Characterization and engineering properties of tectonically undisturbed but lithologically varied sedimentary rock masses

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CHARACTERIZATION AND ENGINEERING PROPERTIES OF TECTONICALLY UNDISTURBED BUT LITHOLOGICALLY VARIED SEDIMENTARY ROCK MASSES

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Abstract

Tectonically undisturbed sedimentary rocks deposited in a quiescent shallow marine environment often include a sequence of strata that may present significant lithological variety at the scale of an engineering structure. Such rock masses exhibit engineering properties that are significantly different from tectonically disturbed rock masses of similar composition. For example, molasse consists of a series of tectonically undisturbed sediments of sandstones, conglomerates, siltstones and marls, produced by the erosion of mountain ranges after the final phase of an orogeny. They behave quite differently from flysch which has the same composition but which was tectonically disturbed during the orogeny. The molasses behave as continuous rock masses when they are confined at depth and the bedding planes do not appear as clearly defined discontinuity surfaces. Close to the surface the layering of the formations is discernible and only then similarities may exist with the structure of some types of flysch. Therefore extreme care is necessary in the use of geotechnical classification systems for the selection of design parameters, in order to avoid penalizing the rock mass unnecessarily. A discussion on the use Geological Strength Index, GSI, for the characterization of such rock masses is presented. Two GSI charts are proposed for estimating the mechanical properties of these masses, one mainly for tunnels and the second for surface excavations. An example is given to illustrate the process of tunnel design in molassic rocks.

1. Introduction

In many mountainous regions a sequence of alternations of clastic and pelitic sediments were deposited during a quiescent period after the main orogenesis. The behaviour of these deposits, known as molasses in Europe, is quite different from that of flysch, a sequence of strata of similar composition associated with the same orogenesis. Although the cases on which this discussion is based come from the molassic formation of Northern Greece, we believe that the proposed characterisation can be of general application to sedimentary rocks deposited in a quiescent shallow marine environment and not associated with significant tectonic disturbance.

2. General geological setting.

Molasse comes from a provincial Swiss name originally given to soft sandstone associated with marl and conglomerates belonging to the Miocene Tertiary period, extensively developed in the low country of Switzerland and composed of Alpine detritus. The term is now applied to all orogenic deposits of similar genesis e.g. to describe sediments produced by the erosion of mountain ranges after the final phase of an orogeny.

The molasse consist of an almost undisturbed sequence of great overall thickness of sandstones and siltstones, mudstones or marls. These rocks can alternate in layers of tens of centimetres or they can be present as massive strata (mainly the sandstones with occasional siltstone intercalations). Conglomerates occur rather commonly, forming thick bands in some cases. Rather restricted limestone horizons may also be present. Due to the facts that the sedimentation of the detritus material took place close to the sea shore line and the ongoing subsidence of the newly formed basin, an alternation of sea, lacustrine and terrestrial deposits, may characterise the molasses together with lateral transitions from one lithological type of layer to the other. A Stratigraphic column and a geologic profile of molassic formations from Greece are presented in Figures 1 and 2.

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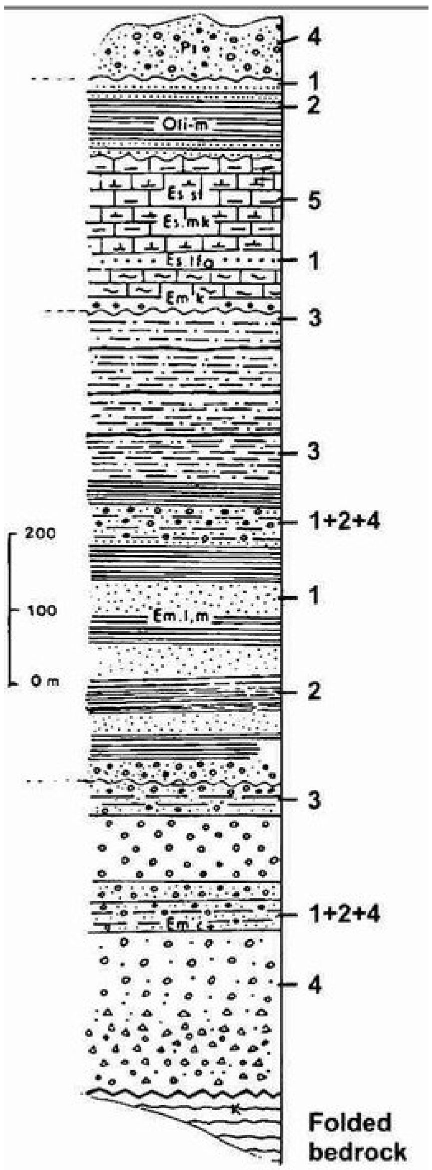


Figure 1: Schematic column of the mollassic formations in the Rhodope basin, NE Greece.

1. sandstones,
2. clay shales or siltstones,
3. sandstones with siltstones or clayey sandstones,
4. conglomerates,
5. limestones, marly limestones or marles.

(from the Geolgical map of Greece, 1:50000, IGME, 1980)

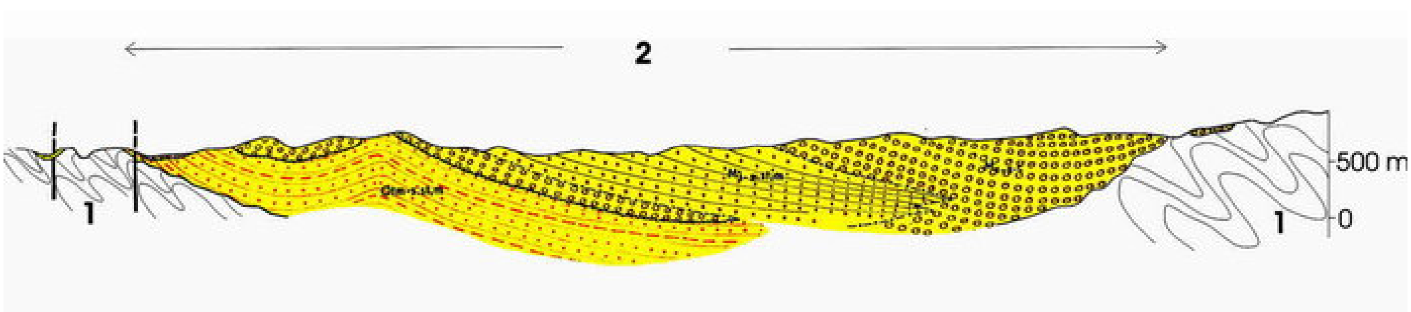


Figure 2: Geologic section in a molassic country, NW Greece (from the Geological map of Greece, sheet Ayiofillo, 1:50.000, IGME, 1979, slightly modified)

1: Bed rock of the already formed mountain belt.
2: Molassic country: alternation of sandstones, conglomerates, siltstones and marls.

In some cases sandstones are very weak and can be assimilated with sands; in such weak molasses, clays and silts are also present and the material can be treated as soil. These types of molasses are not considered in this paper.

As the molasse characterise a series of sediments that were formed and developed after the main orogenesis, they have not suffered from compression or shear. They are thus unfolded or contain mild gravity folds or flexures. Inclination of strata is generally low and cases with dips of more than 30° are infrequent or local. Gravity faults are present, as in all post-tectonic basins but their impact on the deterioration of the quality of the rock is limited. In certain ranges molassic formations may be deformed and overthrust by the final advance of tectonic nappes. Again the decrease of their quality is localised.

3. Molasse vs Flysch

In contrast to molasses, the term flysch is used to describe sediments produced early in the mountain building process by the erosion of uprising and developing fold structures. These are subsequently deformed by later stages in the development of the same fold structures. Flysch is thus produced in front of the advancing orogenesis, folded with the other strata or even overthrust by the advancing mountain belt. On the other hand, molasses in the basins behind the already formed mountain belt remain over the folded belt and are undisturbed by the mountain building process.

Flysch, in contrast to molasses, has more rhythmic and thinner alternations of sandstone and pelitic layers. These suffered strong compressional deformations which produced folds of many scales, sizeable sheared zones and weaker surfaces, primarily in the form of well developed bedding planes.

4. Lithology

The sandstones members of the molasse are often silty or marly and these exhibit low strength values. Their unconfined compression strength may be about 10 MPa if they are marly or silty and more than 50 MPa in their typical granular form. A value of 20 MPa may reasonably describe the typical unconfined compression strength of the sandstone component of the molasses in NW Greece.

The unconfined compressive strength of a typical siltstone can be about 15 MPa. However, siltstones may have a significant presence of clayey materials (mudstones) and in the case of a clayey-siltstone, mudstone or marl, the unconfined compressive strength may be in the range of 5 to 10 MPa.

All of these siltstones are very vulnerable to weathering and development of fissility parallel to the bedding when these rocks are exposed or are close to the surface. In outcrops they appear thinly layered like siltstone shales and when they alternate with sandstones, their appearance resemblances similar alternation in flysch. The weathering of outcrops shown in Figure 3 can be misleading when considering the behaviour of these molassic rocks in a confined underground environment in which the process of air slaking is restricted. This can be seen by comparing the appearance of freshly drilled core in Figure 4 with that of the same core after storage in a core shed for approximately 6 months, shown in Figure 5.

Figure 3: Surface exposure showing alternating sandstone and siltstone layers in a molassic rock mass in NW Greece.

In the freshly drilled core it is sometimes difficult to distinguish between the sandstone and siltstone components of the molasse since the core may be continuous over significant lengths. It is only after exposure that the siltstone cores start to develop a fissile appearance and, after a few months they collapse to a silty-muddy loose mass. This process, which also affects the silty-sandstones, can result in a dramatic misinterpretation of the engineering characteristics of molasses if inspection of the core is not done immediately. Similarly, testing for the unconfined compressive strength must be performed as soon as possible after drilling and, in some extreme cases, it has been found that this testing will only produce reliable results if it is done on site immediately after drilling.

Figure 4: Appearance of molassic rock core immediately after drilling. Sandstones and siltstones are present but the bedding planes (mainly of the siltstone) do not appear as defined discontinuity surfaces.

Figure 5: Appearance of the same core as shown in Figure 4 but after storage in a core shed for six months. The sandstone remains intact but the siltstones exhibit fissility followed by collapse.

Figures 6 and 7 show very similar behaviour to that described above in cores from rock from the site of the Drakensberg Pumped Storage Project in South Africa where site investigations were carried out in the early 1970s. The appearance of surface outcrops resulted in an extremely conservative assessment of the rock mass behaviour. It was only after an exploratory adit was mined and freshly drilled core was inspected and tested and that realistic excavation designs were developed.

Figure 8 shows the main powerhouse cavern of the Drakensberg Pumped Storage Scheme during construction in about 1975. Based on tests carried out on site and on the behaviour of exploration adits on the project [1], a final design was developed using tensioned and grouted rockbolts (6 m long and 25 mm diameter) and a 15 cm thick shotcrete lining. A 5 cm thick protective coating of shotcrete was applied as soon as possible to all exposed rock surfaces in order to prevent air slaking. A further 10 cm of wire-reinforced shotcrete was applied later to complete the lining. No additional lining or reinforcement was used and a suspended steel ceiling was used to catch water drips and to improve the appearance of the interior of the cavern. The system has performed without any problems for more than 25 years.

5. The application of GSI to molassic rock masses

The molasses form rock masses with dramatically different structure when they outcrop or are close to the surface as compared to those confined in depth. This means that care has to be exercised in the use of the Geological Strength Index (GSI) charts for assessment of rock mass properties.

In the undisturbed in situ rock mass encountered in tunnelling, the rock mass is generally continuous as illustrated in the freshly drilled core photographs described above. Even when lithological variation is present the bedding planes do not appear as clearly defined discontinuity surfaces. They are taken into account by the intact strength σ_{ci} of the mass. In such cases the use of the GSI chart for blocky rock [2, 3] reproduced in Figure 9, is recommended and the zone designated M1 is applicable. The fractures and other joints that are present, given the history of the formation, are generally not numerous and the rock mass should be assigned a GSI value of 50 to 60 or more. Due to the benign geological history it is even expected that the molasses will exhibit very few or no discontinuities in several stretches of the tunnels. In these cases GSI values are very high and indeed the rock mass can be treated as intact with engineering parameters given by direct laboratory testing.

Figure 6: Freshly drilled sandstones and siltstones from the Drakensberg Pumped Storage Project in South Africa (1972).

Figure 7: Similar core to that shown in Figure 6 but after storage for 6 months.

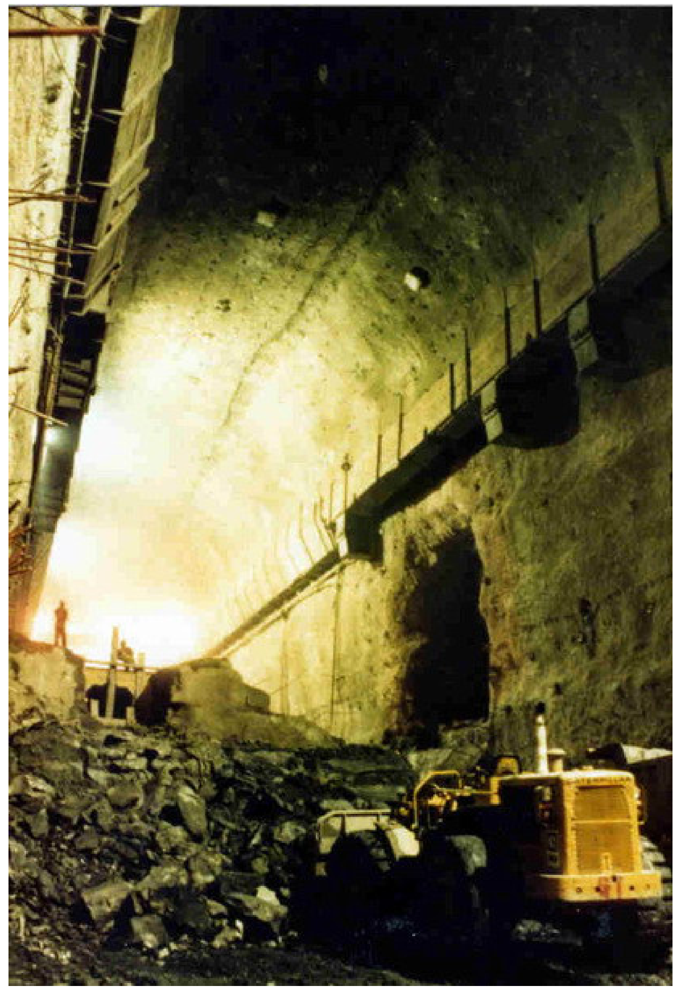


Figure 8: The 17 m span, 32 m high underground powerhouse of the Drakensberg Pumped Storage in South Africa. This cavern was excavated in the undisturbed sedimentary rock mass illustrated in Figure 6 and it was supported using rockbolts and shotcrete only. A 5 cm thick layer was applied immediately to all exposed rock surfaces and this was followed later by a layer of wire mesh and an additional 10 cm of shotcrete. The project has recently completed 25 years of trouble-free operation. Note: The extreme degradation of the Drakensberg rocks was partly because some of the units were tuffaceous

When fault zones are encountered in tunnelling through these molassic rocks, the rock mass may be heavily broken and brecciated but it will not have been subjected to air slaking. Hence the blocky rock GSI chart given in Figure 9 can be used but the GSI value will lie in the range of 25 to 40 as shown by the area designated M2.

In outcrops the heterogeneity of the formation is discernible and similarities exist with the structure of some types of flysch. Hence the GSI chart for heterogeneous rock masses such as flysch [4, 5] can be used with the exclusion of sheared and deformed types and with a slight shifting to the left of the flysch chart categories. This version of the chart, for use with fissile molassic rocks, is presented in Figure 10. The M3 to M7 designations in Figure 10 are largely self-explanatory. However, the user should read the descriptions in both rows and columns carefully and should not rely only on the pictures in choosing GSI values.

6. Estimates of the mechanical properties of molassic rock masses

For massive units of sandstone or siltstone, where no significant bedding planes or discontinuities are present, the rock mass should be treated as intact and the design values for strength and deformation modulus should be taken directly from laboratory tests. Note that these tests have to be performed very carefully in order to obtain reliable results. As mentioned earlier, some of the siltstone units can break down very quickly on exposure and it is essential to test than as soon after recovery of the core as possible. In some cases, testing in the field using portable equipment has been necessary in order to obtain reliable results.

The use of point load tests is not recommended for these low strength materials since the penetration of the loading points can invalidate the results. Compression testing should always be carried out normal to the bedding direction and the results from specimens in which the failure is controlled by structural features should be rejected. A first estimate of the deformation modulus for these massive rock units can be obtained from $E \approx 200\sigma_{ci}$ (all units in MPa).

For molassic rock masses in which significant bedding planes or discontinuities are present the charts presented in Figures 9 and 10 can be used to estimate the GSI values which can then be used to downgrade the strength of the intact rock in accordance with the Hoek-Brown criterion.

7. Brittle failure in massive molassic units

Research over many years, dating back to the pioneering work on the fracture of glass aircraft windshields by Griffith [6, 7], has established that brittle fracture in massive rocks is associated with propagation of tensile cracks which originate at defects such as grain boundaries in the material. These cracks propagate parallel to the major principal stress direction and their length is controlled by the ratio of minor to major principal stresses at the point under consideration. At the excavated boundary of an underground excavation the minor principal stress is zero and hence these tensile cracks propagate parallel to the boundary forming the slabs and spalls. Recent thinking on brittle fracture in hard massive rocks has been summarized by Kaiser et al [8] and by Diederichs [9].

Since the tensile crack propagation described above does not mobilize any frictional forces within the rock mass, the Mohr Coulomb criterion for the initiation of these cracks can be expressed in terms of cohesive strength only, with the friction angle set to zero. Laboratory tests and back analyses of the extent brittle failure in underground excavations show that the appropriate cohesive strength is approximately equal to one third of the uniaxial compressive strength of the intact rock, i.e. $c \approx 0.33\sigma_{ci}$, $\phi = 0$. The broken material that remains within the failure zone surrounding an underground excavation can be characterized as a highly frictional, cohesionless rock mass, i.e. $c = 0$, $\phi \approx 33°$.

This failure process has been used in modelling an unsupported tunnel in molasse and the results are shown in Figure 11. The properties of the sandstone and siltstone layers in this model are as follows:

Sandstone: Intact rock: $c = 7$ MPa, $\phi = 0$,
$E = 4000$ MPa ($\sigma_{ci} = 20$ MPa)
Residual strength: $c = 0$, $\phi = 35°$, dilation angle 5°.
Siltstone: Intact rock: $c = 3$ MPa, $\phi = 0$,
$E = 1800$ MPa ($\sigma_{ci} = 9$ MPa)
Residual strength: $c = 0$, $\phi = 25°$, dilation angle 5°.

Figure 9: GSI chart for confined molasse (mainly applicable for tunnels).

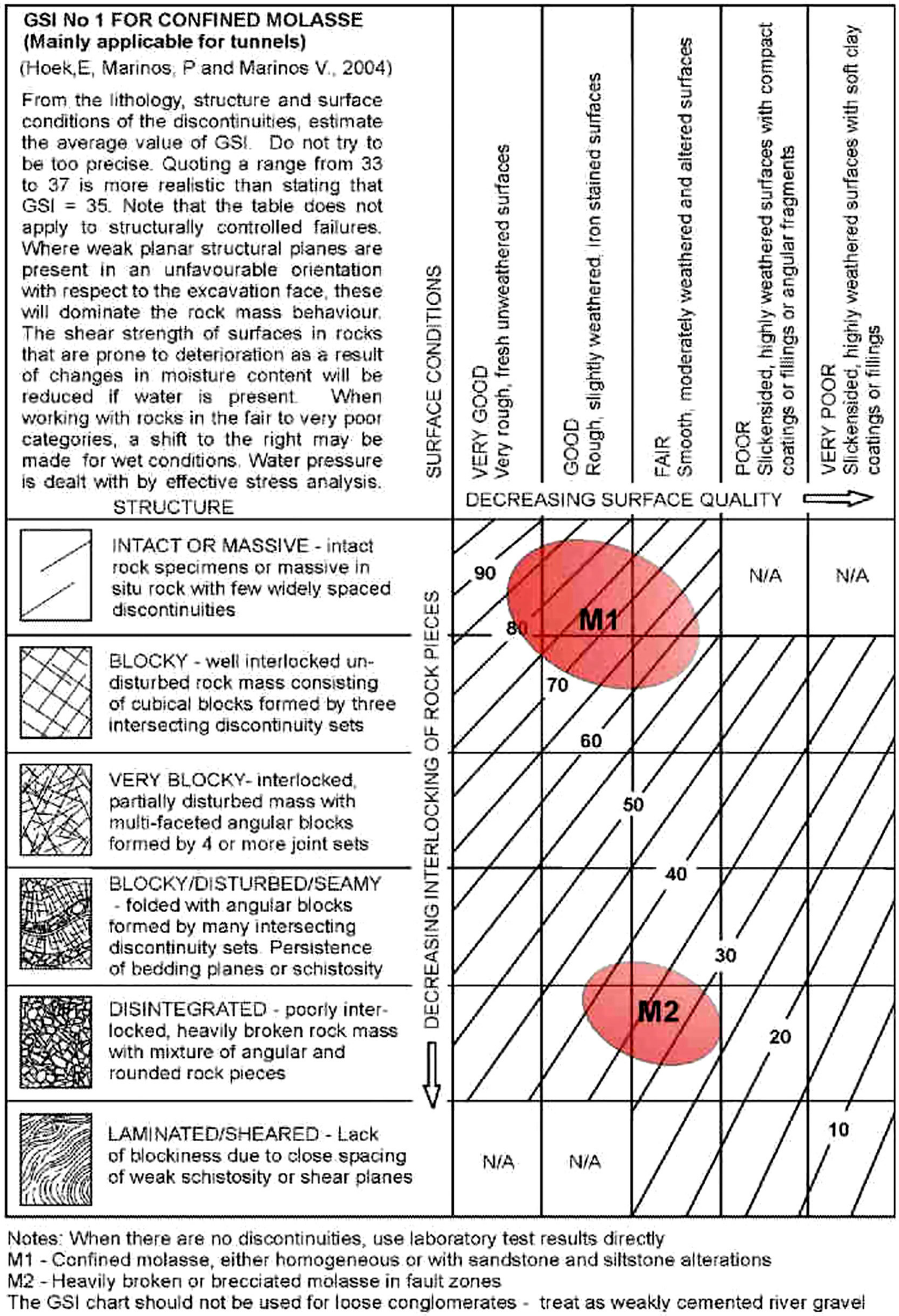
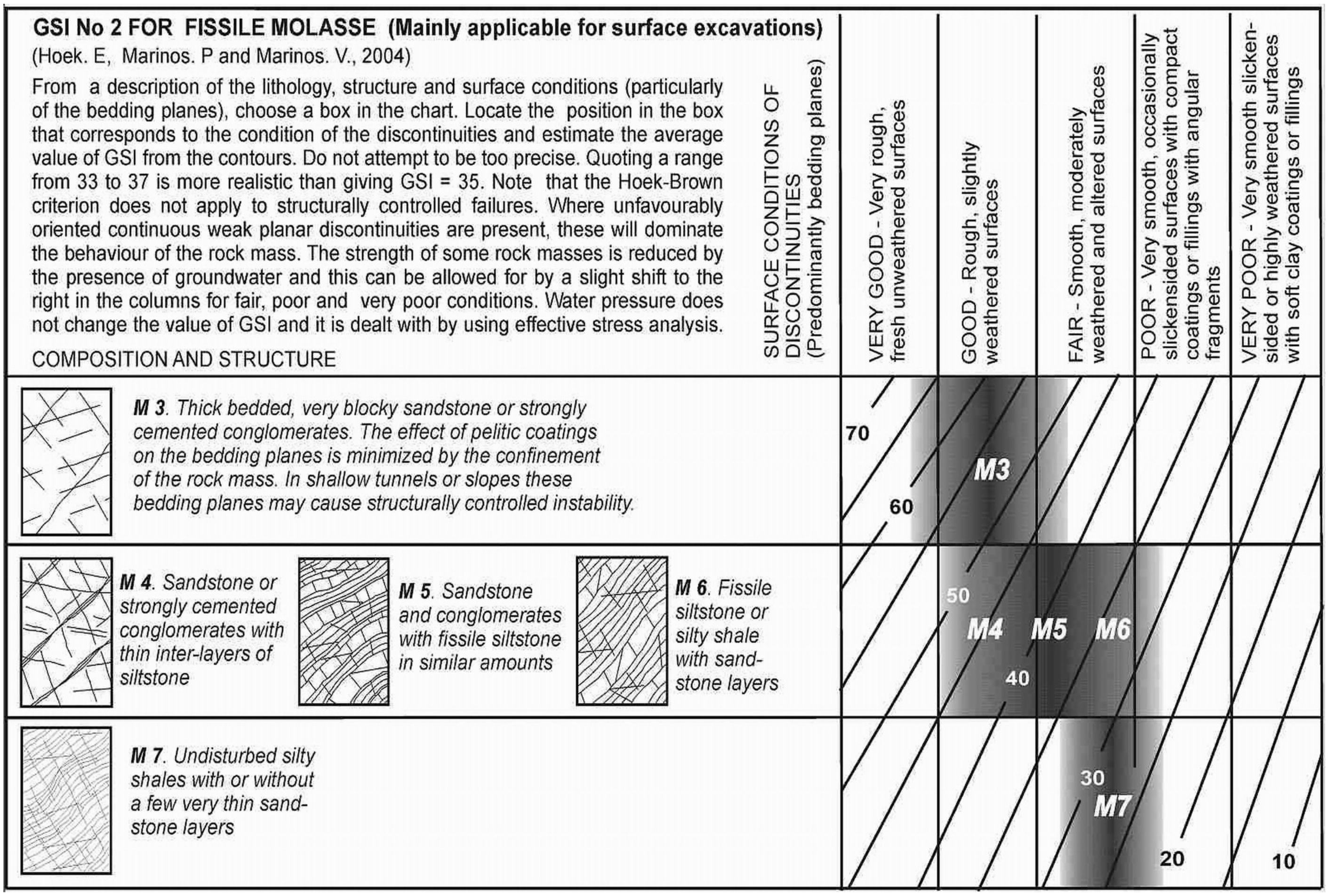


Figure 10: GSI chart for fissile Molasse where bedding planes of siltstones-mudstones are frequent and well defined. (Surface excavations and slopes)



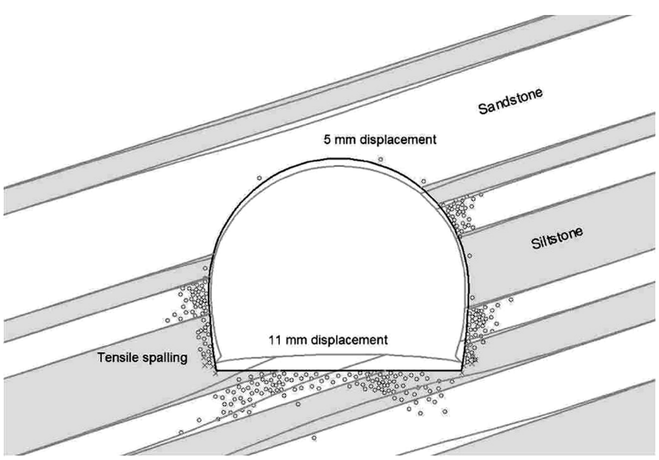


Figure 11: Tensile failure in sandstone and siltstone molasse surrounding an unsupported tunnel.

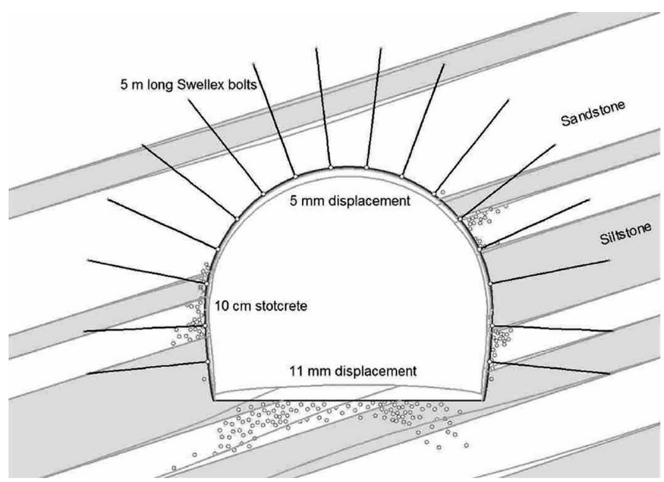


Figure 12: Typical support in molassic rock masses under moderate stress levels.

Zero tensile strength was assumed for all cases. The depth of cover over the tunnel is assumed to be 100 m and the ratio of horizontal to vertical stress $k = 0.5$.

Note that the failure process is almost entirely tensile (denoted by the **o** symbol in Figures 11 and 12) and the propagation of the failure is quite limited and concentrated largely in the sandstone layers which are "dragged" by the softer siltstones. The deformations are also small as would be expected for the relatively high deformation modulus and the modest in situ stress level.

Sensitivity studies showed that the distribution and extent of failure is quite sensitive to the ratio of horizontal to vertical stress. This suggests that, where no in situ stress measurements are available, the designer should check the design for both $k = 0.5$ and $k = 2$ which can be considered reasonable lower and upper bounds for the molassic rock masses under consideration.

The influence of jointing was also checked and found to be not very significant on the results shown in Figure 11. This is because the joints are tensile failures created by differential strains in the sandstone and siltstone layers. Consequently, their surfaces are rough and they exhibit high frictional strength. Obviously there are situations in which the creation of a free surface by the excavation of the tunnel can combine with joints and bedding planes to release blocks and wedges that will fall under gravity. Predicting the location and size of these failures is difficult and, where they are or concern, it is prudent to use pattern rockbolting to stabilize the tunnel roof and walls.

Figure 12 shows the results of an analysis of the same tunnel shown in Figure 11 except that a pattern of 5 m long Swellex rockbolts and a 10 cm layer of shotcrete have been added. It can be seen that, apart from a reduction of spalling and deformation in the lower sidewalls of the tunnel, the support system does not have a dramatic impact upon the behaviour of the tunnel. However, this support plays the following critical roles:

1. The application of a 3 to 5 cm thick layer of shotcrete to exposed rock faces as soon as possible (typically at the and of each excavation round) provides sealing and protection of the siltstone layers against air slaking.
2. The pattern of rockbolts reinforces the rock mass by maintaining the confinement and preventing gravity falls of loose structurally defined blocks or wedges or falls due to decompression of the sealing of bedding planes.
3. The addition of a second layer of shotcrete, with either wire mesh or fibre reinforcement, forms a bridging shell between rockbolts and prevents progressive ravelling from falling of small "key blocks" from the surface of the excavation.

4. Support design for discontinuous, broken and weak molassic rocks

For broken and weak molassic rocks in the vicinity of faults (M2 in Figure 9) or in the cases where discontinuous weak masses occur (M3 to M7 in Figure 10), there is clearly a need to provide heavier support than that shown in Figure 12. In addition, stabilization of the face may be required in order to prevent progressive ravelling and chimney formation. A typical primary support design is illustrated in Figure 13 and this closely resembles the design used in tunnels in flysch in many Alpine highway projects. An important difference is the application of a protective layer of shotcrete to exposed molassic rock surfaces to prevent air slacking and the resultant deterioration of the rock mass.

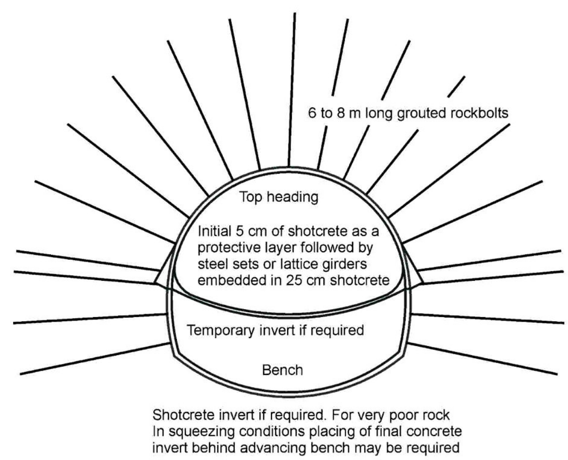


Figure 13: Typical primary support design for broken molassic rocks with frequent and well defined discontinuities, mainly bedding planes. A final concrete lining (not shown) is placed later.

The treatment of the invert depends upon the rock mass characteristics and in situ stress levels. If heavy squeezing conditions are anticipated [4, 10] consideration can be given to the installation of the final concrete invert as close as practicable to the bench face. Where the cover is relatively modest it may be possible to proceed without an invert in the top heading and a relative light final invert (mainly for trafficability).

8. Conclusions

Extreme care has to be taken when classifying tectonically undisturbed sedimentary rock masses with lithological variation formed in a quiescent depositional environment. Although the cases on which this paper are based come from molassic formations, we believe that the proposed characterization can be of general application for this type of undisturbed geologic formation.

For massive units of sandstone or siltstone, where no significant bedding planes or discontinuities are present, the rock mass should be treated as intact and the design values for strength and deformation modulus should be taken directly from laboratory tests.

For rock masses in which bedding planes or discontinuities are present the GSI charts can be used to estimate the values to be used to downgrade the strength of the intact rock in accordance with the Hoek-Brown criterion. The use of the program RocLab[1] is recommended for the estimation of rock mass properties. For the confined conditions encountered in tunnels, these rock masses are generally continuous with few discontinuities and zone M1 in the the basic GSI chart for blocky rock given in Figure 9. Zone M2 in this chart corresponds to broken and brecciated masses as a result of faulting. Typical GSI values of 60 to 70 can be anticipated in the first case and values of 30 to 40 in the second.

In such massive rocks under confined conditions, brittle failure is considered to be the most likely failure mode and this results in spalling and slabbing of tunnel boundaries. Laboratory tests and back analyses of the extent brittle failure in underground excavations show that the appropriate cohesive strength is approximately equal to one third of the uniaxial compressive strength of the intact rock ($c \approx 0.33\sigma_{ci}$) and the friction angle is zero ($\phi = 0$) since the failure process is predominantly tensile and no shear is mobilized. Simple tunnelling conditions and good advance rates can be anticipated for confined masses which can be treated as intact rock or classified as M1 in Figure 9.

An example of the analysis of tunnel behaviour in these rocks is presented in Figures 11 and 12 and a typical primary support design for a 12 m span tunnel excavated by top heading and benching is given in Figure 13.

For surface excavations such as portals and cuts, where air slaking occurs as a result of the exposure of the rock mass, the use of a new GSI chart is recommended. This chart, presented in Figure 10, is derived from a GSI chart for heterogeneous rocks such as flysch with the elimination of the deformation and sheared features that govern the behaviour of flysch.

In surface excavations or where faulting and brecciation have disrupted the rock mass, more conventional rock mass failure characteristics, defined by the Hoek-Brown failure criterion are appropriate.

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