failed resulting in the collapse of the tunnel.

Strut design: Relative vertical displacements

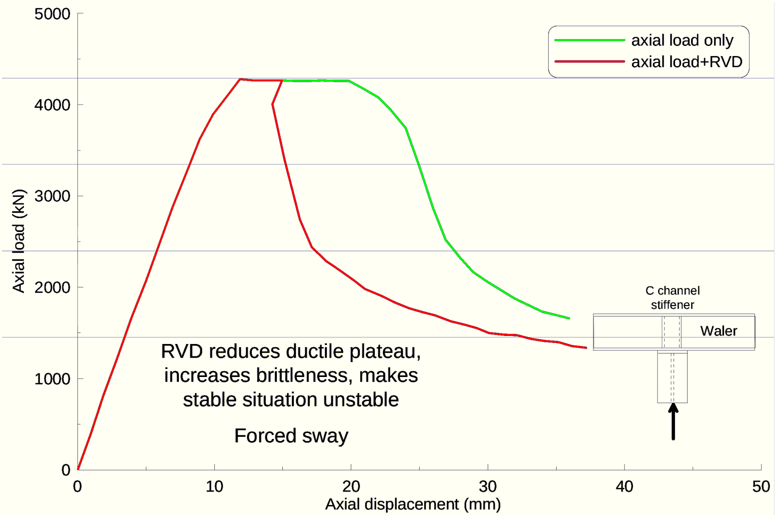


Relative vertical displacement between the King Post and the Dwall

Strut design: C-channel

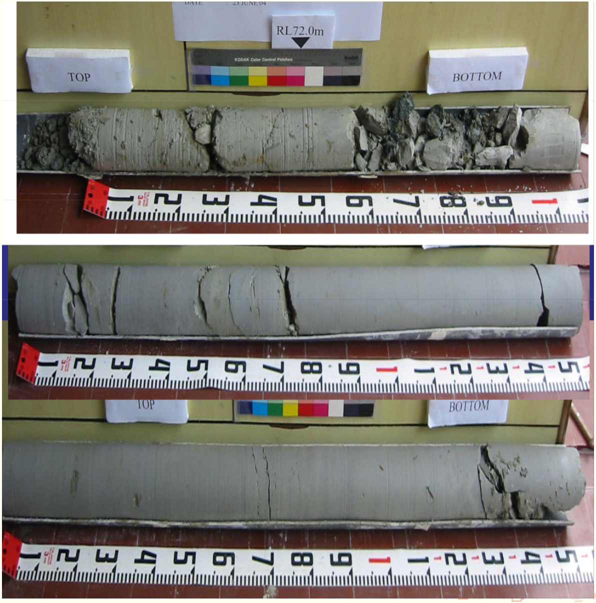


Strut design: C-channel relative vertical displacement

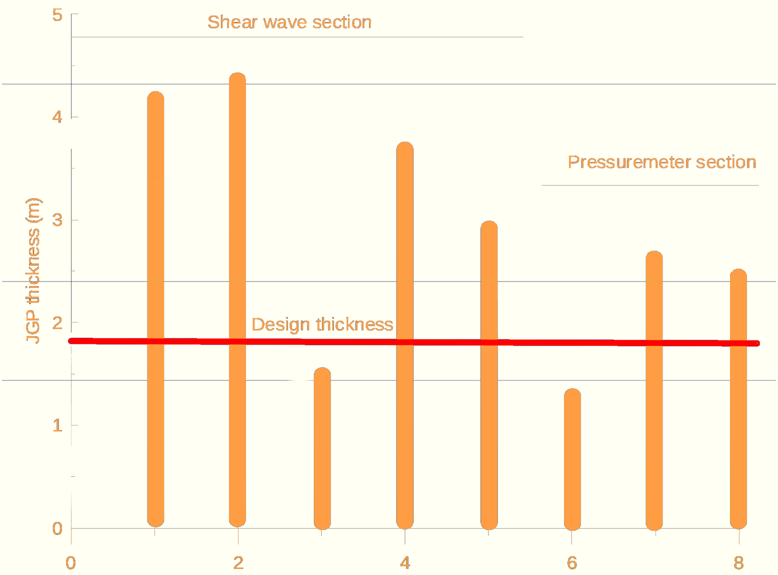


Excavation of sacrificial jet grout

Quality of jet grouting



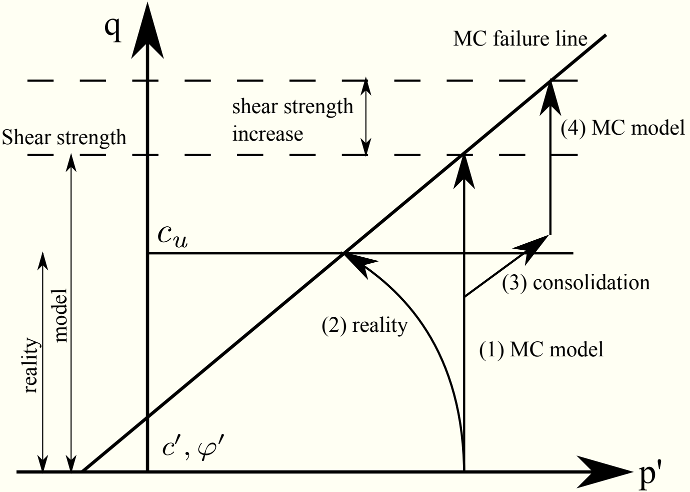
Quality of jet grouting



Undrained analysis

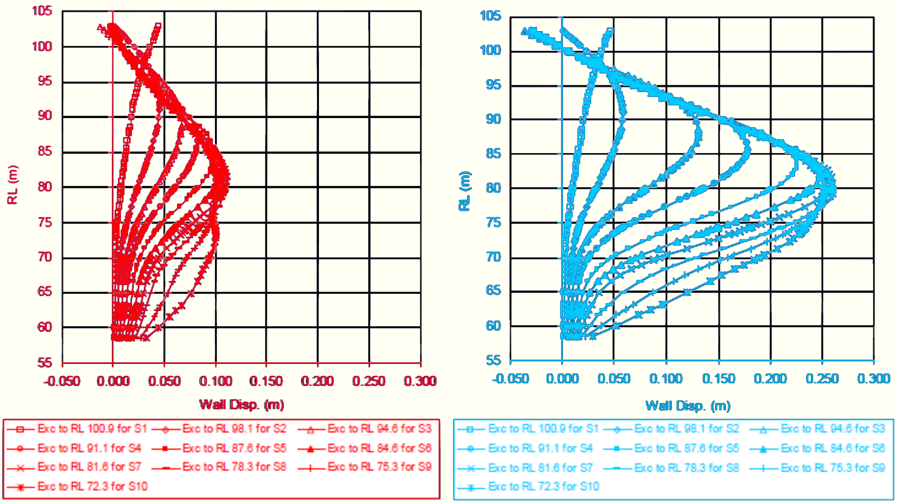
- Method A and Method B refers to 2 alternatives modeling of undrained behaviour in Plaxis.
- **Method A is an effective stress analysis** of an undrained problem it assumes an isotropic elastic behavior and a Mohr-Coulomb failure criterion.
- As a result mean effective stress p' is constant until yield.
- Method A was being applied to marine clays which were of low over-consolidation or even under-consolidated because of recent reclamation.

Undrained effective stress analysis



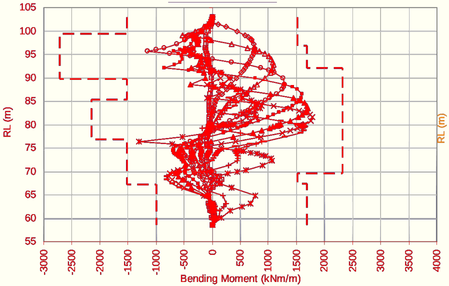
Effect of depth on Su

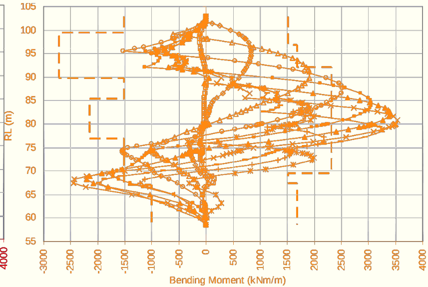
Wall displacements: Effective stress vs Undrained strength



Method A vs Method B

Bending moments: Effective stress vs Undrained strength





Method A vs Method B

Undrained effective stress analysis

- Method A over-estimates the undrained shear strength of normally and lightly overconsolidated clays
- Its use led to a 50% under-estimate of wall displacements and of bending moments and an under-estimate of the 9 th level strut force of 10%
- The larger than predicted displacements mobilised the capacity of the JGP layers at an earlier stage than predicted

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One of the major incidents that happened prior to the collapse was in the tunnel launch shaft area in August 2003. Tunnel launch shaft was located at the eastern end of the cut and cover tunnels. During the excavation of the tunnel launch shaft at about the 7th level of struts severe deflections of the diaphragm walls was observed which exceeded the design limit. Excessive ground settlement in the order of 400 mm was observed in a stadium nearby the south diaphragm wall (COI 2005, pp 36-54). Vertical cracks in some of the diaphragm wall panels were observed. Further excavation of the tunnel launch shaft was stopped immediately.

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A back analysis was performed for the diaphragm walls based on effective stress approach and it was determined that the walls were not capable of withstanding the load if the excavation had proceeded. So the contractor constructed a jet grout pile layer and temporary walls to support the deflected wall and added additional struts to support the excavation before the excavation was resumed.

The critical error of using the effective stress approach in place of total stress approach overestimates the undrained shear strength of the marine clays which results in underestimation of the deflection, bending moment in the walls. This explanation was pointed out by the engineering advisory panel of the owners. They strongly suggested the contractors to reanalyze using the total stress approach. **The contractor declined to reanalyze claiming that their design was sound and the fact that the excavation in other areas based on the effective stress approach behaved as designed.** They eventually agreed to do a back analysis based on the TSA.

2020-04-07

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The contractors were reluctant to use the total stress approach for back analysis as they felt that the method gave large wall deflections, which would result in thicker walls. They also justified the use of effective stress approach based on the fact that the analysis provided similar results as the actual deflection of the walls. However this was partially true as for smaller depths the effective stress method would give close displacement results but as the excavation in the marine clay progresses the deflections calculated become highly un-conservative and wouldn't match the actual deflection and so was still a very unsafe basis for design. Although this was not very clear at that point of time so the owners agreed to the further excavation of the various tunnel sections.

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The critical error of using effective stress method for analysis and subsequent back analysis became more evident as the excavation progressed deeper. As the excavation of different sections of the tunnel were carried out in parallel, the excavation didn't reach to such levels were the diaphragm walls would have failed. But the tunnel section near the Nicoll highway had to be excavated quicker than other sections as the construction and excavation work hindered with the traffic flow on the highway.

Undrained effective stress analysis

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- The larger than predicted displacements mobilised the capacity of the JGP layers at an earlier stage than predicted

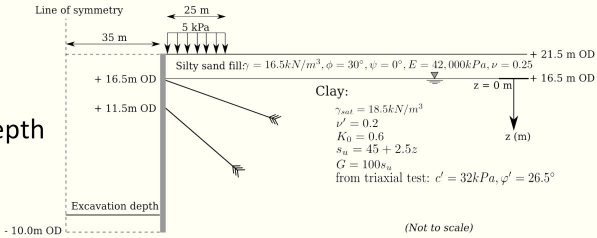
As the excavation of the M3 type walls progressed to about the 6th level of strut the deflection of the walls exceeded the design limit in February of 2004. The section was back analyzed using effective stress approach and the design limits were revised. The excavation progressed with the new revised deflection limits. By the end of March of 2004 the deflection of the walls had exceeded the revised deflection limits. A second back analysis was done based on the same effective stress method and the design deflection limits were further revised and remediation measures were taken to keep the excavation open. The owners accepted the second back analysis and permitted the further excavation on 3rd April 2004. Between 3rd April and the 20th April there were periods of time when the inclinometer readings were not monitored by the contractor.

Overview

1. Total vs Effective stress
2. Nicoll Highway Collapse, Singapore
   - The Collapse
   - Post collapse investigation
3. Project on FEA of an excavation

Total stress evaluating varying K_0

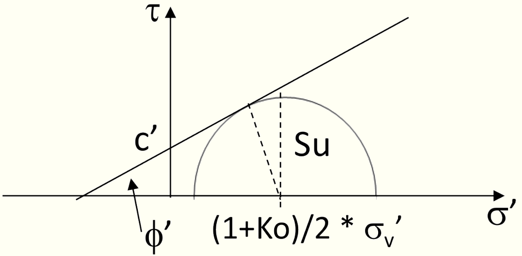
- *K_0* Varies with depth



Depth	σ_v	u	σ_v'	$k_0\sigma_v'=\sigma_h'$	σ_h	K_0	K_0 Layer
0	82.5	0	82.5	49.5	49.5	0.6	
6.5	202.75	65	137.75	82.65	147.65	0.728	0.664
11.5	295.25	115	180.25	108.15	223.15	0.756	0.742
16.5	387.75	165	222.75	133.65	298.65	0.77	0.763

Effective stress Method A

- Define c' and ϕ' in terms of the real effective stress parameters, assuming zero dilation.
- ν' is the effective Poisson ratio
- E and K_0 should be '*effective stress*' based.

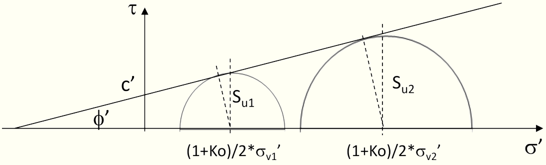


Mohr-Coulomb failure criteria: $\tau = c' + \sigma' \tan\phi'$

$$s_u = c' \cos\phi' + (1 + K_0)/2 \cdot \sin\phi' \cdot \sigma'_{v0}.$$

Effective stress Method B

- Define c' and ϕ' in terms of the "*equivalent*" effective stress parameters, with zero dilation. Parameters are defined based on the strength profile with depth.
- ν' is the effective Poisson ratio
- E and K_0 should be '*effective stress*' based.



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$$s_u = c' \cos\phi' + (1 + K_0)/2 \cdot \sin\phi' \cdot \sigma'_{v0}.$$

Methods of undrained analysis for Mohr-Coulomb clay

undrained analysis	material type	deformation parameters	strength parameters	initial conditions
Total stress	Non-porous / drained	E_u, ν_u	$c_u, \phi_u = 0$	$K_{0,u}$
Effective stress	Undrained (triaxial parameters)	E', ν'	c', ϕ'	K_0
Equivalent Effective stress	Undrained (strength profile)	E', ν'	c', ϕ'	K_0