CE394M: Finite Element Analysis in Geotechnical Engineering

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Overview

1. Geotechnical FEA
   - Element types
   - Discretization
   - Boundary conditions
   - Errors in FEA

2. Nicoll Highway Collapse, Singapore
   - The Collapse
   - Post collapse investigation

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.

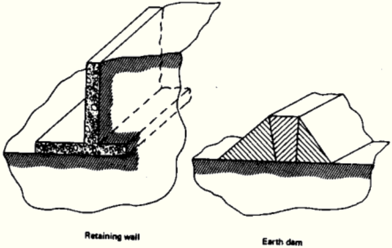
3

Consistent system of units

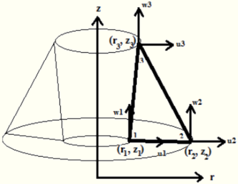
	SI			
Length	m	m	m	cm
Density	kg/m^3	10^3 kg/m^3	10^6 kg/m^3	10^6 g/cm^3
Force	N	kN	MN	Mdynes
Stress	Pa	kPa	MPa	bar
Gravity	m/sec^2	m/sec^2	m/sec^2	cm/s^2
Stiffness*	Pa / m	kPa / m	MPa / m	bar / cm

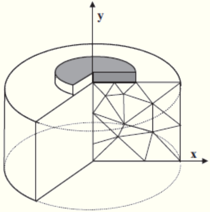
Problem definition

Plane Strain: Strain normal to x-y plane is zero $\varepsilon_z = 0$ and shear strains γ_{zy} and γ_{zx}



Axisymmetric:

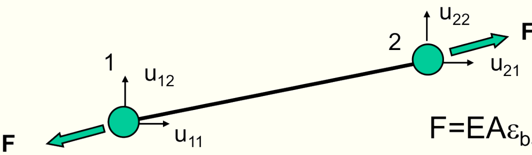




Take advantage of symmetry

1D Finite Elements: Bar element

Two node element with axial stiffness only (no flexural or shear resistance).Examples of this type of structure are cables, reinforcing bars.



$F = EA\varepsilon_{bar}$

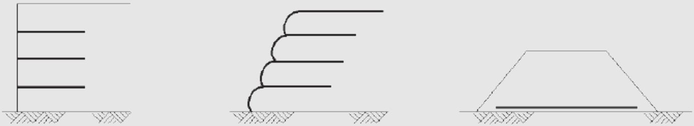
Two node element with axial stiffness only (no flexural or shear resistance).Examples of this type of structure are cables, reinforcing bars.

1. Node to Node: are springs that are used to model ties between two points.
2. It's not recommended to draw geometry line at position where node-to-node anchor is to be placed.
3. It's a 2 node elastic spring element with normal stiffness (Spring constant)
4. Element can sustain both tensile forces (anchors) as well as compressive forces (struts).
5. Fixed-End anchors: Modelling of struts or props to sheet pile walls.

1D Finite Elements: Bar element

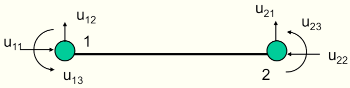
Two node element with axial stiffness only (no flexural or shear resistance).Examples of this type of structure are cables, reinforcing bars.

1. Geogrids are slender structures with a normal stiffness but no bending stifness.
2. Geogrids can only sustain **Tensile forces** and no compression!
3. Structures involving geotextiles.



1D Finite Elements: Beam element

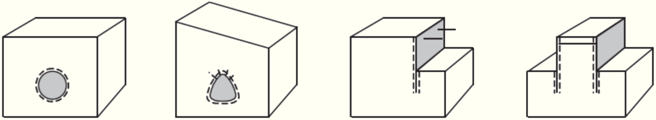
two node structure element with axial and bending stiffness (no transverse shear deformation). Three degrees of freedom for 2D beam element (1, 2 displacements and a moment). Examples are sheet pile walls, structural foundation beams, structural facing for reinforced soil walls.

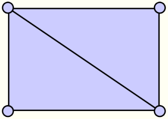


$F_a = EA\,\varepsilon_a$

$V = -EI(u_{12}-u_{21})/L^3 - 6EI(u_{13}+u_{23})/L^2$

$M = EI(u_{13}-u_{23})/L$



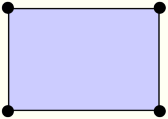


3 nodes element

linear variation of displacement within the element = constant strain in the element

$$d_1 = \alpha_1 + \alpha_2 x + \alpha_3 y$$

$$d_2 = \beta_1 + \beta_2 x + \beta_3 y$$

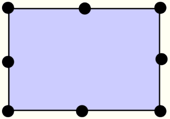


4 nodes element

linear variation of displacement in both x and y directions

$$d_1 = \alpha_1 + \alpha_2 \xi + \alpha_3 \eta + \alpha_4 \xi\eta$$

$$d_2 = \beta_1 + \beta_2 x + \beta_3 y + \beta_4 \xi\eta$$



8 nodes element

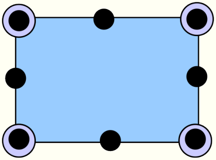
quadratic variation of displacement in both x and y directions.

$$d_1 = \alpha_1 + \alpha_2 \xi + \alpha_3 \eta + \alpha_4 \xi^2 + \alpha_5 \xi\eta + \alpha_6 \eta^2 + \alpha_7 \xi^2\eta + \alpha_8 \xi\eta^2$$

$$d_2 = \beta_1 + \beta_2 \xi + \beta_3 \eta + \beta_4 \xi^2 + \beta_5 \xi\eta + \beta_6 \eta^2 + \beta_7 \xi^2\eta + \beta_8 \xi\eta^2$$

2D/3D Finite elements

2D Consolidation element

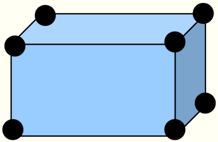


Pore pressure and displacements

Displacements

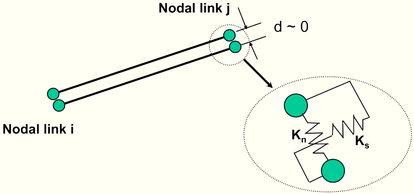
Linear variation of pore pressures and quadratic variation of displacements in x and y directions

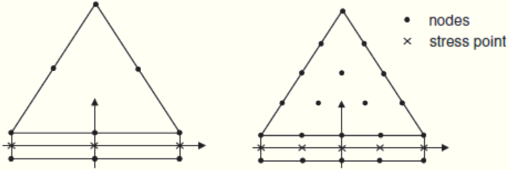
8 node 3D brick element



Linear variation of displacements in x, y and z directions

Interface element



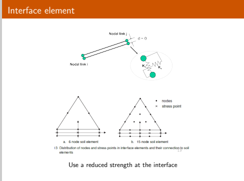


a. 6-node soil element

b. 15-node soil element

13 Distribution of nodes and stress points in interface elements and their connection to soil elements [13]

Use a reduced strength at the interface



This element allows relative displacement between elements. It is capable to model soil/structure interface conditions, shear planes within a soil mass. The element is 'fictitious' four node element made up of two independent nodal links. Each link consists of two nodes connected by a normal and shear spring as shown below. The stiffness of the springs can be non-linear, modelling frictional slip behaviour. The thickness of the element is assumed to be negligibles

Interface elements for Soil Structure Interactions

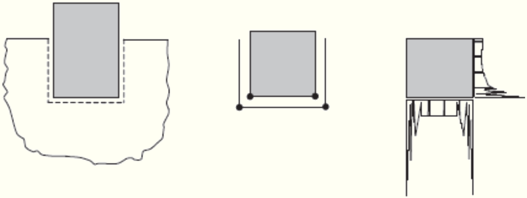


Figure 3.14 Inflexible corner point, causing poor quality stress results

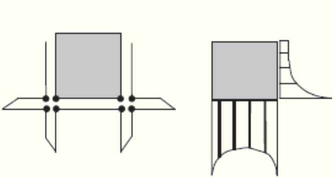
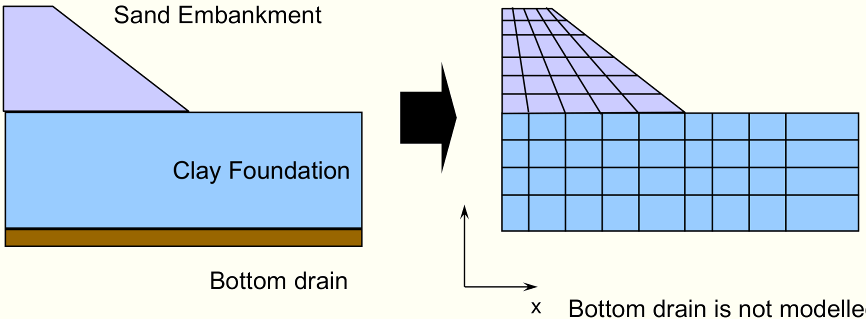


Figure 3.15 Flexible corner point with improved stress results

FE discretization



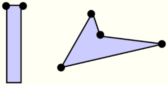
Bottom drain is not modelled because it is assumed to behave as a rigid material.

FE discretization

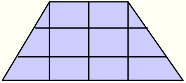
(1) Use big graph paper / draw in boundaries

(2) Keep elements as “Square” as possible.

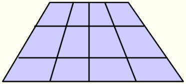
Do not want :



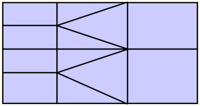
(3) For 4 node elements, try to avoid triangular



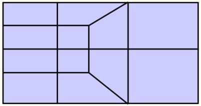
Not recommended



Better

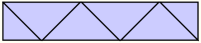


Not recommended



Better

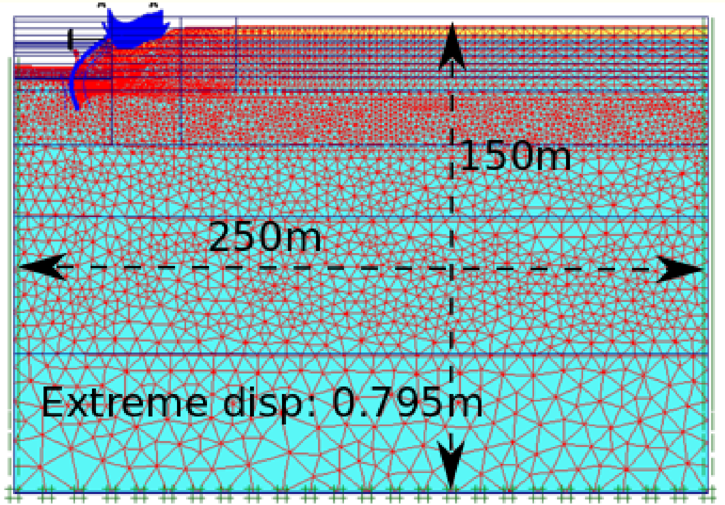
(4) Use regular mesh



Not recommended

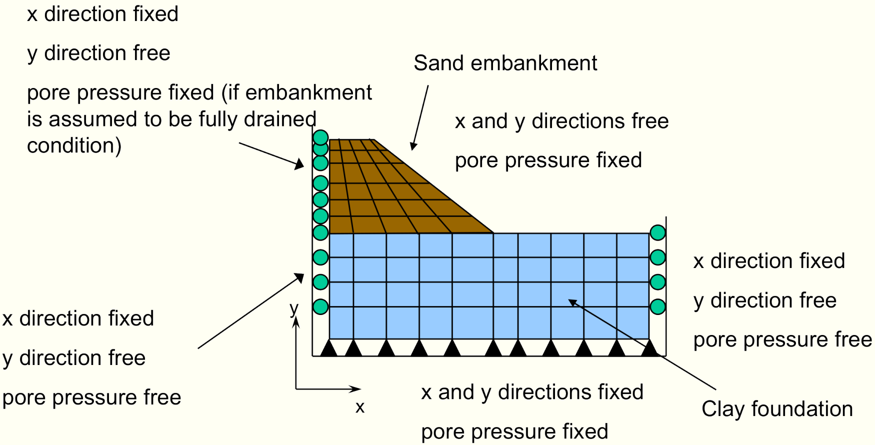
Better

FE discretization: Refining



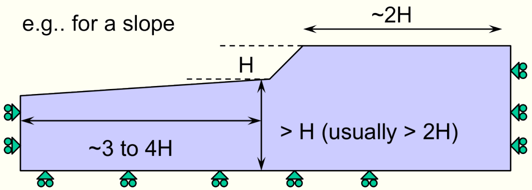
Avoid large jumps in element size: size jump should be < 3

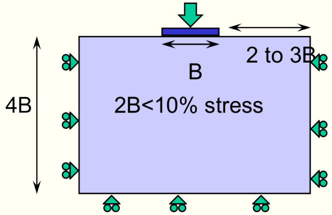
FE boundary conditions



Pore pressure free = no water flow perpendicular to the boundary

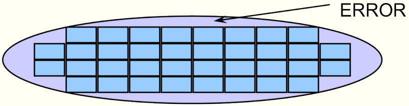
FE boundary conditions





FE errors

(1) Creating elements



(2) Numerical errors - e.g. finite element approximation (variation within the elements), numerical integration and time integration, mesh locking

(3) Constitutive model - dominant error for us.

Liner elastic model?

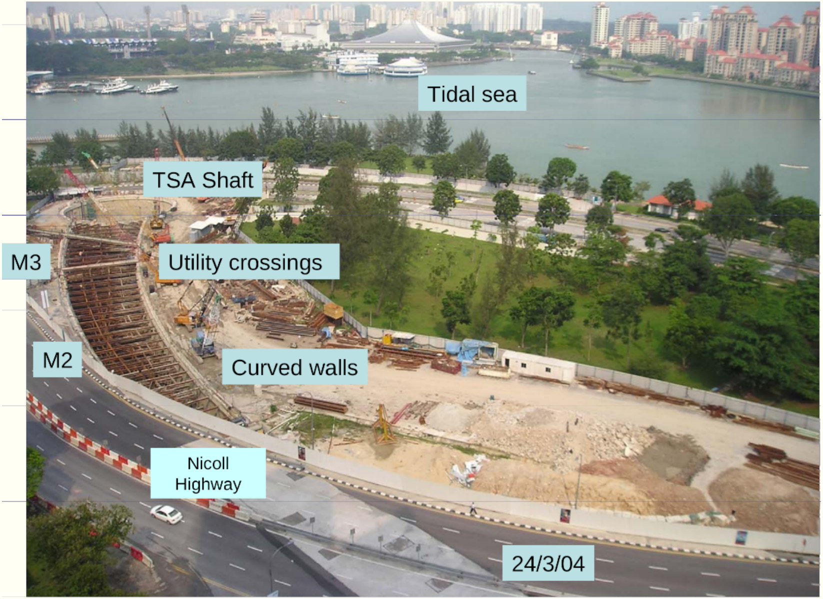
Non-linear elastic model?

Cam-clay model?

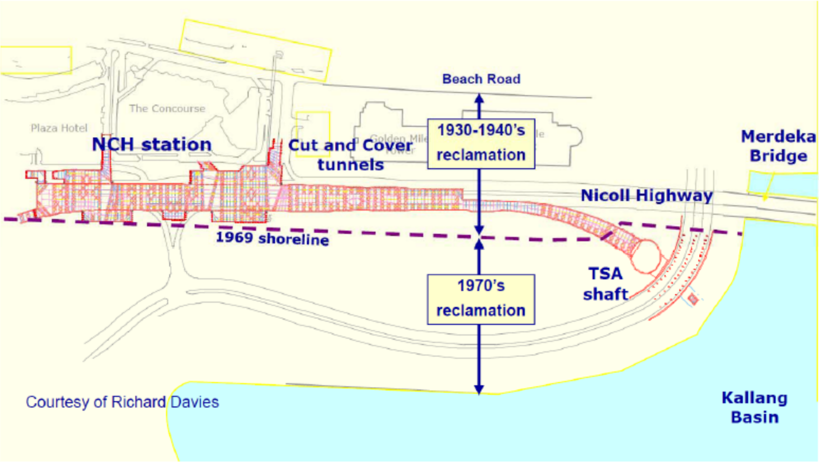
More sophisticated models?

(4) Modelling the boundary conditions - anything we do to approximate boundary conditions introduces error.

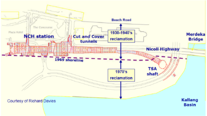
Nicoll Highway, Singapore



Nicoll Highway, Singapore

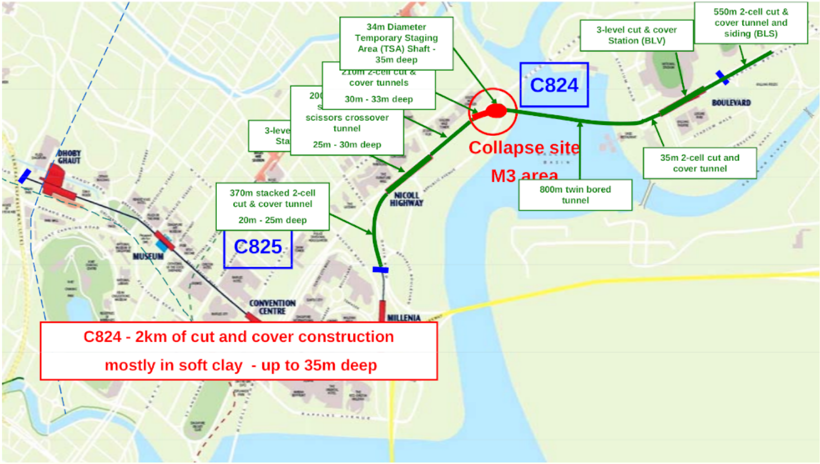


Nicoll Highway, Singapore



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.

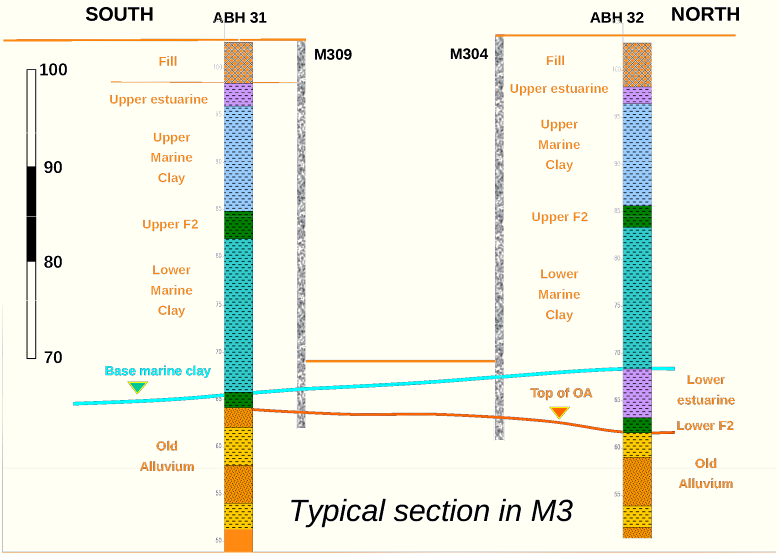
Cut and cover construction

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.

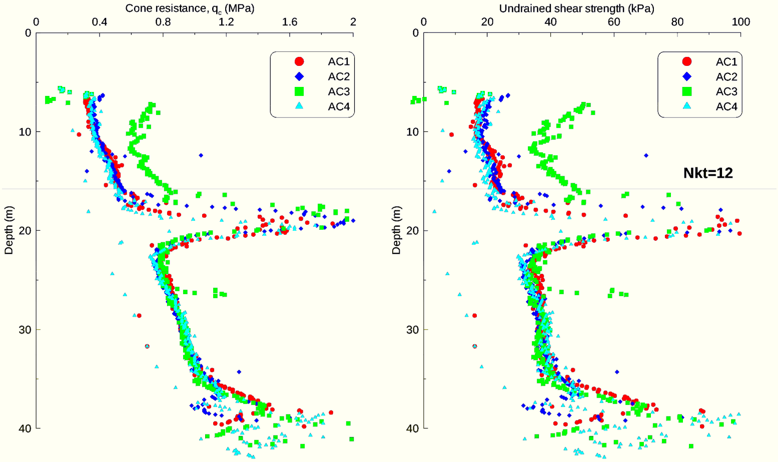
Bottom-Up method Bottom-up method involves excavation of a trench from the surface. The sides of the excavation can be sloped or vertical. The tunnel is then constructed in the trench and after the completion of the tunnel the trench is backfilled. Generally in urban areas due to limited availability 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.

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.

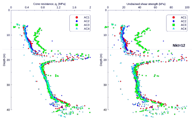
Soil profile



Soil profile

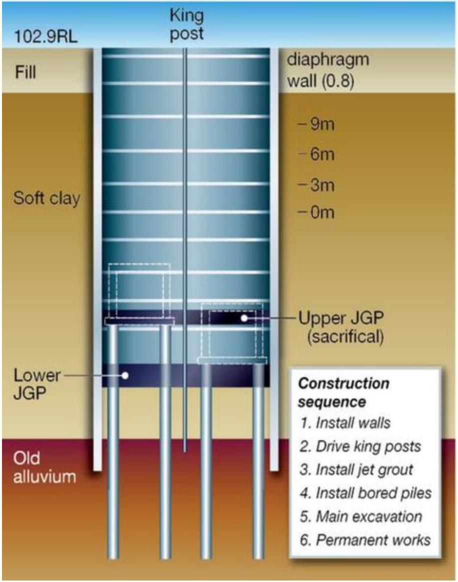


Soil profile

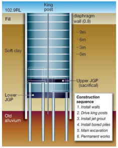


The investigation included 14 boreholes and cone penetration tests along with collection of soil samples and testing them in laboratories. After the contract was awarded an extensive testing was conducted which included 72 boreholes and cone penetration tests to determine the alignment of the tunnel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

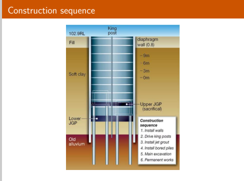
Construction sequence



Construction sequence



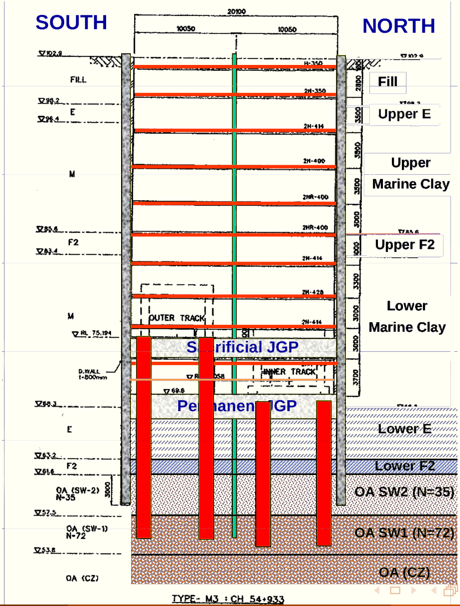
For design purposes diaphragm walls were divided into 40 different wall sections. Wall sections were designed based on the worst soil parameters obtained from the nearest borehole. The temporary retaining wall system consisting of diaphragm walls and struts was modeled, analyzed and designed using Plaxis, 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.

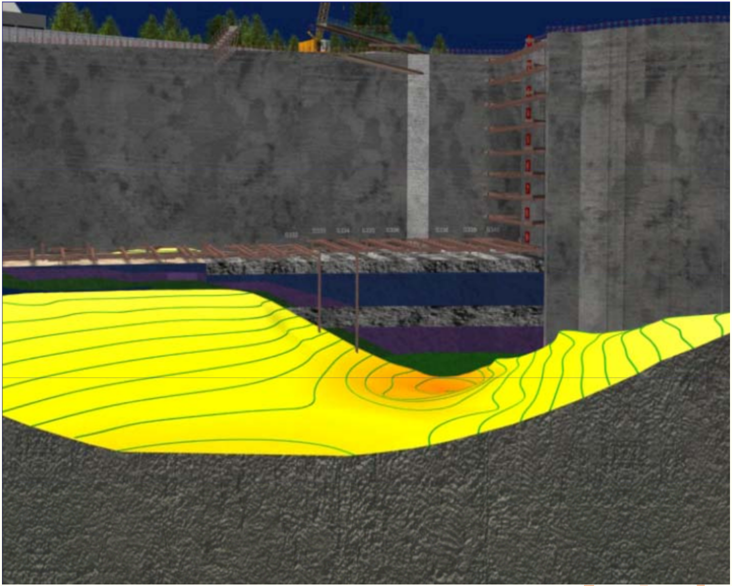
Construction sequence



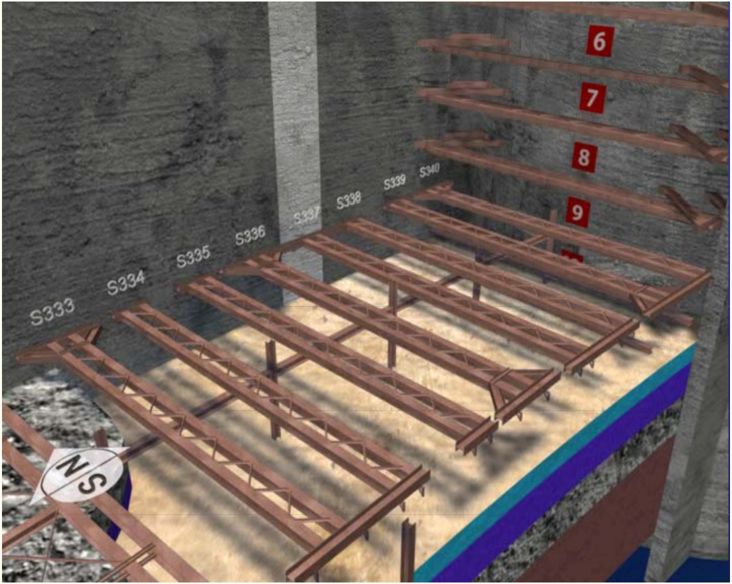
Nicoll Highway: Excavation

South side 13 March 2004

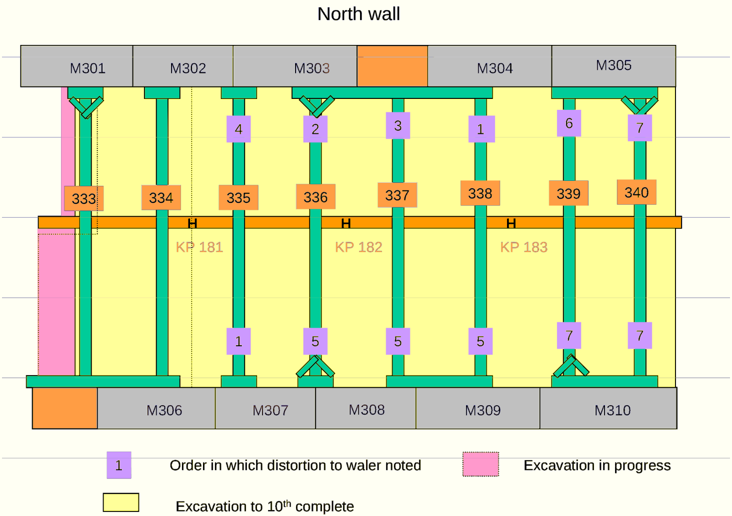
Excavating 10th level of struts



On the morning of collapse



On the morning of collapse



Location of walers, splays and Dwall gaps

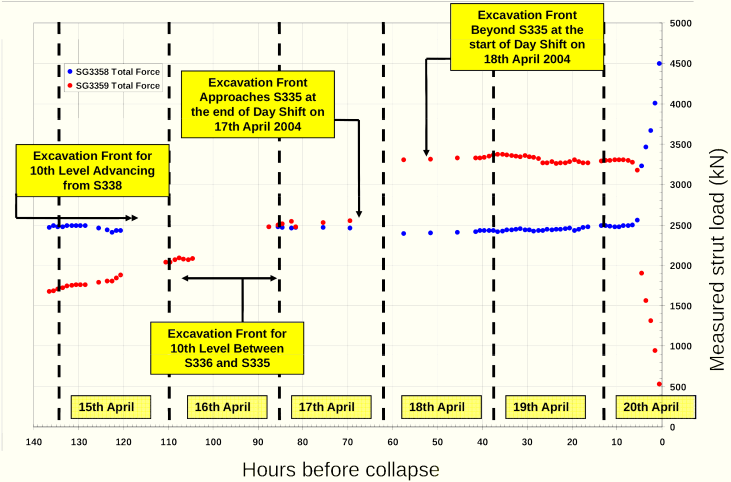
On the morning of collapse

Strut 338 North side

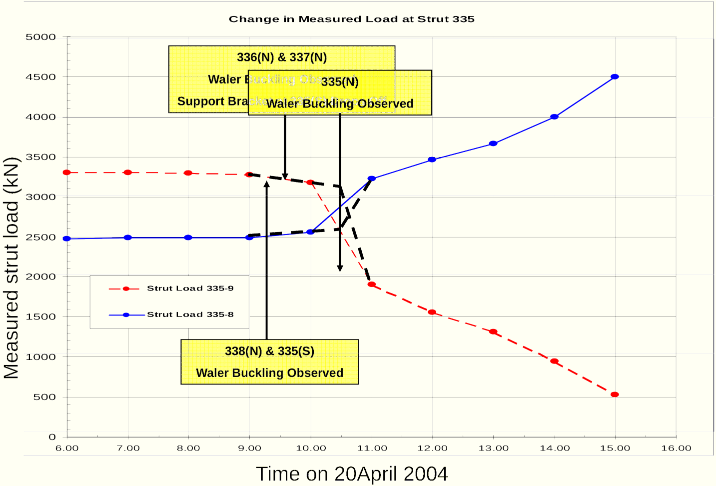
On the morning of collapse

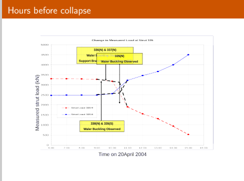
Strut 335 Sorth side

Hours before collapse



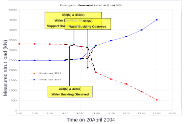
Hours before collapse





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.

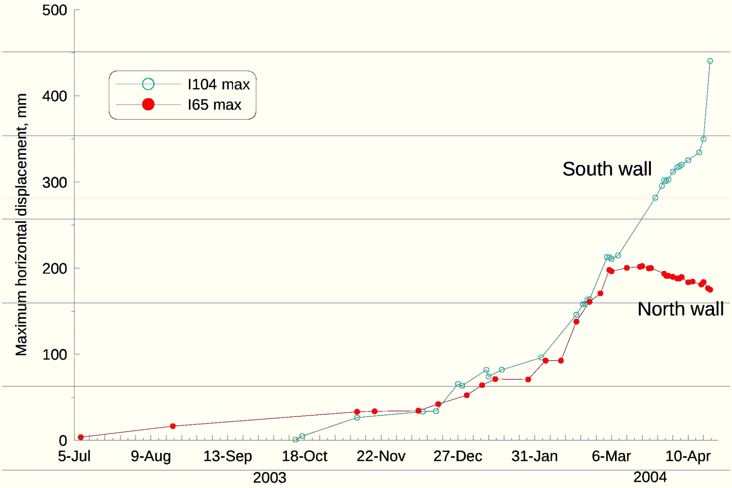
Hours before collapse



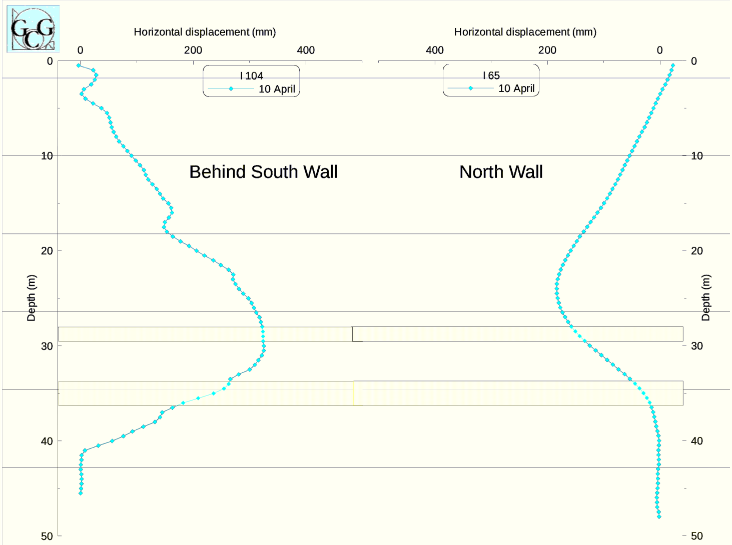
By noon engineers from the owner's side were also present at the worksite and they along with the contractor's engineers decided to pour concrete at the 9th level of the strut to stabilize the excavation and prevent it from caving in. By 3:30 pm the temporary system gave away and the tunnel had caved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

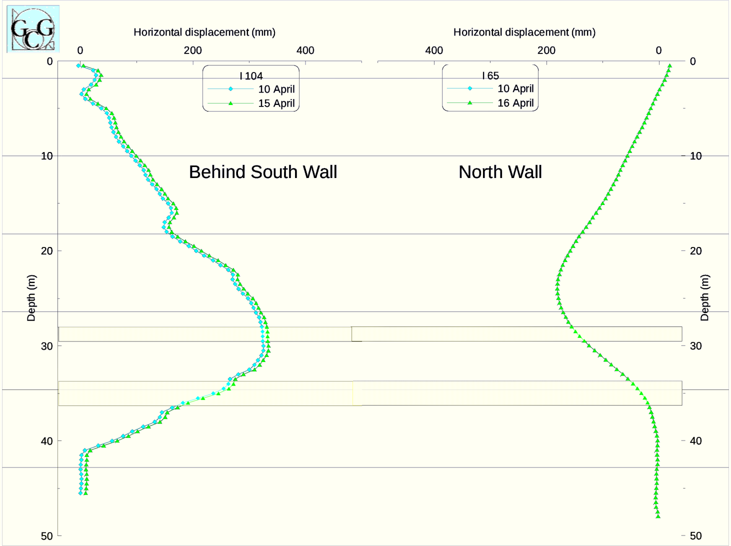
Leading up to the collapse: Inclinometer



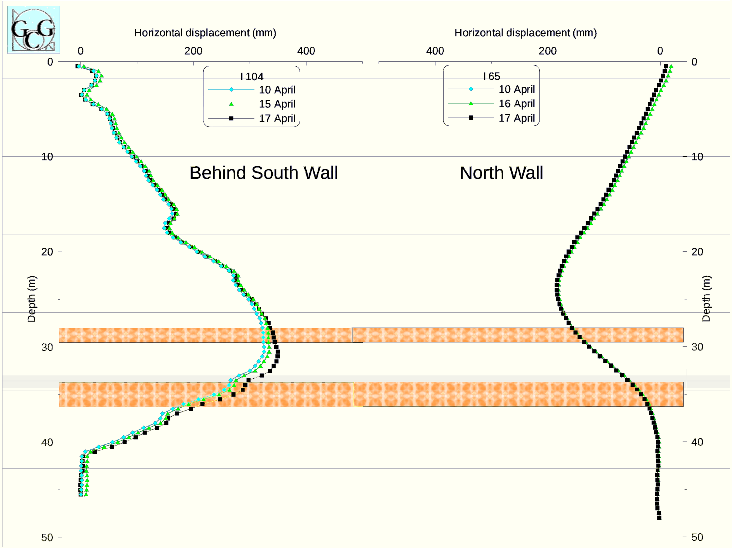
Leading up to the collapse: Wall displacements



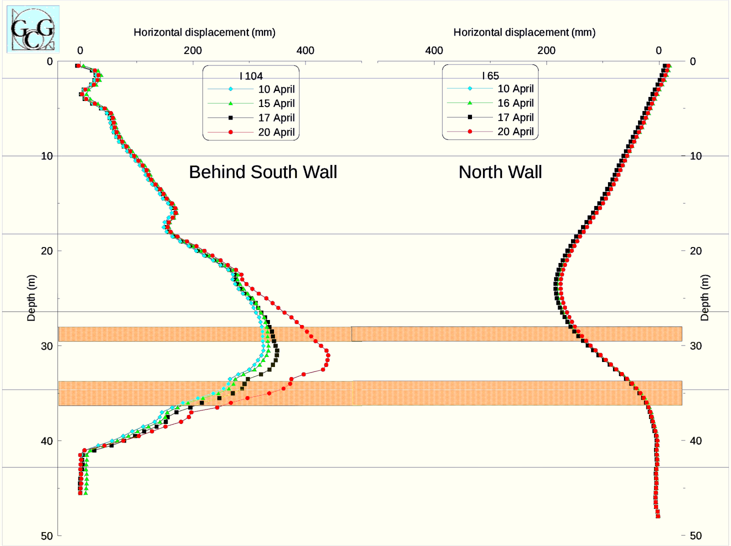
Leading up to the collapse: Wall displacements



Leading up to the collapse: Wall displacements



Leading up to the collapse: Wall displacements



The collapse

The collapse

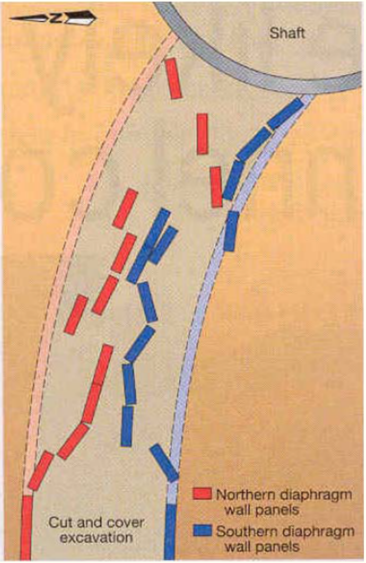
The collapse

The collapse

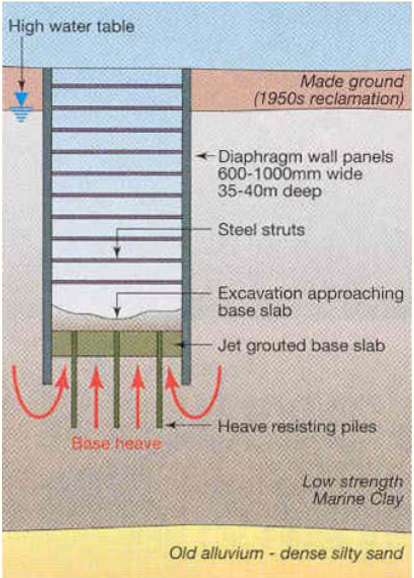
The collapse

Post collapse

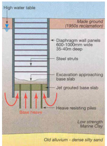
Post collapse



Reasons for collapse



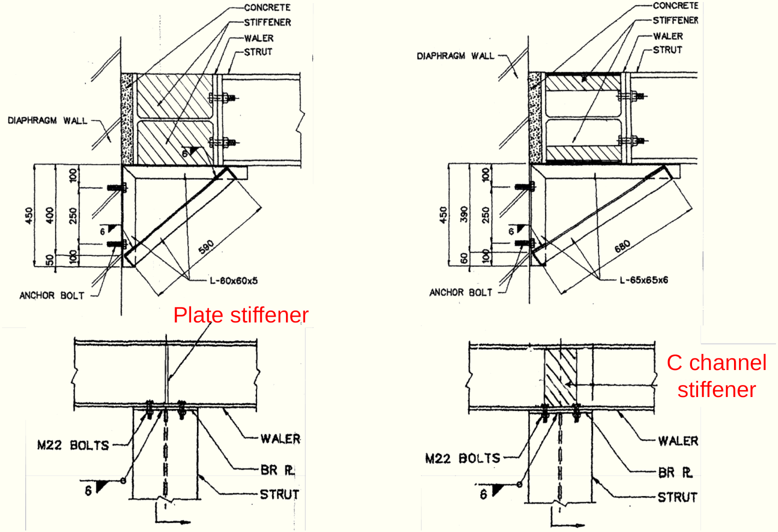
Reasons for collapse



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

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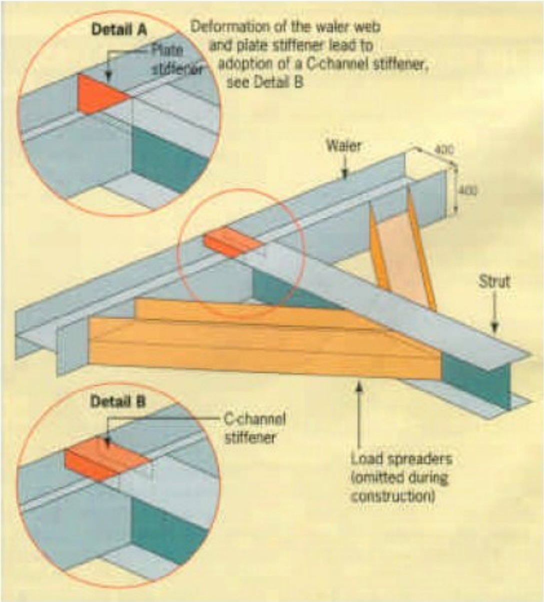
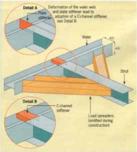
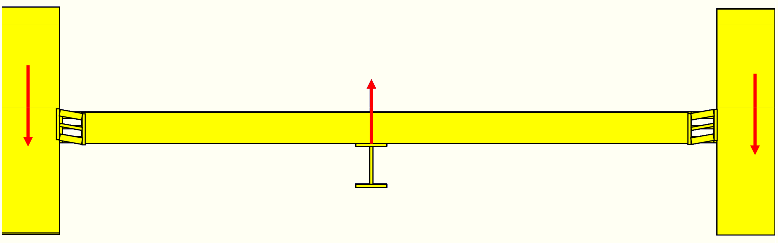


Strut design: Waler connection



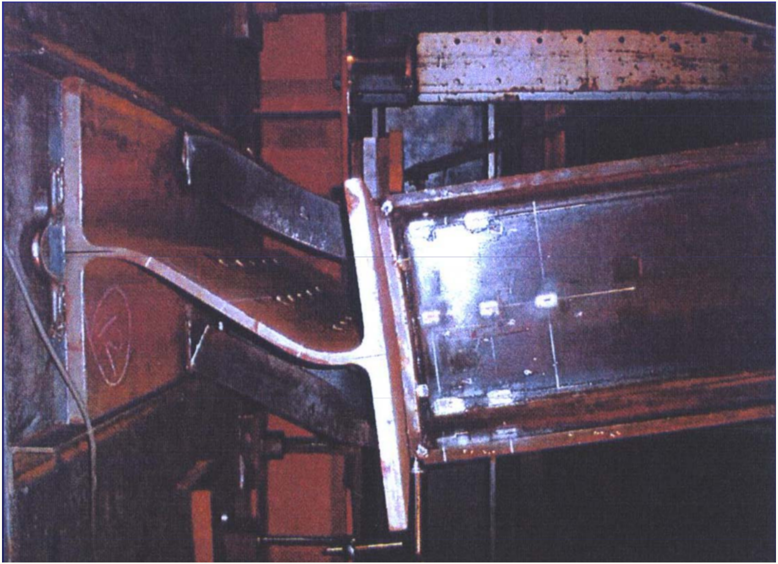
Another chief reason for the failure was the under design of the waler-strut connection. Designers misinterpreted the **stiff bearing length for the C-channels as per BS 5950 to 400 mm instead of using 65 mm**. Waler connections with C-channels were designed with the effective length factor of 0.7, where 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

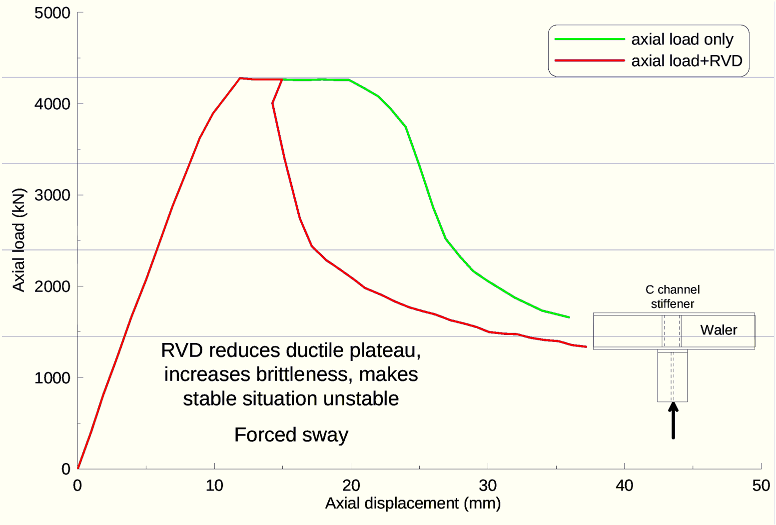


Relative vertical displacement between the King Post and the Dwall

Strut design: C-channel

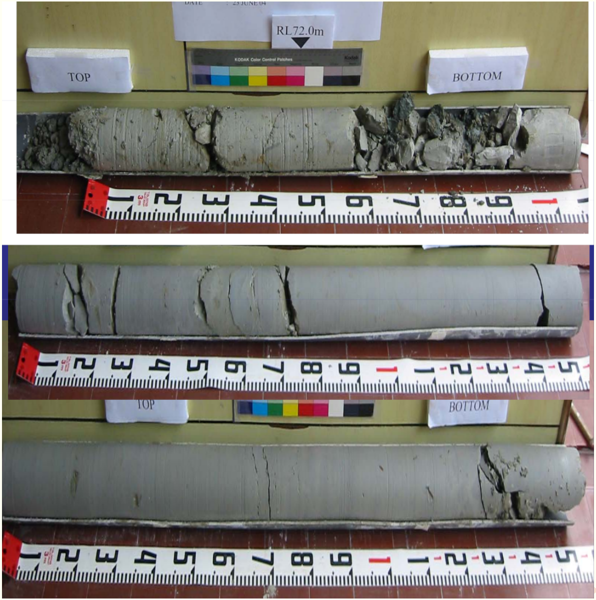


Strut design: C-channel relative vertical displacement

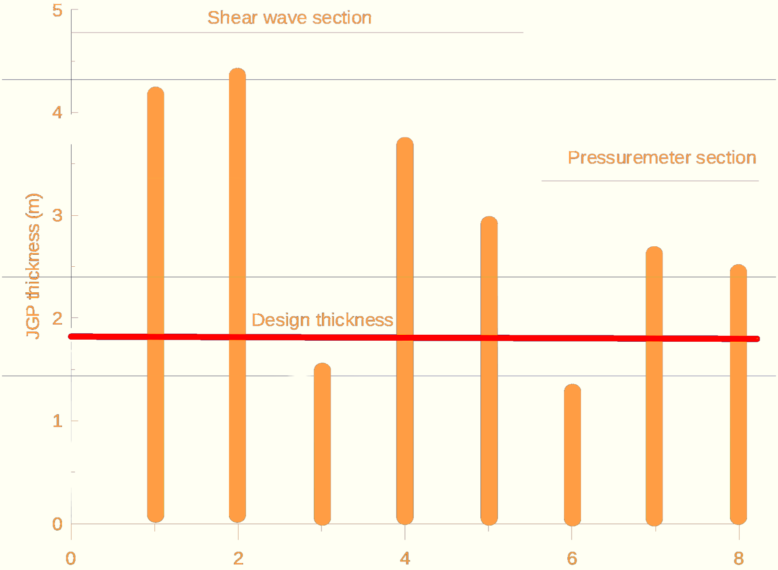


Excavation of sacrificial jet grout

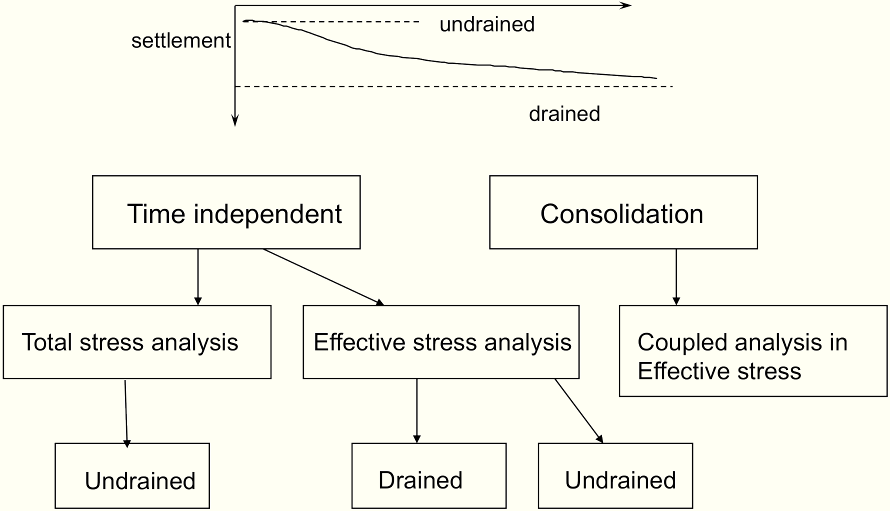
Quality of jet grouting



Quality of jet grouting



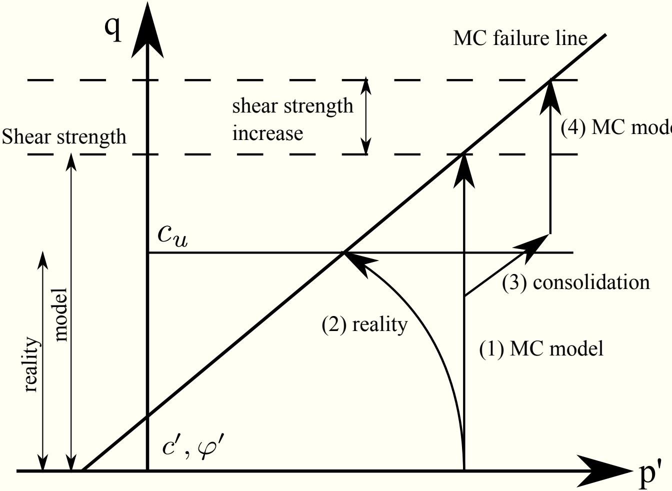
Porepressure analysis in geotechnical engineering

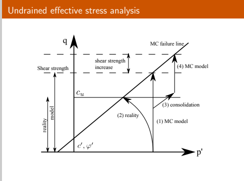


Undrained analysis

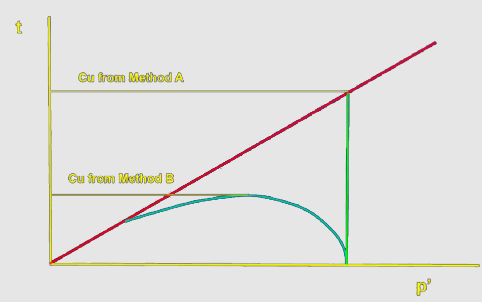
- Method A and Method B refers to 2 alternatives modeling of undrained behaviour in Plaxis.
- **Method A is an effective stress analysis** of an undrained problem it assumes an isotropic elastic behavior and a Mohr-Coulomb failure criterion.
- As a result mean effective stress p' is constant until yield.
- 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

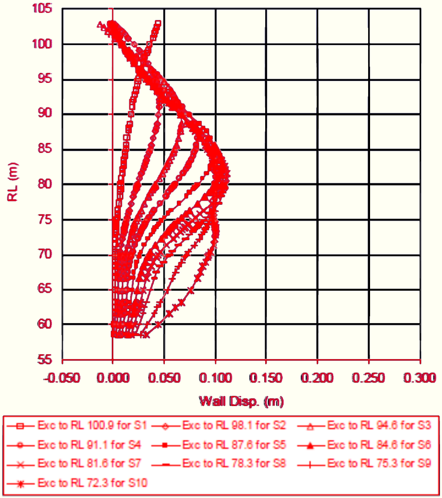


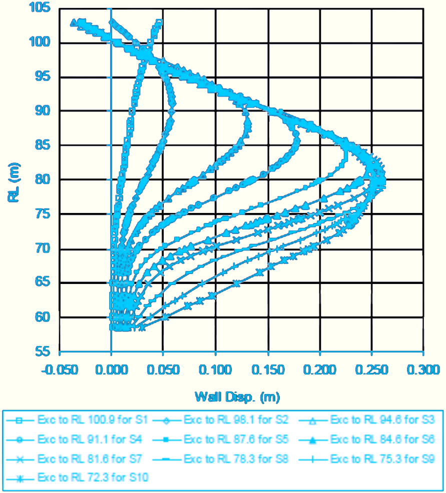


Discuss the effect of depth on the increase in shear strength



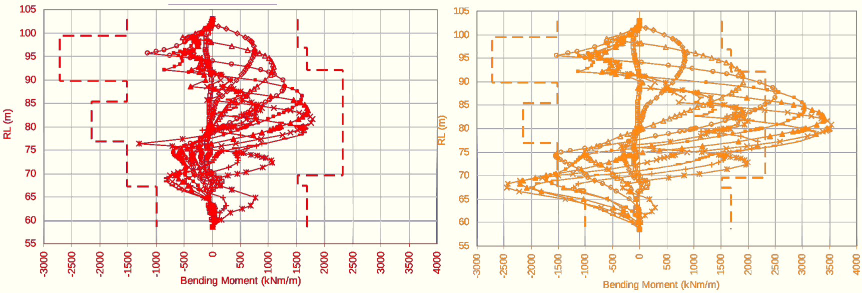
Wall displacements: Effective stress vs Undrained strength





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.

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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.

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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.