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Rock Mechanics Screening of Surface Redevelopment Impacts on Hagalundstunneln

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Summary

The simplified elastic analytical model provides a upper-bound estimate of the stress evolution under demolition and new-build conditions at Hagalundstunneln.

It confirms that, while the post-construction load case approaches the rock-mass capacity, the system remains globally stable under the assumed parameters.

Nevertheless, because the approach is assumption-driven, a numerical simulation calibrated to site-specific data is recommended for final verification of tunnel stability under the planned redevelopment.

1. Scope and Assumptions

The objective of this analytical study is to evaluate the potential effect of demolition and reconstruction of the surface building located above Hagalundstunneln, on the underlying twin tunnel system. The site currently includes five tunnel openings; however, for simplification and clarity, the system was focused on twin-tunnel. This approach captures the primary interaction between adjacent openings while maintaining analytical simplification. This is an analytical, elastic-based method, assuming isotropic, continuous rock mass behavior and neglecting time-dependent, plastic, or discontinuum effects. Because of the simplifications (elastic assumption, two-tunnel equivalence, infinite-area loading), the results are likely conservative in elastic state as they represent upper bound stress conditions on the tunnel lining.

1.1 Analytical Methodology

The assessment was carried out using closed-form solutions for circular tunnels, complemented by Hoek–Brown rock mass strength and a Convergence–Confinement (CCM) interpretation for the lining–ground interaction. The analytical solution consisted of:

1. Baseline in-situ stress estimation based on overburden weight and hydrostatic assumption ($k_0 = 1$).
2. Surface load transfer to tunnel depth using Boussinesq formulation. Because the building footprint was not defined, the load effect was approximated using an infinite-area upper-bound assumption.
3. Determination of the far-field stress per construction phase (pre-demolition, demolition, and new build).
4. Application of Kirsch's elastic solution for a circular opening to calculate wall and crown stresses.
5. Evaluation of rock-mass capacity using the Hoek–Brown criterion.
6. Inclusion of a twin-tunnel interaction factor to account for spacing.
7. Comparison of the resulting maximum hoop stress with the Hoek–Brown capacity to obtain a factor of safety (FoS).
8. Construction of the ground reaction curve (GRC) and support characteristic curve (SCC) to identify the operating point.

2 Input Parameters

Table 1-Input Data

Category	Parameter	Value	Symbol	Source / Note
Geometry & Loads	Tunnel diameter (m)	8	D	Extracted from design drawings
	Cover to crown (m)	9	z	
	Center spacing (m)	12	S	
	Old surface stress (kPa)	3000	q_old	Provided by the client
	New surface stress (kPa)	4500	q_new	Assumed and confirmed by the client
Rock & In-situ Conditions	Unit weight (kN/m³)	27	γ	Typical value for crystalline rock
	Stress ratio $k_o = \sigma h_o/\sigma v_o$ (–)	1	k_o	Hydrostatic assumption
Hoek–Brown Parameters	Intact UCS (MPa)	100	σci	Derived in RSData using SGU data
	Material constant (–)	18	m_i	
	Geological Strength Index (–)	60	GSI	
	Disturbance factor (–)	0.2	D	
Support / Twin Interaction	Support pressure (kPa)	0	p_i	Typical for reinforced rock mass
	Twin-tunnel interaction α (–)	0.1	α	Screening parameter
Lining (CCM)	Elastic modulus (MPa)	25 000	E_l	Typical shotcrete stiffness
	Thickness (m)	0.1	t	Assumed lining thickness
	Poisson's ratio (–)	0.08	ν_l	Typical value for concrete lining
	Unsupported convergence (mm)	10	u_o	Estimated GRC input
	GRC exponent (–)	1	n	Linear assumption

3 Analytical Calculations and Results

3.1 Baseline InSitu Stress

The baseline stress field represents the initial state prior to any surface loading. Vertical stress (σ_{v0}) is derived from the overburden weight, and horizontal stress is estimated using the at rest coefficient k0. A hydrostatic stress condition ($k_0 = 1.0$) was assumed, producing an isotropic field at tunnel depth.

Compute vertical and horizontal far field stresses at tunnel depth gives:
$\sigma v0 = \gamma \cdot z, \quad \sigma h0 = k0 \cdot \sigma v0.$
σv0 = 243.0 kPa, σh0 = 243.0 kPa.

3.2 Surface Load Effect at Tunnel Depth

The stress influence of surface structures was estimated using elasticity theory for semi-infinite continua (Boussinesq/Newmark).

$$I(a,z) = 1 - 1/(1 + (a/z)2)3/2, \Delta\sigma v = q \cdot I(a,z).$$

As the actual building footprint is unknown, an infinite area approximation was adopted ($\Delta\sigma_v \approx q$), providing an upper-bound estimate of additional vertical stress at tunnel level; conservative for design screening.

$$\Delta\sigma v(demolition) \approx -qold, \qquad \Delta\sigma v(new) \approx -qold + qnew$$
$$\Delta\sigma v(demolition) = -3000kPa; \ \Delta\sigma v(newbuild) = 1500kPa$$

Far Field Stresses per stage thus updated as:

$$\sigma v \ = \ \sigma v0 \ + \ \Delta\sigma v, \ \ \sigma h \ \approx \ \sigma h0 \ (centred \ load)$$

3.3 Circular Opening — Kirsch Solution

Stress concentration around the tunnel boundary was evaluated using the Kirsch equations for a circular cavity in an elastic, isotropic medium under biaxial far-field loading.

These equations provide the circumferential (hoop) stress distribution at the spring line and crown.

For the demolition case, where unloading produced $\sigma_v < 0$, vertical stress was capped at zero to avoid non-physical tensile confinement in the rock mass.

$$\sigma_\theta^{(spring)} \ = \ -\sigma h \ + \ 3\,\sigma v \ - \ pi$$
$$\sigma_\theta^{(crown)} \ = \ 3\,\sigma h \ - \ \sigma v \ - \ pi$$
$$\sigma_{\theta,max} \ = \ max(\,\sigma_\theta^{(crown)}\,, \sigma_\theta^{(spring)}\,)$$

3.4 Rock Mass Strength — Hoek–Brown (2002)

Rock-mass strength parameters (m_b, s, a) were derived from the empirical Hoek–Brown failure criterion using the assumed Geological Strength Index (GSI = 60) and disturbance factor (D = 0.2).

$$mb \ = \ mi \ \cdot \ exp(\,(GSI \ - \ 100)\,/\,(28 \ - \ 14\,D)\,)$$
$$s \ = \ exp(\,(GSI \ - \ 100)\,/\,(9 \ - \ 3\,D)\,)$$
$$a \ = \ 0.5 \ + \ (1/6) \cdot (\,e^{\,-GSI/15} - \ e^{-20/3}\,)$$

Rock mass strength envelope:

$$\sigma 1 \ = \ \sigma 3 \ + \ \sigma ci \ \cdot \ (\,mb \ \cdot \ \sigma 3\,/\,\sigma ci \ + \ s\,)^a$$ (Use σ3 = pi, σ1 = σθ,max)

gives: mb = 3.681, s = 0.00855, a = 0.503,

σci = 100000 kPa, capacity at wall σ1_cap = 9122.0 kPa.

3.5 Twin Tunnel Interaction

The influence between adjacent tunnels was represented through a dimensionless interaction factor (Twin), which amplifies wall stresses when the spacing to diameter ratio (S/D) is small; Following Einstein & Schwartz (1979),

$$Twin \ = \ 1 \ if \ S/D \ \geq \ 3; \ else \ Twin \ = \ 1 \ + \ \alpha \ \cdot \ (3 \ - \ S/D).$$

gives: S/D = 1.50 → Twin = 1.15.

3.6 Factor of Safety

The Factor of Safety (FoS) quantifies the ratio of rock-mass capacity to acting boundary stress:

$$FoS_{\,rock} = \ \sigma_{1,cap}\,/\,(\,\sigma_{\theta,max} \ \cdot \ Twin\,).$$

where $\sigma_{\theta,max}$ is the maximum circumferential stress from the Kirsch solution. FoS > 1 indicates stable equilibrium.

3.7 Convergence–Confinement Method (Linear GRC)

CCM evaluates the equilibrium between ground deformation and support stiffness by intersecting the Ground Reaction Curve (GRC) and Support Characteristic Curve (SCC). This defines the operating point (p*, u*) — the combination of wall pressure and deformation representing stable tunnel behaviour.

$$u = u0 \cdot (1 - p / p0), \quad p_{GRC}(u) = p0 \cdot (1 - u / u0)$$
$$Ks = (El \cdot t) / [a \cdot (1 - \nu_2^1)], \quad pSCC(u) = Ks \cdot u$$
$$p^* = (K_s\, u_0\, p_0) / (K_s\, u_0 + p_0), \quad u^* = p^* / K_s, \quad p_0 = (\sigma_{v0} + \sigma_{h0}) / 2$$

gives: a = 4.000 m, p0 = 243.0 kPa, Ks = 629.026 kPa/mm, (Panet; 1995)

p* = 234.0 kPa, u* = 0.372mm

3.8 Results

All analytical calculations and stage-based stress evaluations have been compiled in a complimentary Excel file submitted together with this report. The spreadsheet includes input parameters, intermediate derivations, Hoek–Brown strength evaluation, Kirsch stress calculations with capped vertical stress, and the GRC-SCC analysis. It is intended to provide full transparency of the analytical workflow and allow independent verification or sensitivity testing of the assumed parameters.

Table 2-FoS Calculation

State	σv (kPa)	σv_eff (kPa)	σh (kPa)	Hoop_wall (kPa)	Hoop_crown (kPa)	Hoop_max (kPa)	Twin	σ1 (kPa)	FoS_rock
Baseline	243	3243	243	9486	-2514	9486	1.15	10000	9.16
Post-Demolition	-2757	0	243	-243	729	729	1.15	9122.02	10.88
Post-New-Build	1743	1743	243	4986	-1014	4986	1.15	9122.02	1.59

Table 3-GRC-SCC analysis

u (mm)	0	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7	7.5	8	8.5	9	9.5	10
p_GRC(u) (kPa)	243	230.85	218.7	206.55	194.4	182.25	170.1	157.95	145.8	133.65	121.5	109.35	97.2	85.05	72.9	60.75	48.6	36.45	24.3	12.15	0
p_SCC(u) (kPa)	0	314.5129	629.0258	943.5386	1258.052	1572.564	1887.077	2201.59	2516.103	2830.616	3145.129	3459.642	3774.155	4088.667	4403.18	4717.693	5032.206	5346.719	5661.232	5975.745	6290.258

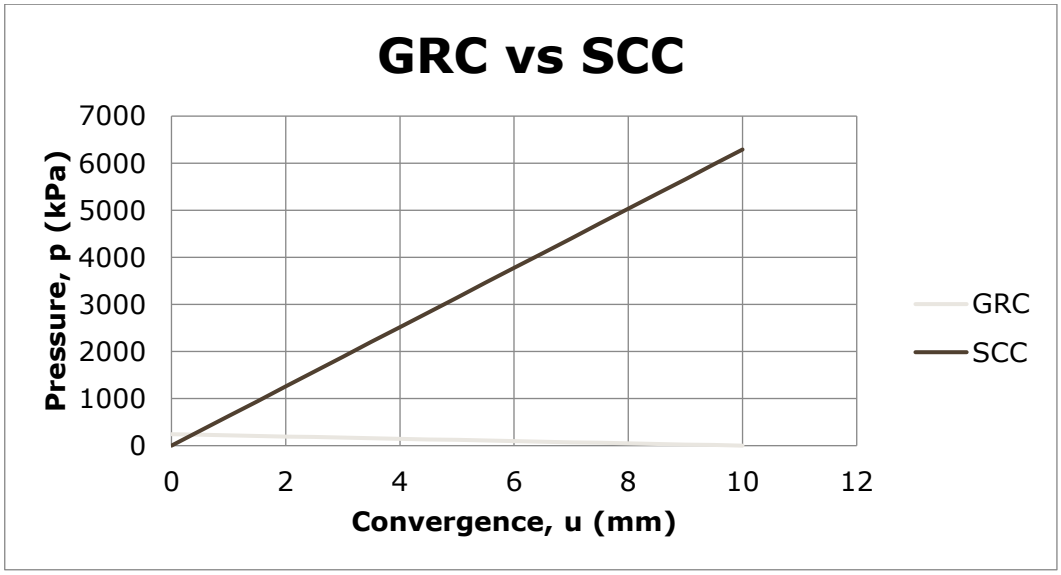

Figure 1-GRC-SCC plot

4 Interpretation of Analytical Results

The analytical assessment of the Hagalundstunneln twin-tunnel model indicates that the overall system remains stable, but the magnitude of the calculated safety margins changes substantially depending on loading conditions.

1. **Baseline condition:**
   The rock mass experiences moderate, hydrostatic stress ($\sigma_v \approx \sigma_h \approx 243$ kPa). The resulting factor of safety **(FoS ≈ 9.16)** indicates a stable condition, with stresses well below the rock mass capacity.

2. **Demolition stage:**
   The removal of the existing structure corresponds to an unloading of approximately 3 MPa at surface. Although this theoretically leads to a tensile stress, the analytical model limits σ_v to zero (no negative confinement). the computed **FoS ≈ 11** therefore reflects the system's stability. Physically, this stage represents a period of **stress relaxation** and should not pose a risk of instability, though localized joint dilation or minor movement may occur near the crown if discontinuities are present.

3. **New-build stage:**
   The new structure introduces a higher average surface load (4.5 MPa), producing a stress increase of about +1.5 MPa at tunnel level.
   The corresponding analytical result gives a **FoS ≈ 1.6**, indicating that the rock mass would be approaching its plastic limit according to the Hoek–Brown envelope, therefore **nonlinear deformation or localized yielding** could develop around the crown and sidewalls. Thus, this represents the **lower bound of acceptable stability** and signals the need for refined assessment.

The **Convergence–Confinement Method (CCM)** shows that the intersection between the GRC and SCC occurs at $p \approx 234$ *kPa** and $u \approx 0.37$ *mm**, showing that under the assumed stiffness, the lining would mobilize only a small portion of its load-

carrying capacity. The predicted convergence is minimal, consistent with a stiff support system and limited excavation-induced deformation.

However, this equilibrium is sensitive to both the assumed rock modulus and the support parameters (E_l, t, v_l). If the rock mass is weaker (e.g., GSI < 50) or the lining thinner, the operating pressure would shift upward, possibly reaching the lining's design limit.

5 Limitations and Recommendations

The analytical model represents a simplified system and should be viewed as an indicative approach. Elastic–isotropic assumptions and the use of an infinite-area surface load generally lead to upper-bound stress estimates, but geometric simplifications—such as representing multiple openings by a twin-tunnel system—may either over- or underestimate local interaction effects depending on the actual spacing and alignment. Material properties and in-situ stresses are generalized values without site calibration; therefore, while the approach ensures that results do not underestimate risk, it also limits quantitative accuracy. It should also be noted that no permanent rock support other than an assumed 100 mm shotcrete lining has been included in the analysis. In practice, tunnels typically include additional support elements (rock bolts, fiber reinforcement, or thicker linings) that strengthen the rock mass and would increase the overall factor of safety.

This assessment is therefore a first-order estimation, yet it indicates that the expected changes in the stress regime, associated with the demolition of the existing building and construction of the new structure, are unlikely to pose a risk to the existing tunnels. **The analytical solution confirms the viability of the planned redevelopment, while highly emphasizing the need for further refinement** and a more detailed investigation is recommended, ideally through a 3D numerical model (e.g. FLAC3D or RS3) to incorporate the actual building footprint, all tunnel openings, and nonlinear material behavior. Updated site data and monitoring should be established to verify assumptions and capture actual ground response. If future conditions deviate from those assumed, support enhancement or design adjustment may be required.

In addition, any redevelopment above or adjacent to Hagalundstunneln must consider Trafikverket's protection zones (skyddszon), within which no construction, excavation, or other ground-altering activities may be carried out without prior approval. The proposed works appear to fall within this regulatory zone and must therefore be coordinated with Trafikverket to ensure compliance with tunnel safety requirements.