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Lessons from Full-and Partial-Face Excavation in Difficult Rock Conditions

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Abstract

The selection between full-face and partial-face excavation methods is a decisive factor in the success and efficiency of conventional tunnelling projects. This choice is shaped by a complex interplay of geological and geotechnical rock mass conditions, environmental and urban constraints, regional design and construction standards, and permissible surface settlement thresholds. Each excavation method imposes distinct implications for project cost, schedule, and safety, highlighting the critical importance of informed decision-making during the early design phase. Over the years, continuous advancements in construction materials, excavation machinery, numerical modelling, and real-time monitoring technologies have significantly enhanced the feasibility of full-face excavation. These advancements enable the safe and efficient construction of tunnels with cross-sectional areas exceeding 150 m², pushing the limits of conventional tunnelling. Nonetheless, partial-face excavation remains an essential and often preferred strategy when staged excavation is required to ensure ground stability, manage deformations, or address specific site constraints. This study presents a comparative analysis of full-face and partial-face excavation methods, drawing on selected complex case histories from one of the major Alpine base tunnels and international projects under low overburden. Critical factors, including rock mass properties, excavation techniques, tunnel geometry, alignment, and construction sequencing, are evaluated to identify their influence on method selection and project outcomes. The case studies provide valuable insights into the practical advantages, limitations, and applicability of each approach, establishing a reference framework for conventional tunnelling practice in challenging geological and urban contexts. The paper highlights key considerations for optimizing excavation strategy, balancing safety, cost, and schedule, and leveraging modern construction technologies, providing engineers with guidance to select the most effective excavation method for complex tunnelling scenarios.

Keywords

Tunnelling, Conventional Tunnelling, Full-Face Excavation, Partial-Face Excavation.

1 Introduction

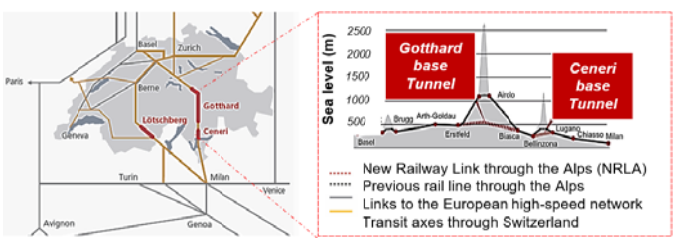
The choice between full-face and partial-face excavation is crucial for tunnel project success, impacting costs, schedule, and safety. This decision depends on geological, geotechnical factors, and environmental and urban constraints. Recent advances in materials, equipment, and monitoring have expanded the use of full-face excavation, even for sections over 150m² in complex ground conditions. According to Gamper (2014), Lunardi et al. (2024), modern full-face tunnelling methods offer significant advantages over traditional partial-face excavation in terms of cost efficiency, safety, and construction performance. By maintaining a continuously reinforced core-face, allowing immediate lining along the entire perimeter, and promoting uniform excavation profiles (i.e., circular or oval geometry), full-face excavation improves face stability and reduces stress concentrations. On the other side, the NATM emphasizes that the rock mass should be maintained in a load-bearing condition to prevent unnecessary loosening and softening. However, static and deformative analyses indicate that total deformation is lower for full-face excavation because of its capacity to improve the face stability by rapid ring closure and pipe arch pre-support systems. Operationally, full-face excavation enables higher levels of industrialization, reduced workforce exposure, and simplified logistics, resulting in improved safety and efficiency, and thus represents a major advancement in conventional tunnelling. Nevertheless, partial-face excavation may still be required in cases constrained by equipment size or tunnel geometry, or in certain geological settings, particularly with highly variable or unstable ground conditions, or where local constraints such as nearby structures require staged excavation. This method allows for better control over ground deformation in complex geologies, though it may involve more intensive support measures and longer construction durations. This paper compares the two methods through complex case histories, offering a framework for assessing the applicability of each one to illustrate the conditions under which each method is appropriate (Tab.1). As the case studies themselves demonstrate well, the method superiority is strongly context-dependent.

Table 1. Main features of the three case studies.

	Minimum Overburden	Unit	Cross section	Constrains	Excavation method
Ceneri BaseTunnel (Switzerland)	5-35m	Gneiss	≅ 80m^2	Historical building / nearby existing tunnels	Partial-face
Road60 Tunnel Project (Israel)	2-8m	Dolomite	≅212m^2	Nearby existing tunnels	Partial-face
Sotra Link Project (Norway)	6-50m	Moraine/Weathered bedrock	≅130m^2	Existing bridge and viaduct	Full-face

2 Ceneri Base Tunnel (Switzerland): partial-face excavation for vibration mitigation at Vezia South Portal

The Ceneri Base Tunnel (CBT) represents the southernmost section of the New Railway Link through the Alps (NRLA), crossing the Swiss Alps from North to South. Connecting the Swiss cities of Lugano and Bellinzona (Fig.1).The CBT lies within the crystalline bedrock of the southern Alps. An extensive geological survey involved boreholes as deep as 700 meters, with a total drilling length almost equivalent to the tunnel itself, allowing the identification and characterization of homogeneous geological units to define geological and geotechnical conditions (Fig.2).



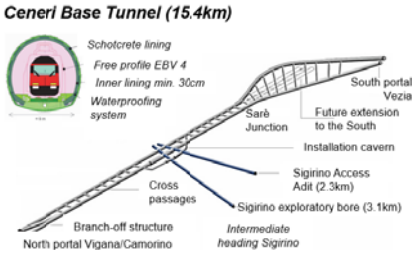


Fig. 1 General overview of the Ceneri Base Tunnel (Merlini et al. 2018).

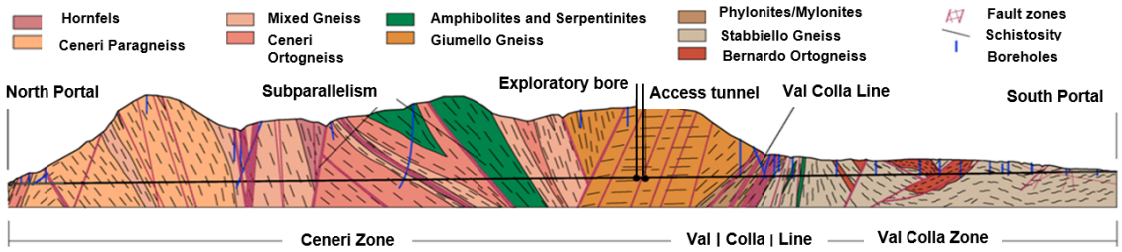


Fig. 2 CBT: geological longitudinal section.

The south portal located within Lot 854 was positioned in a critical urban environment near Vezia, where the minimum overburden was 11 meters, and included the presence of a historical building. It consists of three sub-sectors: 1) two single-line tubes (303m and 340m in length, cross-sectional area of 80m², overburden 5m-35m) excavated with the D&B method, including a 34m long by-pass; 2) two separate cut-and-cover tunnels connecting East and West tubes with the portal (Merlini et al. 2018, Gugelmann 2011); 3) the portal zone, which is the connection of the new section to the rest of the line. The entire site was constrained within a narrow corridor, surrounded by the A2 motorway, the existing Vedeggio-Cassarate road tunnel (distance approx. 4-5m from the Est tube), and the existing federal railway line (Fig.3). The geological formation in the project area, investigated through exploratory boreholes and seismic methods, is composed of Stabiello gneiss. This formation consists of a heterogeneous, jointed sequence of schistose gneisses and sericitic-chloritic schists.

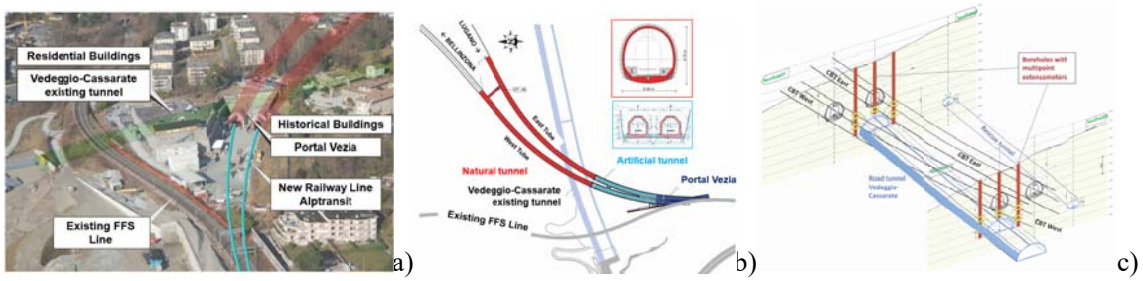


Fig. 3 Overall view of the CBT's southern portal: a) overview; b) plan view; c) axonometric plan of the Vedeggio-Cassarate road tunnel and the CBT south opposed tunnel intersection zone: multipoint extensometers installed in boreholes are able to transmit measurements continuously (Merlini et al. 2018).

A predictive analysis of the behaviour of the interfering structures, along with the implementation of special excavation and support techniques, together with advanced tunnel monitoring systems for subsidence and vibration control, was necessary to evaluate this interference. During the design phase, numerical analyses were performed using the 2D UDEC™ distinct element code, which enabled the prediction of the rock mass's stress-strain response and the surface-level impacts of the excavations (Fig.4).

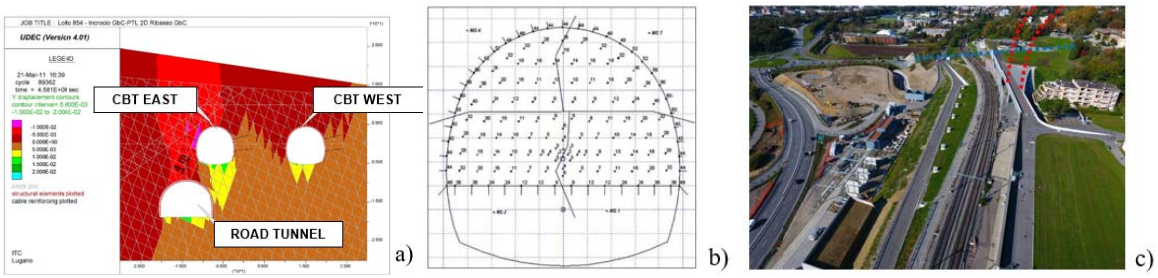


Fig. 4 a) DEM calculations; b) blasting plan for Roof 3A; c) final phase of the CBT's southern portal (Merlini et al. 2018).

To mitigate the risk of blast vibrations, the following measures were implemented: 1) technical restrictions on the D&B process, including: drilling and shot-firing allowed only during specific periods of the day, a general limit on the amount of explosive per initiation stage to less than 0.25 kg·delay^{-1}, subdivision of the excavation cross-section, and a reduction in round length; 2) stricter limits than those specified by Swiss standards (i.e., a minimum value of 6 mm·s^{-1} for frequencies below 30 Hz); 3) a comprehensive monitoring system using geophones, continuously assessed in real-time. The results of the design analyses demonstrated that a full-section excavation would have

exceeded the allowable vibration thresholds. A good agreement between the numerical simulation (5-10mm) and deformations obtained from monitoring (6-7mm) was recorded. The excavation was successfully completed approximately ten months ahead of the scheduled date outlined in the work program, while maintaining both environmental and economic sustainability.

3 Road 60 Tunnel Project (Israel): partial-face excavation to minimize the impact on the adjacent existing tunnel

The Road 60 Tunnels Project (R60TP) is situated along Highway 60 located between the city of Bethlehem and Jerusalem, Israel. R60TP involved the construction of two new road tunnels in addition to the refurbishment of two existing tunnels. The new ones, 870m and 270m in length, respectively, run almost parallel to the existing tunnels. The maximum overburden amounts to 35m and to 70m for the short and long tunnels, respectively, with a minimum of 2m to 8m close to the portal areas. The external width of the new tunnels amounts to about 17m, while their height amounts to about 10 m. Both long and short new tunnels accommodate three traffic lanes and two emergency pedestrian lanes (safety walks). The project also included the construction of four cross passages between the new and existing tunnels, as well as six emergency bays along the road alignment (Schuerch et al. 2025). R60TP provided a challenging case for the application of partial-face excavation. The presence of a highly heterogeneous and weathered "Transition Zone" between the Amminadav and Moza formations (Fig.5), together with the minimal separation between the new and existing tunnels (as low as 3m), imposed severe constraints on the excavation process. The Transition Zone, composed of brecciated dolomite blocks embedded in marl and clay, exhibited rapid variations in mechanical behaviour - from competent rock to cohesionless sandy dolomite or swelling marl - making full-face excavation unsafe and unpredictable. The parameters reported in Table 2 were assigned based on available site investigation data, supported by literature values for comparable rock masses and engineering judgement, with the cohesion values representing equivalent rock mass properties adopted for numerical modelling. To mitigate these geotechnical uncertainties and reduce the risk of damaging the operational existing tunnel - based on the results provided by numerical analyses carried out by the RS2 Finite Element code (Fig.6) - the excavation was performed in multiple partial phases (Sequential Excavation Method).

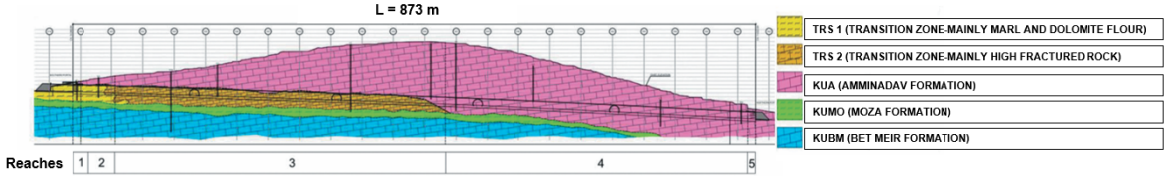


Fig. 5 R60TP: Longitudinal and geological profile of the long tunnels.

Table 2. Geotechnical properties of the soil/rock units.

Unit	**6a**	**4**	**5a, 5b**	**1**	**2**	**3**	**6b**
Geology	**Transition zone 1 (Portal)**	**Transition Zone 2**		**Amminadav Formation**			
Unit volume weight, γ [kN/m^3]	27	27	25	27	27	27	27
Cohesion, c'[kPa]	150	1500	26	6995	5828	3148	1300
Friction angle, φ [°]	30	35	33	40	40	40	35
Young's Modulus, E [GPa]	0.25	2.65	0.15	8	6.5	3.5	0.6

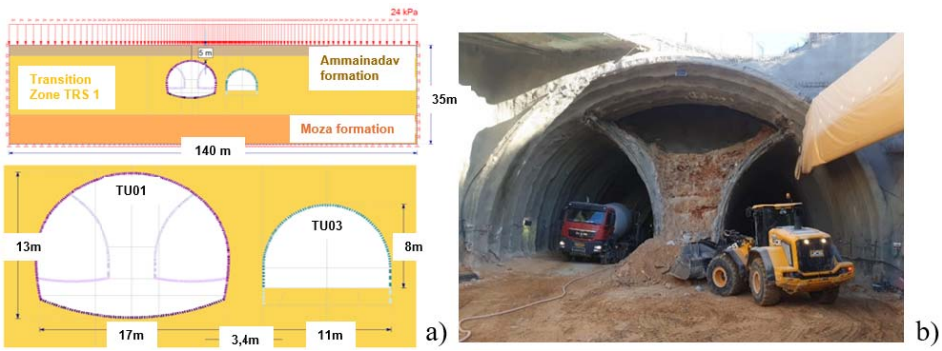


Fig. 6a) RS2 model; b) excavation phase (Schuerch et al. 2025).

The sequence began with a side drift adjacent to the rock pillar separating the two tunnels, followed by one to three additional stages (top heading and benching), the number of which varied according to local ground conditions. This staged approach allowed for progressive assessment of the rock mass and real-time adaptation of support measures. In the most critical sections within the Transition Zone, four excavation stages were implemented to maintain tunnel stability and minimize deformation: two side drifts, top heading, and benching, including invert closure. The use of partial-face excavation offered several advantages: it enabled direct geological mapping of the rock pillar before enlargement, reduced vibration transmission to the existing tunnel, and allowed pre-reinforcement through grouting or transverse bolting when karstic voids or weak zones were encountered. At the peculiar Emergency Bay South section, where the new and existing tunnels intersected under a 55m overburden, partial-face excavation proved essential. The combination of large spans (up to 35m), ongoing traffic in the adjacent tube, and poor ground conditions required controlled excavation stages supported by steel ribs, shotcrete up to 40cm thick, spiling, and fiberglass face bolts. This sequenced construction ensured both safety and service continuity, demonstrating how partial-face excavation within the Sequential Excavation Method can provide a flexible and reliable solution under complex geological and operational constraints. Started in 2019, R60TP was successfully completed in 2024, with the commissioning of the refurbished existing tunnels.

4 Sotra Link Project (Norway): full-face excavation of Drotningsvik Tunnel at the Kipledalen section

The Sotra Link Project (SLP) is one of Norway's priority infrastructure projects. It encompasses the design, financing, construction, and maintenance of a range of surface and underground facilities, connecting the city of Bergen and the Sotra island. The SLP comprises 9km of highway, a 900m-long suspension bridge, 12.5km of tunnels (Kolltveit, Straume, Knarrvika, Drotningsvik), 19 roads and pedestrian underpasses, 3 tunnel portals, 22 bridges and viaducts, and 14km of pedestrian and bicycle paths (Barbieri et al. 2025). Construction works started in March 2023, with ongoing activities across all four main tunnels. The Drotningsvik area, located in the Eastern section, features the most complex underground structure of the project. Excavated near the Kiplevatnet lake, the most complex section of the Drotningsvik tunnel includes both the North tube and the South tubes. This section spans a total length of 30m, with a maximum width of 19m, and a front height reaching up to 9m.The main engineering challenges with this section can be summarized as follow: i) very limited overburden, with a minimum cover of approximately 6m; ii) potential interference with an existing viaduct which pillar load is founded in the ground upon the North tube; iii)interference with the pillar of a planned pedestrian bridge to be built after the tunnel excavation; iv) poor quality of fractured rock mass infillings, which may induce swelling phenomena and consequently affect the long-term performance and durability of the permanent structures (Tab.3). The excavation design solutions for Kipledalen section have been developed to minimize the possible impact of the tunnel construction on the upper viaduct to be maintained fully operative and to avoid draining the lake's water resources.

Bidimensional finite element analyses have been performed by RS2 code, under plane strain conditions, to estimate stresses and displacements on the concrete support during the full-face excavation phases (Fig.7a). An elastic-perfectly plastic Mohr-Coulomb criterion was adopted as the constitutive law. Supports consist of rock bolts (ϕ25, L=4-5m), primary lining (FRC B35 thickness 250mm + steel mesh K335),spiling bolts (73/56 L=8m), fiberglass bolts at tunnel face (ϕ25, L=6m, spacing 1.1x1.1m). Reinforced ribs of shotcrete RRS (spacing 1.50m, shotcrete B35: thickness 600mm, steel bars ϕ20 B500NC) have been adopted as a permanent lining for stabilizing very poor rock mass conditions. The rib-reinforced shotcrete arches provide a superior stiffness if compared to lattice girders, because of an increased adherence with the tunnel perimeter and a systematic bolting of the arches (Fig.7b, Barton and Grimstad 2014). This solution provides a general optimisation if compared to an equivalent primary liner,e.g. in terms of section efficiency and higher capacity to bending moment, under compression. However, a higher load capacity to pure tensile force is given by an equivalent lining, but this represents an infrequent condition in tunnelling.

Table 3. Geotechnical properties of the soil/rock units.

	γ [kN/m^3]	φ [°]	c' [kPa]	E [MPa]	v [-]	Ψ [°]	Q [-]
Rockfill	20	42	10	75	0.25	12	-
Weathered bedrock	29	30	70	1500	0.20	0	0.14
Good rock mass	29	40	4500	6000	0.17	10	290

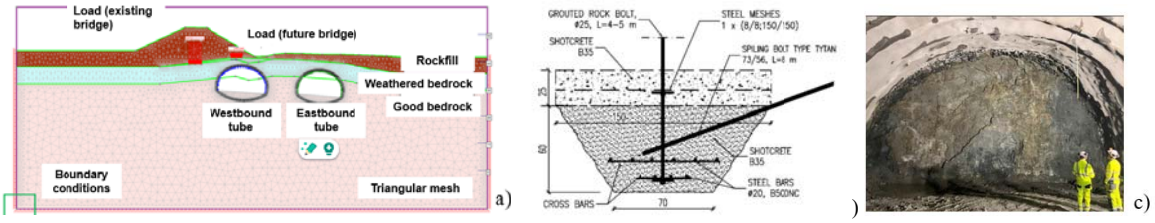


Fig. 7 a) FE model, geometry loads and boundary conditions (km 8+126); b) reinforced ribs of shotcrete detail; c) tunnel face during the excavation of the Kipledalen section (Barbieri et al. 2025).

Monitored displacements during the excavation phase were then considered for the model validation to calibrate the deconfining ratio. The presence of loosened surficial deposits at the tunnel and the lake required grouting of curtains and crowns around and above the excavations to improve the effectiveness of the radial bolting and the waterproofing. Before the tunnel excavation, pre-grouted columns were also executed all along the lake perimeter to avoid water inflow towards the tunnel. The excavation of the Kipledalen section has been completed, and the RRS - as a rigid, permanent lining - has been successfully adopted under critical geotechnical conditions, which do not guarantee complete effectiveness of radial bolting. Under such conditions, traditional tunnelling practice would adopt either a double-shielded support or a trenching methodology, such as cut and cover.

5 Conclusions

The case histories shown in this paper demonstrate that, while full-face excavation enhances efficiency and safety in stable or reinforced ground, partial-face excavation remains indispensable for managing uncertainty, minimizing vibration, or protecting adjacent structures. The integration of observational methods, real-time monitoring, and adaptable support systems is critical in both approaches. The combination of full-face advancement and rapid ring closure has proven especially effective in controlling deformation. The authors would like to highlight the following main conclusions: (1) Full-face excavation offers superior deformation control, safety, and productivity when adequately supported by pre-confinement and reinforcement systems; (2) Partial-face excavation remains essential in complex geological settings or urban areas with strict vibration and deformation constraints; (3) The choice between methods should be based on quantified geomechanical parameters and integrated design tools such as ADECO-RS and NATM; (4) Contract models should encourage early collaboration among designers, contractors, and clients to allow method optimization and risk sharing; (5) Continuous monitoring and data interpretation are vital for real-time decision-making during excavation.

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