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Devils Slide Tunnel

Geohydrologic Study

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1 INTRODUCTION

1.1 Purpose and Scope

Groundwater is one of the factors with the highest impact during and after tunnel construction. In order to evaluate (a) the impact of groundwater on tunnel construction and (b) the long and short term impact of tunnel construction on the groundwater system analytical and numerical calculations were conducted.
Analytical calculations were applied to (a) estimate flush inflow and (b) steady state inflow for one tunnel bore. In order to calculate the transient inflow and to estimate the influence of the second tube on the groundwater inflow a numeric model was applied.

2 GEOLOGY OF THE PROJECT AREA

2.1 Description of Geology

Two stratigraphic series are expected along the tunnel alignment, namely the Montara granodiorite in the south and Paleocene age sedimentary rocks in the centre and the north. The contact between these two units has been reported to be depositional in some areas and faulted in other areas, but was interpreted in GIR [7] as a fault contact. Montara granodiorite is commonly coarse-grained and may range in composition from quartz diorite to granite, containing pegmatitic veins and hornblende-rich intrusions.
The sedimentary rocks include interbedded sandstones, siltstone and claystone as well as conglomerates with individual layers ranging from less than half a meter to perhaps several tens of meters thick. The sedimentary rocks have been extensively folded and faulted. Near the ground surface these rocks are pervasively weathered.

2.2 Lithological Units

Five lithological units are identified along the proposed alignment of the two Devil´s Slide Tunnel bores. The mineralogy, intact rock properties, and discontinuities identified for those units are summarized in EMI documents [4] to [9]. The rock mass description below is a brief compilation of this data.

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2.2.1 Granitic Rock

Moderately to very intensely fractured granitic rock. Rock mass is fresh to strongly weathered with partial disintegration of the rock mass. Within the highly fractured rock sections, the appearance of clayey fault gouge is common. Spacing of discontinuities is varying between <3 to 50 cm. The persistence of the discontinuities is in a range of <3 to >6 m.

2.2.2 Claystone

Slightly to intensively fractured claystone. Rock mass is fresh to intensely weathered, in highly fractured and sheared sections, intensely weathered to disintegrated. Spacing of discontinuities is varying between <3 and 40 cm. The persistence is in the range of <3 to >6 m. Highly fractured and sheared sections show soil like rock mass characteristics.

2.2.3 Siltstone

Slightly to intensively fractured siltstone. Rock mass is fresh to intensely weathered, in highly fractured and sheared sections, intensely weathered to disintegrated. Spacing of discontinuities is varying between <3 and 40 cm. The persistence is in a range from <3 to >6 m. Highly fractured and sheared sections show soil like rock mass characteristics.

2.2.4 Sandstone

Slightly to intensively fractured, fine to coarse grained sandstone. The degree of weathering is basically controlled by the intensity of fracturing and varies from fresh to intensely fractured. Spacing of discontinuities is varying between 3 and 40 cm. The persistence is in a range from <3 to 6 m.

2.2.5 Conglomerate

Slightly to intensively fractured, fine to coarse grained conglomerate with transition to coarse grained sandstones. The grain size of the matrix varies from clay to sand. The degree of weathering is controlled by the intensity of fracturing and varies from fresh to intensely fractured. Spacing of discontinuities is varying between 3 and 40 cm. The persistence is in a range from <3 to >6 m.

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2.3 Type of Aquifer

2.3.1 Joint Aquifer System

Since the bulk of the rock mass to be expected along the proposed tunnel alignment is bedrock, waterflow is basically taking place along discontinuities (secondary porosity). Therefore it is a function of discontinuity density, aperture and persistence of discontinuities.

2.3.2 Porous Aquifer System

Fault zones, where bedrock is pervasively fractured, constitute soil like rock mass with corresponding hydrogeological properties. Therefore this type of rock mass constitutes a porous aquifer, with a hydraulic conductivity varying from highly permeable (tectonic breccias) to impermeable (fault gouge) depending on the composition of fine grained soils.

3 DATA BASES

3.1 Hydraulic Conductivity K

The values for the hydraulic conductivity *K* were taken from the well logs and pumping tests as presented in [4], [5] and [11]. ILF has allocated the available data to the lithological units to be encountered along the tunnel alignment. The geological model was obtained from Fig 1-6 of the GIR [9]. The following table shows the *K-values* of the associated lithological units and their distribution along the tunnel alignment.

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Table 1: Localization, Number of Samples, Hydraulic Conductivity (*K-Values*) and Range of *K-Values* in orders of magnitude for the individual Lithological and Tectonic Units

Tectonic Unit	**Intervall [m]**			**Lithology**	**K-Value [cm/s]**				
	No.	Beginning	End		No. of Tests	Min	Mean	Max	Span
South Block - Footwall	1	11844	12015	Km (fw)	4	2.3E-07	5.6E-06	1.9E-05	1.92
South Block - Hanging Wall	2	12015	12355	Km (hw)	4	1.1E-05	6.3E-05	1.2E-04	1.04
Central Block: Fault B	3	12355	12375	Km/Tc	5	8.6E-07	4.1E-05	9.7E-05	2.05
Central Block	4	12375	12430	Tcs_a	4	7.5E-06	6.1E-05	9.5E-05	1.10
	5	12430	12480	Tss_d	3	3.9E-05	1.4E-04	3.4E-04	0.94
	6	12480	12600	Tc	5	8.6E-07	4.1E-05	9.7E-05	2.05
	7	12600	12700	Tss_a	4	2.5E-05	7.6E-05	1.9E-04	0.88
	8	12700	12750	Tcs_a	4	7.5E-06	6.1E-05	9.5E-05	1.10
	9	12750	12870	Tss_c	4	1.3E-06	2.9E-05	6.0E-05	1.66
	10	12870	12880	Tss_a	4	2.5E-05	7.6E-05	1.9E-04	0.88
	11	12880	12930	Tc	5	8.6E-07	4.1E-05	9.7E-05	2.05
	12	12930	12950	Tcs_a/Tss_a	4	7.5E-06	6.1E-05	9.5E-05	1.10
North Block: Fault C	13	12950	12958	Tcs	1	3.5E-03	3.5E-03	7.0E-03	0.30
	14	12958	12970	Tcs	1	1.0E-05	1.0E-05	2.0E-05	0.30
North Block	15	12970	13095	Ts	6	4.5E-05	1.7E-04	4.0E-04	0.95

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Since values for the hydraulic conductivity *K* should be regarded as point-specific data, their significance is spatially limited. Even on short distances the *K-value* may vary over several orders of magnitude for the same lithology. To take this into account, the analytical calculations have been conducted for different *K-value* to meet these variations:

- The smallest measured value: the inflow calculated with this value has to be expected even under otherwise ideal conditions. The calculated inflow for this value is considered to be the most favorable case.
- The arithmetic mean for all measured values: the arithmetic mean is the sum of all arguments divided by the number of arguments. Its value is always above the ones of the geometric or the harmonic means and it is influenced more strongly by outliers.
- The highest measured value: the calculated inflow for this value is considered to be the most unfavorable case.

It should be kept in mind that the actual variation of the *K-value* can vary from the available min and max values, as insitu testing can only be carried out randomly. In cases where only single *K-value* are available, these were assumed to represent the average condition. Instead of a minimum value, the average *K-value* has been used, while the maximum *K-value* was assumed to range up to one third of an order of magnitude above the average value.

For the favorable/unfavorable case scenarios, the minimum/maximum water level and the minimum/maximum value for the hydraulic conductivity for the individual sections were applied. The results for average-based calculations were derived using the mean of the water level and the arithmetic mean for the lithology's hydraulic conductivity.

3.2 Hydraulic Heads

Data regarding hydraulic heads along the tunnel alignment were obtained from the GIR, Fig. 1-6 [9]. These data constitute a snap shot of the groundwater table taken during the soil investigation. Eighteen boreholes have been equipped with piezometers for long term groundwater monitoring. From the available monitoring data [13] it becomes apparent that the annual fluctuation of the groundwater table is small. The standard deviation is, with the exception of BH 02-5B (2.78 m), below 2 m (for most piezometers between 0.5 and 2 m).

The available groundwater monitoring data including hydrographs are summarized in appendix 1. The below table shows minimum and maximum groundwater levels for each of the tectonic and lithological units.

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Table 2: Groundwater Levels applied for Calculations (the shown water levels are groundwater levels above the crown)

Tectonic Unit	Intervall [m]				Lithology	Tunnel-Water Level [m]		
	No.	Beginning	End	Length		Min	Max	Δh
South Block - Footwall	1	11844	12015	171	Km (fw)	0	10	10
South Block - Hanging Wall	2	12015	12355	340	Km (hw)	34	65	31
Central Block: Fault B	3	12355	12375	25	Km/Tc	23	34	11
Central Block	4	12375	12430	50	Tcs_a	20	146	126
	5	12430	12480	50	Tss_d	102	120	18
	6	12480	12600	120	Tc	82	118	36
	7	12600	12700	100	Tss_a	65	116	51
	8	12700	12750	50	Tcs_a	56	113	57
	9	12750	12870	120	Tss_c	50	109	59
	10	12870	12880	10	Tss_a	50	108	58
	11	12880	12930	20	Tc	42	108	66
	12	12930	12950	20	Tcs_a/Tss_a	40	107	67
North Block: Fault C	13	12950	12958	8	Tcs	18	40	22
	14	12958	12970	12	Tcs	16	18	2
North Block	15	12970	13095	20	Ts	3	16	13

3.3 Hydrological Data

Hydrological data are needed in order to calculate the groundwater recharge rate of the project area. This value, which represents the max. possible steady state tunnel inflow, is used for a plausibility crosscheck of the calculated inflow data. Furthermore hydrological data are essential input for the numerical modeling.

The hydrological data were basically obtained from the Geoscience report [11]. Where possible, other sources were used to check their plausibility. For further analysis Geoscience divided the project area into 4 subareas, estimating values for evapotranspiration, surface runoff and groundwater recharge for each of these subareas. ILF used the mean values of the 4 subareas. Groundwater recharge was calculated by subtracting the evaportranspiration and the surface runoff from the precipitation.

Precipitation

Precipitation data from the San Pedro Valley Park Station of the period between 1989 and 1998 have been used to estimate the mean precipitation of the project area. The period covers wet and dry cycles with a min. value of 501 mm and a max. value of 1515 mm. The mean value of 922 mm is expected to cover the historical average.

Since the San Pedro Valley Park Station is situated 3 km inland of the project site, Geoscience has calculated adjustment factors for the project area. Applying these factors, the annual precipitation in the project area is estimated to range between 700 and 825 mm, depending on the elevation. The derived mean value of 763 mm meets the long term average of the Santa Cruz Station of 758 mm (California Cooperative Snow Surveys [3]).

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Evapotranspiration

Geoscience has estimated a potential evapotranspiration rate of 1016 mm (based on information from the NOAA and the mean annual evaporation map in the United States). This corresponds to the information available from the CIMIS (California Irrigation Management Information System) [2], according to which a value of 1003 mm can be estimated (assuming that the project area including the catchment area is situated in evapotranspiration zones 1 to 3).

Based on the potential evapotranspiration an actual transpiration rate with a mean value of 457mm has been calculated.

Surface Runoff

Surface runoff is a function of precipitation, infiltration rate and the slope of an area. Geoscience estimated infiltration rates for the project area based on information provided by the United States Department of Argriculture's (USDA's) State Soil Geographic (STATSGO) with a mean value of 5.84 mm/h.

Based on this estimate a surface runoff with an average value of 241 mm was calculated.

Catchment Area

According to the watershed model of Geoscience the catchment area of the Devil's Slide Tunnel covers 6.1 km². ILF estimated a catchment area of app. 6.9km² considering the following boundaries:

The catchment area roughly forms a triangle (see Figure 1). The north and west boundaries are formed by the 250 feet elevation line running parallel to the Pacific Ocean and the San Pedro Valley. Since the tunnel is situated above 250 feet elevation, groundwater level below this elevation will not be affected by tunnel construction. Furthermore, precipitation falling below this elevation cannot contribute to the tunnel inflow, unless artesic groundwater conditions occur (not assumed on project area scale). The southeastern boundary is formed by the Martini Creek, draining towards the SW into the Pacific Ocean and the South Fork of the San Pedro Creek, draining towards the NW into the Pacific Ocean.

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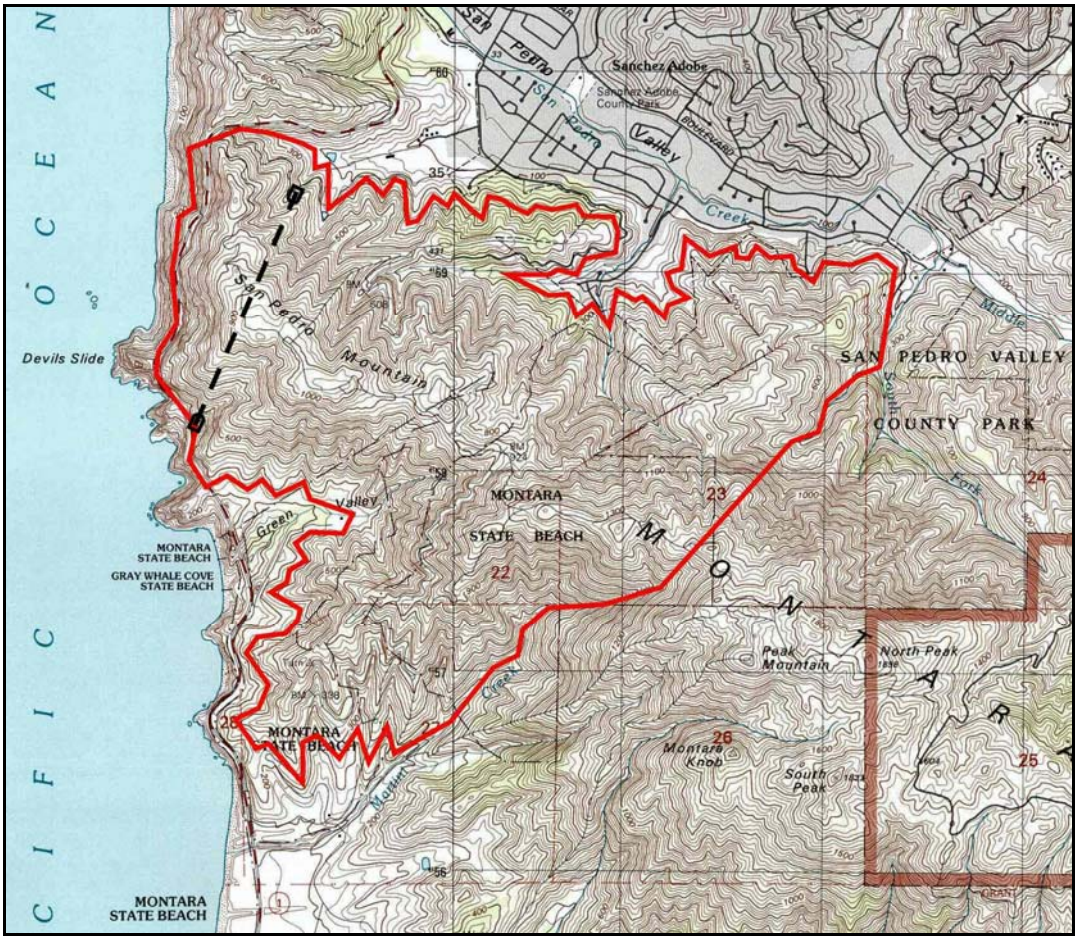
Figure 1: Geomorphological Catchment Area for the Devil's Slide Tunnel Project (red framed)



Areal Groundwater Recharge

Based on the above estimates ILF has calculated a groundwater recharge rate for the project area of 65 mm which equals 2.1 l/s*km². For comparison Geoscience calculated an average groundwater recharge rate of 70 mm for the project area which equals 2.2 l/s*km².

Considering a catchment area of 6.9 km² (Geoscience: 6.1 km²) the areal groundwater recharge is approximately **14.5 l/s** (Geoscience: 13.4 l/s).

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4 ANALYTIC CALCULATIONS

4.1 Methodology

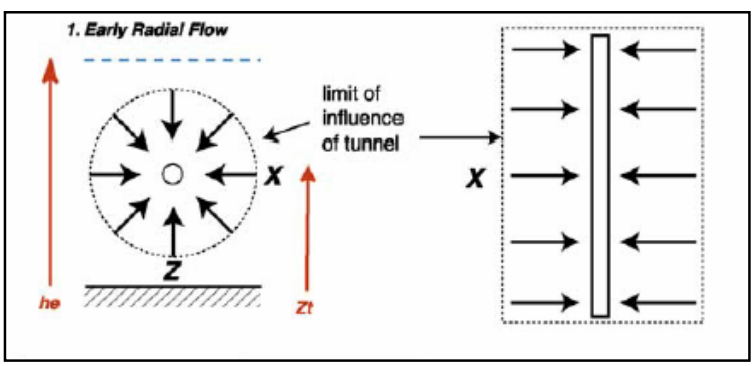
In order to predict groundwater inflow into the tunnel analytical calculations were conducted. ILF calculated tunnel inflow using different analytical solutions and included sensitivity analyses for the different tunnel sections. Four different approaches were applied:

- Method by Goodman (1965)
- Method by Jacob & Lohmann (1952).
- Cylinder formula
- Method by Herth & Arndts (1973)

For tunnel construction three basic stages of tunnel inflow were considered:

Stage one (see Figure 2) shows radial flow, assuming that that the tunnel has been built instantaneously. The radial flow acts as long as the drawdown of the hydraulic head induced by the draining tunnel does not reach the groundwater table or any other flow boundary [17]. This initial stage of tunnel inflow is equated with flush inflow.

Figure 2: Early radial inflow (flush inflow) during stage 1



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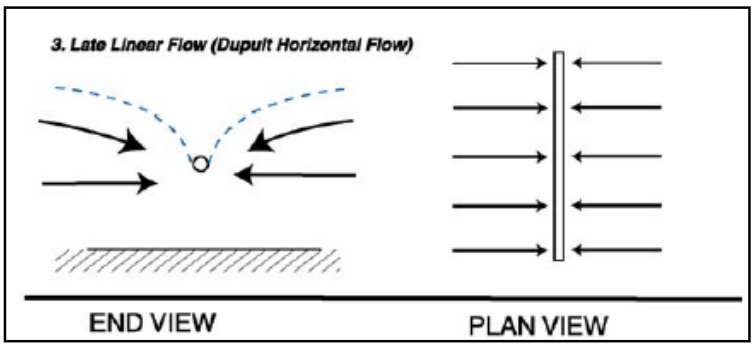
Once the water table drawdown drops to the elevation of the tunnel crown, the inflow can be approximated by purely horizontal flow [17]. This stage of tunnel inflow is equated with steady state inflow.

Figure 3: Late linear inflow (steady state) during stage 3



The transient stage 2 (i.e. between the time when the drawdown cone first affects the groundwater table and the time when the drawdown reaches tunnel elevation) has not been considered in the analytical calculations due to the lack of necessary input data for the analytical calculations. The transient stage has been modeled using numerical modeling.

For calculating the flush inflow the approaches of Goodman [14] and Jacob & Lohmann (in [17]) were used. Steady state inflow was calculated using the cylinder formula (in [18]) and the approach by Herth & Arndts [15].

The calculations were carried out as a parameter study using the minimum, the average and the maximum value of the input data and applying a tunnel radius of 5.6 m.

4.2 Goodman's Approximation

Polubarinova-Kochina (1962) [19], Goodman (1965) [14], Lei (1999) [16] and El Tani (1999) [12] provide different ways for deriving the same equation of approximating the tunnel inflow for tunnels that are overlain by a constant water column that is much larger than the tunnel's radius (eq. 1).

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The equation itself resembles Thiem's well formula [20], but it assumes a special geometry with respect to the drawdown.

$$Q = 2\pi \cdot K \frac{\Delta h}{\ln\left(\frac{2 \cdot \Delta h}{r}\right)} \qquad \text{eq. 1}$$

Q: tunnel inflow [m^3 s^{-1}]
K: hydraulic conductivity [m/s]
Δh: distance between the center of the tunnel and the groundwater table [m]
r: tunnel radius [m]

Goodman's formula is probably the most commonly used approximation for calculating early tunnel inflow rates. It also serves as the basis of the calculations of the empirical method proposed by Heuer [10].

4.3 Asymptotic Solution by Jacob and Lohmann

A second approach for calculating the first phase radial flow is described by the asymptotic constant head solution of Jacob & Lohmann (1952) (in [17]):

$$Q(t) = \frac{4\pi \cdot K \cdot L \cdot \Delta h}{2.3 \cdot \log\left(\frac{2.25 \cdot K \cdot L \cdot t}{S \cdot r^2}\right)} \qquad \text{eq. 2}$$

Q: tunnel inflow [m^3 s^{-1}]
t: point of time [s]
K: hydraulic conductivity [m/s]
L: length of the tunnel section under construction [m]
Δh: *distance between the center of the tunnel and the groundwater table [m]*
S: *elastic storativity*
r: *tunnel radius [m]*

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4.4 Cylinder Formula

Tunnel inflow can be estimated with a very basic approach by assuming the tunnel to be a perfect cylindrical horizontal drain (eq 3). The inflow has been estimated for the actual tunnel surface with an infinitesimal interface.

$$Q = 2\pi \cdot r \cdot i \cdot K \qquad \text{eq. 3}$$

Q: tunnel inflow [$m^3\ s^{-1}$]
i: hydraulic gradient [-]
K: hydraulic conductivity [m/s]
r: tunnel radius [m]

For solid rock with a low hydraulic conductivity K, the gradient i is usually assumed to be i=1. This of course is a strong simplification, since the hydraulic gradient changes significantly along the drawdown's surface. While the gradient is steep and therefore i>>1 close to the tunnel itself, it's declining towards the effective radius of the drawdown. For most of the surface of the drawdown, the gradient is i<<1.

4.5 Herth & Arndts Empiric Approach

Another approach for calculating late linear flow of the steady state is the empirical formula by Herth & Arndts (1973) [15] as stated in eq. 4.

$$Q = \left(0{,}73 + 0{,}27 \cdot \frac{h_2 - h_1}{h_2}\right) \cdot \frac{K}{r_2} \cdot \left(h_2^{\,2} - h_1^{\,2}\right) \qquad \text{eq. 4}$$

Q: tunnel inflow [$m^3\ s^{-1}$]
K: hydraulic conductivity [m/s]
h_1: hydraulic head for the proximate observation point with respect to the tunnel center [m]
h_2: hydraulic head for the remote observation point with respect to the tunnel center [m]
r_2: distance from the tunnel center to the remote observation point [m]

4.6 General Assumptions

All of the analytical solutions for the tunnel inflow describe the hydraulic conductivity as a function of the depression cone's geometry.

El Tani [12] suggests that the drainage area for a tunnel section without major fluctuations of the groundwater table can be assumed to be four times as wide as the

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groundwater head measures vertically. The results for the calculations according to Goodman are based on this assumption.

Similar assumptions are implied for most of the analytical solutions. Assumptions for the interrelationship of the depression cone and its effective radius had to be made only for the application of Herth & Arndt's approach. As the available data does not cover site-specific data from corresponding pumping tests, values were back-calculated from the results of Geoscience's study [11].

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5 RESULTS OF THE ANALYTICAL CALCULATIONS

5.1 Flush Tunnel Inflow

Table 3: Inflow Calculations for One Tube

			Goodmann, 1965			Jacob & Lohman 1952			Arith. Av.			Weighted Av.			ILF	Heuer 1995
Tectonic Unit	**Intervall**		**Q for 100m tunnel progress [l/s]**													
	No.	Length	Fav.	Av.	Unfav.	Fav.	Av.	Unfav.	Fav.	Av.	Unfav.	Fav.	Av.	Unfav.		
South Block - Footwall Block	1	171		0,30	0,94			0,36	0,00	0,30	0,65	0,53	4,80	12,25	5,0	5,0
South Block - Hanging Wall Block	2	340	0,94	6,85	15,59	0,66	7,28	20,57	0,80	7,07	18,08					
Central Block: Fault B	3	25	0,06	3,17	8,30	0,02	1,98	6,29	0,04	2,57	7,30	1,22	8,31	26,02	8,5	9,5
Central Block	4	50	0,48	11,01	22,02	0,21	9,09	26,35	0,34	10,05	24,18					
	5	50	6,97	27,13	68,25	6,13	8,81	85,13	6,55	17,97	76,69					
	6	120	0,13	7,22	19,22	0,08	6,35	21,84	0,10	6,78	20,53					
	7	100	3,25	12,35	37,19	2,37	11,40	41,92	2,81	11,88	39,55					
	8	50	0,88	11,15	18,22	0,58	9,25	20,39	0,73	10,20	19,31					
	9	120	0,14	4,31	11,22	0,07	3,39	10,59	0,11	3,85	10,91					
	10	10	2,72	11,22	35,30	1,82	5,34	39,03	2,27	8,28	37,16					
	11	50	0,08	5,89	18,02	0,04	5,16	19,80	0,06	5,52	18,91					
	12	20	0,71	10,11	17,51	0,41	4,38	7,22	0,56	7,24	12,37					
North Block: Fault C	13	8	212,73	272,81	660,00	180,37	180,37	427,00	196,55	226,59	543,50	12,38	17,54	45,03	18,0	30,4
	14	12	0,58	0,59	1,21	0,23	0,23	0,49	0,40	0,41	0,85					
North Block	15	125	3,17	8,17	23,07	0,31	3,44	11,67	1,74	5,80	17,37					

Legend:

Fav.: Favorable Scenario, using min. values of hydraulic conductivity and hydraulic heads within the considered tunnel section

Av.: Average Scenario, mean values of hydraulic conductivity and hydraulic heads within the considered tunnel section

Unfav.: Unfavorable Scenario, max. values of hydraulic conductivity and hydraulic heads within the considered tunnel section

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5.2 Steady State Tunnel Inflow

Table 4: Steady State Inflow Calculations for One Tube

			Cylinder Formula						Herth & Arendts 1973						ILF	Heuer 1995
Tectonic Unit	**Intervall**		**Q for tunnel section [l/s]**													
	No.	Length	Fav.	Av.	Unfav.	Fav.	Av.	Unfav.	Fav.	Av.	Unfav.	Fav.	Av.	Unfav.	incl. Safety Factor of 2	
South Block - Footwall Block	1	171	0,01	0,34	1,14	1,33	7,91	15,50		0,05	0,32	1,27	10,70	26,85	3	8,8
South Block - Hanging Wall Block	2	340	1,32	7,57	14,36				1,27	10,65	26,53					
Central Block: Fault B	3	25	0,01	0,36	0,85	2,09	12,50	26,53	0,00	0,16	0,44	4,13	26,22	69,89	6	24,0
Central Block	4	50	0,13	1,26	1,67				0,06	2,18	5,08					
	5	50	0,69	2,52	5,98				2,00	7,94	20,41					
	6	120	0,04	1,73	4,10				0,06	3,29	9,16					
	7	100	0,88	2,66	6,69				1,63	4,56	14,69					
	8	50	0,13	1,26	1,67				0,14	3,02	5,36					
	9	120	0,05	1,22	2,53				0,08	2,75	7,85					
	10	10	0,09	0,27	0,67				0,13	0,60	2,05					
	11	50	0,02	0,72	1,71				0,01	1,03	3,49					
	12	20	0,05	0,50	0,67				0,04	0,70	1,35					
North Block: Fault C	13	8	NOT APPLICABLE						3,36	5,41	14,90	3,52	6,81	20,26	7	10,8
	14	12							0,01	0,01	0,03					
North Block	15	125							0,15	1,38	5,33					
Sum												8,92	43,72	117,00	16	43,6

Legend:

Fav.: Favorable Scenario, using min. values of hydraulic conductivity and hydraulic heads within the considered tunnel section

Av.: Average Scenario, mean values of hydraulic conductivity and hydraulic heads within the considered tunnel section

Unfav.: Unfavorable Scenario, max. values of hydraulic conductivity and hydraulic heads within the considered tunnel section

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5.3 Discussion of Results

5.3.1 General Comments

The results of the analytical calculations produced by ILF were compared with the results of the analytical calculations using Heuers approach (see Appendix 4), carried out independently by Greg Korbin [10]. It has to be mentioned that the results represent tunnel inflow for one tube only. The results of this comparison are provided below.

5.3.2 Flush Tunnel Inflow

Table 3 summarizes the results of the calculations of flush tunnel inflow for 100 m long tunnel sections. To compare ILF results with the results of the Heuer method, ILF calculated the arithmetic mean of the results according to Goodman and Jacob & Lohman for each lithological unit (average values). In a second step the results were summarized for each of the three tunnel sections by calculating the weighted average. These average results are generally in accordance with the results provided by the Heuer method.

Only the calculated tunnel inflow for the North Block is app. 2/3 of the value according to Heuer. This deviation is due to the fact that the Heuer approach does not weight the influence of the length of the tunnel section in the same way this is done by calculating the weighted average. The impact of the tunnel length increases with the increase of the variation of the *K-values*. It was found that tunnel length has little impact on the results in the South and Central Block since the *K-values* show only minor variations compared to the setting in the North Block. Here the variation of *K-values* is high due to the occurrence of the highly permeable zone along fault C.

Choosing 100 m length has been found to be unreasonable for calculating flush tunnel inflow since, especially in highly permeable zones such as fault zones, this approach results in unrealistic high values of several 100 l/s. The extent of fault zones in the project area is limited to few meters. Hence, such high values are will not be encountered during actual tunneling. This fact should be considered when reviewing the results shown in Table 3.

5.3.3 Steady State Tunnel Inflow

The applied numerical calculations are simple solutions for calculating both flush and steady state groundwater inflow into tunnels. However all those methods are based on assumptions which have to be considered assessing the results.

The analytical methods generally assume that the groundwater body is laterally unconfined. In contrary the groundwater's body at the Devil's Slide Tunnel Project area is

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laterally confined due to the triangular shape of the geomorphological catchment area (see Figure 1). Based on the current standard of knowledge it is believed that the hydrogeological catchment area does not exceed the morphological catchment area. Assuming a catchment area of 6.9 km², an average groundwater recharge ratio of 14.5 l/s can be expected (see Chapter 3.3). This value can be understood as a maximum value for steady state tunnel inflow (the assumption is on the safe side since it implies that all groundwater recharge drains towards the tunnel). Therefore ILF proposes to use the average of the calculated "Favorable Values" (see Table 4) for average steady state tunnel inflow prediction to one tunnel tube, applying an uncertainty factor of two. This factor of safety is found reasonable due to uncertainties in connection with groundwater inflow along fault zones and the occurrence of unknown highly permeable zones. The results show a total discharge for one tunnel tube of 16 l/s.

Due to experiences it is not reasonable to double the proposed inflow value in order to consider the second tunnel tube. In fact numerous tunneling projects have shown that the groundwater inflow increases by the factor of 20% due to the second tube ([10] and [17]). This would result in a steady state inflow of app. 20 l/s for both tunnels. In order to consider the fluctuation of the annual precipitation ratio (dry and wet years) a supplement of 50 % is added, leading to a final figure for steady state inflow of **30 l/s**.

6 NUMERICAL MODELING

6.1 Evaluation of the 2001 Geoscience Model

6.1.1 Overview

The model is generated and calibrated based on the findings in Woodward-Clyde (1996) [4].

The more recent data provided by EMI ([5] to [9]) have improved and changed the understanding of the geological structure as well as the spatial distribution of the hydraulic heads. The new findings with respect to the general ground water flow direction or the impact of faults (e.g. fault C and fault 02-5) are not represented within the 2001 numeric model.

Numerical modeling was carried out in order to determine tunnel inflow for the second tube.

6.1.2 Watershed Model

Four subareas are accounted for and named Eastern, Southern, Western and Northern Subarea. For the precipitation, meteorological data from the station at San Pedro Valley Park between 1989 and 1998 is used and fitted to the model area's morphology by

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calculating an adjustment factor from the isohyetal map. In addition, the potential evapotranspiration is calculated. Estimates of the infiltration rate of soil in each subarea are derived from the United States Department of Agriculture's State Soil Geographic database.

The hydrologic budget for the subareas is computed using the program 'Hydrological Simulation Program – Fortran' (HSPF; Bicknelli et al. 1997 [1]). It uses the given precipitation and potential evapotranspiration to estimate surface runoff, evapotranspiration and ground water recharge within the watershed as shown in Table 5.

Table 5: Results of the watershed model

	Eastern	Southern	Western	Northern
Adjustment Factor	0.89	0.83	0.76	0.81
Calculated Precipitation [mm/a]	821	765	701	747
Surface Runoff [mm/a]	279	254	203	229
Evapotranspiration [mm/a]	483	457	431	457
Ground Water Recharge [mm/a]	76	76	51	76
Mass balance error [%]	2.1%	2.9%	-2.3%	2.0%
Infiltration Rate [mm/h]	5.84	8.13	6.35	4.83

6.1.3 Numeric Model

6.1.3.1 Boundary Conditions

For the Southern, Western and Northern model boundaries, boundary conditions of the first kind (= Dirichlet boundary condition) are applied. Neither piezometric heights nor mass transfer rates for the calculated in-/outflow are documented in the Geoscience report.

For the Eastern model boundary, a boundary condition of the second kind (= Neumann boundary condition) is applied. The inflow rate is assumed to be 4,600 m³/d (843 gpm). It is reported in the Geoscience Report to act similar to a boundary condition of first kind, but again, no piezometric elevation is mentioned.

The tunnel sections are considered as drains with an initial drain capacity of 77% of their final conductance. The Geoscience Report does not mentioned at what point in time the total drain capacity is applied.

6.1.3.2 Parameterization

The model generation and manipulation is performed using the MODFLOW-based software packages Ground Water Vistas 3.0 and GMS 3.1.

The finite-difference model's grid covers an area of approximately 6 km² and consists of 267,150 cells (137x195 cells in 10 layers), 117,357 of which are considered to be active

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during the computations. Each model cell is approximately a cube, measuring 15 m on a side.

The hydraulic conductivities in Y-direction for the different lithological units are estimated during the calibration by PEST (calibration module of MODFLOW). The values for the hydraulic conductivities in X-direction are lower than the ones in Y-direction by a factor of 5 with respect to the steep gradient.

The hydraulic conductivity in vertical direction is assumed to be 0.02 m/d (2.5E-05 cm/s) independent from the lithological units, including the faults. This assumption is not sustained and it does not correspond with the standard interpretation of the material properties, which would suggest to assume igneous rocks to behave almost isotropic, sedimentary rocks being less permeable in the bedding direction by an order of one magnitude and open faults being highly permeable in their direction of strike and normal to their dip direction, but showing a low permeability in their dip direction.

The storativity is assumed to be independent from the lithological units as well and to be a decreasing function of the depth instead.

6.1.3.3 Calibration Results

The steady state model calibration is performed against ground water level elevation data from 8 boreholes from July and August 1996. The results diverge from the measured ground water levels within a range of 1.2 m (P-10-96) to 16.2 m (P-9-96) with an average of 5.8 m.

Table 6 shows the hydraulic conductivities in Y-direction for the different lithological units estimated by the calibration module PEST.

Table 6: Hydraulic conductivities estimated during the calibration

	Estimated Hydraulic Conductivity in Primary Direction	
Lithologic Unit	[m/day]	[cm/s]
Granitic Rocks	0.028	3.2E-05
Sedimentary Rocks	0.079	9.1E-05
Contorted Sedimentary Rocks	0.065	7.5E-05
Landslide Deposits	0.013	1.5E-05
Fault B Zone	0.305	3.5E-04
Fault C Zone	0.574	6.6E-04
Fault D Zone	0.174	2.0E-04

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6.1.3.4 Results

The actual model is computed for three different stages: the construction phase, the finalization of the construction after 1.6 years and a near-equilibrium stage 5 years after the construction was completed. Two scenarios are applied:

- Scenario 1 – Average Conditions. An areal recharge of 50-70 mm/a is applied along with values for the effective porosity of 0.02-0.1 and for the storativity of 0.0001-0.02.
- Scenario 2 – Unfavorable Conditions. The values of storativity and effective porosity are increased to account for the uncertainty of these parameters to ranges between 0.0005-0.05 respectively 0.05-0.12. The values for the recharge under wet year conditions are doubled. These assumptions are not sustained by calculations.

For either scenario, the hydraulic head above the tunnel at the end of the tunneling process is predicted to be less than 5 m along the entire tunnel and within 1 m for most of the tunnel. Further predictions include that the hydraulic heads will have lowered to within 2 m of the tunnel elevation along the entire tunnel alignment by five years after the end of construction. The suggested tunnel inflow rates are shown in Table 7.

Table 7: Tunnel inflow rates (for both tunnel tubes)

	Scenario 1		**Scenario 2**	
	[gpm]	[l/s]	[gpm]	[l/s]
Total Tunnel Inflow during Construction	339	21.4	495	31.2
Total Tunnel Inflow after Completion	119	7.5	183	11.5
Total Tunnel Inflow at Near-Equilibrium	55	3.5	101	6.4

6.1.4 Recommendations

Geoscience already recommended that the monitoring of the existing boreholes and additional ground water levels in new borings are collected on a regular basis to improve the understanding of the regional ground water table's behavior and establish baseline conditions before the start of construction. These recommendations include also collecting data from monitoring any streamflow or spring in the model area.

During tunneling, it is recommended that all established piezometers, drainages and springs are continuously monitored in order to asses the effects that the tunneling may have on the ground water discharge.

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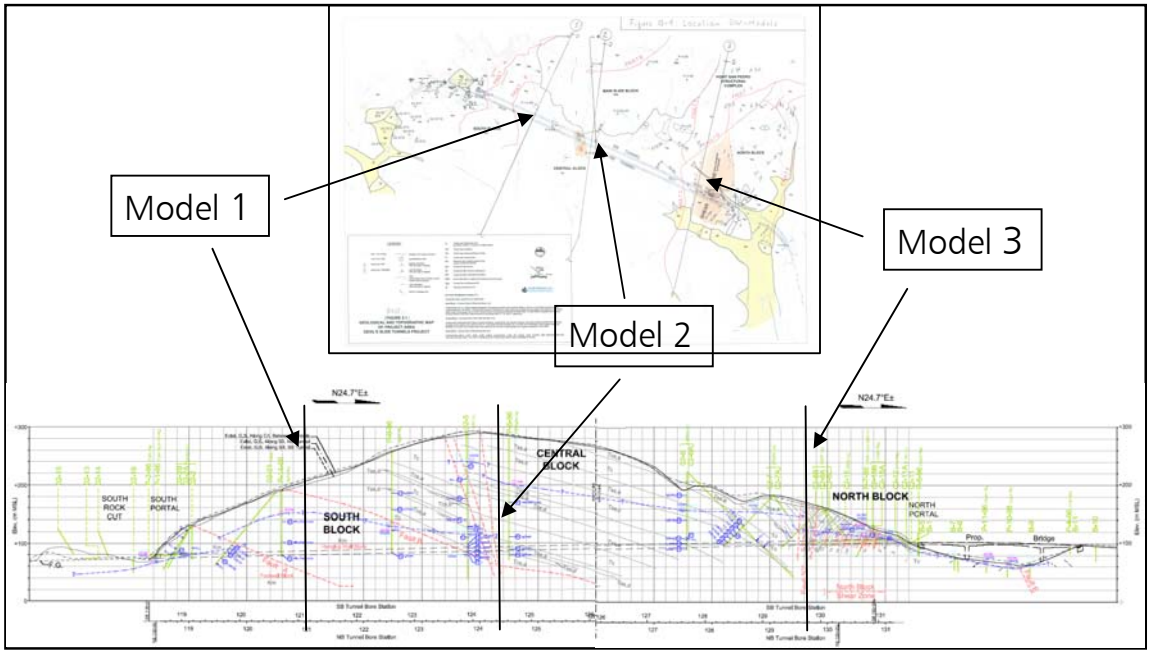
6.2 Vertical Models

6.2.1 General

Three vertical models situated at different hydrological units have been generated for the estimation of groundwater table – drawdown and inflow to both tunnel tubes. The models are orientated rectangular or nearly rectangular to tunnel axis, groundwater flow will be forced to this direction by the drainage effect of the tunnels (see figure 0-1 in Appendix 3).

Figure 4: Position of the numerical models



The vertical models are simulated as a slice of 1 m thickness. The results can therefore be regarded as inflow for a 1 m long tunnel section. Tunnel inflow for the entire tunnellength is obtained by interpolating and extrapolating these results.

Finite-Element-Technology (FE) based on the software "Feflow" (developed by Wasy-Berlin) was used for modeling. Each model consists of approximately 5000 triangular elements. FE allows a good fit to curved and polylined boundaries like tunnel sections, topography or faults. The baseline of the models was set to a level of –60 m.

The tunnels were simulated by setting the conductivity of the elements within the tunnel boundary to a value of 100 m/s and integrating a constant head boundary at a level close to the tunnel invert. Using this method no numerical instabilities due to internal holes occurred.

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First a steady-state calculation for simulating the undisturbed situation (before tunnel excavation) was performed. The calibration of conductivity and groundwater recharge showed that the existing groundwater table could only be simulated by applying low conductivity values (lower bound values) and a groundwater recharge of 150 mm/year (upper bound value).

Using the calibrated models a transient calculation was performed including the following time steps and boundary conditions:

Time step 0:	Start of NB-Tunnel excavation
Period 0 – 30 days:	300 steps with length 0.1 d each
Time step 30 days:	Start of SB-Tunnel excavation
Period 30 – 400 days:	370 steps with length 1 d each

Note that the following stations refer to stationing of the NB tunnel.

6.3 Model 1 – South Block, km 121+50

Ref.: Figures 1-1 to 1-6 in Appendix 3.

6.3.1.1 Geometry

The model is situated at km 121+50 in the South Block, with a groundwater table of 150 m above sea level. The orientation of the model is perpendicular to the tunnel axis. The model extends from the shoreline 1415 m to the east. The two tunnels are situated 405 respectively 431 m to the east of the shoreline.

6.3.1.2 Boundary Conditions

<u>Left side (shore):</u> Constant head level is 0 to 15 m

The condition was chosen for simulating groundwater outflow of the hanging wall layer with higher conductivity, indicated by springs close to the shoreline in the vicinity of fault A (see Figure 4).

<u>Right Side:</u> Constant head level is 180 m

This level corresponds to results of the 2001 Geoscience model. The flow direction at this boundary is towards the east and it can be assumed that the groundwater table will not to be affected by the tunnel over this distance.

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6.3.1.3 Material Parameters

South Block Footwall: K = 2.3E-7 cm/s, Porosity: 1%

South Block Hanging Wall: K = 2.5E-5 cm/s, Porosity: 2%

The two blocks are separated by Fault A which extents between level 0 to 30 m (see Appendix 3 Figure 1-3).

During the calibration process the K-value of the hanging wall formation was varied between 1.1 and 6.3 E-5 cm/s. At a K-value of 2.5 E-5 cm/s the groundwater table showed good coincidence.

6.3.1.4 Results

Drawdown of the Groundwater Table

Figure 1-4 in Appendix 3 shows the groundwater table at time steps 0, 30, 100 and 400 days. After 30 days groundwater drawdown is as much as approx. 8 m, while after 100 days drawdown has reached tunnel level (= 79 m above sea level). The level of undisturbed groundwater table is 150 m above sea level, the magnitude of drawdown 71 m.

Tunnelinflow

Tunnelinflow is shown on Figure 1-6 (Appendix 3). The hydrograph of the inflow to the NB tunnel shows an initial inflow of 13.2 $m^3/d*m$ followed by a rapid decrease of the inflow rate to approx. 2.7 $m^3/d*m$ after 30 days. After the start of the excavation of SB-Tunnel (timestep 30 days) an initial inflow of 6.6 $m^3/d*m$ to the SB tunnel occurs (total tunnel inflow of 8.9 $m^3/d*m$), also followed by a rapid decrease. After 400 days the total inflow reaches 1 $m^3/d*m$, 1/3 from the SB-Tunnel and 2/3 from the NB-Tunnel.

6.3.2 Model 2 – Central Block, km 124+38

Ref.: Figures 2-1 to 2-6 in Appendix 3

6.3.2.1 Geometry

The model is situated at km 124+38 close the southern boundary of the Central Block at the location of the higher groundwater table (157 m). The orientation of the model is parallel to the sub vertical fault zone 02-5 (= 8° to north and 79° to tunnel axis) and perpendicular to tunnel axis (see Figure 4). The model extends from shoreline 1450 m to

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the east. The two tunnels are situated 442 respectively 471 m to the east of the shoreline.

6.3.2.2 Boundary Conditions

Left side (shoreline): Constant head level 0 m

Right side: Constant head level 180 m

This level corresponds to results of the 2001 Geoscience model. The flow direction at this boundary is towards the east and it can be assumed that the groundwater table will not to be affected by the tunnel over this distance.

6.3.2.3 Material Parameters

South Block Hanging Wall: K = 0.8E-5 cm/s; Porosity: 2 %

Central Block: K = 2.0E-5 cm/s; Porosity: 2 %

The two blocks are separated by Fault B which extents between level 15 and 90 m, the tunnels are located at a level of 60 m, see Figure 2-3 (Appendix 3).

A groundwater table of 160 m was achieved by using relatively small K-values.

6.3.2.4 Results

<u>Drawdown of the Groundwater Table</u>

Figure 2-4 in Appendix 3 shows the groundwater table at time steps 0, 30, 100 and 400 days. After 30 days groundwater drawdown is approx. 8m, after 100 days approx. 50 m, while after 200 days drawdown has reached tunnel level (=84 m above sea level). The level of undisturbed groundwater table is 157 m above sea level, the magnitude of drawdown 73m.

<u>Tunnelinflow</u>

Figure 2-6 in Appendix 3 shows the hydrographs of the inflow to the NB and SB tunnel as well as the sum of both. The hydrograph for the NB tunnel shows an initial inflow of 13.5 m^3/d*m followed by a rapid decrease to 2.4 m^3/d*m after 30 days. The initial inflow to the SB tunnel (time step 30 days) reaches 6.8 m^3/d*m, also followed by a rapid decrease. After 400 days the total inflow reaches 1 m^3/dm, 1/3 from the SB-Tunnel and 2/3 from the NB-Tunnel.

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6.3.3 Model 3 – North Block Fault C, km 129+56

Ref.: Figures 3-1 to 3-6 in Appendix 3.

6.3.3.1 Geometry

Model 3 is situated at km 129+56 within the high permeable zone of Fault C at the southern boundary of the north block. The orientation is almost parallel to Fault C (= 78.5° to tunnel axis). The model extends from the shoreline 1079 m to the east. The tunnels are situated 585 respectively 614 m east of the shoreline.

6.3.3.2 Boundary Conditions

Left side (shore):	Constant head level 0 m.
Right side:	Cauchy-boundary condition level 90 m
Spring 100 m east of tunnel:	Cauchy-boundary condition level 122 m

With the help of the Cauchy boundary condition (setting inflow transfer rate to 0) physically incorrect inflow into model can be avoided during groundwater drawdown.

<u>Concentration of Groundwater Recharge</u>

The calibration of the model resulted in much higher inflow rates (factor 10) than a recharge of 150 mm would provide. This can be explained by the draining effect of the permeable Fault C zone, which dewaters the adjacent northern part of the Central Block with its higher groundwater table.

6.3.3.3 Material Parameters

North Block Fault C: see Figure 3-3 in Appendix 3

Central and right profile section (area of fault C): K = 3.5 E-3cm/s

Left profile section (relatively undisturbed rock mass): K = 1E-5 cm/s

For the left profile section the lower k-value was applied in order to raise the groundwater table to 123 m.

6.3.3.4 Results

<u>Drawdown of the Groundwater Table</u>

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Figure 3-4 in Appendix 3 shows the groundwater drawdown at time steps 0, 2, 40 and 400 days. After 2 days, drawdown already reaches tunnel level (= 97 m above sea level). The level of the undisturbed groundwater table is 123 m above sea level; the magnitude of drawdown is 26 m. The model shows that groundwater drawdown takes place much faster due to the high conductivity of Fault C.

Tunnelinflow

Figure 3-6 in Appendix 3 shows the tunnel inflow. The initial inflow to the NB tunnel of 95.6 $m^3/d*m$ is followed by a rapid decrease to 29.2 m^3/dm after 30 days. The initial inflow to the SB tunnel is 20 $m^3/d*m$ followed by a rapid decrease of the inflow rate. After 400 days the tunnel inflow drops down to 10 $m^3/d*m$ for both tunnels. These inflow rates are about a factor of 10 higher than the inflows rates of model 1 and 2. It has to be mentioned that these high inflow rates only occur within the highly permeable zone of Fault C (estimated length of approx. 8 m in tunnel direction) and not along the entire North Block section.

7 PROPOSED GROUNDWATER INFLOW

7.1 Flush Inflow during tunnel excavation

The following aspects are essential in order to determine the flush flow during the tunnel driving process:

- Tunnel driving is a dynamic process with stepwise actions between excavation and installation of various support elements.
- The water ingress starts during drilling of boreholes for blasting and is further activated by the drillholes for rock dowels.
- By application of shotcrete the rock surface is partially sealed. However, due to the fact that the shotcrete lining is not designed to carry water loads, it will be drained by weep holes.
- Furthermore, the invert of the heading will not be sealed allowing water to ingress. Therefore, most of the groundwater ingress takes place within a distance of several diameters of excavation from the face.
- Due to the fast decrease of groundwater inflow, near steady state conditions will be reached 20 m behind the tunnel face (assuming an average tunneling progress of 3 m per day excavation rate).
- At fault zones, the lowering of the water table will take longer to develop. Therefore, it is assumed that the flush inflow extends over 100 m

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Based on these considerations, the following data are derived for inclusion in the GBR:

- The flush inflow to the first tunnel during heading excavation of the granite and sandstone / marl of the North, Central and South Block is derived by extrapolating the initial inflow to the NB tunnel in the South and Central Block (model 1 and 2) for a 20 m long tunnel section.
- The expected inflow for the 2nd tube was extrapolated from the initial inflow to the SB tunnel in model 1 and 2 for a 20 m long tunnel section. These data shall be regarded as maximum for a 20 m long tunnel section starting at the tunnel face.
- Additional tunnel inflow has to be considered when major fault zones occur within the 20 m tunnel heading section. For fault C and the North Block (both tubes) the available data of model 3 were extrapolated. The values for maximum tunnel inflow along fault zones in the South and Central Block were estimated from the initial inflow to model 3 considering the available conductivity data, the groundwater table above crown and the geological description of fault zones in these blocks. For the second tube, the figures of the first tube were reduced. The following table show the flush tunnel inflow for sections with and without fault zones for both tunnel tubes.

Table 9: Flush tunnel inflow

	Heading 1st Tube		**Heading 2nd Tube**	
Tectonic unit	**General [l/s]**	**Fault [l/s]**	**General [l/s]**	**Fault [l/s]**
South block	3	8	1,5	3.5
Central block	3	12	1,5	3.5
North Block	3	15	1	6

With the current state of knowledge, 2 major faults are expected in the South Block (fault A and B), 1 major fault in the Central Block (fault 02-5) and 3 major faults respectively shear zones in the North Block (fault C plus 2 major shear zones).

During construction a maximum *steady state* inflow behind the 20m heading section of **app. 50 l/s** is expected.

7.2 Steady State Groundwater Inflow

After completion of the final lining, the ground water table is expected to reach the steady state even for layers with high hydraulic conductivity.

The result of the analytical method for the long time inflow is about 16 l/s for one tunnel, which fits to the ground water recharge of 14,5 l/s.

Experiences indicate that it is unreasonable to double the proposed inflow value in order to consider the second tunnel bore. In fact numerous tunneling projects have shown that

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the groundwater inflow increases by the amount of 20% due to the second bore ([10] and [17]). This would result in a steady state inflow of approximately 20 l/s for both tunnels. In order to consider the fluctuation of the annual precipitation ratio (dry and wet years) a supplement of 50 % is added, resulting in a final figure for steady state inflow of **30 l/s** for both tunnel tubes.

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Appendix 1: Groundwater Monitoring Data

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Groundwater Monitoring Data of the 1996 Piezometers [1]

WATER SURFACE ELEVATION - ft (APPROX.)																
DATE	P-2-96		P-7-96		P-9-96		P-10-96		P-11-96		B-5		B-7		B-9	
	[ft]	[m]	[ft]	[m]	[ft]	[m]	[ft]	[m]	[ft]	[m]	[ft]	[m]	[ft]	[m]	[ft]	[m]
12.09.2001	216.84	66.11	373.48	113.86	424.90	129.54	199.37	60.78	217.00	66.16						
24.10.2001	218.93	66.75	372.70	113.63	425.11	129.60	198.69	60.57	216.60	66.03						
27.11.2001	220.09	67.10	372.19	113.47	425.15	129.62	199.32	60.77	216.82	66.10						
29.01.2002	223.40	68.11	375.00	114.33	425.65	129.77	203.47	62.03	220.68	67.28						
26.02.2002	224.95	68.58	375.00	114.33	425.62	129.76	203.77	62.12	222.76	67.91						
27.03.2002	222.67	67.89	375.00	114.33	426.00	129.87	203.85	62.15	221.36	67.49						
29.04.2002	221.33	67.48	375.00	114.33	426.20	129.94	202.95	61.87	220.10	67.10						
30.05.2002	220.56	67.24	375.00	114.33	426.35	129.98	202.32	61.68	219.38	66.88						
28.06.2002	220.99	67.37	375.00	114.33	426.40	130.00	201.75	61.51	218.85	66.72						
26.07.2002	218.71	66.68	375.00	114.33	426.40	130.00	201.15	61.32	218.48	66.61						
30.08.2002	219.24	66.84	375.00	114.33	426.37	129.99	201.00	61.28	218.62	66.65						
27.09.2002	219.76	67.00	375.00	114.33	426.65	130.07	200.59	61.15	218.33	66.56						
04.11.2002	219.68	66.97	375.00	114.33	426.48	130.02	200.00	60.97	217.48	66.30	284.14	86.63	250.90	76.49	212.82	64.88
26.11.2002	220.51	67.23	375.00	114.33	426.67	130.08	200.35	61.08	217.58	66.33						
30.12.2002	229.00	69.81	375.00	114.33	426.82	130.12	203.32	61.99	221.62	67.57						
31.01.2003	223.15	68.03	375.00	114.33	426.80	130.12	203.52	62.05	220.32	67.17						
28.02.2003	232.38	70.85	375.00	114.33	426.88	130.14	203.89	62.16	222.80	67.93						
28.03.2003	223.33	68.09			427.25	130.26										
30.04.2003	223.27	68.07	375.00	114.33	427.12	130.22	203.89	62.16	221.09	67.40						
30.05.2003	222.19	67.74	375.00	114.33	427.30	130.27	203.55	62.06	220.64	67.27						
30.06.2003	219.24	66.84	375.00	114.33	427.21	130.24	202.36	61.69	219.46	66.91						
31.07.2003	220.05	67.09	375.00	114.33	427.35	130.29	201.42	61.41	218.82	66.71						
29.08.2003	219.30	66.86			427.19	130.24	200.69	61.18	218.32	66.56						
30.09.2003	219.45	66.90			427.12	130.22	200.15	61.02	217.97	66.45						
30.10.2003	217.82	66.41			427.18	130.23	199.48	60.82	217.60	66.34						
26.11.2003	219.62	66.96			427.00	130.18	199.30	60.76	217.36	66.27						
05.01.2004	237.99	72.56			427.04	130.19	202.90	61.86	222.59	67.86						
28.01.2004	223.09	68.01			426.98	130.17	203.12	61.93	220.12	67.11						
27.02.2004	241.83	73.73			426.98	130.17	204.65	62.39	223.57	68.16						
31.03.2004	223.04	68.00			427.08	130.20	203.70	62.10	220.72	67.29						
30.04.2004	219.31	66.86			426.86	130.14	202.70	61.80	219.67	66.97						
30.06.2004	218.02	66.47			427.78	130.42	201.18	61.33	218.50	66.61						
29.07.2004	219.70	66.98			427.95	130.47	200.67	61.18	218.10	66.49						
31.08.2004	217.95	66.45			427.83	130.43	200.27	61.06	217.71	66.37						
Minimum	216.84	66.11	372.19	113.47	424.90	129.54	198.69	60.57	216.60	66.03						
Maximum	241.83	73.73	375.00	114.33	427.95	130.47	204.65	62.39	223.57	68.16						
Mean	222.28	67.77	374.68	114.23	426.70	130.09	201.80	61.52	219.42	66.90						
Median	220.30	67.16	375.00	114.33	426.87	130.14	201.75	61.51	218.85	66.72						
Stand. Deviation	5.47	1.67	0.82	0.25	0.75	0.23	1.72	0.52	1.90	0.58						

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Groundwater Monitoring Data of the 2002 Piezometers [1]

WATER SURFACE ELEVATION - ft										
DATE	02-2B	02-5A	02-5B	02-5C	02-6	02-7	02-8C	02-18A	02-18B	02-20
	[m]	[m]	[m]	[m]	[m]	[m]	[m]	[m]	[m]	[m]
27.09.2002					158.11	155.63			112.83	
04.11.2002		241.06	121.53	79.58	157.46	155.06			112.45	45.32
26.11.2002		240.45	118.77	79.58	157.14	154.73			112.25	45.69
30.12.2002		233.43	114.35	79.60	156.76	154.34			112.15	46.84
31.01.2003		233.12		79.58	156.69	154.20			115.37	47.56
28.02.2003		239.12	111.37	79.52	156.67	154.18			114.80	47.77
28.03.2003		239.25	110.83	79.56	157.05	154.53			115.45	47.99
30.04.2003		239.21	110.79	79.54	157.48	154.85			114.89	47.89
30.05.2003		239.15	111.13	79.48	158.04	155.35			115.31	47.50
30.06.2003		239.08	111.44	79.48	158.39	155.61			114.54	46.95
31.07.2003		239.15	111.62	79.46	158.39	155.55			113.87	46.43
29.08.2003		239.26	111.74	79.46	158.20	155.35			113.41	46.08
30.09.2003		239.12	111.84	79.44	157.90	155.10			112.95	45.77
30.10.2003		239.12	111.86	79.40	157.56	154.77			112.44	45.53
26.11.2003		239.17	111.90	79.50	157.34	154.55			112.19	45.57
05.01.2004		239.02	111.96	79.50	156.95	154.24			111.77	46.47
28.01.2004		239.04	112.02	79.52	156.86	154.16			113.75	46.92
27.02.2004		238.99	112.06	79.44	156.95	154.24			115.60	47.29
31.03.2004		239.02	111.84	79.42	157.88	155.08	125.30		115.42	47.60
30.04.2004		239.00	111.68	79.46	158.32	155.47	125.26		114.62	47.19
30.06.2004		239.04	111.62	79.42	158.07	155.20			113.50	46.27
29.07.2004		239.10		79.42	157.79	154.94			113.02	45.92
31.08.2004		239.06		79.44	157.48	154.65			112.54	45.63
Minimum		233.12	110.79	79.40	156.67	154.16	125.26		111.77	45.32
Maximum		241.06	121.53	79.60	158.39	155.63	125.30		115.60	47.99
Mean		238.73	112.65	79.49	157.54	154.86	125.28		113.70	46.64
Median		239.11	111.84	79.48	157.48	154.85	125.28		113.50	46.66
Stand. Deviation		1.83	2.78	0.06	0.58	0.50	0.03		1.28	0.87

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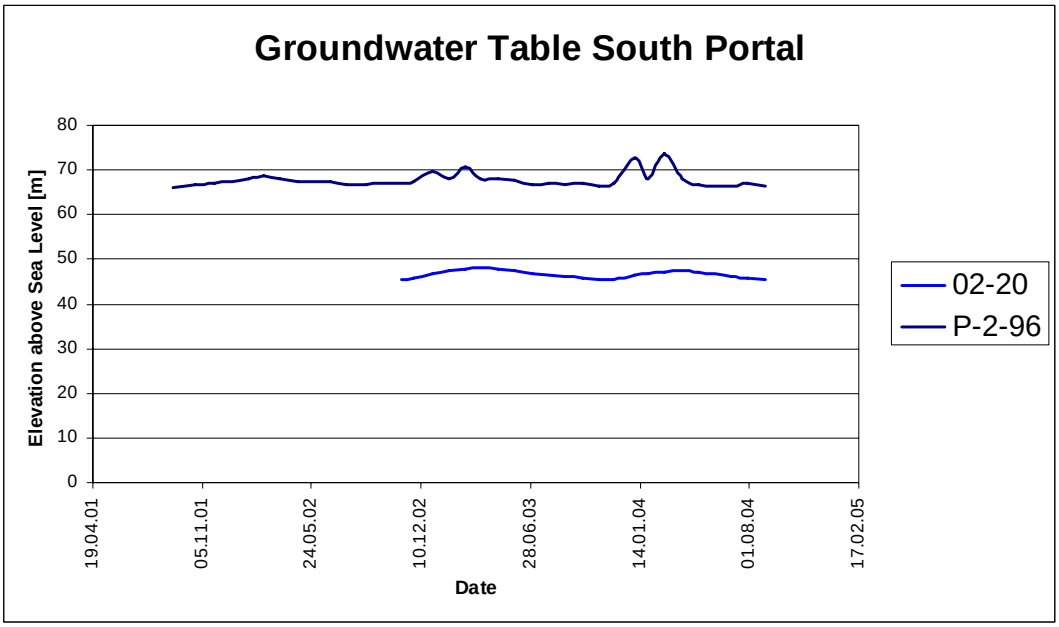
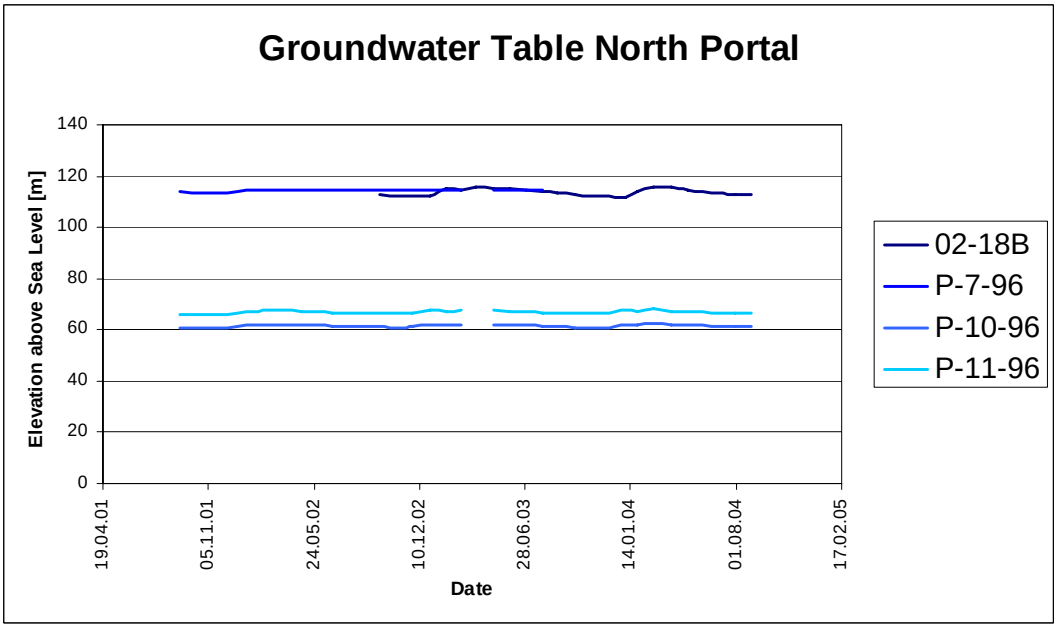
Hydrographs of Piezometers at the South Portal Area



Hydrographs of Piezometers at the North Portal Area



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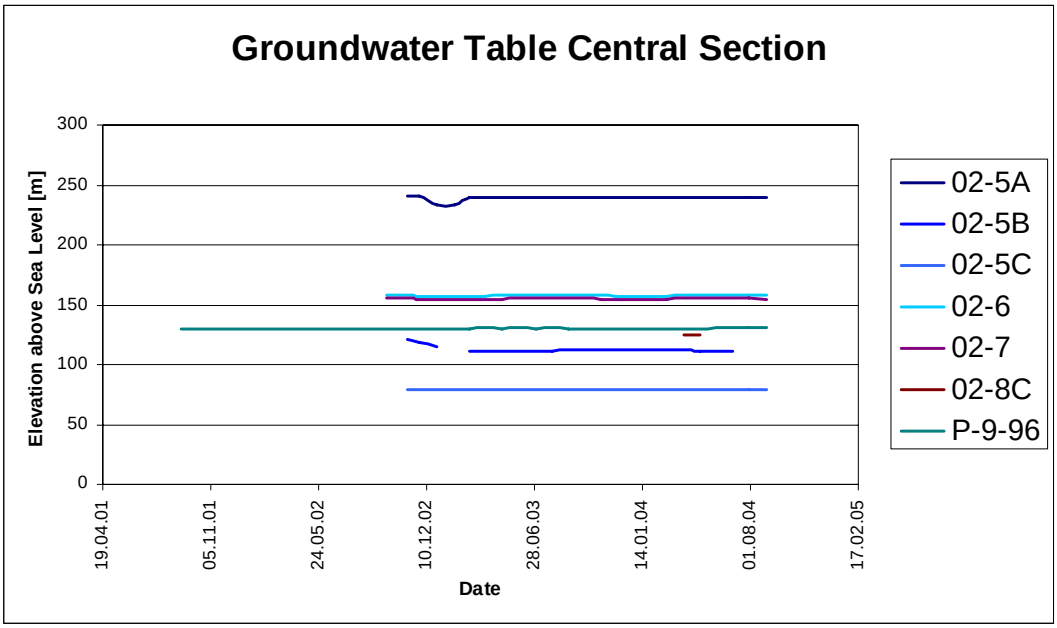
Hydrographs of Piezometers along the Central Tunnel Section



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Appendix 2: Results of the Analytical Tunnel Inflow Calculations

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Flush Tunnel Inflow Calculations after Goodman 1965

Tectonic Unit	Intervall		Q for 1m [l/s] (after Goodman 1965)			Q for 100m tunnel progress [l/s]			Q for 100m tunnel progress [l/s]		
	No.	Length	Favorable	Average	Unfavorable	Favorable	Average	Unfavorable	Favorable	Average	Unfavorable
South Block - Footwall Block	1	171		3.03E-03	9.38E-03		0.30	0.94	0.94	4.66	10.68
South Block - Hanging Wall Block	2	340	9.41E-03	6.85E-02	1.56E-01	0.94	6.85	15.59			
Central Block: Fault B	3	20	5.90E-04	3.17E-02	8.30E-02	0.06	3.17	8.30	1.38	24.67	0.01
Central Block	4	55	4.79E-03	1.10E-01	2.20E-01	0.48	11.01	22.02			
	5	50	6.97E-02	2.71E-01	6.83E-01	6.97	27.13	68.25			
	6	120	1.31E-03	7.22E-02	1.92E-01	0.13	7.22	19.22			
	7	100	3.25E-02	1.24E-01	3.72E-01	3.25	12.35	37.19			
	8	50	8.81E-03	1.11E-01	1.82E-01	0.88	11.15	18.22			
	9	120	1.42E-03	4.31E-02	1.12E-01	0.14	4.31	11.22			
	10	10	2.72E-02	1.12E-01	3.53E-01	2.72	11.22	35.30			
	11	50	8.38E-04	5.89E-02	1.80E-01	0.08	5.89	18.02			
	12	20	7.09E-03	1.01E-01	1.75E-01	0.71	10.11	17.51			
North Block: Fault C	13	8	2.13E+00	2.73E+00	6.60E+00	212.73	272.81	660.00	14.52	22.14	56.40
	14	12	5.77E-03	5.92E-03	1.21E-02	0.58	0.59	1.21			
North Block	15	125	3.17E-02	8.17E-02	2.31E-01	3.17	8.17	23.07			

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Flush Tunnel Inflow Calculations after Jacob & Lohmann 1952

Tectonic Unit	Intervall		Storativity	Q for 1m [l/s] (after Jacob & Lohmann 1952)			Q for 100m tunnel progress [l/s]			Q for 100m tunnel progress [l/s]		
	No.	Length		Favorable	Average	Unfavorable	Favorable	Average	Unfavorable	Favorable	Average	Unfavorable
South Block - Footwall Block	1	171	0.01			3.61E-03			0.36	0.44	4.85	13.81
South Block - Hanging Wall Block	2	340	0.01	6.57E-03	7.28E-02	2.06E-01	0.66	7.28	20.57			
Central Block: Fault B	3	25	0.05	2.20E-04	1.98E-02	6.29E-02	0.02	1.98	6.29	1.06	6.92	27.49
Central Block	4	50	0.05	2.06E-03	9.09E-02	2.64E-01	0.21	9.09	26.35			
	5	50	0.1	6.13E-02	8.81E-02	8.51E-01	6.13	8.81	85.13			
	6	120	0.05	7.84E-04	6.35E-02	2.18E-01	0.08	6.35	21.84			
	7	100	0.1	2.37E-02	1.14E-01	4.19E-01	2.37	11.40	41.92			
	8	50	0.05	5.77E-03	9.25E-02	2.04E-01	0.58	9.25	20.39			
	9	120	0.1	7.05E-04	3.39E-02	1.06E-01	0.07	3.39	10.59			
	10	10	0.1	1.82E-02	5.34E-02	3.90E-01	1.82	5.34	39.03			
	11	50	0.05	4.01E-04	5.16E-02	1.98E-01	0.04	5.16	19.80			
	12	20	0.05	4.12E-03	4.38E-02	7.22E-02	0.41	4.38	7.22			
North Block: Fault C	13	8	0.2	1.80E+00	1.80E+00	4.27E+00	180.37	180.37	427.00	10.24	12.93	33.66
	14	12	0.05	2.27E-03	2.27E-03	4.91E-03	0.23	0.23	0.49			
North Block	15	125	0.05	3.08E-03	3.44E-02	1.17E-01	0.31	3.44	11.67			

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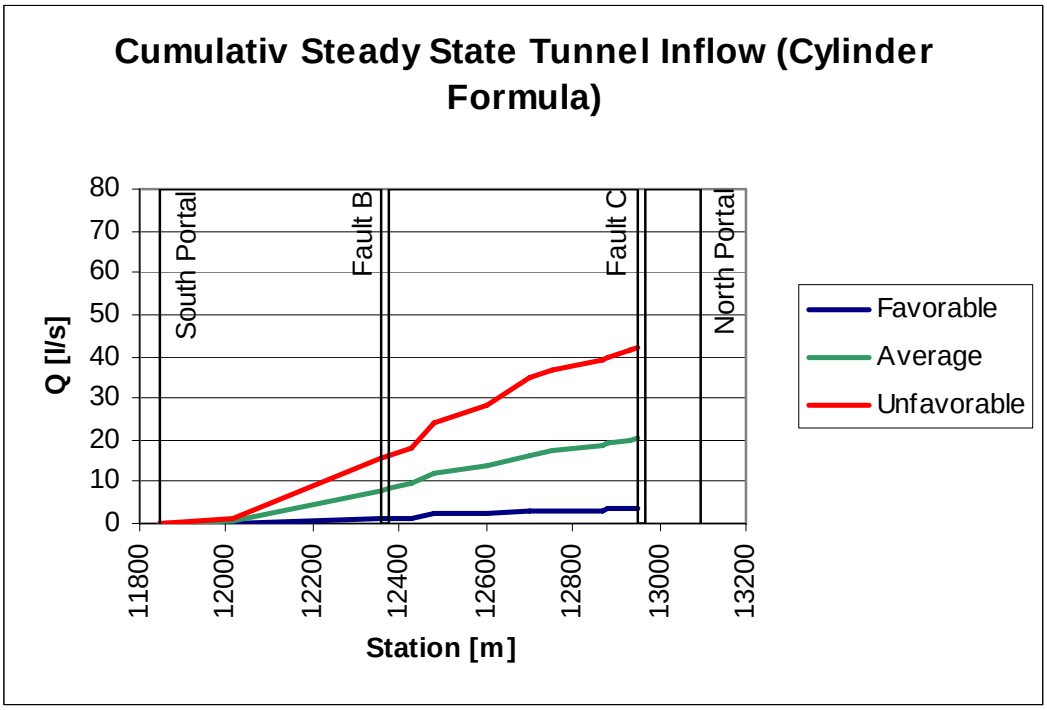
Steady State Tunnel Inflow Calculations using Cylinder Formula

Tectonic Unit	Intervall		Q for 1m [l/s] (Cylindric tunnel)			Q for tunnel section [l/s]		
	No.	Length	Favorable	Average	Unfavorable	Favorable	Average	Unfavorable
South Block - Footwall Block	1	171	8.09E-05	1.97E-03	6.69E-03	0.01	0.34	1.14
South Block - Hanging Wall Block	2	340	3.87E-03	2.23E-02	4.22E-02	1.32	7.57	14.36
Central Block: Fault B	3	25	3.03E-04	1.44E-02	3.41E-02	0.01	0.36	0.85
Central Block	4	50	2.64E-03	2.52E-02	3.34E-02	0.13	1.26	1.67
	5	50	1.38E-02	5.04E-02	1.20E-01	0.69	2.52	5.98
	6	120	3.03E-04	1.44E-02	3.41E-02	0.04	1.73	4.10
	7	100	8.80E-03	2.66E-02	6.69E-02	0.88	2.66	6.69
	8	50	2.64E-03	2.52E-02	3.34E-02	0.13	1.26	1.67
	9	120	4.57E-04	1.02E-02	2.11E-02	0.05	1.22	2.53
	10	10	8.80E-03	2.66E-02	6.69E-02	0.09	0.27	0.67
	11	50	3.03E-04	1.44E-02	3.41E-02	0.02	0.72	1.71
	12	20	2.64E-03	2.52E-02	3.34E-02	0.05	0.50	0.67
North Block: Fault C	13	8	NOT APPLICABLE			NOT APPLICABLE		
	14	12						
North Block	15	125						
Sum								

Cumulative Steady State Tunnel Inflow using Cylinder Formula



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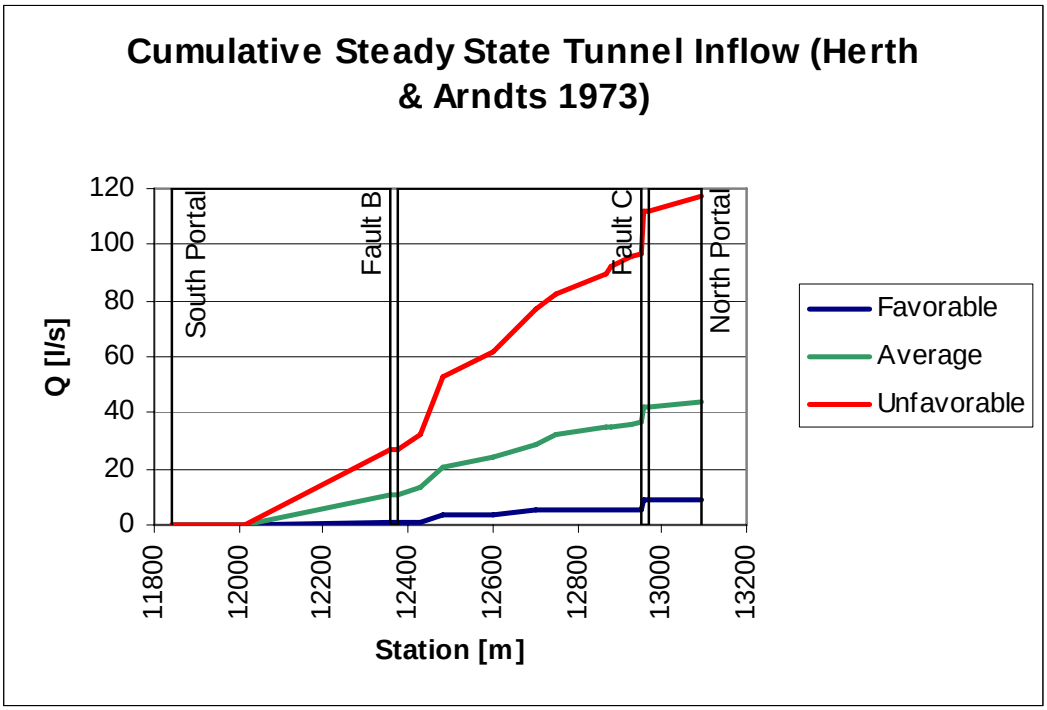
Steady State Tunnel Inflow Calculations after Herth & Arndts 1973

Tectonic Unit	Intervall		Q for 1m [l/s] (after Herth & Arndts 1973)			Q for tunnel section [l/s]		
	No.	Length	Min	Mean	Max	Min	Mean	Max
South Block - Footwall Block	1	171		2.80E-04	1.90E-03		0.05	0.32
South Block - Hanging Wall Block	2	340	3.74E-03	3.13E-02	7.80E-02	1.27	10.65	26.53
Central Block: Fault B	3	20	1.32E-04	7.80E-03	2.20E-02	0.00	0.16	0.44
Central Block	4	55	1.00E-03	3.96E-02	9.24E-02	0.06	2.18	5.08
	5	50	3.99E-02	1.59E-01	4.08E-01	2.00	7.94	20.41
	6	120	4.70E-04	2.74E-02	7.63E-02	0.06	3.29	9.16
	7	100	1.63E-02	4.56E-02	1.47E-01	1.63	4.56	14.69
	8	50	2.80E-03	6.04E-02	1.07E-01	0.14	3.02	5.36
	9	120	6.50E-04	2.29E-02	6.54E-02	0.08	2.75	7.85
	10	10	1.25E-02	5.96E-02	2.05E-01	0.13	0.60	2.05
	11	50	2.41E-04	2.05E-02	6.98E-02	0.01	1.03	3.49
	12	20	2.00E-03	3.51E-02	6.77E-02	0.04	0.70	1.35
North Block: Fault C	13	8	4.20E-01	6.77E-01	1.86E+00	3.36	5.41	14.90
	14	12	1.07E-03	1.13E-03	2.39E-03	0.01	0.01	0.03
North Block	15	125	1.20E-03	1.10E-02	4.27E-02	0.15	1.38	5.33
Sum						8.92	43.72	117.00

Cumulative Steady State Tunnel Inflow using Cylinder Formula



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Appendix 3: Results of the Numerical Tunnel Inflow Calculations

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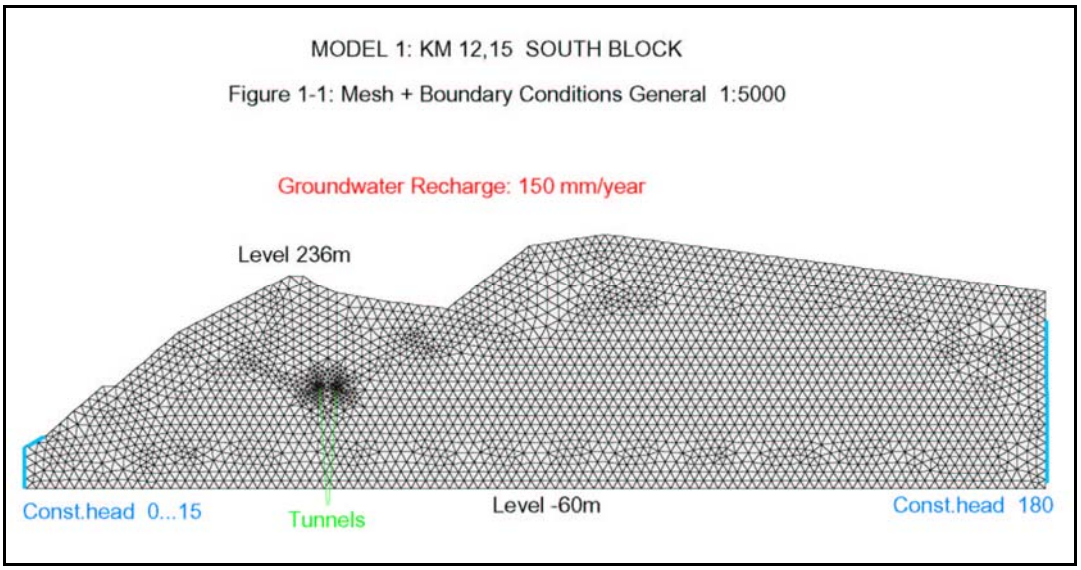
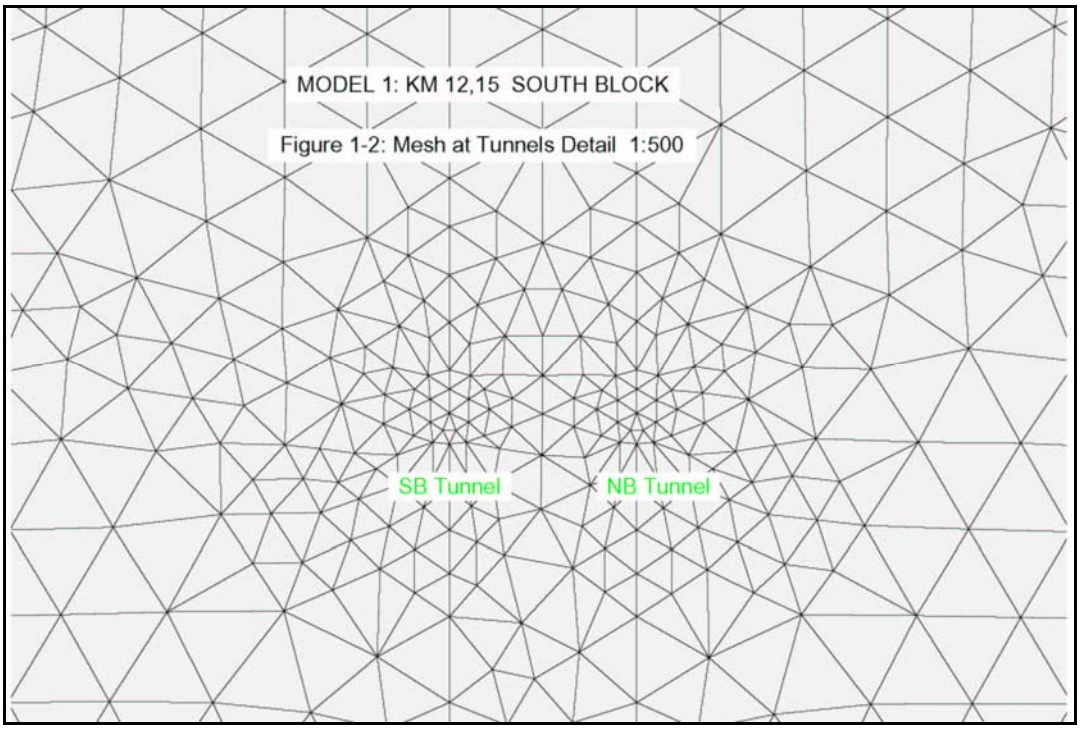

MODEL 1: KM 12,15 SOUTH BLOCK
Figure 1-1: Mesh + Boundary Conditions General 1:5000
Groundwater Recharge: 150 mm/year
Level 236m
Const.head 0...15
Tunnels
Level -60m
Const.head 180


MODEL 1: KM 12,15 SOUTH BLOCK
Figure 1-2: Mesh at Tunnels Detail 1:500
SB Tunnel
NB Tunnel

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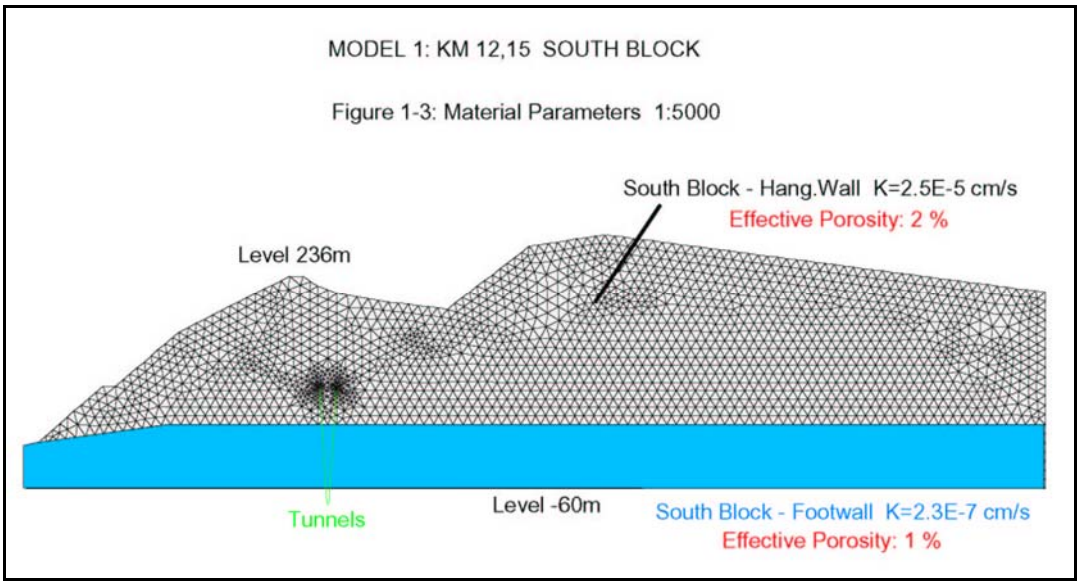
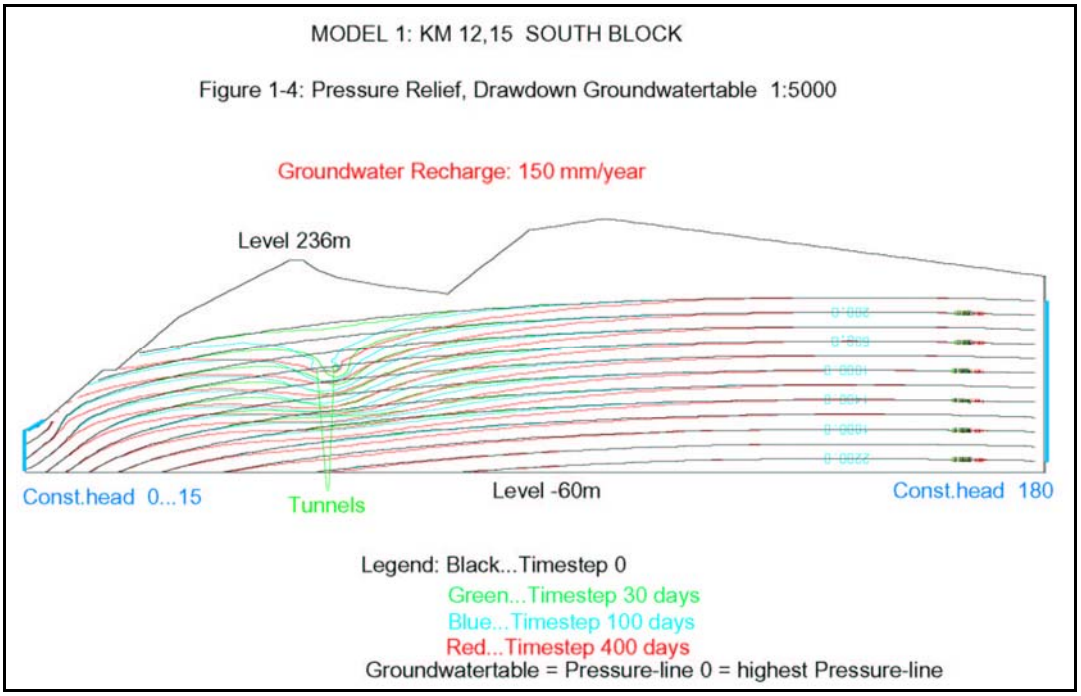

MODEL 1: KM 12,15 SOUTH BLOCK
Figure 1-3: Material Parameters 1:5000
South Block - Hang.Wall K=2.5E-5 cm/s
Effective Porosity: 2 %
Level 236m
Tunnels
Level -60m
South Block - Footwall K=2.3E-7 cm/s
Effective Porosity: 1 %


MODEL 1: KM 12,15 SOUTH BLOCK
Figure 1-4: Pressure Relief, Drawdown Groundwatertable 1:5000
Groundwater Recharge: 150 mm/year
Level 236m
Const.head 0...15
Tunnels
Level -60m
Const.head 180
Legend: Black...Timestep 0
Green...Timestep 30 days
Blue...Timestep 100 days
Red...Timestep 400 days
Groundwatertable = Pressure-line 0 = highest Pressure-line

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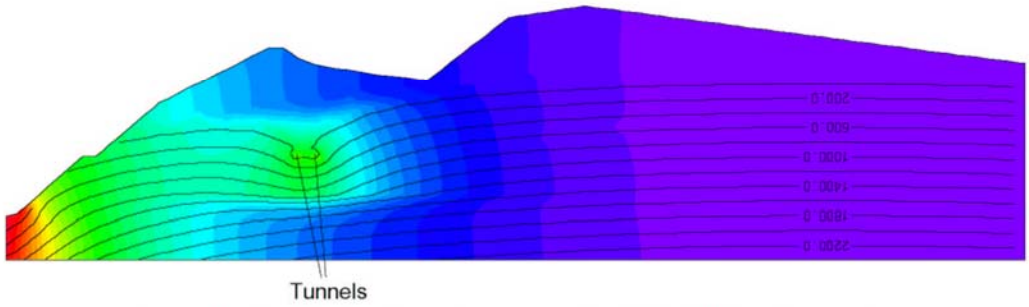
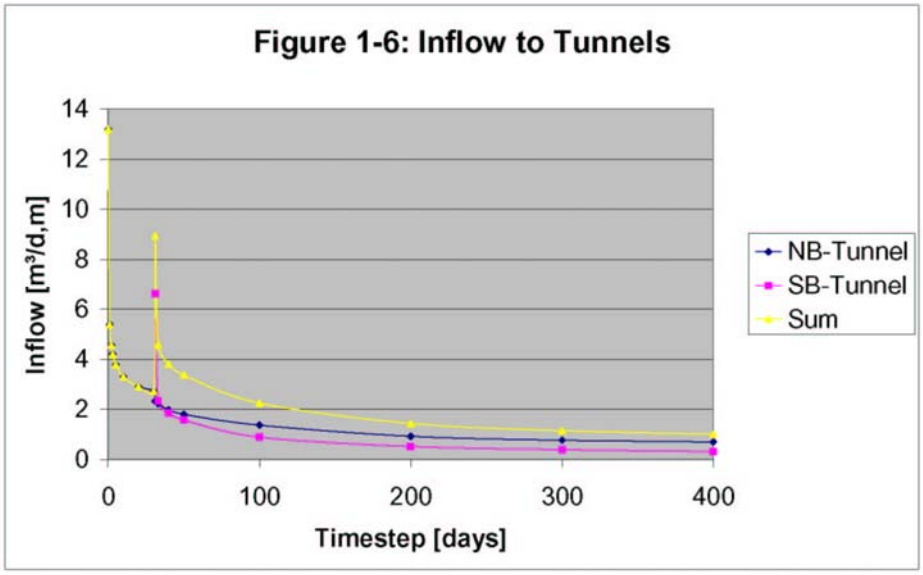
MODEL 1: KM 12,15 SOUTH BLOCK

Figure 1-5: Hydraulic Head Relief, Pressure Contour Lines 1:5000

Timestep: 100 days



Hydralic Head = 0 (red, sea)....79 (green, tunnel)....180 (darkblue, right boundary)



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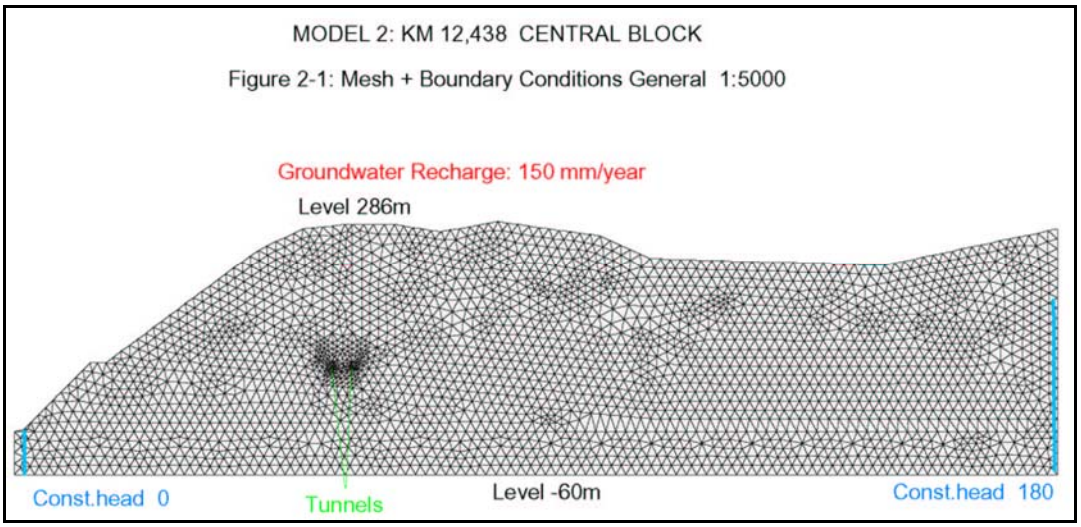
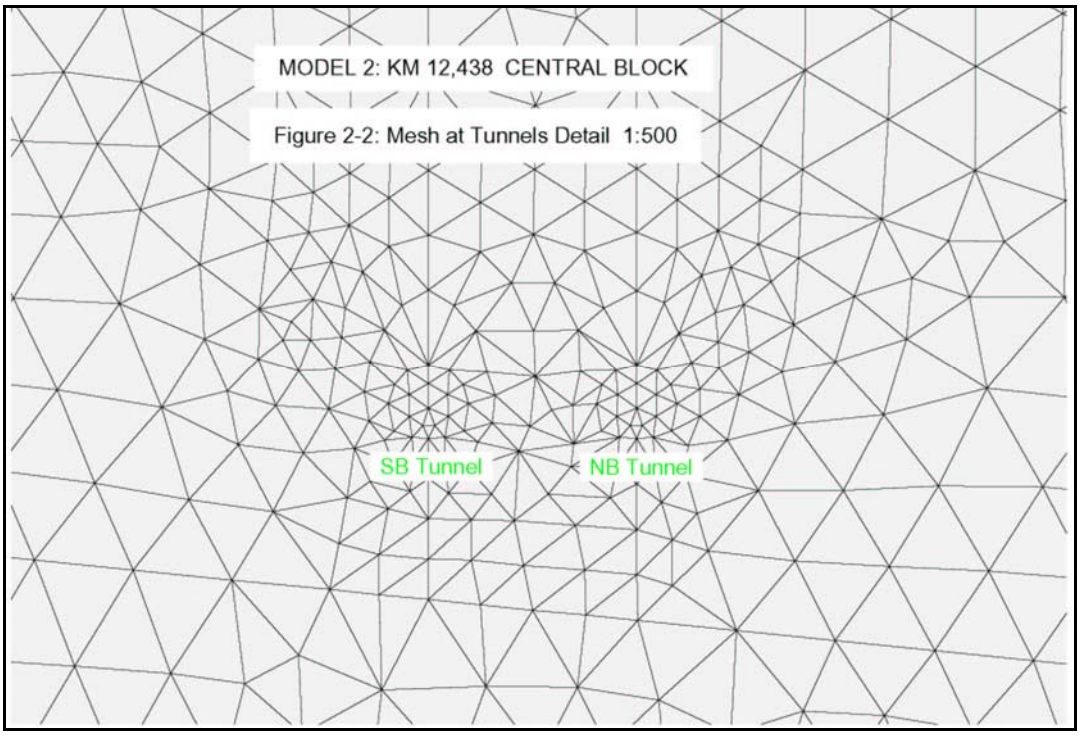

MODEL 2: KM 12,438 CENTRAL BLOCK
Figure 2-1: Mesh + Boundary Conditions General 1:5000
Groundwater Recharge: 150 mm/year
Level 286m
Const.head 0
Tunnels
Level -60m
Const.head 180


MODEL 2: KM 12,438 CENTRAL BLOCK
Figure 2-2: Mesh at Tunnels Detail 1:500
SB Tunnel
NB Tunnel

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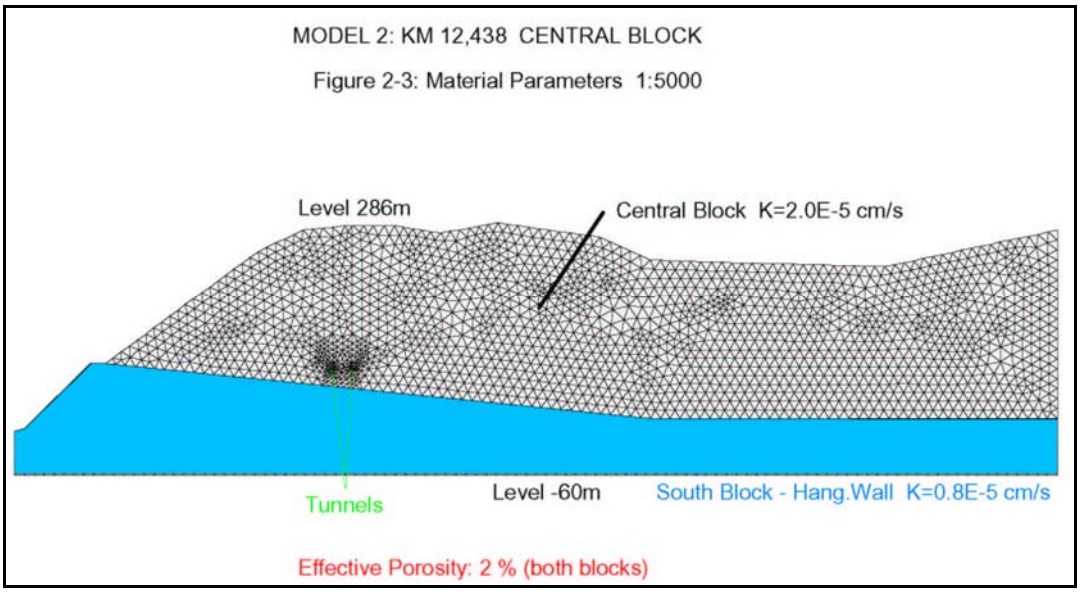
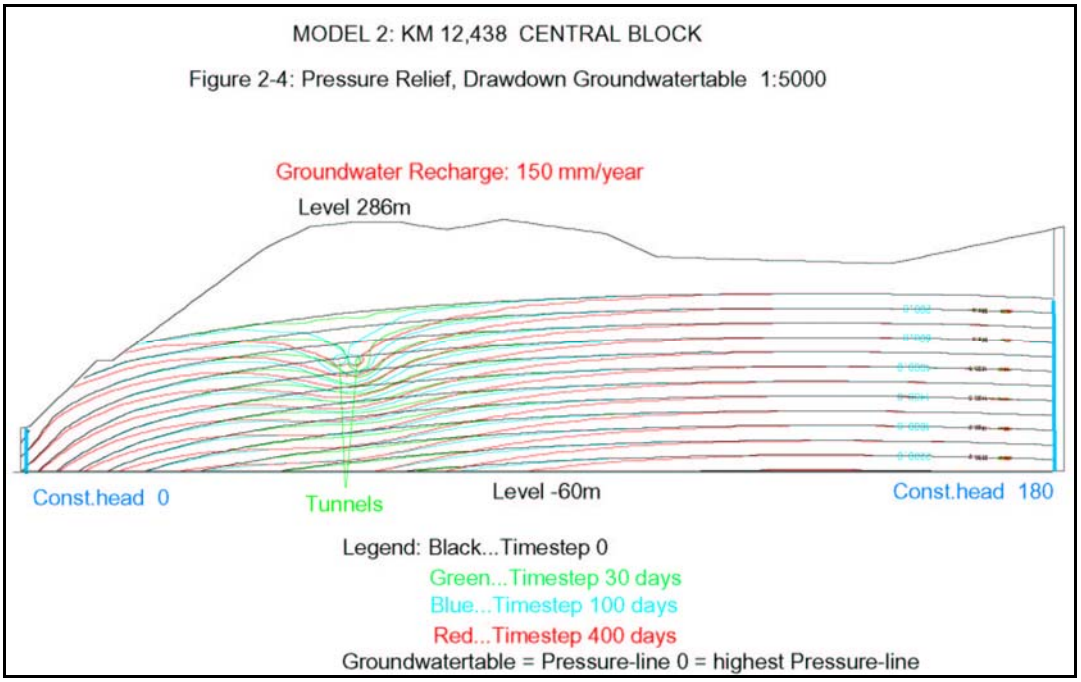

MODEL 2: KM 12,438 CENTRAL BLOCK
Figure 2-3: Material Parameters 1:5000
Level 286m
Central Block K=2.0E-5 cm/s
Tunnels
Level -60m
South Block - Hang.Wall K=0.8E-5 cm/s
Effective Porosity: 2 % (both blocks)


MODEL 2: KM 12,438 CENTRAL BLOCK
Figure 2-4: Pressure Relief, Drawdown Groundwatertable 1:5000
Groundwater Recharge: 150 mm/year
Level 286m
Const.head 0
Tunnels
Level -60m
Const.head 180
Legend: Black...Timestep 0
Green...Timestep 30 days
Blue...Timestep 100 days
Red...Timestep 400 days
Groundwatertable = Pressure-line 0 = highest Pressure-line

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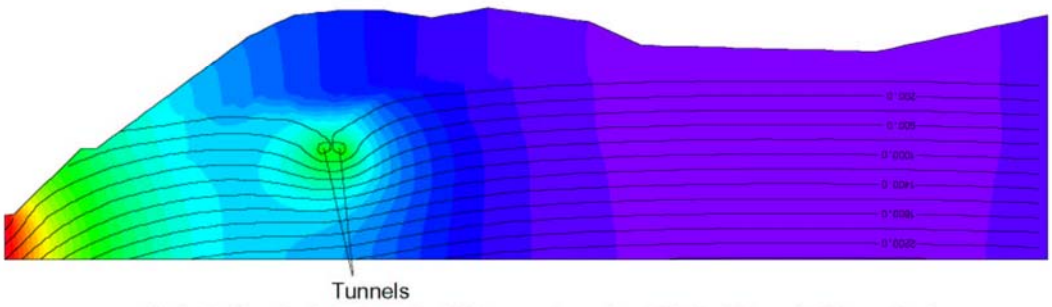
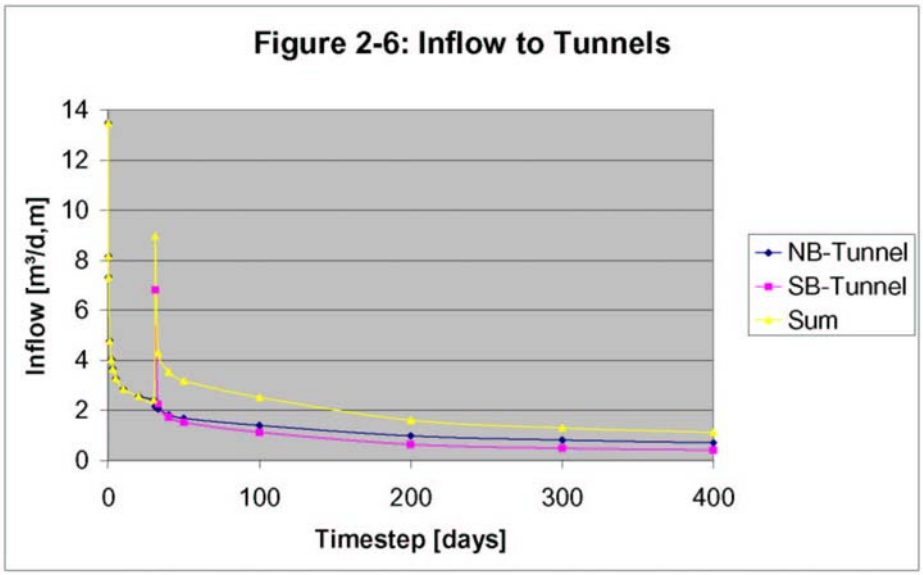
MODEL 2: KM 12,438 CENTRAL BLOCK

Figure 2-5: Hydraulic Head Relief, Pressure Contour Lines 1:5000

Timestep: 100 days



Hydralic Head = 0 (red, sea)....86 (green, tunnel)....180 (darkblue, right boundary)



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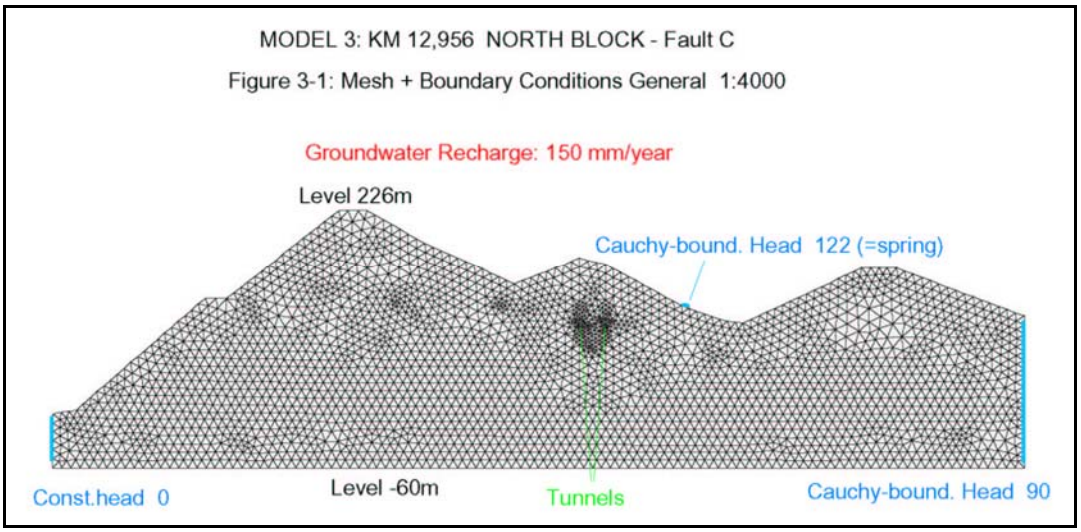
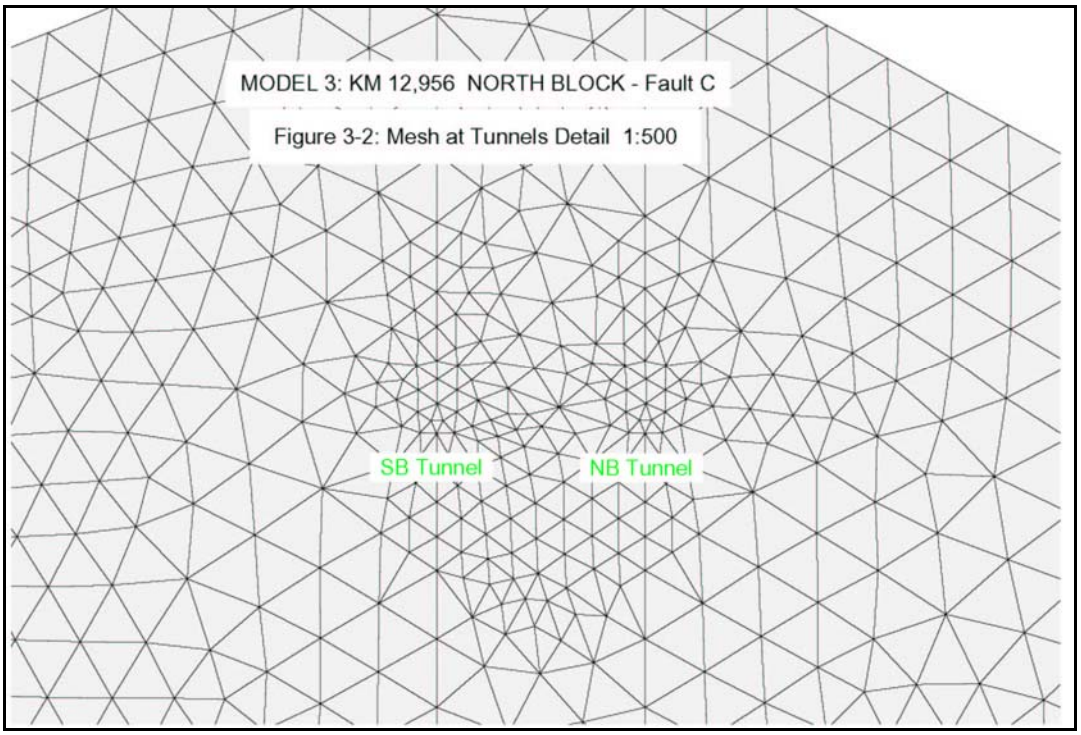

MODEL 3: KM 12,956 NORTH BLOCK - Fault C
Figure 3-1: Mesh + Boundary Conditions General 1:4000
Groundwater Recharge: 150 mm/year
Level 226m
Cauchy-bound. Head 122 (=spring)
Const.head 0
Level -60m
Tunnels
Cauchy-bound. Head 90


MODEL 3: KM 12,956 NORTH BLOCK - Fault C
Figure 3-2: Mesh at Tunnels Detail 1:500
SB Tunnel
NB Tunnel

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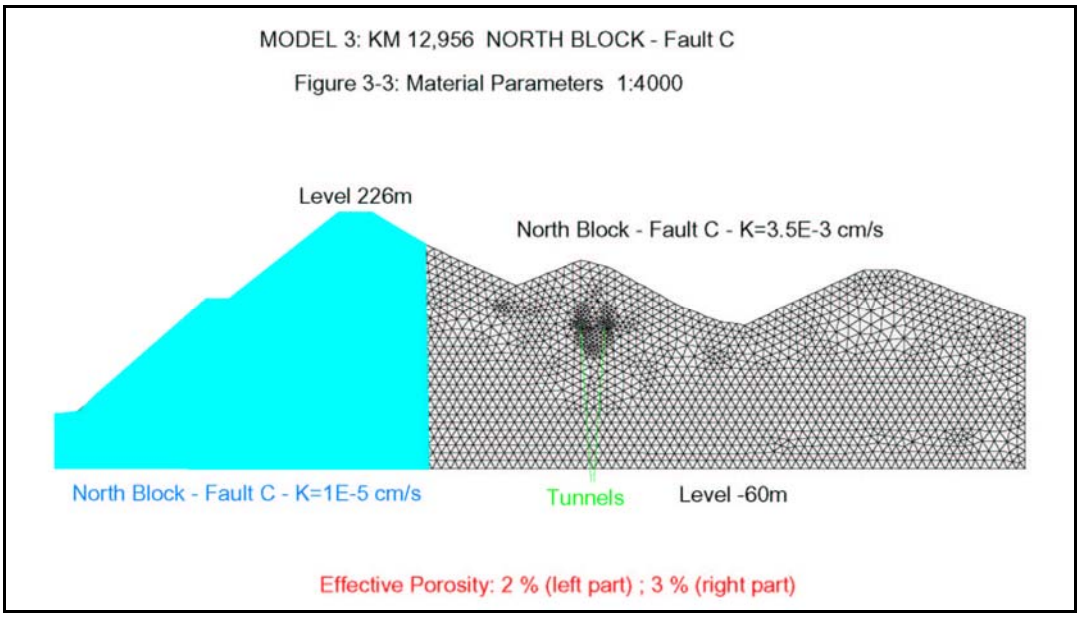
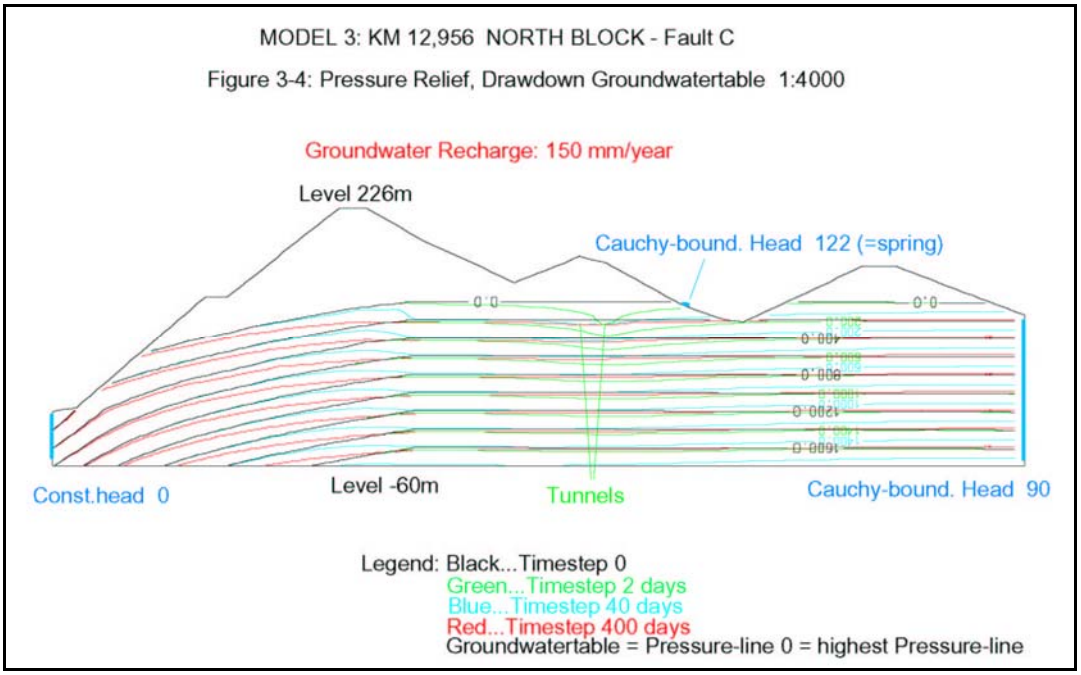

MODEL 3: KM 12,956 NORTH BLOCK - Fault C
Figure 3-3: Material Parameters 1:4000
Level 226m
North Block - Fault C - K=3.5E-3 cm/s
North Block - Fault C - K=1E-5 cm/s
Tunnels
Level -60m
Effective Porosity: 2 % (left part) ; 3 % (right part)


MODEL 3: KM 12,956 NORTH BLOCK - Fault C
Figure 3-4: Pressure Relief, Drawdown Groundwatertable 1:4000
Groundwater Recharge: 150 mm/year
Level 226m
Cauchy-bound. Head 122 (=spring)
Const.head 0
Level -60m
Tunnels
Cauchy-bound. Head 90
Legend: Black...Timestep 0
Green...Timestep 2 days
Blue...Timestep 40 days
Red...Timestep 400 days
Groundwatertable = Pressure-line 0 = highest Pressure-line

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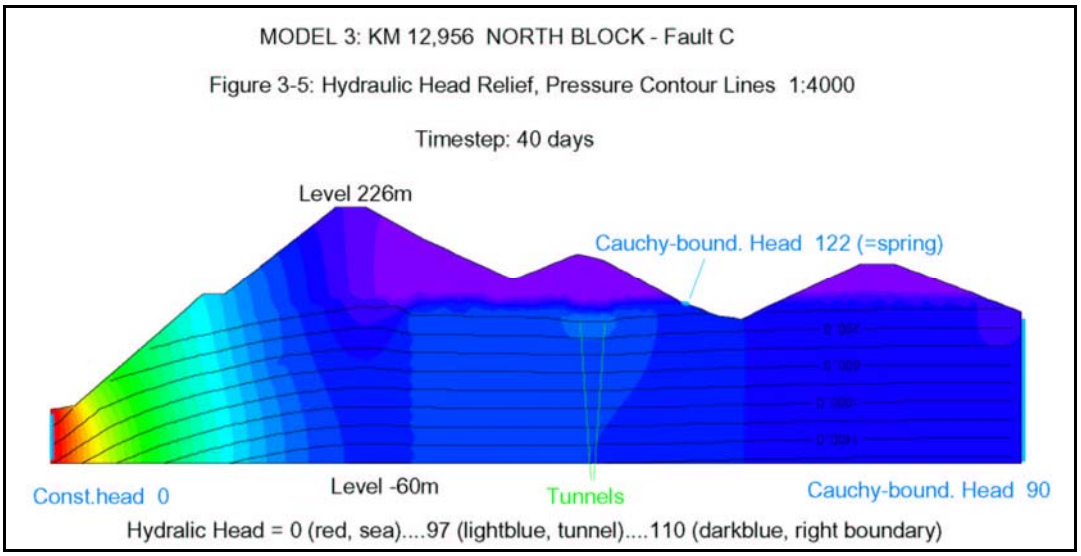
MODEL 3: KM 12,956 NORTH BLOCK - Fault C
Figure 3-5: Hydraulic Head Relief, Pressure Contour Lines 1:4000
Timestep: 40 days
Level 226m
Cauchy-bound. Head 122 (=spring)
Const.head 0
Level -60m
Tunnels
Cauchy-bound. Head 90
Hydralic Head = 0 (red, sea)....97 (lightblue, tunnel)....110 (darkblue, right boundary)

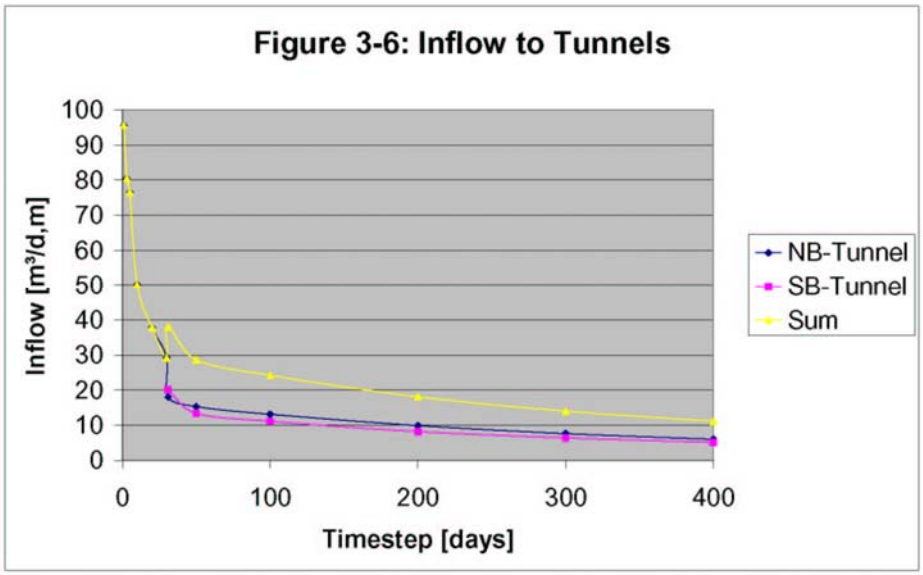
Figure 3-6: Inflow to Tunnels
Inflow [m³/d,m]
100
90
80
70
60
50
40
30
20
10
0
0
100
200
300
400
Timestep [days]
NB-Tunnel
SB-Tunnel
Sum

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Appendix 4: Tunnel Inflow Calculations by Greg Korbin using Heuer's Method

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