Coupled Stress-Seepage Numerical Design of Pressure Tunnels

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Abstract: The design and construction of pressure tunnels are most complicated among all types of tunnels. Therefore, special attention is required in the design and construction of these tunnels to prevent failure. This research introduces PLAXIS 2D finite element program as a tool for numerical modeling of plain concrete pressure tunnels. The numerical model cannot completely portray the nature, but should have the capability to simulate the materials and the loading cases. The rock behaviour is approximated by using the elasto-plastic Mohr - Coulombs model. The shotcrete and final lining are assumed permeable and elastic. The structural analysis is based on plain strain condition. Different phenomena in terms of loading cases during construction stage and operational loading case with internal water pressure are simulated. Special concern is taken on the modeling of the contact between the shotcrete and the final lining where during shrinkage and temperature change by first filling with cold water a gap can open and later closed during prestressing. Plain concrete lining of pressure tunnels is not absolutely tight and water seeps out of the tunnel. The seeped water losses energy, but can also cause stability problems in the surrounding rock mass. Additionally, if the rock mass around the tunnel is tight (originally or tightened by grouting) the seeped water stays in the vicinity of the tunnel and increases the external water pressure. Such increased external water pressure decreases the gradients between internal and external pressure and reduces the seepage and losses. Modeling of this phenomenon is performed by coupled stress-seepage calculation performed by the same model. The coupling of stress - seepage is carried out by superimposing results of consolidation and water flow analyses. The pressure tunnel of the HPP Ermenek Turkey is taken as a practical example in the numerical simulation. The results illustrate the applicability of the present method.

Keywords: *pressure tunnel, plain concrete lining, internal water pressure, stress-seepage analysis*

Coupled stress-seepage numerical design of pressure tunnels

Presented by
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OUTLINE

- Introduction
  - Research objectives
- Methodology
  - Results and discussion of results
- Conclusions
- Recommendations

INTRODUCTION

- Design and construction of pressure tunnels are among the most complicated of tunnels.
- The construction of the power waterway for high-head hydro-power plants is generally one of the most expensive project parts.
- Special attention is required to prevent failure.
- Most especially the used of plain concrete as a liner in pressure tunnels.
- No method have been found to include all important parameters

  for its construction and operational loading.
- The bearing of IWP is limited by low tensile strength of concrete.
- Plain concrete lining of pressure tunnels is not absolutely tight and water seeps out of the tunnel.
- The seeped water is lost of energy, but can also cause stability problems in the surrounding rock mass.
- If the rock mass around the tunnel is tight (originally or tighten by grouting) the seeped water stays in the vicinity of the tunnel and increases the external water pressure. Such increased external water pressure decreases the gradients between internal and external pressure and reduce the seepage and losses.

RESEARCH OBJECTIVES

- **Overall objective**

  To review common practice design methods with special attention on plain concrete lined power waterways.

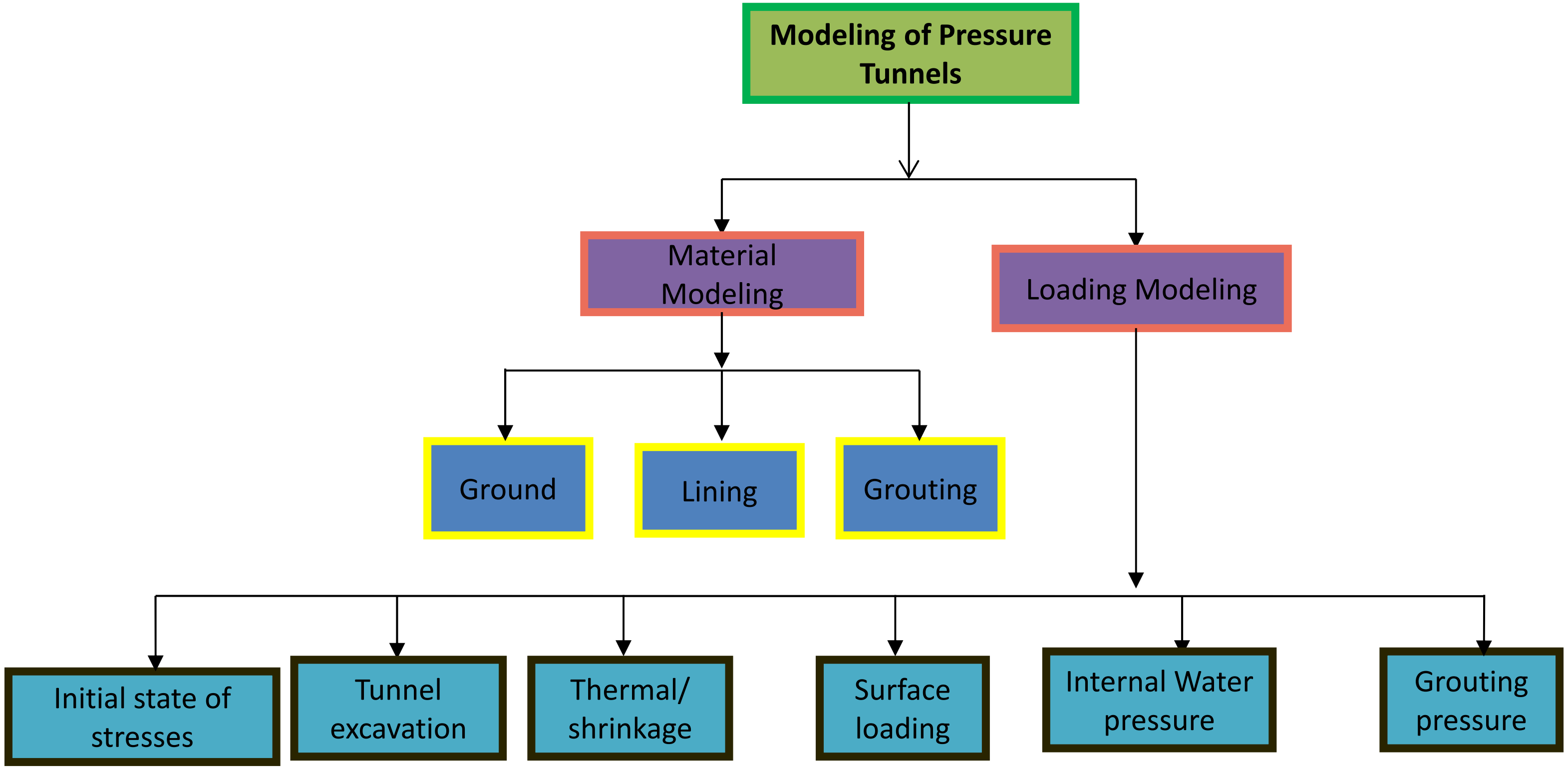
- **Specific objectives**

  - To define a numerical methods that include all important parameters and loading cases occurring during excavation, lining construction, grouting and operational loading by internal water pressures.

  - To give a unique and effective method for design of the permeable pressure tunnel linings.

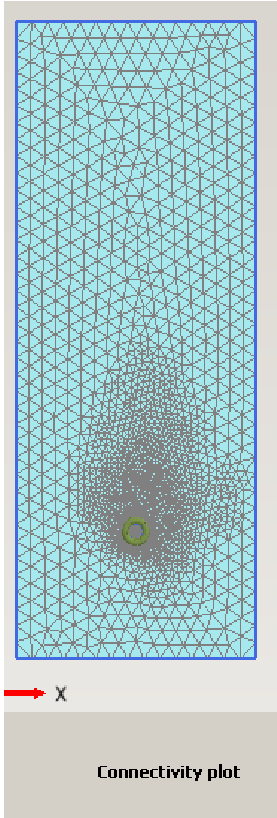
MATERIAL AND LOADING MODELLING

- **Material behaviour**
  - Understanding the behaviour of rock mass and concrete will allow proper modelling of the materials and well as the interaction during and after loading.
- **(Material and loading) Modelling**

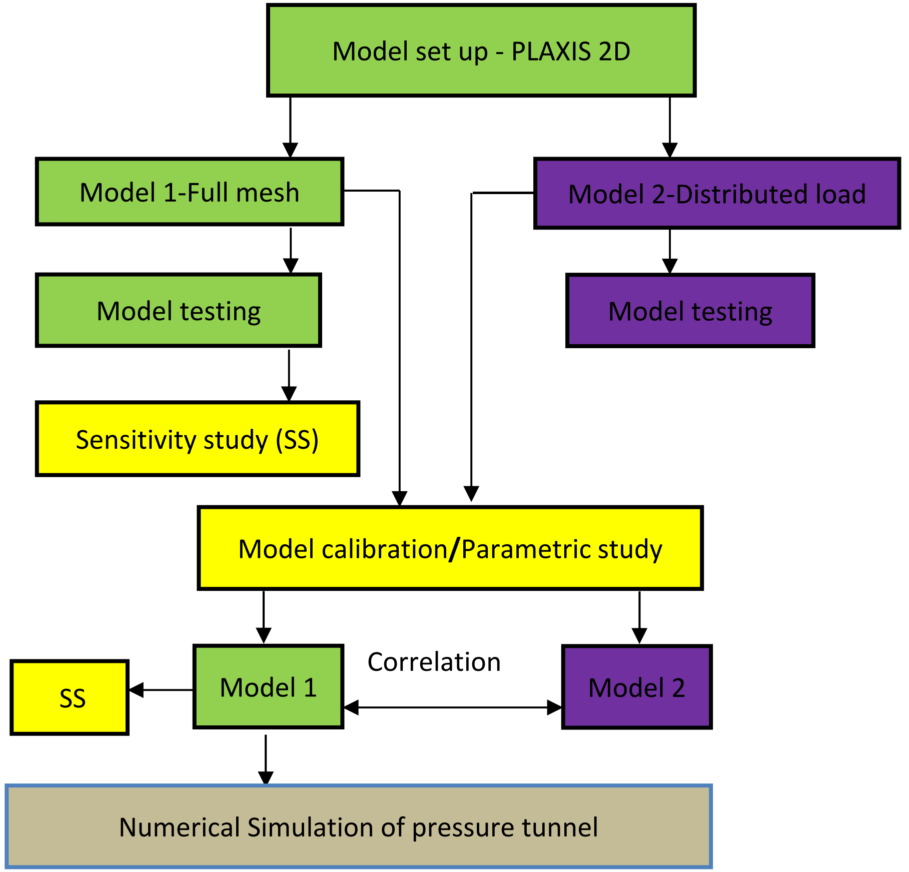

Modeling of Pressure Tunnels
Material Modeling
Loading Modeling
Ground
Lining
Grouting
Initial state of stresses
Tunnel excavation
Thermal/ shrinkage
Surface loading
Internal Water pressure
Grouting pressure

Schematic diagram of material & loading Modeling

METHODOLOGY I


X
Connectivity plot

Full model (FE meshing)



Model set up - PLAXIS 2D

Model 1-Full mesh

Model 2-Distributed load

Model testing

Model testing

Sensitivity study (SS)

Model calibration/Parametric study

SS

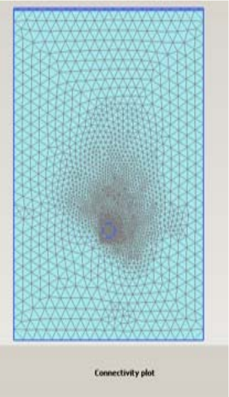
Model 1

Correlation

Model 2

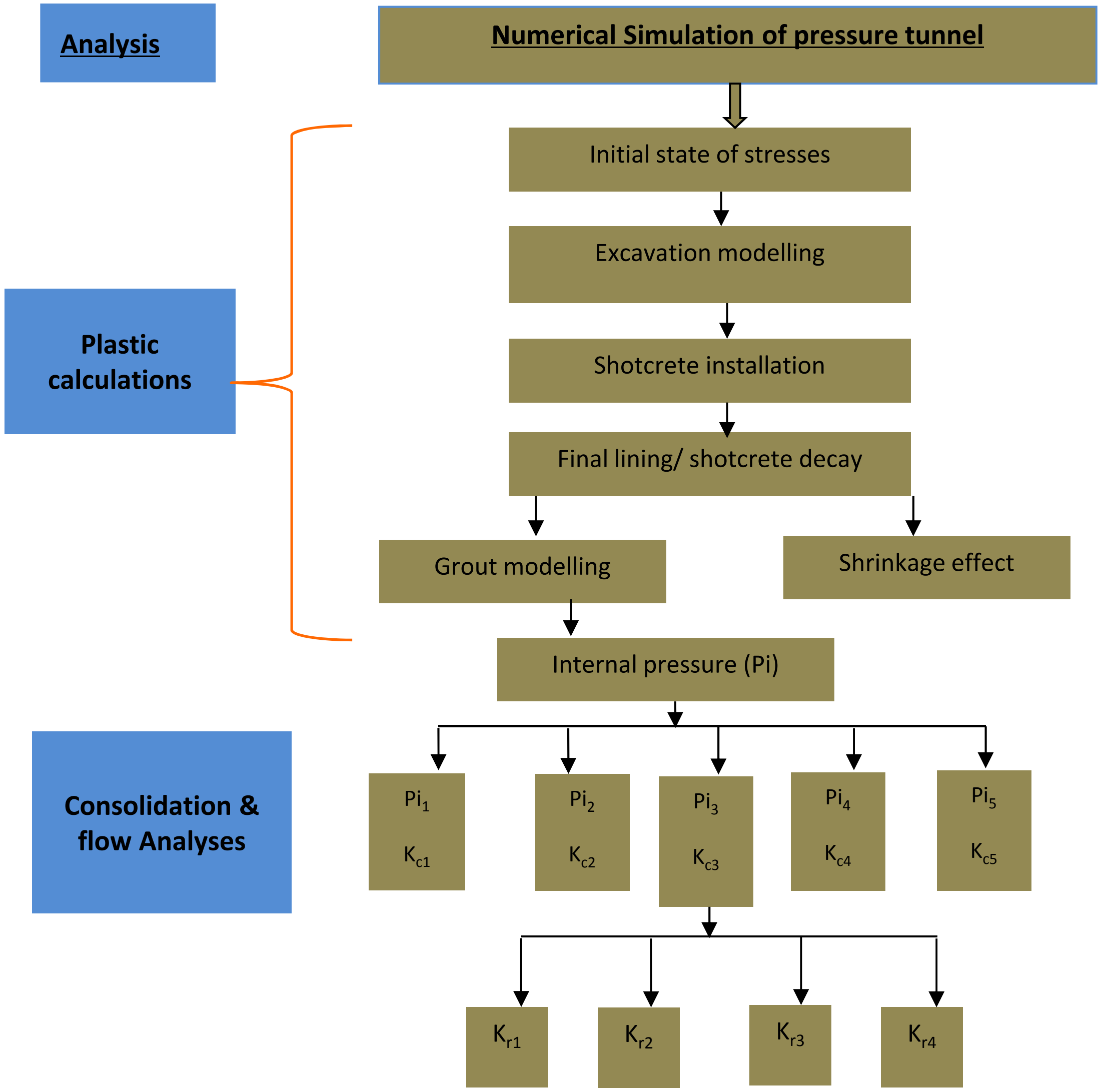
Numerical Simulation of pressure tunnel

Methodology flow chart


Connectivity plot

Distributed load
(FE meshing)

METHODOLOGY II



Analysis

Numerical Simulation of pressure tunnel

Plastic calculations

Initial state of stresses

Excavation modelling

Shotcrete installation

Final lining/ shotcrete decay

Grout modelling

Shrinkage effect

Internal pressure (Pi)

Consolidation & flow Analyses

Pi_1 K_{c1}

Pi_2 K_{c2}

Pi_3 K_{c3}

Pi_4 K_{c4}

Pi_5 K_{c5}

K_{r1}

K_{r2}

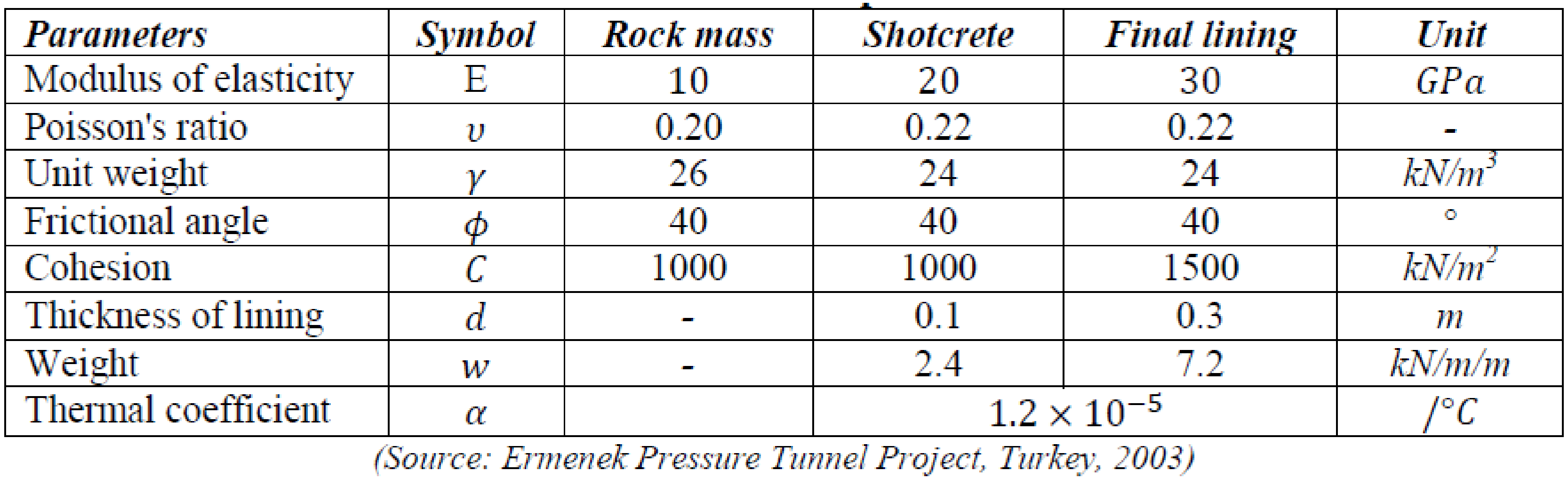
K_{r3}

K_{r4}

Flow chart for the numerical analysis of pressure tunnels

DESIGN DATA

Material properties

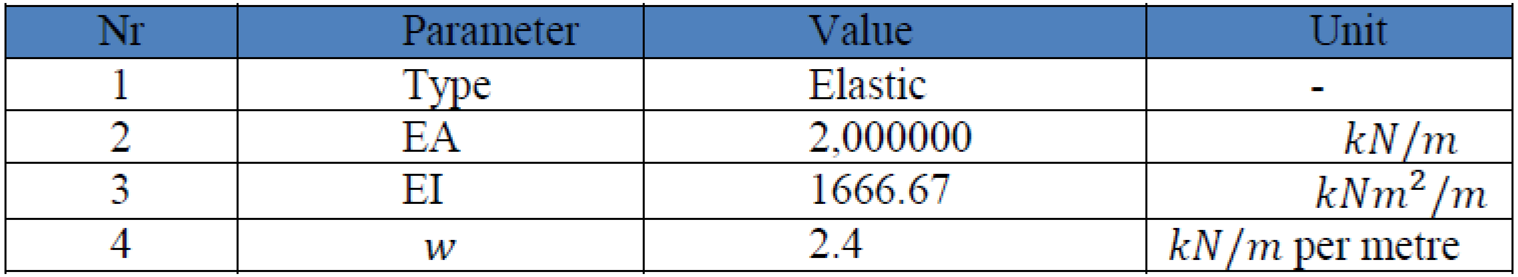


Parameters	*Symbol*	*Rock mass*	*Shotcrete*	*Final lining*	*Unit*
Modulus of elasticity	E	10	20	30	GPa
Poisson's ratio	υ	0.20	0.22	0.22	-
Unit weight	γ	26	24	24	kN/m^3
Frictional angle	ϕ	40	40	40	°
Cohesion	C	1000	1000	1500	kN/m^2
Thickness of lining	d	-	0.1	0.3	m
Weight	w	-	2.4	7.2	$kN/m/m$
Thermal coefficient	α		1.2×10^{-5}		$/°C$

(Source: Ermenek Pressure Tunnel Project, Turkey, 2003)

Overburden height =h=200m

Material properties of shotcrete (calculated values)



Nr	Parameter	Value	Unit
1	Type	Elastic	-
2	EA	2,000000	kN/m
3	EI	1666.67	kNm^2/m
4	w	2.4	kN/m per metre

MODEL ASSUMPTIONS

❖Rock mass behaviour is assumed to be in drained conditions.

❖Lining material is elastic.

❖The model is based on condition of plane strain.

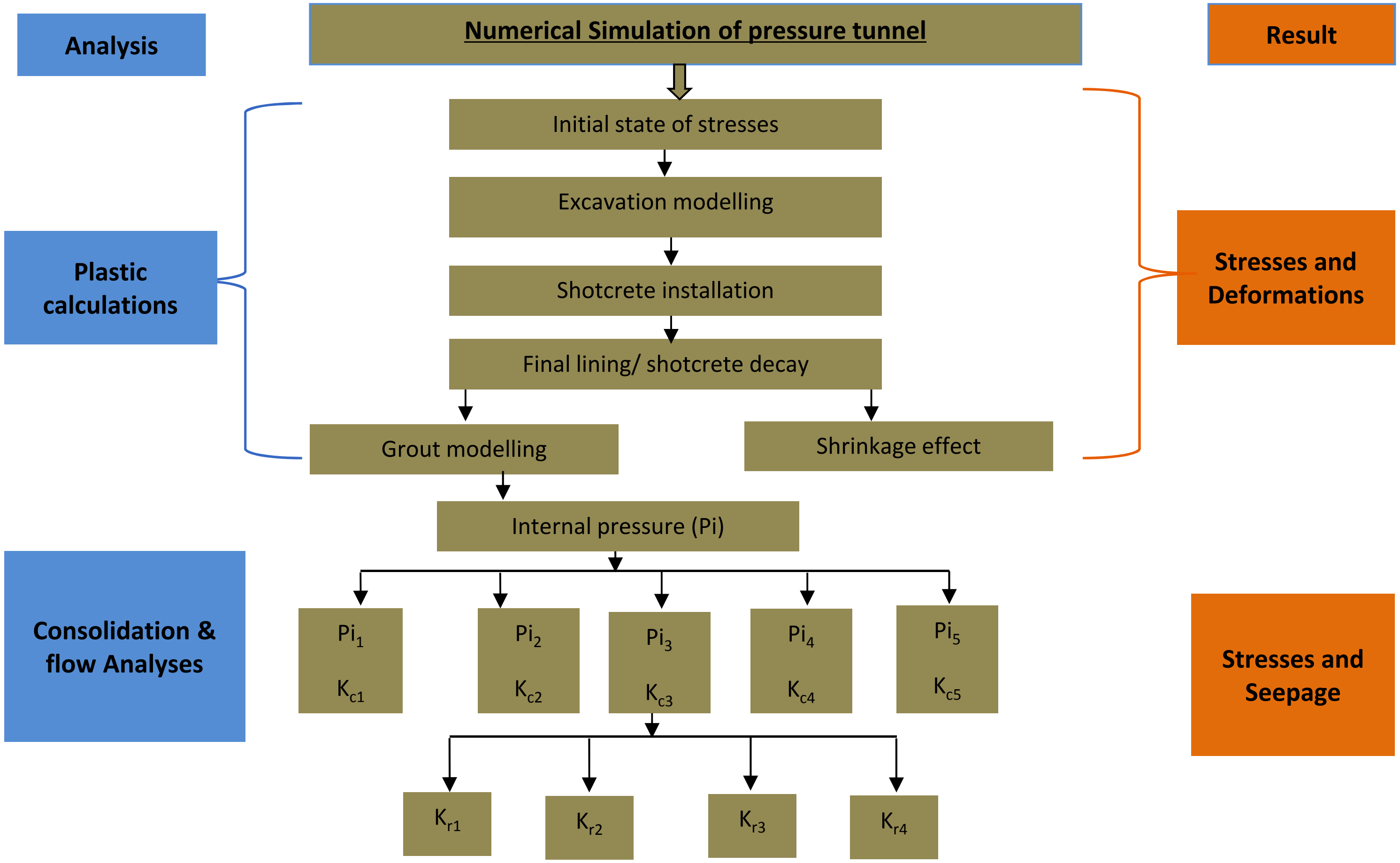
❖Rock mass is defined as elasto-plastic material, with yield functions defined by Mohr-Coulomb strength law.

❖The stresses existing in the rock mass are related to the weight of the overlying strata and geological history. No geotechnical stresses are expected and the vertical stress is assumed as a weight of overburden. The horizontal stress is defined as a portion of the vertical stress for elastic solution with restrained horizontal movements.

Boundary condition:

Horizontal fixity, Ux =0
Vertical fixity, Uy = 0

RESULTS AND DISCUSSION OF RESULTS



Analysis

Numerical Simulation of pressure tunnel

Result

Initial state of stresses

Excavation modelling

Shotcrete installation

Final lining/ shotcrete decay

Grout modelling

Shrinkage effect

Plastic calculations

Stresses and Deformations

Internal pressure (Pi)

Pi_1 K_{c1}

Pi_2 K_{c2}

Pi_3 K_{c3}

Pi_4 K_{c4}

Pi_5 K_{c5}

K_{r1} K_{r2} K_{r3} K_{r4}

Consolidation & flow Analyses

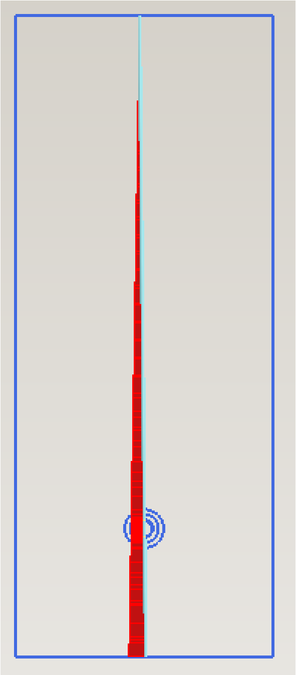
Stresses and Seepage

Flow chart for results of numerical design of plain concrete pressure tunnels

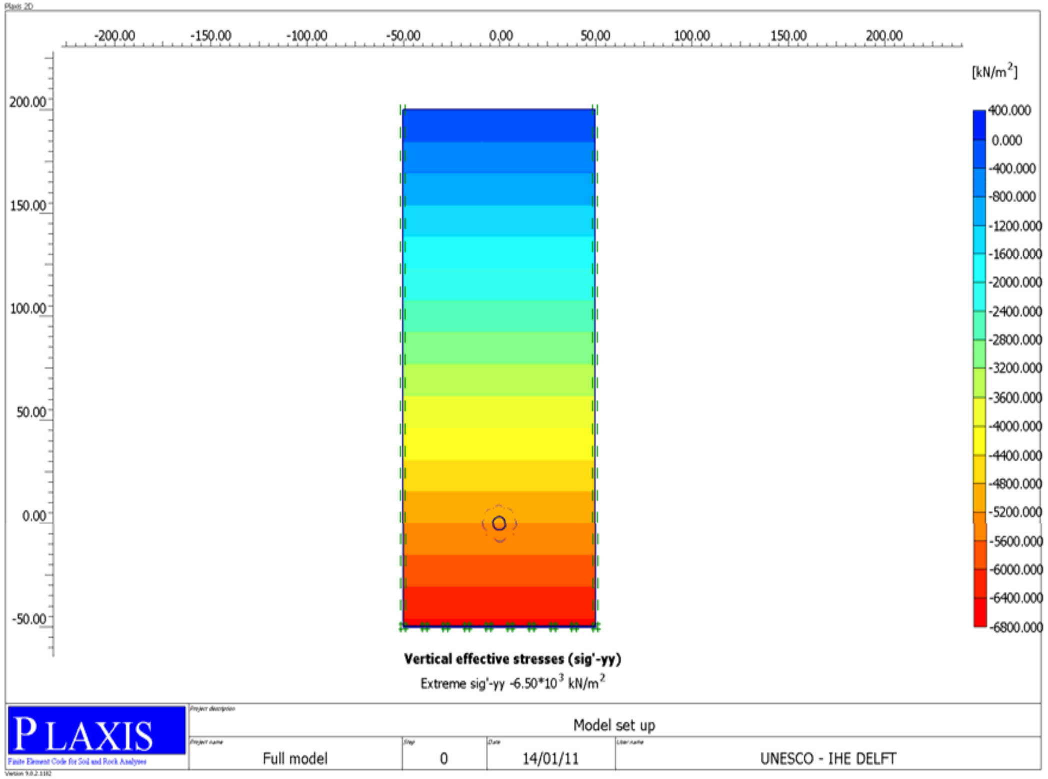
RESULTS AND DISCUSSION OF RESULTS

1. Initial state of stress

1.1 vertical stresses (σ_{yy})



Cross section


-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
150.00
200.00
200.00
150.00
100.00
50.00
0.00
-50.00
[kN/m^2]
400.000
0.000
-400.000
-800.000
-1200.000
-1600.000
-2000.000
-2400.000
-2800.000
-3200.000
-3600.000
-4000.000
-4400.000
-4800.000
-5200.000
-5600.000
-6000.000
-6400.000
-6800.000
PLAXIS

Vertical effective stresses (sig'-yy)

Extreme sig'-yy -6.50*10^3 kN/m^2

Model set up

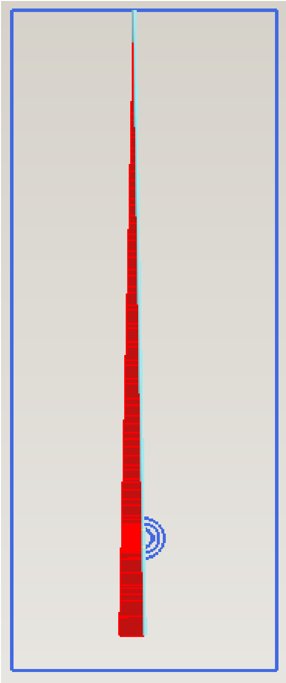
Full model | 0 | 14/01/11 | UNESCO - IHE DELFT

Deformation is uniform and equals to zero

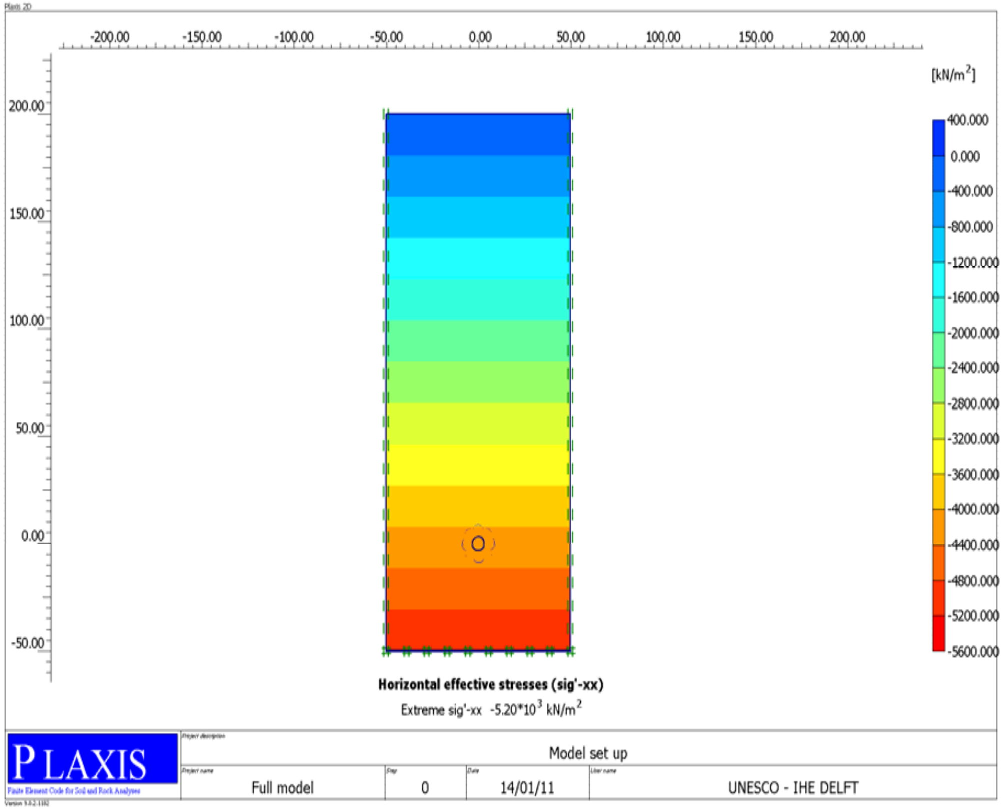
RESULTS AND DISCUSSION OF RESULTS

1. Initial state of stress

1.2 Horizontal stresses (σ_{xx})



Cross section



Horizontal effective stresses (sig'-xx)

Extreme sig'-xx -5.20×10^3 kN/m^2

Model set up

Full model

0

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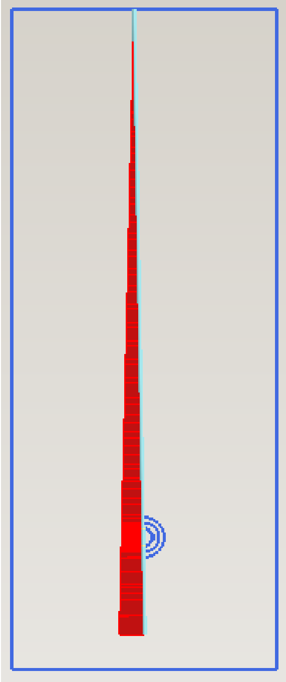
RESULTS AND DISCUSSION OF RESULTS

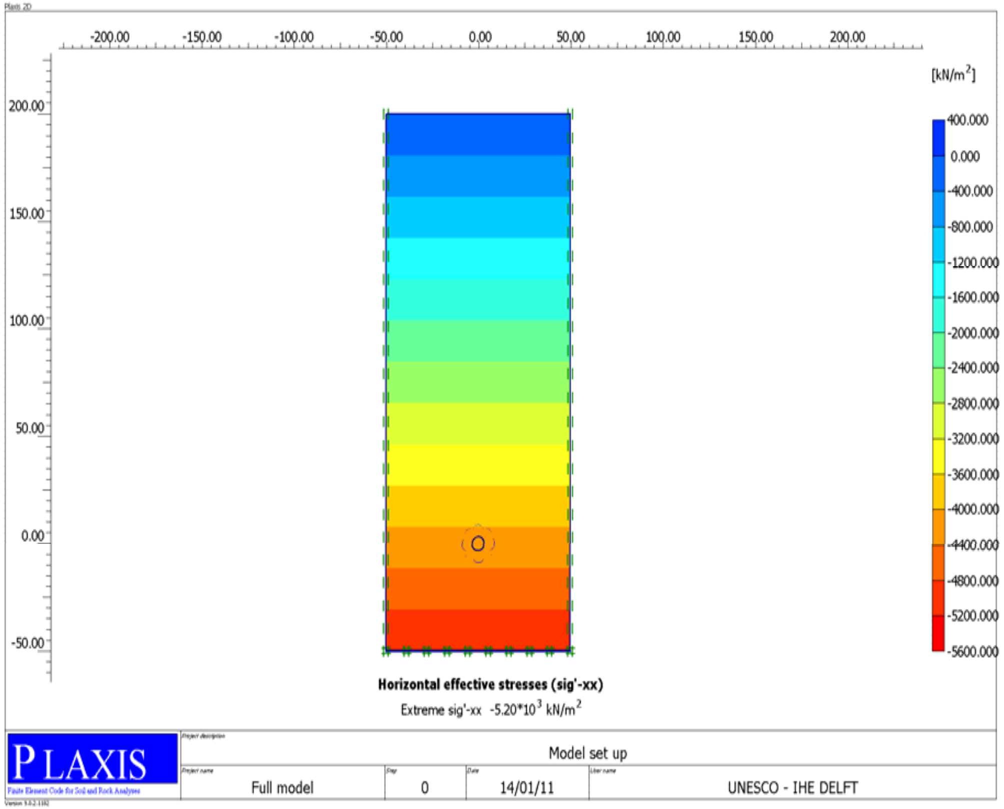
Loading -0-

1. Initial state of stress

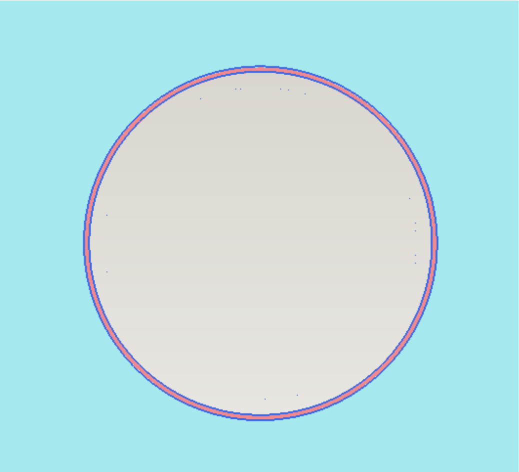
1.3 Horizontal stresses (σ_{zz})



Cross section



RESULTS AND DISCUSSION OF RESULTS



Loading -1- Stress relief

Model calibration (β- method)
Excavation and shotcrete installation
(2D simulation of 3D arching effect).

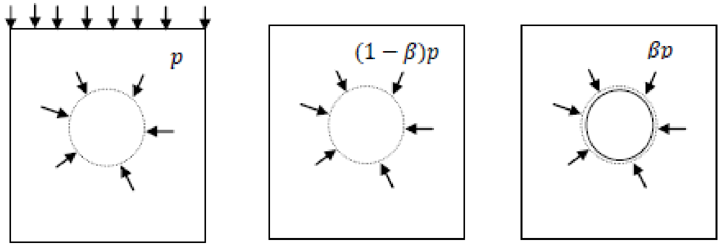
Before excavation, initial radial stress = initial state of stress in the rock.

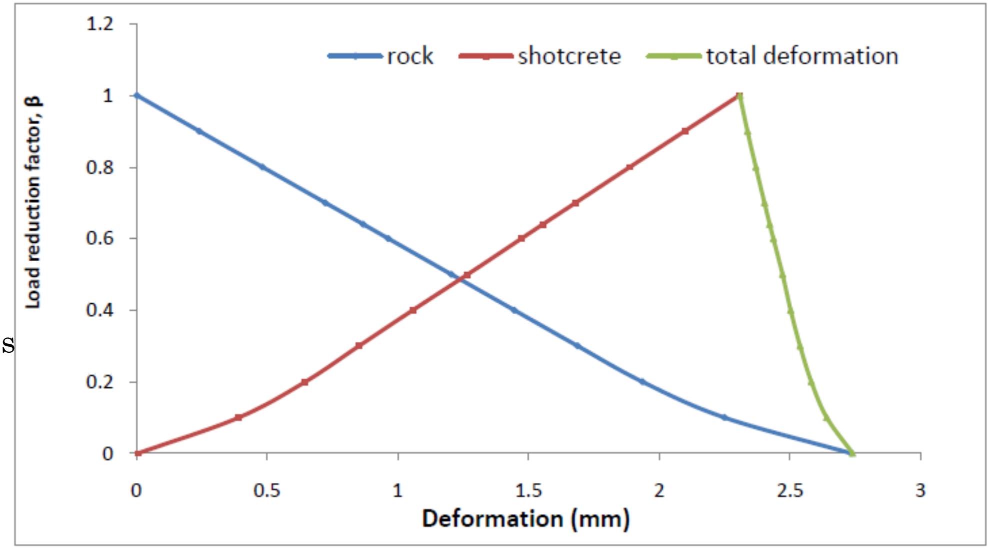
As excavation starts, the reduction factor applies because the natural equilibrium of the rock is disturbed.

β increases as the excavation progresses leading to decrease in deformation around the opening as the shotcrete takes part of the rock pressure.

The load reduction factor , **β** defines the degree to which the lining is unloaded due to rock mass being allowed to converge.

This implies that the shotcrete lining partly confines the convergence and reduces deformation.





Rock mass -lining characteristic curve (Element 7, Node 2873, Crown).

RESULTS AND DISCUSSION OF RESULTS

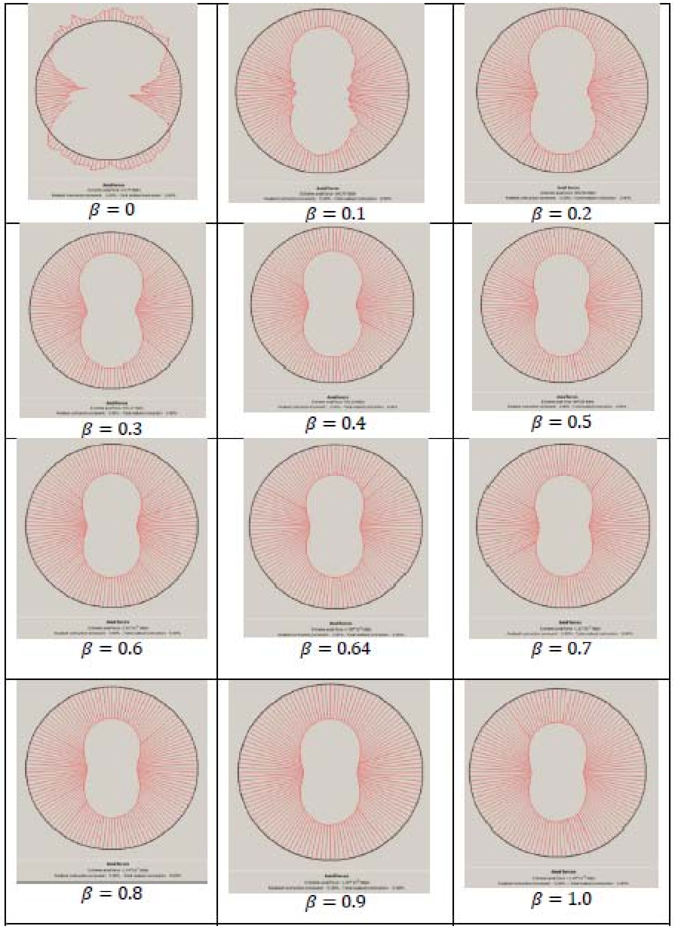
Model calibration (β- method)

β in the range of (0.6 - 0.64) is found to have given a normal compressive force in range of 1000kN as measured at the project site and in the geological conditions similar to the selected rock mass parameters.

Normal forces and deformations
for 0≤ **β** ≤ 1

C1.2: Full Model Approach Results
Parametric study
Full model (d=100mm)

S/n	*β*	*A (kN/m)*	*Displacement (mm)*
1	0	14.74	2.76
2	0.1	341.97	2.66
3	0.2	552.50	2.60
4	0.3	691.37	2.57
5	0.4	781.83	2.53
6	0.5	895.09	2.49
7	0.6	1010.00	2.46
8	0.64	1050.00	2.45
9	0.7	1120.00	2.43
10	0.8	1240.00	2.40
11	0.9	1360.00	2.37
12	1	1480.00	2.35

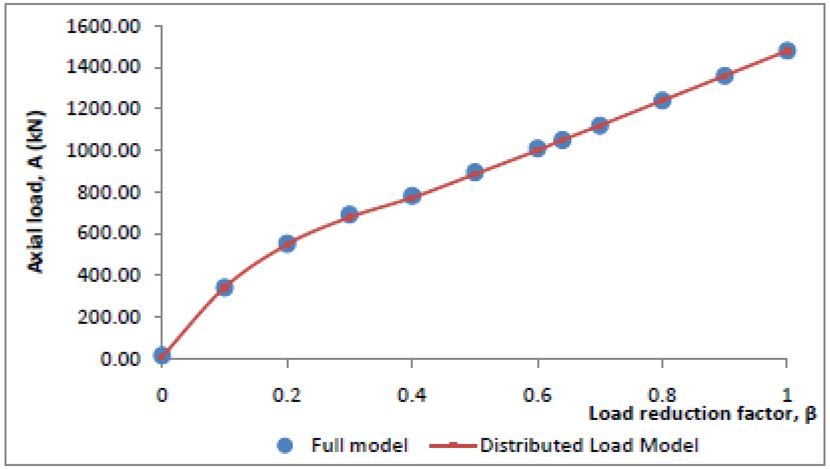


$\beta = 0$ $\beta = 0.1$ $\beta = 0.2$
$\beta = 0.3$ $\beta = 0.4$ $\beta = 0.5$
$\beta = 0.6$ $\beta = 0.64$ $\beta = 0.7$
$\beta = 0.8$ $\beta = 0.9$ $\beta = 1.0$

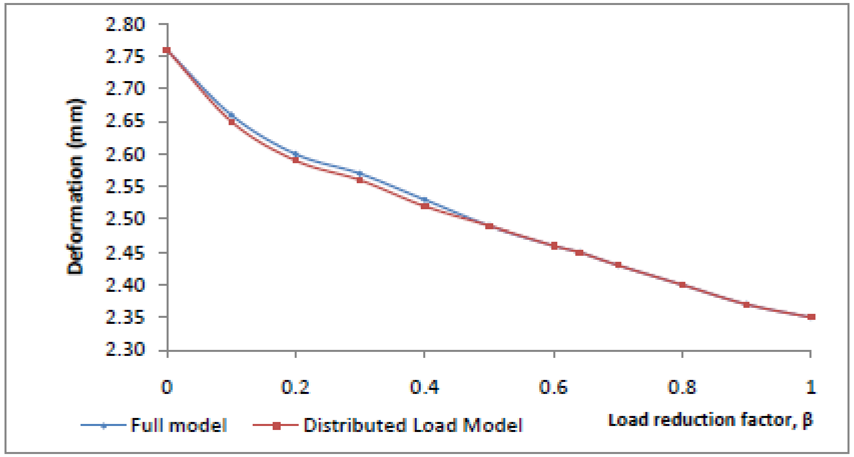
Normal force envelope
Note: diagram not to scale

RESULTS AND DISCUSSION OF RESULTS

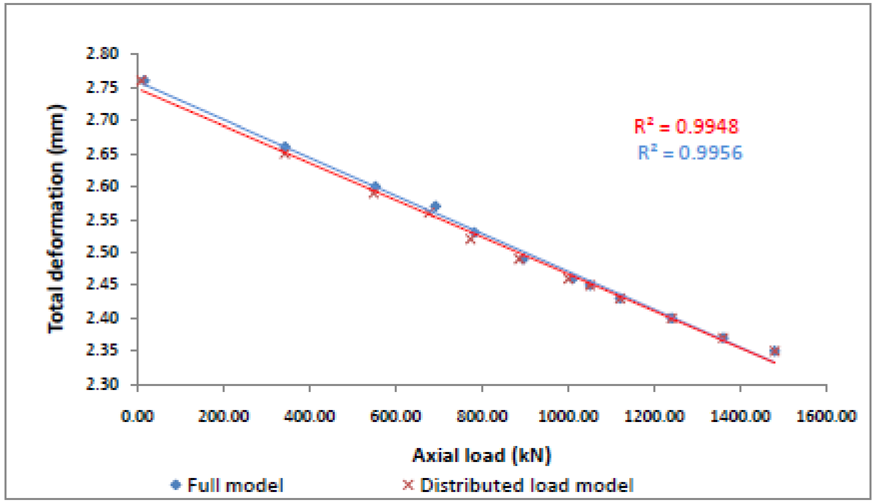
Performance of models



Effect of β on internal forces in the shotcrete lining



Effect of β on total deformation in the tunnel



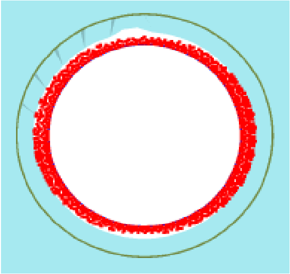
Relationship between inner forces and deformations

RESULTS AND DISCUSSION OF RESULTS

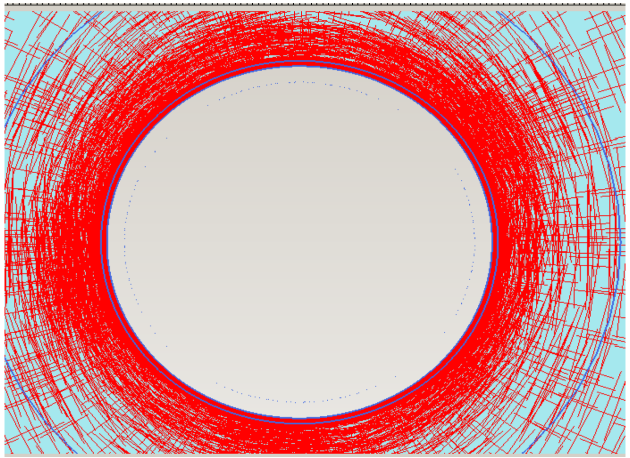
Loading -2- Excavation

Stresses around excavation

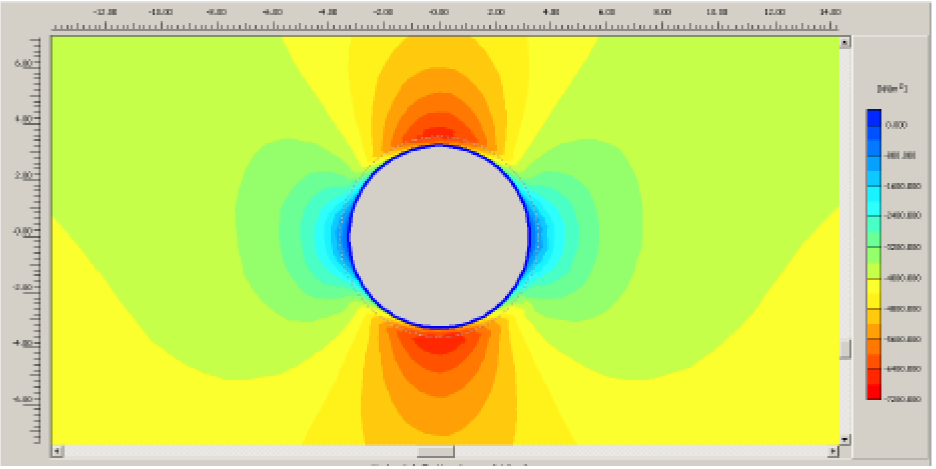
- pressure reduction in the rock mass
- transfer of stresses
-Reduction in deformation due to pressure transfer.



Plastic zone



Direction of principal stresses


-12.00
-10.00
-8.00
-6.00
-4.00
-2.00
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
6.00
4.00
2.00
0.00
-2.00
-4.00
-6.00
[kN/m²]
0.000
-800.000
-1600.000
-2400.000
-3200.000
-4000.000
-4800.000
-5600.000
-6400.000
-7200.000

Redistribution of stresses

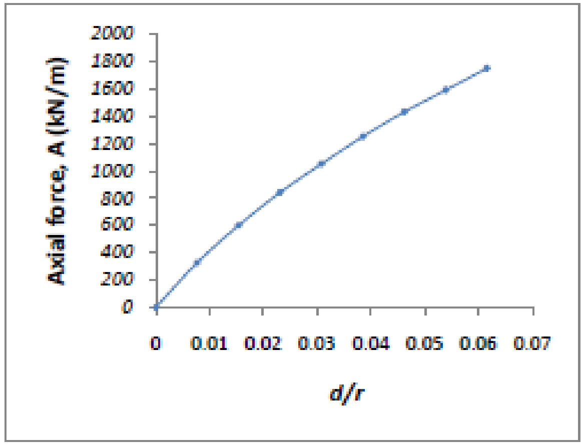
Loading -3- Shotcrete lining

Stresses around shotcrete

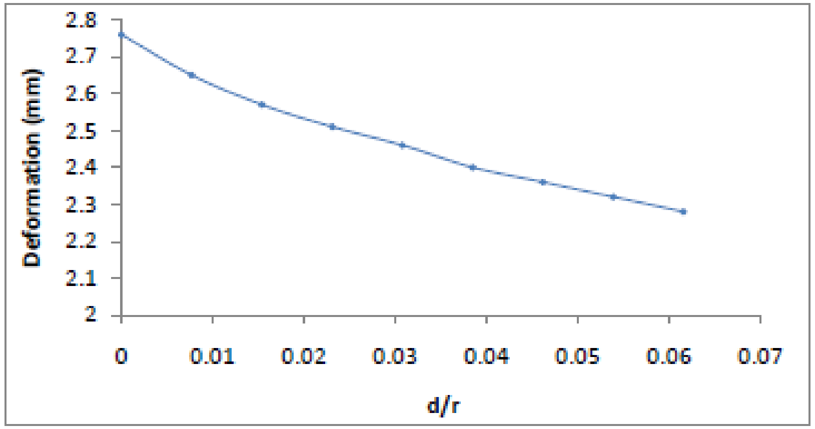
- shotcrete becomes a load bearing member.
-redistribution of stresses
-stress concentration increases due to confinement.
- stresses becomes negligible at a distance of about 3 times tunnel radius
- 12% reduction in deformation
-25% of rock pressure taken by the shotcrete.
- rock around shotcrete in plastic point.

RESULTS AND DISCUSSION OF RESULTS

Sensitivity analysis



Effect of thickness variation on axial forces in the shotcrete



Effect of thickness variation on liner deformation

- Increase in thickness, increase in stiffness and normal comp. stresses at E=constant produce increase in inner forces.

- Moment of resistance tend to increase with forces

- The forces reduced the impact of pressure from rock mass,

- Part of 3D arching effect produces deformation and stresses in shotcrete

- Decrease in total deformation unlike unsupported tunnel.

RESULTS AND DISCUSSION OF RESULTS

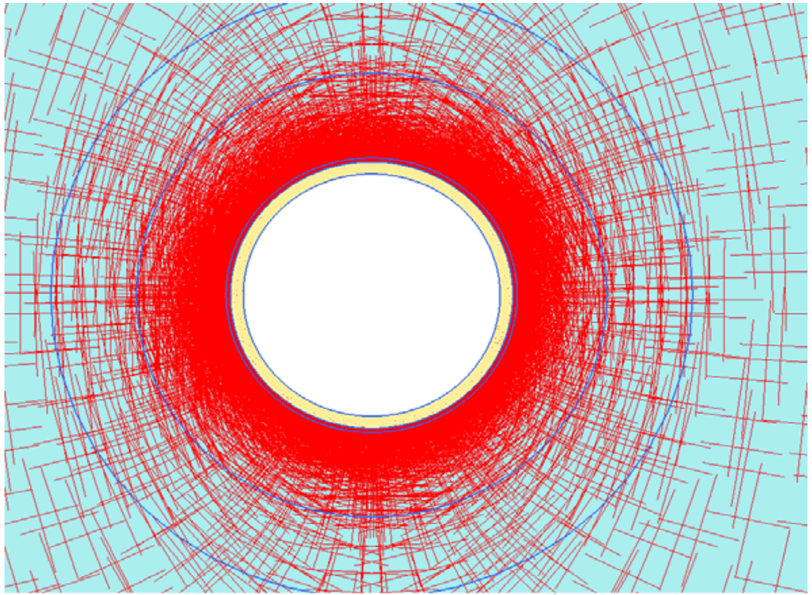
Loading- 4- Final lining installation

- Final lining is not subjected to any significant stresses

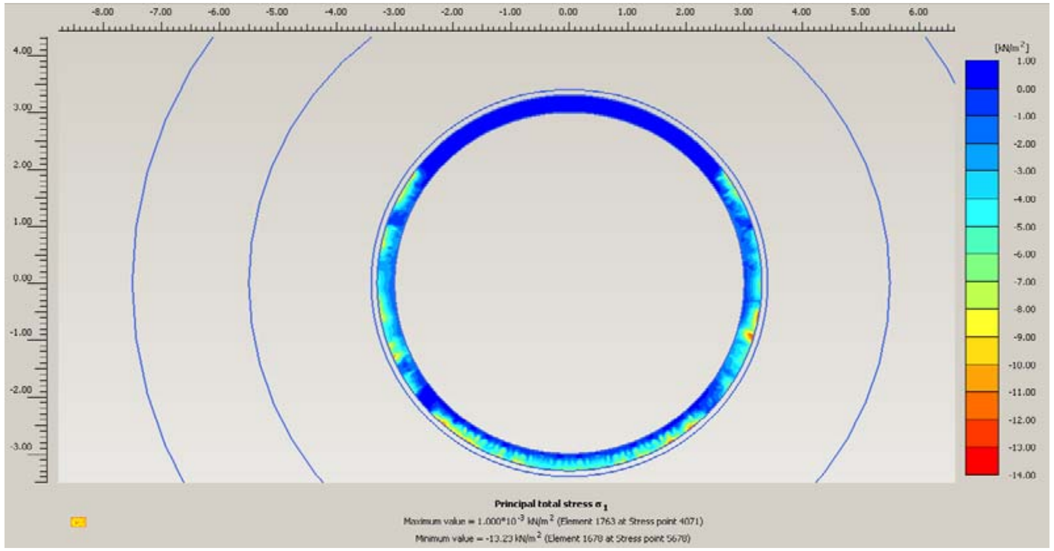
- Stresses in lining can be attributed to self weight of lining

- Stresses are maximum at extrados and minimum at intrados.

- Normal stresses and shear stresses at the interface boundary are negligible.



Principal total stress directions (scaled up $0.500*10^{-3}$ times)

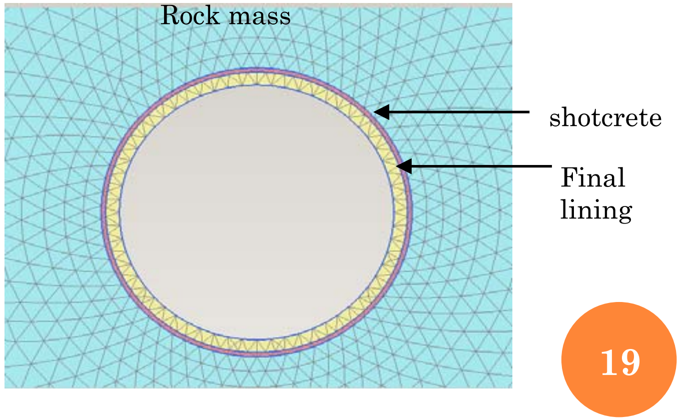


Principal total stress σ_1

Maximum value = $1.000*10^{-3}$ kN/m^2 (Element 1763 at Stress point 4071)

Minimum value = -13.23 kN/m^2 (Element 1678 at Stress point 5678)

Principal stresses in lining



RESULTS AND DISCUSSION OF RESULTS

Loading - 5 – Temperature and shrinkage effect

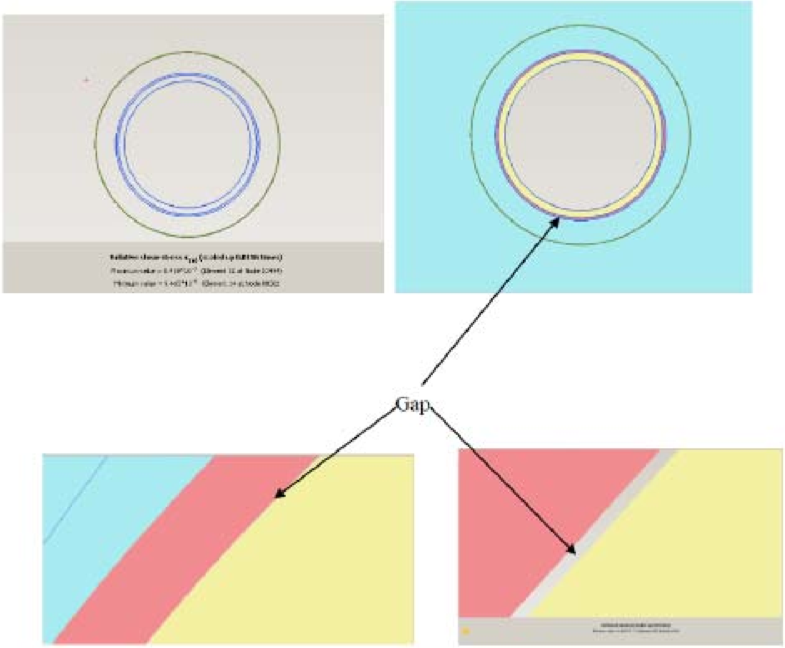
Volumetric strain calculation:

The % volumetric strain (ε_v) = $\alpha \Delta T \times 100\%$ = 0.022%

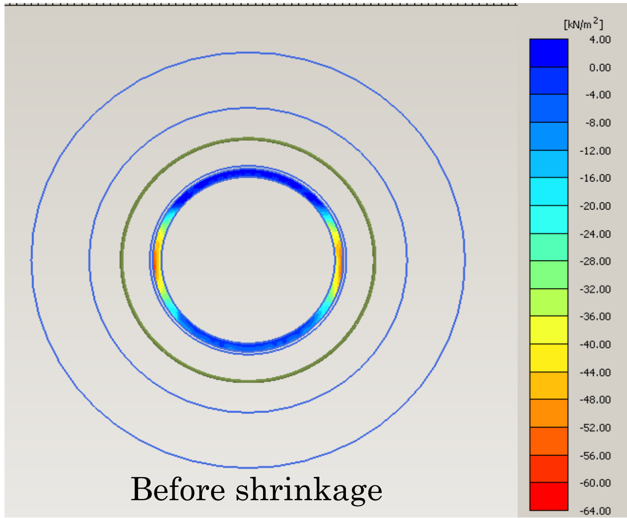
where $\alpha = 1.2 \times 10^{-5} /°C$ and $\Delta T = 18\ °C$

- Gap formation under 250 scale factor
- Zero normal stresses at the crown and sides of tunnel interface – no external forces.
- Interface as bond breaker
- compressive normal stresses in the max range of 6kN/m2 found at the invert.
- Shear stresses are negligible.

The gap indicates decoupling of the final lining from shotcrete due to temperature change – curing and initial filling with cold water.

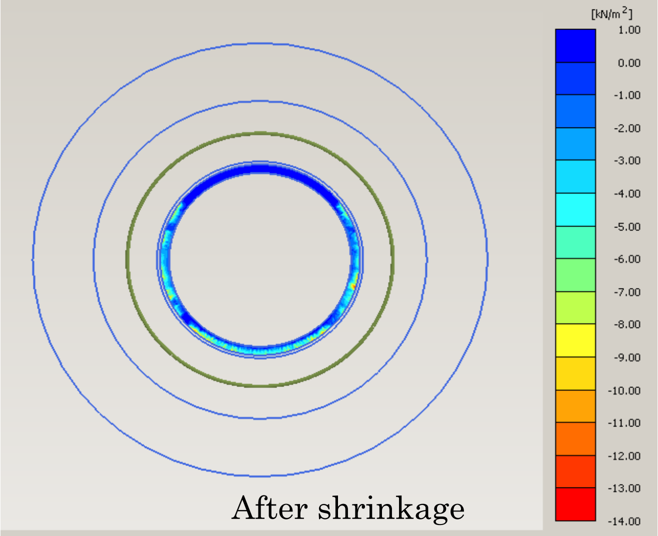


Decoupling



Before shrinkage

Principal stresses



After shrinkage

RESULTS AND DISCUSSION OF RESULTS

Loading – 6 – Shotcrete decay

$$E_c/E_r = 2 \text{ to } E_c/E_r = 0.2.$$

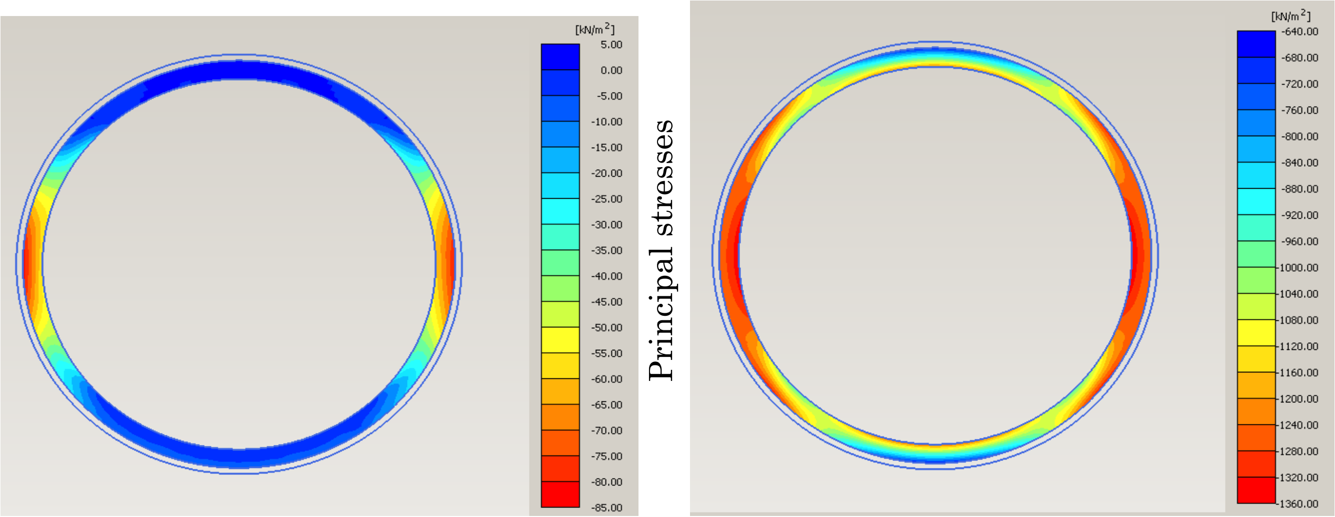
The analysis simulates the shotcrete element decay by reducing the bearing contribution of the shotcrete .

- Increase in stresses in the lining
- Increase in deformation.

- Forces in lining is approx. 30% of forces in shotcrete

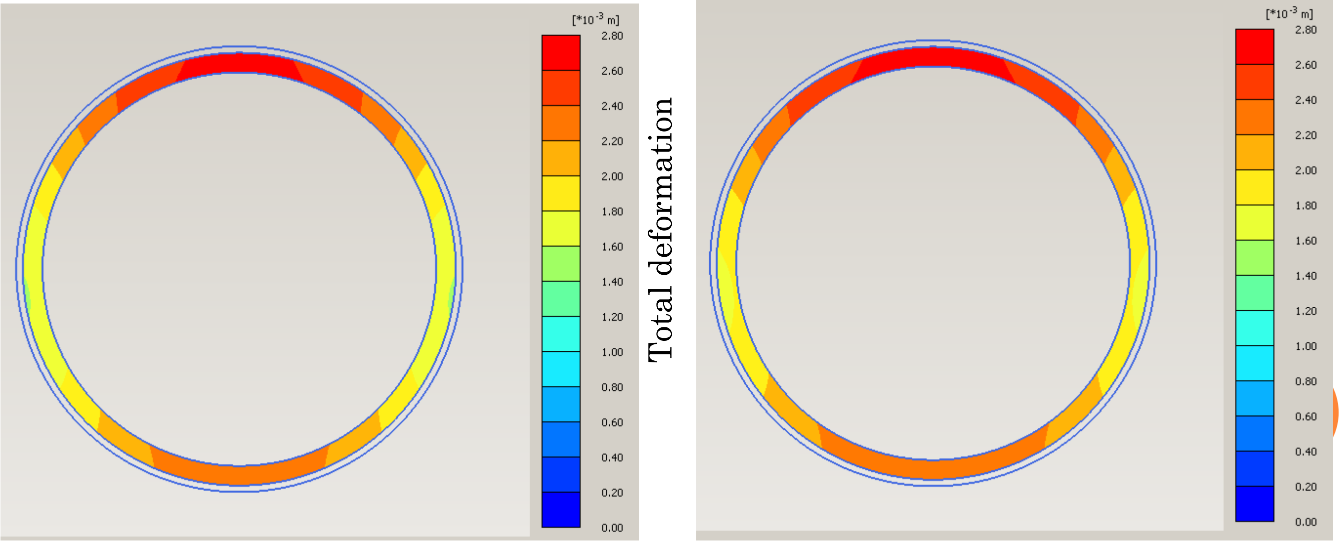
- Stresses in lining is approx. 10% of shotcrete

- The lining becomes a load bearing in the long term.



Without decay

With decay



RESULTS AND DISCUSSION OF RESULTS

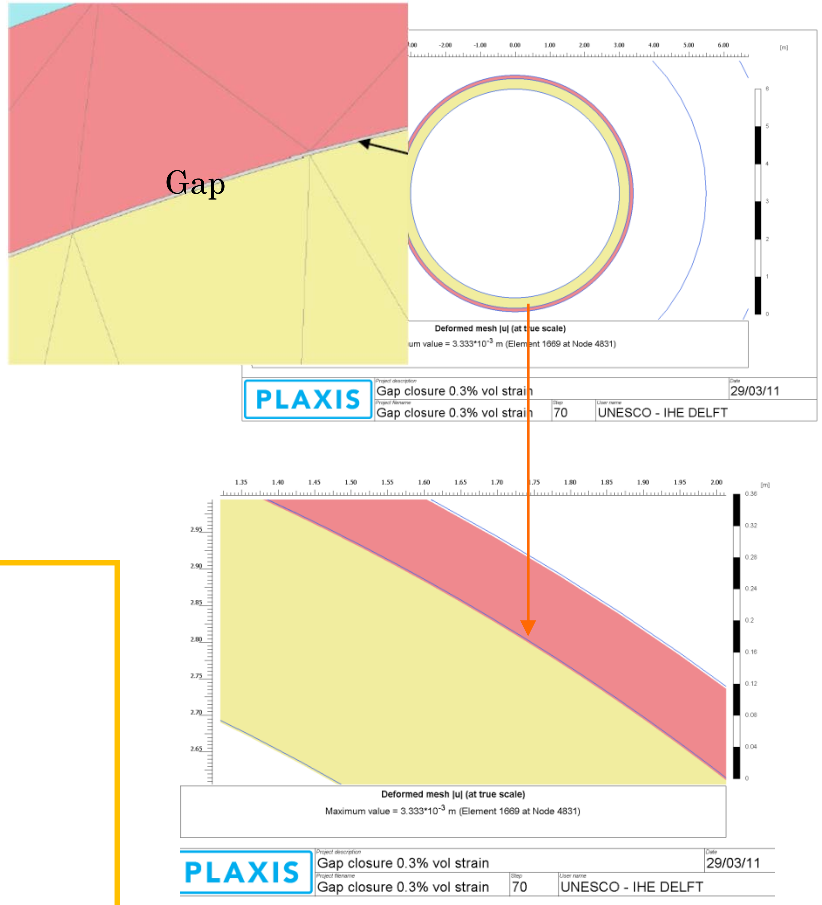
Loading – 7 – Grouting

Contact grouting

The filling of gap by numerical simulation of injecting cement grout in the void between shotcrete and final lining was impossible.

The prestressing of rock mass behind shotcrete with 0.3% reconstituted the elements.

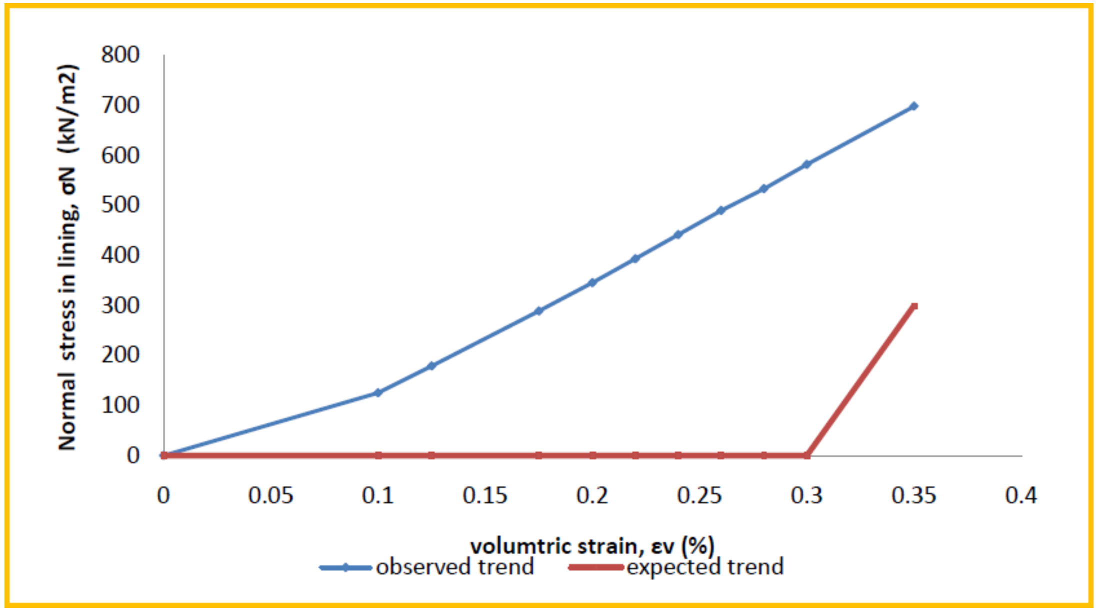
Contrary to expectation, the normal stress increased with volumetric strain.



Deformed mesh |u| (at true scale)
Maximum value = 3.333*10^-3 m (Element 1669 at Node 4831)

Project description: Gap closure 0.3% vol strain | Date: 29/03/11
Project filename: Gap closure 0.3% vol strain | Step: 70 | User name: UNESCO - IHE DELFT

Gap reconstitution with 0.3% volumetric strain



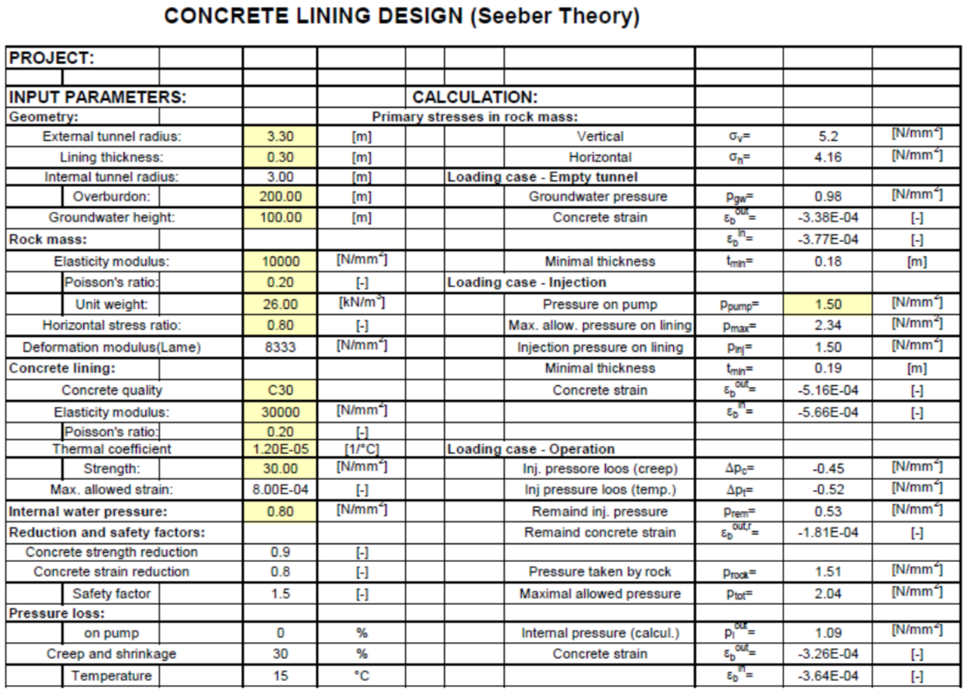
Note: expected values are just arbitrary numbers

RESULTS AND DISCUSSION OF RESULTS

Loading – 7 – Grouting

Consolidation grouting

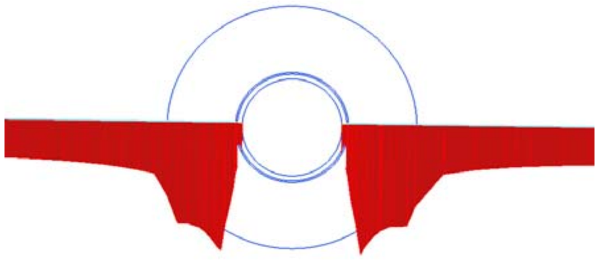
- increase the rock mass strength
- Increase the stiffness.
- Prestressing of the tunnel- increase external forces acting upon lining.
- Reduce permeability of the rock mass
- Liner becomes relatively tight.



CONCRETE LINING DESIGN (Seeber Theory)

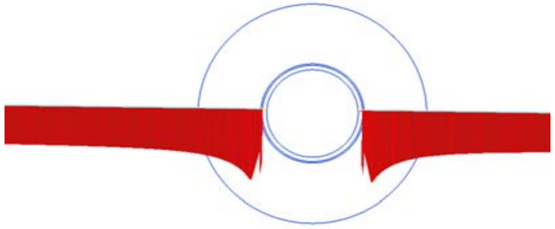
PROJECT:						
INPUT PARAMETERS:			**CALCULATION:**			
Geometry:			**Primary stresses in rock mass:**			
External tunnel radius:	3.30	[m]	Vertical	$\sigma_v=$	5.2	$[N/mm^2]$
Lining thickness:	0.30	[m]	Horizontal	$\sigma_h=$	4.16	$[N/mm^2]$
Internal tunnel radius:	3.00	[m]	**Loading case - Empty tunnel**			
Overburdon:	200.00	[m]	Groundwater pressure	$p_{gw}=$	0.98	$[N/mm^2]$
Groundwater height:	100.00	[m]	Concrete strain	$\varepsilon_b^{out}=$	-3.38E-04	[-]
Rock mass:				$\varepsilon_b^{in}=$	-3.77E-04	[-]
Elasticity modulus:	10000	$[N/mm^2]$	Minimal thickness	$t_{min}=$	0.18	[m]
Poisson's ratio:	0.20	[-]	**Loading case - Injection**			
Unit weight:	26.00	$[kN/m^3]$	Pressure on pump	$p_{pump}=$	1.50	$[N/mm^2]$
Horizontal stress ratio:	0.80	[-]	Max. allow. pressure on lining	$p_{max}=$	2.34	$[N/mm^2]$
Deformation modulus(Lame)	8333	$[N/mm^2]$	Injection pressure on lining	$p_{inj}=$	1.50	$[N/mm^2]$
Concrete lining:			Minimal thickness	$t_{min}=$	0.19	[m]
Concrete quality	C30		Concrete strain	$\varepsilon_b^{out}=$	-5.16E-04	[-]
Elasticity modulus:	30000	$[N/mm^2]$		$\varepsilon_b^{in}=$	-5.66E-04	[-]
Poisson's ratio:	0.20	[-]				
Thermal coefficient	1.20E-05	[1/°C]	**Loading case - Operation**			
Strength:	30.00	$[N/mm^2]$	Inj. pressore loos (creep)	$\Delta p_c=$	-0.45	$[N/mm^2]$
Max. allowed strain:	8.00E-04	[-]	Inj pressure loos (temp.)	$\Delta p_t=$	-0.52	$[N/mm^2]$
Internal water pressure:	0.80	$[N/mm^2]$	Remaind inj. pressure	$p_{rem}=$	0.53	$[N/mm^2]$
Reduction and safety factors:			Remaind concrete strain	$\varepsilon_b^{out,r}=$	-1.81E-04	[-]
Concrete strength reduction	0.9	[-]				
Concrete strain reduction	0.8	[-]	Pressure taken by rock	$p_{rock}=$	1.51	$[N/mm^2]$
Safety factor	1.5	[-]	Maximal allowed pressure	$p_{tot}=$	2.04	$[N/mm^2]$
Pressure loss:						
on pump	0	%	Internal pressure (calcul.)	$p_i^{out}=$	1.09	$[N/mm^2]$
Creep and shrinkage	30	%	Concrete strain	$\varepsilon_b^{out}=$	-3.26E-04	[-]
Temperature	15	°C		$\varepsilon_b^{in}=$	-3.64E-04	[-]

0.2% volumetric strain ≈ 15 bars of injection pressure



Lining with grouting

Principal total stress σ_1 (scaled up $0.500 \cdot 10^{-3}$ times)

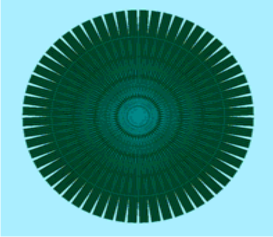


Lining without grouting

Principal total stress σ_1 (scaled up $0.500 \cdot 10^{-3}$ times)

RESULTS AND DISCUSSION OF RESULTS

Loading – 8 – Internal water pressure

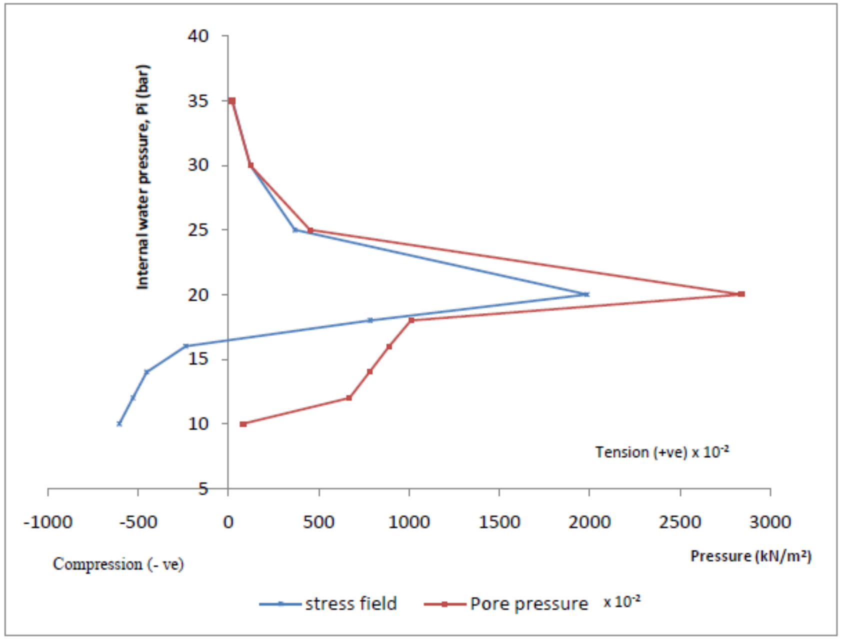


•at 18 bars of pressure, cracks occur as the magnitude of tensile stress in the liner is increasing.

•With the prestressing, as pi increases, the net pressure drive the water through the liner.
•The magnitude of increase in hydraulic pressure start to reduce.
•The hydraulic pressure generate uniform, all round counter-pressure and increase external pressure.

•The hydraulic pressure in the grouted rock mass tend to reach equilibrium value (equilibrium of flow).

•the increased external pressure decrease the gradient between internal and external pressure.

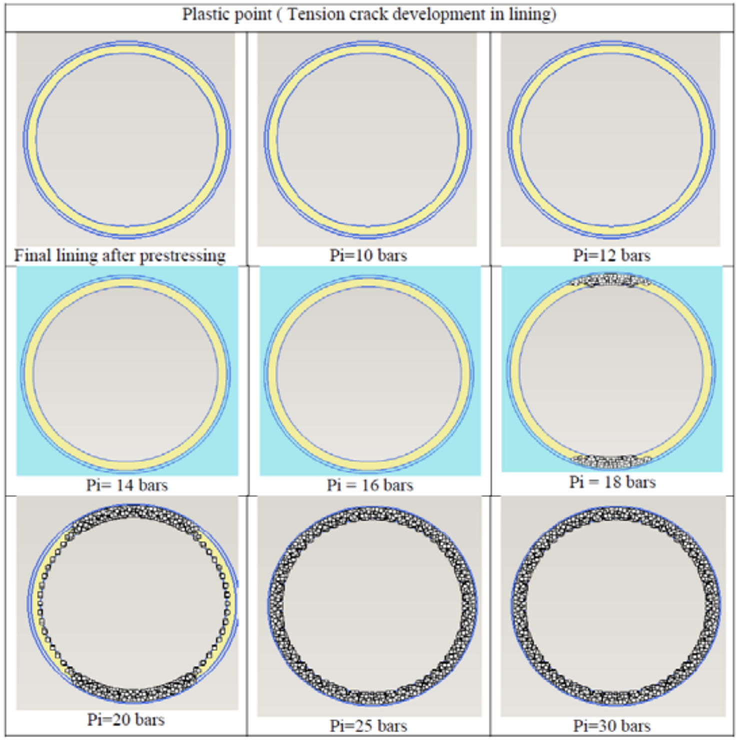


Pore pressure, Stress transformation in lining and internal water pressure

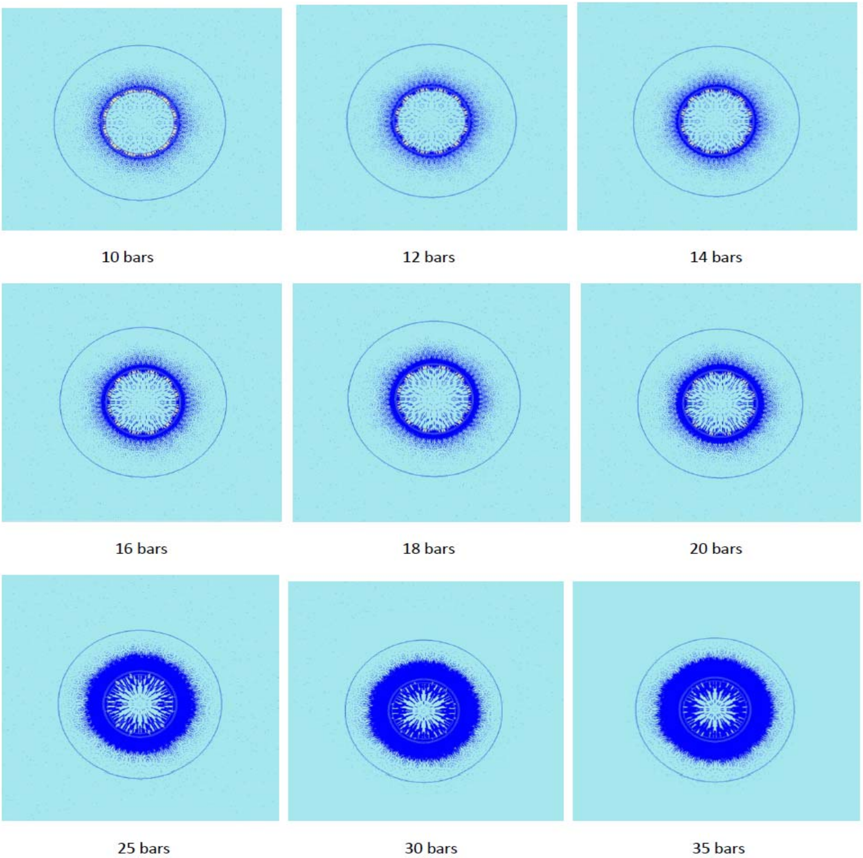
RESULTS AND DISCUSSION OF RESULTS

Loading – 8 – Internal water pressure

Consolidation analysis


Plastic point (Tension crack development in lining)
Final lining after prestressing
Pi=10 bars
Pi=12 bars
Pi= 14 bars
Pi = 16 bars
Pi = 18 bars
Pi=20 bars
Pi=25 bars
Pi=30 bars

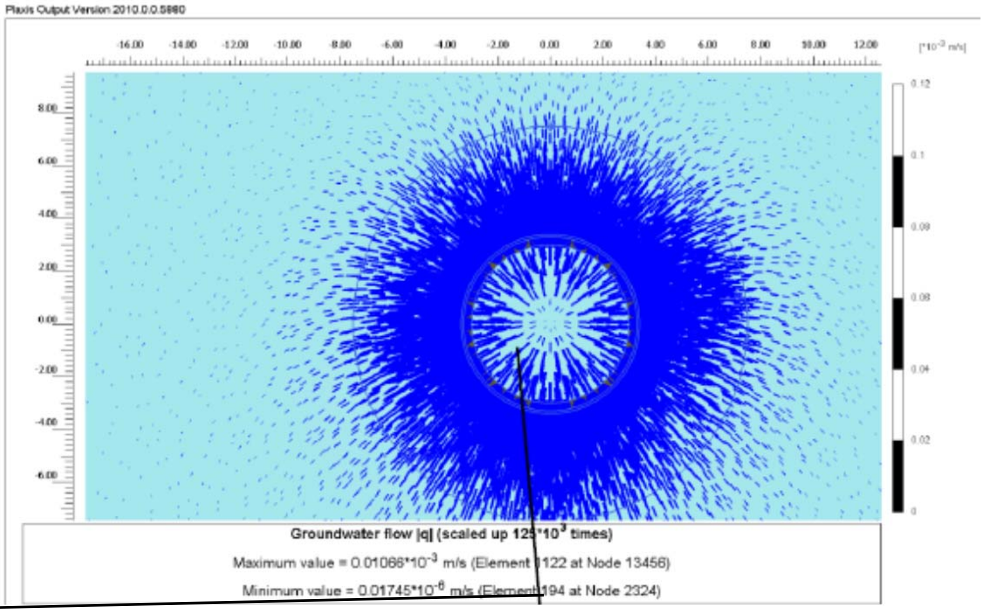
Water flow analysis


10 bars
12 bars
14 bars
16 bars
18 bars
20 bars
25 bars
30 bars
35 bars

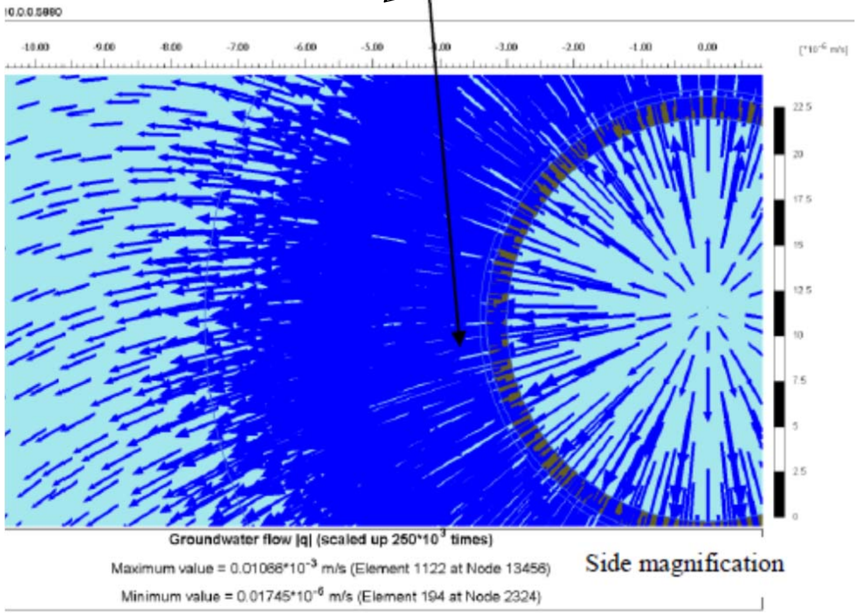
RESULTS AND DISCUSSION OF RESULTS

Loading – 8 – Internal water pressure

Water flow analysis



Zoom in

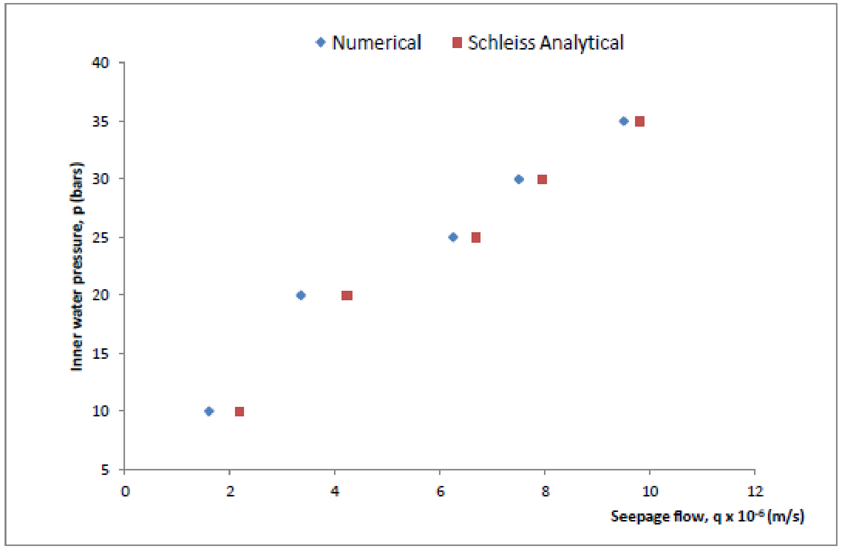


Side magnification

Seepage losses without prestressing (18 bars)

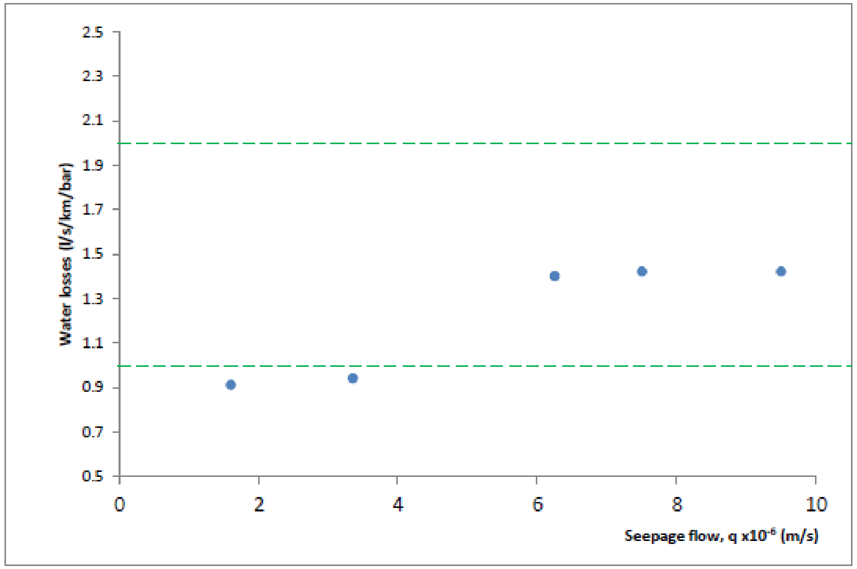
RESULTS AND DISCUSSION OF RESULTS

Loading – 8 – Internal water pressure



Internal water pressure and seepage flow in pressure tunnel

$$\frac{p_i}{\rho_w g} - \left(\frac{3}{4} r_g\right) = \frac{q}{2\pi K_r} ln \frac{q}{\pi K_r r_g} + \frac{q}{2\pi}\left[\frac{1}{K_c} ln\left(\frac{r_a}{r_i}\right) + \frac{1}{K_g} ln\left(\frac{r_g}{r_a}\right)\right]$$



Water losses in the pressure tunnel

CONCLUSIONS

•The hydraulic-mechanical interaction due to change in stress in cracked liner change permeability which results to change seepage flow in the rock zone.

•The stress field and seepage field affect each other while trying to attain a state of equilibrium.

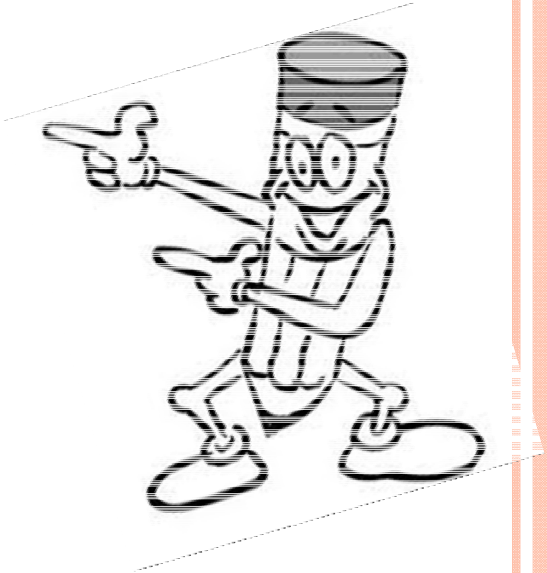
•The overall assessment showed that the program is capable of evaluating the rock mass behaviour around pressure tunnels in terms of stresses and deformation

• a tool for faster simulation of 3D arching effect, assessing the performance of lining, predicting the effect of internal pressure on the lining, reach of seepage through lining to the surrounding rock mass as well as estimating the leakage in pressure tunnels.

• The program remained not independent in the modeling of prestressing as the volumetric strain needs to be related to the injection pressure using analytical method.

•The method will cost effective as reduce much more expensive steel lined section in power waterways.

•Above all, this work recommends the finite element program as a tool for observing and/or predicting phenomena in pressure.



THANK YOU

New buddies, new idea

Consolidation and Water flow Analysis

Governing equations are:

Darcy's equation:

$$\nabla p_w + \rho_w \underline{g} + \underline{\varphi} = 0$$

Where $(g = 0, -g, 0)^T$ is a vector of gravitational acceleration.

$\underline{\varphi}$, is the vector of friction force per unit volume, between flowing fluid and soil skeleton.

$$\underline{\varphi} = -\,\underline{m}^{int}\underline{q}$$

$\underline{q}$ is the specific discharge (fluid velocity) and $\underline{m}^{int}$ is:

$$\underline{m}^{int} = \begin{bmatrix} \mu/\kappa_x & 0 & 0 \\ 0 & \mu/\kappa_y & 0 \\ 0 & 0 & \mu/\kappa_z \end{bmatrix}$$

μ is the dynamic viscosity of the fluid and κ_i is the intrinsic permeability of the porous medium.

The basic form of darcy's equation will be of form:

$$\underline{q} = \frac{k_i}{\rho_w g}\left(\nabla p_w + \rho_w \underline{g}\right)$$

Continuity equations:

$$\underbrace{\nabla^T\left(\rho_w \frac{\kappa_{rel}}{\rho_w g}\underline{\kappa}^{sat}(\nabla p_w + \rho_w \underline{g})\right)}_{\text{Mass of water flowing out from a control volume.}} = \underbrace{-\frac{\partial}{\partial t}(\rho_w n S)}_{\text{Change in mass concentration}}$$

For steady state:

$$\nabla^T\left(\rho_w \frac{\kappa_{rel}}{\rho_w g}\underline{\kappa}^{sat}(\nabla p_w + \rho_w \underline{g})\right) = 0$$

Deformation equation:

$$\bar{\bar{L}}^T\left[\bar{\bar{M}}(\bar{\bar{L}}d\underline{u}) + S_e d p_w \underline{m}\right] + d\rho\underline{g} = \underline{0}$$

And

$$\underline{d\sigma'} = \bar{\bar{M}}d\underline{\varepsilon}$$

$\bar{\bar{M}}$ *is the material stress – strain matrix*

$\bar{\bar{L}}$ *is the differential operator*

ρ *is the density of multiphase medium*

Consolidation theory:

Biot's theory, under assumption of Darcy's law of fluid flows and elastic behaviour of soil skeleton $\underline{\sigma} = \underline{\sigma}' + \underline{m}(p_{steady} + p_{excess})$

Where $\underline{m}$ $is\ a\ vector\ containing\ unit\ terms\ of\ normal\ stresscomponents\ and$

$zero\ terms\ for\ shear\ stress\ components$

Further detail (see scientific manual)

Coupled problem (see PLAXIS Flow 2010)