This is Chapter 11 “Discussion” from my PhD thesis

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Enjoy reading.

Best regards,
Alexander Kluckner

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Tunnelling at greater depths: Study on the ground and system behaviour when passing a stiff rock block in a weak zone

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Abstract

Title: **Tunnelling at greater depths: Study on the ground and system behaviour when passing a stiff rock block in a weak zone**

Keywords: deep tunnelling, conventional, brittle fault zone, block-in-matrix, stiff block, shear bands

A stiff block in brittle, weak fault zones can lead to unfavourable ground behaviour when being approached by a tunnel drive. It attracts stresses and may fail when it is close to the tunnel face endangering the tunnel stability. The thesis investigates the ground behaviour with a quasi-two-dimensional parametric study. The tunnel diameter is 10 m, the block height is 2 m, 5 m, or 10 m, and the distance between the block and the tunnel is 1 m, 5 m, or 10 m. One critical case is analysed in three dimensions. Another study simulates a real tunnel drive that crosses a block with a height of over 25 m. Strain data from a lining segment measured at the construction site with a distributed fibre optic sensing system is used to set up the Burgers-Mohr model simulating the shotcrete material behaviour. All simulations consider an interface between the block and the matrix material. From the block, shear bands form towards the tunnel. In the parametric study, even if the block-matrix stiffness contrast is high or the block is close to the tunnel, differences in the tunnel displacements between cases with block and related cases without block are little. Of the cases analysed, those with a hydrostatic primary stress state are least favourable. If the primary stresses are anisotropic, the effect of the block on the ground behaviour strongly depends on the block distance. The case study suggests that the block must be not too high to be hazardous if it fails. Otherwise, stresses redistributed because of the tunnel drive cannot concentrate at the block's top and bottom. In case the stresses are high enough at the moment of block failure, large-scale shear failure of the rock mass close to the tunnel may occur. If the location of blocks is unknown, state-of-the-art approaches to evaluate tunnel displacements must be applied to increase the probability of identifying blocks in time during tunnelling. Making the system stiffer of less stiff (e.g., by adapting the moment of ring closure) may not lead to a less hazardous situation. It is advised to minimise the unreinforced rock mass volume close to the tunnel face to prevent shear bands from reaching the tunnel.

Contents

Chapter 11

Discussion

The following sections discuss the most important considerations that originate from the results of the thesis' studies. Note that the studies and related descriptions focus on the hazardous nature a stiff block can feature during tunnelling. But a block can also have positive effects. If it does not fail, it can reduce the ground deformation resulting in a lower utilisation of the support.

11.1 Primary stress

The direction of the secondary major principal stresses relative to the block surface determines the interface slip. In the cases analysed, the block is vertical and next to the side wall or in front of the tunnel face. In terms of the lateral pressure coefficient, k_0, the primary stress states considered have been:

- 1, 0.5, and 2 in the parametric study (cf. Chapter 6 on p. 81);
- 1 in the supplementary case (cf. Chapter 7 on p. 159);
- 0.5 . . . 0.75 in the validation case (cf. Chapter 10 on p. 243).

Cases where k_0=1 or k_0=0.5 have been least favourable. Depending on the block distance, secondary major principal stresses close to the block top and bottom are then often sub-vertical promoting interface slip at the block front. Exceptions are cases or intermediate excavation stages where k_0=0.5 and where the block is outside the yield zone. Here, the directed stress state pushes some material within a particular region outside the yield zone away from the tunnel. If the block is part of this region, interface slip at the block front remains small. Most favourable have been cases where k_0=2. The high major principal stresses passing around the tunnel and crossing the block are sub-horizontal and prevent most interface slip. Exceptions are where the block is very close to the tunnel or to the yield front. Here, the boundary conditions at the tunnel periphery force the highest stresses to be tangential to the tunnel. Then, also in k_0=2 cases, the stress trajectories close to the block align more vertically. Note that only in the k_0=1 and k_0=0.5 cases the elastic energy state increases when the tunnel is excavated.

If the block is horizontal and above the crown (not analysed), k_0=1 remains unfavourable but the situation for k_0=0.5 and k_0=2 changes. If k_0=0.5, highest stresses are then sub-vertical and prevent much of the interface slip along the block above the crown. And for k_0=2, the situation is unfavourable if the block is close to the tunnel. It is favourable if the block is outside the yield zone and farther away because here, too, the directed stresses push material away from the tunnel (not shown). The latter will be less pronounced than next to the side wall in the k_0=0.5 case because of gravity.

An analytical calculation or a simple numerical simulation considering the actual excavation shape can help to identify where the highest stresses will develop (temporarily and finally). Analytical calculations cannot consider for the block. But the analysis of the matrix-only case still gives a good idea. Also literature can provide relevant information (e.g., [100] for the development of the secondary principal stresses along the longitudinal tunnel profile).

11.2 Block distance and size

Since the stiff block usually features a higher strength, increased stresses are required to cause block failure. In tunnelling, highest stresses pass around the yield zone. Thus, the yield zone which enlarges as the tunnel drive continues must at least come very close to the block.

In the supplementary case (cf. Chapter 7 on p. 159), the mean value of the secondary major principal stresses in the block exceeds the initial value by over 50% the first time when the tunnel face distance to the block is 7.5 m (cf. Fig. 7.4 on p. 165). The distance between the block and the yield zone in front of the tunnel is then approx. 1 m.

When the block is close to the yield front, in the event of block failure, it is more likely that shears developing from the block towards the tunnel cross through the volume of intact rock mass and connect with the yield zone.

The farther away the block from the tunnel (in-plane consideration), the less likely it is that the block will fail due to the lower stress level. When the block is farther away, the volume between the tunnel and the block is larger providing more resistance against large-scale shearing failure towards the tunnel. The greater the distance of the block from the tunnel, the larger the block must be to have a significant effect on the ground behaviour close to the tunnel.

No statement can be made regarding the minimum size a block must feature so that its failure results in a hazardous situation. Significant differences in the tunnel displacements could be observed only in d1|h5 & 10 cases. Here, the volume between the block and the tunnel will provide least resistance since it has been further damaged by the formation of pronounced shear bands. From this point of view, the minimum block size is $0.5D$.

The validation case (cf. Chapter 10 on p. 243) disclosed the consideration that the block must not be too large in order to experience significant loading in the sub-vertical direction by high stresses passing around the tunnel. Otherwise, stresses cannot concentrate at the block top and bottom and not much interface slip will be introduced at the block front. Then, no pronounced increase in stresses will occur and the block will not be able to hold back material behind and above and below it that is pushing towards the tunnel. Since the highest stresses pass around the yield zone, the size of the yield zone determines the maximum size of a block which will be still loaded significantly. As the yield zone progresses towards or even beyond the block, there must be the moment when the highest stresses cross the block near its top and bottom. If this does not happen—like in the validation case where the block top and bottom are far above and below the tunnel, respectively—, highest stresses will be at the block front only. Interface slip may result but stresses cannot concentrate at the block top and bottom.

Consider the case illustrated in Fig. C.44ac (p. 483). It's already a case where the block should not be higher to remain hazardous. Here, the highest stresses passing around the tunnel subject the block at its upper front. Because of the interface slip, the $\sigma_{1,max}$ path moves towards the block top. If the block would be higher, the block top will be too far away from the highest stresses. Stresses would not concentrate much at the block top. However, if the yield zone enlarges, the block can be also higher and will still attract high stresses and remain hazardous.

Once the block is in the yield zone to its full extent, the block loading will not increase much anymore. If the yield zone further enlarges, the yield front with the highest stresses moves away from the block. In addition, stress concentrations reduce by yielding of already failed matrix material, interface slip, shear band formation, and relaxation and movement of the block towards the tunnel. Note here, e.g., the turning points in Fig. C.47ab/bb (p. 490). So, if the block has not failed up to that moment when it is considerably apart from the yield front, and if the lateral support does not decrease much anymore (e.g., lining is installed), it mostly like will not fail. The most critical situation is when the block is close to the yield front (on both sides of it) and when further excavation increases the loading and reduces the lateral support.

An analytical calculation or a simple numerical simulation considering the actual excavation shape can help to identify the size and shape of the yield zone (temporarily and finally).

11.3 Block stiffness

The stiffness contrast determines the magnitude of the stress increase in the block. The higher the stiffness contrast, the higher are the stresses.

A hazardous situation seems to be the shear failure of the block triggering large-scale shear failure of the rock mass between the block and the tunnel (cf. Section 11.5). The stresses that have accumulated until the block failure then are the driving forces for the large-scale movement. If the block distance is larger, the block then also has to attract more stresses before failure and, thus, needs to be stiffer. It is not a linear relation since the length of the shear surfaces from the block to the tunnel and the mobilisable shear strength increase exponentially.

In the parametric study, differences between a with-block case and the related matrix-only case are more often considerably larger when the stiffness contrast is greater than 2. Note that [254] already suggested a minimum stiffness contrast of 2 that the block affects the ground behaviour considerably (cf. Section 3.2.9 on p. 36).

11.4 Block failure

In the three-dimensional supplementary study (cf. Chapter 7 on p. 159), the stiffness contrast and the strength contrast are both 20 and the ratio of the Young's modulus to the uniaxial compressive strength is 500 for both the matrix material and the block material. The ratio $|\sigma_{1,max}/\sigma_c|$ in the primary state is 0.26, and it is 1 for the ratio σ_1/σ_3. The ratio $|\sigma_{1,max}/\sigma_c|$ exceeds 0.5 (damage initiation) for the first time when the tunnel face is 11.5 m in front of the block. At that moment, the distance between the block and the yield front ahead of the tunnel face is approx. 5 m. If the block features a higher strength, the ratio $|\sigma_{1,max}/\sigma_c|$ will exceed 0.5 later when the tunnel face is closer to the block. If the block features a higher stiffness, the parametric study (cf. Chapter 6 on p. 81) suggests a higher $\sigma_{1,max}$ in the block.[1] Then, the ratio $|\sigma_{1,max}/\sigma_c|$ will exceed 0.5 earlier. The increase in $\sigma_{1,max}$ with increasing stiffness contrast is disproportional and features a decreasing trend. The ratio σ_1/σ_3 exceeds 10 (spalling) for the first time when the distance between the tunnel face and the block is 4.5 m. At that moment, the block also fails in tension. However, in reality, sudden extensional failure may never occur

[1] In the parametric study, the block strength and stiffness increase simultaneously. Since the increase in strength outweighs the increase in $\sigma_{1,max}$, $|\sigma_{1,max}/\sigma_c|$ decreases with increasing stiffness contrast.

if gradual crushing has already been initiated when $|\sigma_{1,max}/\sigma_c| > 0.5$. Further, the parametric study suggests that the spalling limit is exceeded earlier when the stiffness contrast is higher. This trend is valid only as long as the block is outside the yield zone. Shear failure in the block occurs when the tunnel drive is 1 m farther ahead. The distance between the tunnel face and the block is then 3.5 m. Also in the parametric study shear or tensile failure occurs very late (i.e., during one of the last four main relaxation steps). If the block strength is lower than simulated (the ratio E/σ_c is then higher)[2], failure occurs earlier.

The above example perfectly illustrates the problem when trying to answer the question of when the block will fail: the stiffness contrast determines at which rate the block attracts stresses but the block strength and intactness decide about the moment of failure initiation. Since in tunnelling, both the stress state (primary or secondary) and ground characteristics are usually known approximately only (if at all regarding the characteristics of any stiff blocks), defining the exact moment of failure is not possible.

In the validation case (cf. Chapter 10 on p. 243), the contrasts and ratios are: $(E_b/E_m)_{min} = 464$, $(\sigma_{c,b}/\sigma_{c,m})_{min} = 91$, $(E_m/\sigma_{c,m})_{min} = 298$, $E_b/\sigma_{c,b} = 1516$, $|\sigma_{1,max}/\sigma_c|_{primary} \approx 0.04$ (theoretically)[3], $(\sigma_1/\sigma_3)_{primary} = 0.5$ (theoretically)[3]. In the simulation, the tunnel drive is approx. 5 m before the chainage where the block enters the excavation area when $|\sigma_{1,max}/\sigma_c|$ exceeds 0.5 for the first time.

Let's assume the block location next to the tunnel is known. An analytical study (e.g., with the approach by [117] used in this thesis) then can give at least the stresses at the block location in the matrix-only case depending on the internal support pressure. A combination with an approach accounting for the spatial displacement development (cf., e.g., [305, 411] and references cited there-in) yields the stresses depending on the distance of the tunnel to the block in the direction of the tunnel drive (being equivalent to a particular internal support pressure). In reality, the maximum tangential stresses in the block will be always higher, at least if the stiffness contrast is significant (i.e., $rYbm > 2$) (cf., e.g., Fig. C.38 on p. 470 and Fig. C.39 on p. 471)[4]. Also $\sigma_{1,max}$ and the maximum ratio σ_1/σ_3 will be higher (cf., e.g., Tab. C.45 on p. 469 and Fig. C.37 on p. 443). The situation is less favourable if the distance of the block to the tunnel is small. If, for example, the situation shown in Fig. C.38bb exists at an intermediate or final excavation stage, in the matrix-only case, the maximum ratio σ_1/σ_3 of approx. 7 is at the block front. $\sigma_{1,max}$ at the block back is approx. -14 MPa. Assuming a block strength of 30 MPa, $|\sigma_{1,max}/\sigma_c| = 0.47$. In the with-block case, both ratios will be less favourable and have approached or even exceed the thresholds (here, 10 and 0.5, respectively). Thus, in reality, the block will have been damaged already at that excavation stage. Consider that the rock blocks investigated in the project may feature lower damage initiation and spalling thresholds (cf. Section 6.3.11 on p. 118). Note also that in contrast to the setting in the parametric study, the strength contrast may be lower than the stiffness contrast. Then, up to a particular moment, the block attracts the same amount of stresses as in the parametric study but will fail sooner.

To have the stiff block become hazardous in terms that it fails when it is close to the tunnel face, it requires considerable strength. If the stiffness contrast is significant but the strength contrast is rather low, the block will gradually fail before the tunnel comes close to it.

[2]Consider here the large natural range of the ratio E/σ_c cited in Section 3.2.9 (p. 36).

[3]In the numerical simulation, the stress initialisation approach resulted in tensile stresses in the block in the primary state. The theoretical primary stress state comprises compressive stresses only.

[4]The figures only show the tangential and radial stresses. But along the symmetry plane they equal the secondary in-plane principal stresses.

This thesis lacks of any fittings providing approximations of secondary stresses in the block since almost all cases analysed in the parametric study feature different matrix material properties.[5]

11.5 Hazardous ground behaviour

All numerical studies in this thesis suggest that damage of the block occurs early. The initial assumption that sudden rock-burst like failure of a block close to the tunnel is the most hazardous scenario is unrealistic.

Rather a large stiff block attracts stresses and holds back a large volume of weaker material behind it that may lead to a problematic situation in case of block failure. Increasing stresses and the reduction of the lateral support will gradually damage the block. However, this damaging process may be local only and the block continues to attract stresses. At any moment, shear failure along a single weak plane in the block may trigger large-scale ground failure. Shear bands that have formed before towards the tunnel now serve as weak shear surface for the moving masses. Material behind the block can now relax and moves towards the tunnel. Stresses that have concentrated at the block top and bottom are the driving forces of the large-scale movement. The block must be large to have a large volume involved in the movement when shear failure occurs. Otherwise, the moving masses will not be able to push the material ahead towards the tunnel.

The situation becomes more critical if sub-vertical weak discontinuities like faults or slickensides surround the block on its sides facing away from the tunnel. Stresses then cannot redistribute also to material beyond the block but concentrate even more in the block.

11.6 At site actions

If a tunnel drive is close to or inside a rock mass zone where engineers expect to encounter stiff blocks but the location and the characteristics of those blocks are unknown, continuous observation may help to identify them in time and the tunnelling method decides about whether block failure occurs and about the consequences.

The following subsections give some considerations and recommendations related to the displacement monitoring, the tunnel support, and the tunnelling sequence. Some recommendations match with those the experts have formulated after the collapse at the *Galgenberg* tunnel (cf. Section 1.1 on p. 1).

11.6.1 Displacement monitoring

Since the location of a potentially hazardous block is unknown, it can be all around the excavation volume or partly or fully in the excavation volume ahead of the advancing tunnel face. Thus, one does not know beforehand which monitoring target and which of its displacement components will be affected first and most by the block. There is no alternative but to analyse all displacement components of all targets and to apply all evaluation approaches. In the parametric study (cf. Chapter 6 on p. 81), the evaluation of both the total in-plane displacements and the in-plane displacement vector orientation disclosed the existence of the block in significant cases.[6] It

[5]The matrix material properties between two cases are equal if only the stiffness contrast or the block height (in $R_{pl,c1}$ cases only) differs.

[6]The parametric study lacks of comparing tunnel displacements of the right tunnel side (next to which the block is) with tunnel displacements of the left tunnel side. Anyway, since displacements of the right half are compared with those from the related matrix-only case introducing significance thresholds, the same conclusions result.

was the longitudinal displacement and the out-of-plane displacement vector orientation in the supplementary case (cf. Chapter 7 on p. 159). And in the validation case (cf. Chapter 10 on p. 243), it is only the difference in the vertical displacement between the side walls that gives clear signs for the block.

In contrast to what was expected before performing the parametric study, the block does not result in a decrease in the tunnel displacements by preventing zones behind to move towards the tunnel. This blocking effect could not be observed. Shears forming in front of the block reduced any such effect. It is the same in the supplementary case. In the validation case, it is different. However, there, the block is extraordinarily large compared to the tunnel. Still, not the decrease in the individual displacement component was significant but the increase in the difference between the left and right side wall.

In order to be able to identify the block by interpreting displacement monitoring data the evaluating engineer must have an idea about the tunnel displacements that will develop during the excavation of the next rounds. This idea usually develops by comparing the geology of rock mass zones along already excavated tunnel sections (showing particular displacement characteristics) with the geology of the rock mass zones ahead of the current tunnel face. Then, also an idea about the geological conditions ahead must exist. Published trends for when the tunnel drive approaches weaker or stiffer zones ahead help to narrow down possibilities (cf., e.g., [59, 139, 218, 382]). Another option is to compare with data from finished projects. However, such data is rarely available to be processed easily.

For the case that the block is ahead of the tunnel face and for its major part in the excavation area, shears may develop between the block and the tunnel face resulting in increased displacements of zones close to the shears in the longitudinal direction. Then, one might think of monitoring the displacements of the tunnel face. However, the tunnel face is a temporary feature. Strain increments developing while a tunnel face exists[7] will be too small in most cases to cause a significant increase in tunnel face displacements.

The general rules for monitoring in tunnelling apply also here:

- the smaller the distance between monitoring cross sections,
- the earlier the initial reading is taken after the installation of the monitoring cross section,
- and the higher the reading frequency,

the better the comparisons will work and the more likely it is that changes in the geological conditions will be identified in time. This is especially important when tunnelling through difficult or little investigated ground. At the end, it is a cost-benefit consideration. If the distance between monitoring cross sections is too large, the probability is high that the last monitoring cross section before the block is too far away to be affected by the block. If the initial reading is performed too late, valuable information is lost and the dependency on estimations of the pre-displacements increases and inaccuracies probably too.

Equally important is the selection of thresholds to distinguish between significant and insignificant changes or differences in tunnel displacements when performing comparisons. Here, the considerations by [140] may be of help.

Overall, considering that the parametric study did not account for pre-displacements, both the parametric study and the supplementary study did not account for a tunnel lining, and that in reality the rock mass is usually heterogeneous and anisotropic, the thesis' results suggest that a

[7] In terms of being stable and free of any machines in front of it for a longer period to allow a continuous measurement.

hazardous block with a maximum in-plane size of 1D is too small to be recognised. To determine the minimum size, more case studies or theoretical studies with tunnel linings are required.

11.6.2 Tunnel support

Because the initial utilisation of a block and the stresses it attracts during the excavation process are unknown or can be roughly approximated only, the moment of block failure cannot be precisely controlled by adapting the support (cf. also Section 11.4 on p. 279). The actual moment will range between the two extremes: (1) If the support is very soft and flexible, it allows for rock mass deformation. This leads to early interface slip, stress concentrations in the block, and shear band formation. The block will also fail early (if it does at all). At that moment, the tunnel face may be or may be not far from the block. (2) If the support is very stiff and inflexible, it allows for small rock mass deformation as long as it does not fail. Interface slip, stress concentrations, and shear band formation occur late, and so does block failure (if it fails at all). The block may then be close to the tunnel face or even behind it (if it is next to the tunnel).

Since support adaption towards both extremes can be unfavourable, no minimum or maximum support pressure recommendations can be given.

The most critical situation is when the block is close to the tunnel face and has not failed yet. Any of the following actions of the excavator may trigger the collapse. Now, stresses build up in the block if interface slip proceeds. Shear band formation between the block and the tunnel promote interface slip. Rock bolts can reduce shear band formation and, thus, also interface slip if they cross relevant rock mass sections close to the block: (a) above and below the block to the reduce the formation of the main shear bands, and (b) in front of the block to reduce the formation of the central shear bands. The formation of subsidiary shear bands will then be reduced too. Since the block location is unknown, and also the block shape and orientation which determines the orientation of forming shear bands, the bolting pattern needs to be systematic rather than selective. This is already standard in deep tunnelling. Longer bolts are better because the probability of having shears or blocks crossed increases. However, this is costly. Thus, a bolting pattern comprising shorter and longer bolts may suffice as well (cf. Fig. 11.1a). The distance between bolts should be not too large. Otherwise, there is space for shears to propagate up to the tunnel. Anyway, more critical is the temporarily supported working area behind the tunnel face. Here, a large unreinforced rock mass zone exists between the last row of radial bolts and the face bolts (cf. Fig. 11.1b). To minimise the zone size, radial bolts could be tilted towards the direction of the tunnel drive and face bolts outwards in the radial direction. The latter seems to be problematic practically because face bolts closest to the tunnel periphery are lost for activation or re-activation as soon as the next rounds have been excavated. In addition, face bolts extending outside the excavation volume interfere with the installation of radial bolts. Nowadays the installation of grouted or non-grouted spiles is also standard in deep tunnelling. They may be used to minimise the size of the unreinforced zone instead of tilting the face bolts. In any case, the effective distance between the ends of rock bolts and spiles should be small to allow for the formation of stress arches between them. When shears then propagate towards the tunnel, either before or at the moment of block failure, the load-bearing arch formed by the rock bolts, spiles, and stress arches between the reinforcement elements will prevent shears to reach the tunnel. Loads originating from material pushing towards the tunnel (e.g., material held back by the block until its failure) will be distributed uniformly over the load-bearing arch. If the unreinforced rock mass zone is too large, the probability is higher that there shears concentrate and develop up to the tunnel. Large-scale shear failure may occur in the event of block failure.

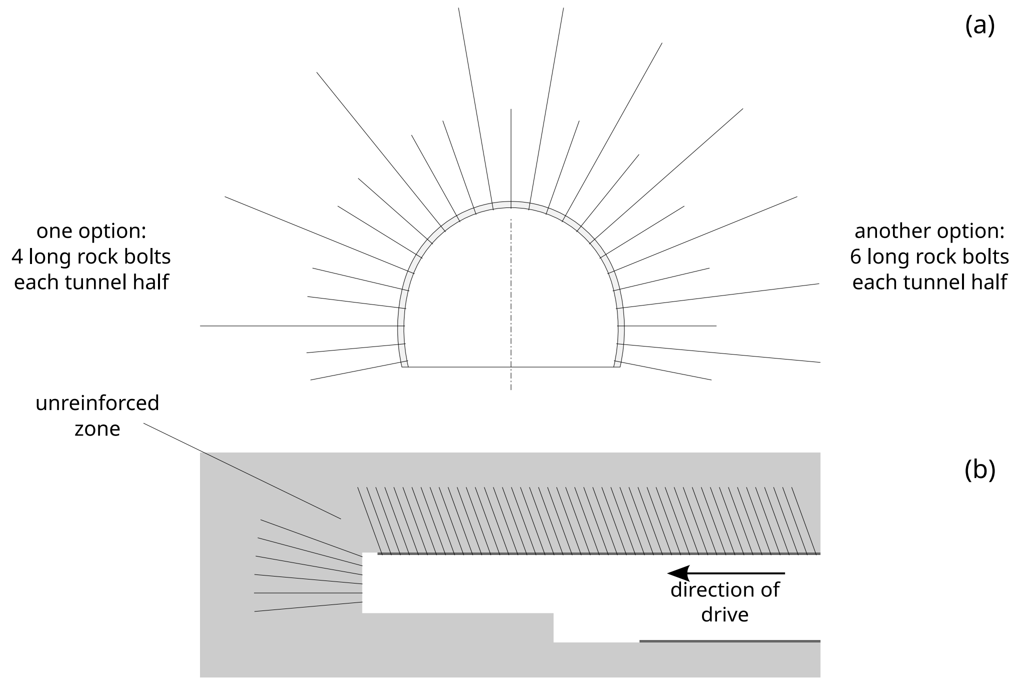

Figure 11.1: Schematic illustration of rock bolt patterns in the cross section (a) and in the longitudinal section (b). Long bolts to increase probability of crossing shear bands developing from stiff blocks. Shorter bolts to minimise the size of unreinforced rock mass zones in the circumferential direction of the tunnel periphery. Inclination of rock bolts in the longitudinal direction (i.e., out-of-plane) to minimise the size of unreinforced rock mass zones above and to the sides of the working area at the tunnel face.

When heading through a zone potentially comprising stiff blocks, the purpose of precautionary measures is not to prevent block failure. One does not know whether and when it will happen. It is rather all about preventing shears to reach the tunnel. In the parametric study, pronounced shears at the tunnel boundary caused displacement vectors to change significantly allowing to identify the block. Thus, the more measures are taken to prevent the development of shears, the more difficult it will be to identify the block by evaluating displacement data.

11.6.3 Tunnelling sequence

Regarding the moment of ring closure the same problem exists as for the stiffness of the support (cf. previous section): an early or late ring closure can promote or mitigate a hazardous situation as it determines the moment of block failure.

It is similar for the number of partial faces. Anyway, reducing the size of temporarily unsupported areas increases the probability that the active support withstands the additional loading when block failure initiates large-scale shear failure.

If there is a block outside the excavation volume, stresses redistributed because of the excavation of the next round may or may not lead to block failure. But if the block fails and there was not much time for the previously installed cement-based support elements to harden—because of a too high tunnelling rate—, the support capacity available may be too low to prevent the large-scale shear failure from happening. Block failure can also occur when the heading is already farther ahead. Then, the utilisation of the lining segments next to the block may increase even if it has decreased before. Unobserved block failure may be the cause for cracks in the lining at a relatively late stage of lining hardening.