Effects of Rock Mass Deformation on Tunnel Performance in Seismic Regions

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Résumé — On a observé que les mouvements sismiques augmentent l'écoulement de l'eau dans les équipements souterrains. L'augmentation apparente de la perméabilité de l'ensemble est problablement causée par les glissements localisés des joints; glissements qui résulteraient en des changements, petits mais irréversibles, dans l'ouverture de ces joints. De tels changements peuvent être observés physiquement et numériquement dans les excavations dans les joints rocheux. On décrit des méthodes pour mesurer les propriétés des joints, pour rendre l'orientation des tunnels la meilleure et pour prévoir les supports nécessaires. Une attention particulière est portée à la relation entre le glissement des joints l'un sur l'autre et leur élargissement (dilatation) et leur effet sur la conductivité et sur l'agencement optimum pour les boulons flexibles utilisés pour renforcer le tunnel.

1. Introduction

Underground structures have a consistent record of suffering much less damage than surface facilities during earthquakes. Generally only portal areas or fault crossings have suffered severe damage. In the case of portals, the combination of poor ground, stiff linings and amplified near surface shaking, make earthquake resistant design very difficult. In the case of fault crossings, associated block motion may be irresistable by any method of support or rock mass reinforcement.

The reduced intensity of shaking experienced at depth and in more competent rock masses, appear to limit damage to occasional rock drops and to cracking of linings. These events may be the result of out-of-phase high frequency shaking, reactivation of joint slip, or positive or negative stress changes adversely affecting existing high or low stress conditions.

In each of these cases the net result may be partially irreversible strain, due to the hysteretic behaviour of jointed rock masses. As suggested above, the impact on stability may be minimal, but the secondary effect on coupled processes such as water inflow or leakage may be marked. Seemingly minor joint displacements can cause radical changes in conductivity.

The present international interest in geological disposal of high level nuclear waste has focussed particular attention on transport velocities through jointed media. Since migration of radionuclides via ground water flow is the only conceivable mechanism for release to the biosphere, any events that could cause radical changes in flow velocities are of potential concern. Reports describing mine flooding and cracking of linings as a result of earthquakes are indications of a potential problem that may have increased impact on design in the future.

1.1 *Influence of depth*

There are several reasons why earthquakes generally result in less damage underground than at the surface. The predominant surface (Rayleigh) waves decay almost exponentially with depth, and below the surface incident and reflected waves interfere so that the total amplitude is usually reduced. A general tendency for increasing modulus with depth, small excavation dimensions relative to the predominant wavelengths, and the general wave de-amplification with depth all contribute to the reduced damage.

Kanai and Tanaka [1] measured ratios of surface displacement to displacement at 300 m depth as high as 6 in the Hatachi copper mine, and up to 10 when surface data from an alluvium site was included. Similar trends are also indicated when monitoring the effects of underground nuclear explosions. Vortman and Long [2] showed mean peak vectors of acceleration, velocity and displacement that ranged in general from 2.5 to 1.9 times larger at the surface than at 500-m depth.

The relative mismatch of wavelengths and most tunnel dimensions suggests that relative strain between rock blocks can only occur with higher frequency waves, when out-of-phase motion is possible across the structure. Dowding [3] suggested that large accelerations at frequencies in the range 30–60 Hz are probably most capable of causing differential block motion and resulting damage in large excavations.

The location of large caverns at shallow depth may be particularly adverse in terms of seismic design. Stevens [4] refers to the case of a large near-surface stope 45 m below the surface at the Tombstone mine in Arizona, which suffered considerable loosening and rockfalls from the hanging wall during the severe jolting, which was witnessed by mine surveyors.

Although many references are made to more damaging effects of shaking at the surface than at depth in the same mines, there are a limited number of cases in which this trend is reversed, with larger displacements observed at depth than at the surface

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[4]. Such occurrences may be due to special geological structures or due to anomalies.

Richter [5] observed that isoseismals drawn from adequate data were rarely circular, and often showed elliptical elongation in the direction of the major structural trends. Regional and locally persistent jointing may have the effect of accentuating the shaking due to the pronounced mismatch of shear and normal stiffness typical of rock joints [6].

1.2. *Water inflow observations*

A number of references to flooding or increased flows of water into mines as a result of earthquakes are given in the literature. There are also occasional references to greatly diminished flows. Unfortunately, details are seldom given as to the exact cause of the flooding; whether one or several levels or an entire mine were subject to flooding. A tabulation of earthquake effects on tunnels and mines given by McClure [7] provides comments such as: "mine filled with water", "mine was flooded", "existing fractures were opened wider causing an increase in water influx and almost flooding mine". Two of these cases were in California, one was in Chile.

Stevens [4] suggested that in such cases the earthquakes may have resulted in renewed movement along existing fractures, or that fracturing resulted from the earthquake and provided new avenues for water inflow into the mines.

A recent earthquake in Idaho (Nov. 2, 1983) registering 6.9 on the Richter scale caused damage to hundreds of buildings and two fatalities. It also caused a 250% increase of water flow into the 1100-ft deep Clayton silver mine. The mine is located 25 miles west of the epicenter. Flow increased immediately from 1000 to 2500 gpm, but has declined over a six month period to about 1500 gpm [8]. Flow rates and pressures reportedly increased in numerous locations in the 800- and 1100-ft levels, while the 500-ft level produced water for the first time in several years. Major jointing in the local quartzite and dolomite strikes approximately N–S, and dips at about 60°. Inflowing water remained clear following the earthquake.

The above-case is an example of joint conductivity enhancement, rather than fault displacement effects. Furthermore, dynamic stress cycling that occurs only perpendicular to the joints is unlikely to cause significant increases or decreases in aperture and conductivity if the joint is already under significant levels of effective normal stress. The essential permanent change in aperture has to have been caused by shear-induced dilation across non-planar joint surfaces. Reversed shear and contraction on subsequent cycles of shaking will be inhibited if a significant level of differential stress already exists. The gradually reduced flows observed in the Clayton silver mine are likely a function of local drawdown of the ground water table, due to the increased permeability of the rock mass.

1.3. *Potential problems from increased conductivity*

Cracking of concrete or shotcrete linings and associated rock mass deformation can have several adverse effects if water transport, or the avoidance of water inflow are important factors in the tunnel design. Cracking and rock mass deformation would adversely affect both the tunnels illustrated in Fig. 1.

The water supply tunnel illustrated in the top diagram is particularly susceptible to earthquake damage if it has a thick unreinforced concrete lining, and is driven above the water table in a weak, deformable rock mass. A stiff concrete lining will attract high stresses when located in low modulus ground. The near-surface location will tend to amplify the shaking, and cracking is most likely to occur where conductivity changes are greatest. Serious leakage is possible.

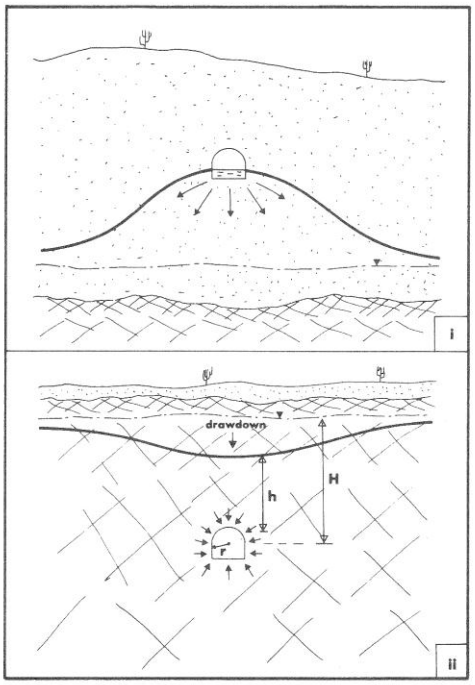

Figure 1. Potential problems of water leakage and increased inflow.

In the second example in Fig. 1, an increase in conductivity due to joint slip around the tunnel would likely result in increased drawdown. In the case of tunnels driven in urban areas, differential settlements in any overlying clay could cause damage to buildings. Such an effect might appear after buildings had been repaired or rebuilt following the earthquake. Resistance to aftershocks would be reduced.

In the case of a nuclear waste repository, increased conductivity surrounding the disposal rooms could be positive or negative, depending on the depth relative to the ground water table. The proposed siting in jointed tuff at the Nevada Test Site is above the present water table, in partially saturated rock. Increased conductivity would reduce the contact time with water, and therefore presumably be beneficial, provided that radical changes in the regional hydrology do not occur in the foreseeable future.

Nuclear waste repositories located below the water table would obviously be adversely affected by increased conductivity, especially if slip occurred along major through-going regional joints. If flow gradients remained constant the transport velocities would be proportional to the square of the theoretical smooth-wall conducting aperture (e) in those parts of the flow path that followed joints. The transport velocity is actually proportional to an even higher power of aperture, when the real mechanical aperture (E) is considered, due to friction losses caused by surface roughness and areas in contact [9]. Shear-induced dilation causes a significant increase in the real mechanical aperture of a joint, and can result in up to three orders of magnitude increase in conductivity.

2. Joint Slip from Dynamic Loading of Models

The possibility that increases in water inflow into a mine may be the result of "renewed movement along existing fractures" [4] is supported by the results of both physical and numerical models. An important prerequisite for this mechanism of conductivity enhancement is the application of a high ratio of horizontal to vertical stress. This has been shown to result in some joint slip during the excavation phase, before either thermal loading (from nuclear waste) or dynamic loading are superimposed.

2.1. *Example from physical models*

Two-dimensional, plane stress models of large near-surface excavations were performed in a joint Norwegian–Swedish study of underground siting for nuclear power plants [10]. The models consisted of a weak, brittle rock simulant, which was divided into 20,000 discrete blocks by two intersecting sets of tension fractures. Four different fracture (joint) orientations were studied, and models of single caverns, two parallel caverns, and four closely spaced caverns were tested.

Physical excavation of the caverns took place after the model rock masses had been fully consolidated with four load-unload cycles, and while they were under either hydrostatic or high horizontal stress. The models were dynamically loaded after excavation, while still under stress. Deformation vectors were generated by photogrammetric analysis in a stereocomparator linked to a computer and plotter.

The dynamic response of the model rock masses to simulated earthquake loading was recorded by two miniature accelerometers. These were buried at simulated depths of approx. 20 m below the surface. Peak horizontal components of motion, which were 4–5 times the vertical, were as follows at prototype scale:

$a = 0.2 - 0.7$ g, $v = 15 - 90$ cm sec^{-1},

$s = 6 - 30$ cm, $t = 0.4 - 12$ Hz

duration = 1 min.

The deformation occurring during this dynamic loading was dependent on three factors: the orientation of the joints, the ratio of horizontal to vertical stress, and the depth below surface. Progressive collapse of walls and pillars occurred in the models with unfavourable, steeply dipping joints. None of the excavations were supported or reinforced in any way.

Figure 2 illustrates the deformation vectors measured during excavation of simulated 25-m span caverns. The width of field shown in the figure is less than one sixth of the whole model; boundaries were distant. The shallow-dipping jointing (shown at the correct relative scale) in combination with the high horizontal stress resulted in slip on one of the joints that intersected the roof of the left-hand cavern. The order in which excavation was performed is shown by the numbers 1–4.

Slip could not be detected after stage 1, but it could be detected after stage 2, and it then increased progressively through to stage 4. The three circles illustrate the vectors measured at reference points straddling the slipping joint. While the major deformation vectors are directed towards the openings, a slip component of at least 50 mm is indicated at prototype scale.

In Fig. 3 the net effect of dynamic loading is shown, with each group of four vectors drawn from the mean position. Settlement, increased deformation towards the openings, and additional joint slip are evident. The divergence of the vectors shown circled indicates additional relative slip of 40–80 mm (full scale) as a direct result of the simulated earthquake loading. This would be sufficient to cause major additional increases in conductivity.

Several other features are illustrated by these physical models. While there was settlement above the openings, little was evident below them. Deformation tended to occur upwards towards the excavations, despite the 150 m of simulated rock mass beneath them. In general the zone around the openings that was sensitive to dynamic loading was more or less the same zone that suffered deformation during excavation.

Cyclic compressive loading tests performed on jointed models by Brown and Hudson [11], led these authors to conclude that rock masses were unlikely to be susceptible to fatigue when loaded at low ratios of stress to strength. In the models

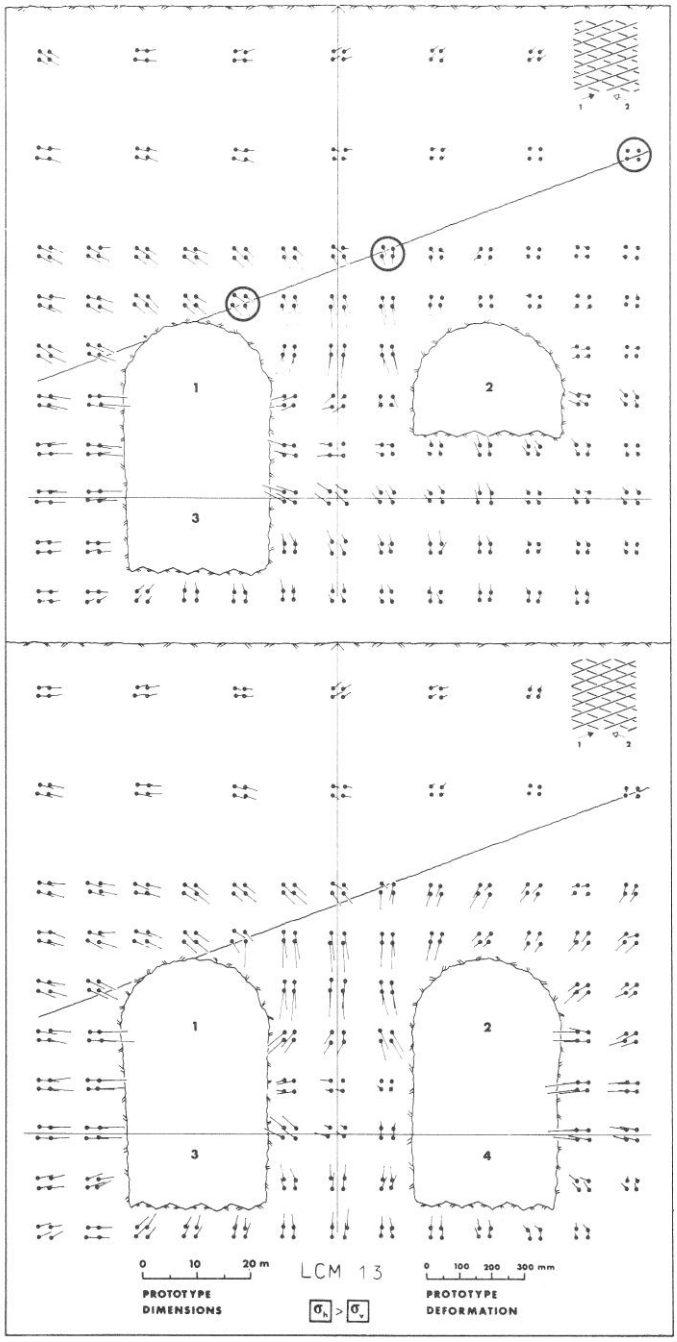

Figure 2. Progressive joint slip caused by excavation in a high horizontal stress field [10].

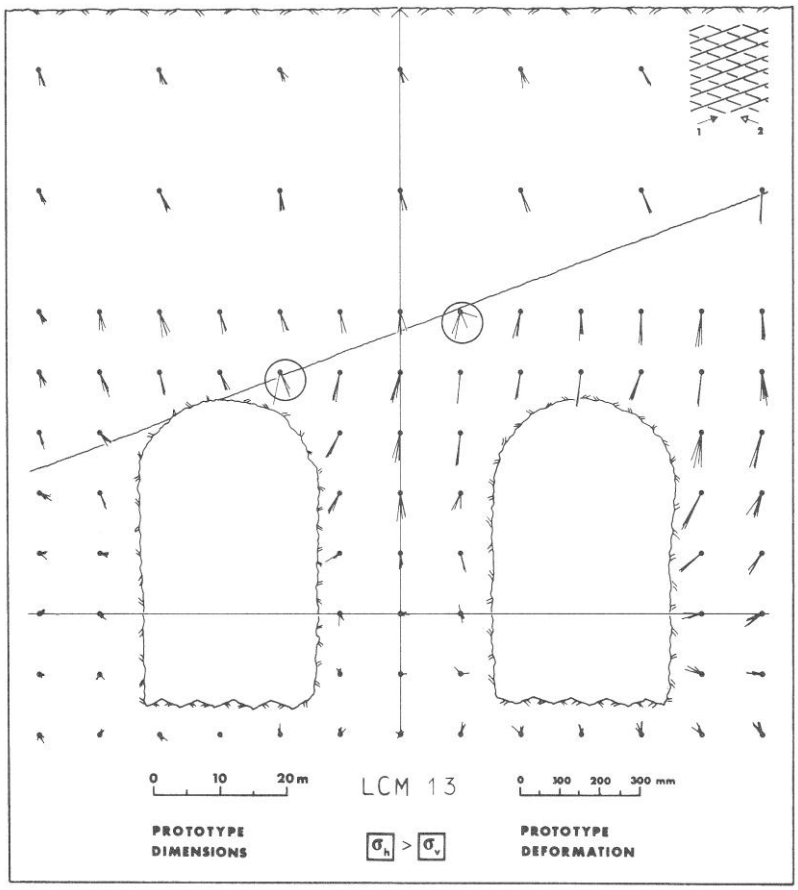

Figure 3. Net deformation caused by simulated earthquake loading.

illustrated in Fig 2–4 the joints were under quite high ratios of shear stress to shear strength due to high initial stress anisotropy and subsequent excavation. They were therefore susceptible to irreversible, progressive accumulation of slip after relatively few cycles. Figure 4 demonstrates how block falls develop progressively during shaking. These events do not occur immediately shaking begins.

2.2. *Example from numerical models*

Wahi *et al.* [12] performed an extensive series of numerical simulations of earthquake effects on tunnels, during generic studies of nuclear waste repositories. A Stealth code was used to simulate excavation, heating and dynamic loading effects. The non-linear behaviour of salt, jointed granite and jointed shale were simulated. The effect of joints was modelled by adjusting the stresses in the directions of the joints. Slip and dilation were simulated in an approximate manner.

In the runs with high horizontal stress (horizontal/vertical = 2.0) major slips occurred on the 45° dipping joints during the simulation of excavation. Dynamic loading exaggerated the slip. Some slip also occurred during the simulation of heating, and cracking of the intact rock around the openings was also indicated. When the Oroville earthquake aftershock was simulated (a = 0.41 g horizontal, 0.25 g vertical) in a hydrostatically loaded model, some additional slip occurred on joints that had slipped during the simulation of heating in the granite.

Joint openings of up to 0.6–0.8 mm due to shear-induced dilation were registered in some of the runs. The maximum joint slips ranged from approx. 1–30 mm on different runs. The average dilatancies in the vicinity of the simulated tunnels in jointed granite ranged from 0.16 to 0.4 mm. In most of the models joint slip and dilation did not occur away from the tunnels. However, high horizontal stress caused slip and dilation to occur throughout the mesh.

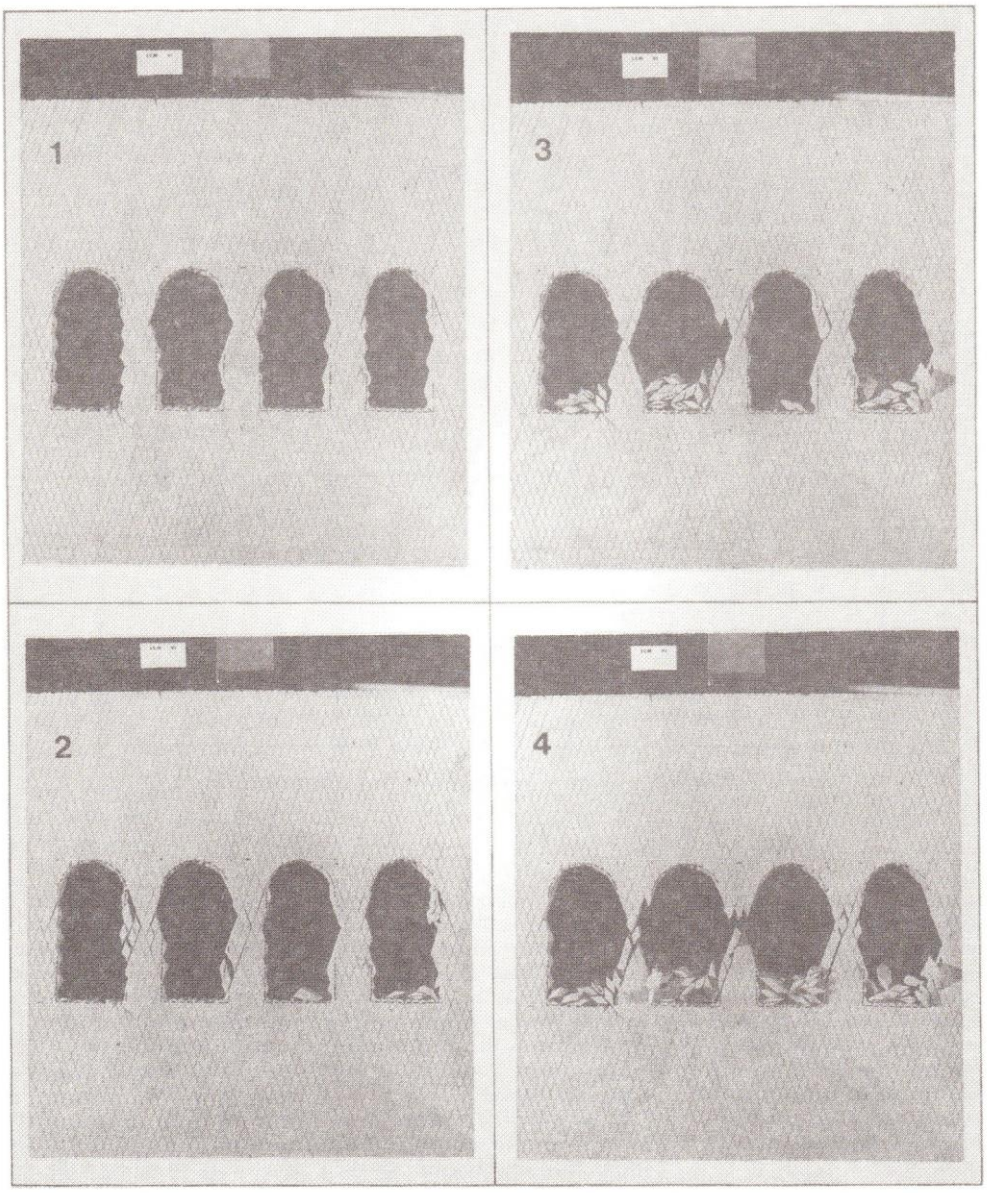

Figure 4. Progressive collapse of walls and pillars when jointing is unfavourable.

3. Seismic Design Strategies

From the foregoing review of real and simulated seismic effects on tunnels, it appears feasible to suggest some general design strategies. It appears obvious that locations having high ratios of principal stress in combination with obliquely-dipping persistent jointing, will be least favourable, due to the likelihood of high shear stress components and potentially low shear strength. This will depend on the degree of joint roughness and on the persistence.

A high virgin level of shear stress, locally accentuated by excavation, would provide the unwanted driving force for progressive, irreversible accumulation of shear displacement during seismic shaking. If such sites are unavoidable, but cavern orientation can be varied, then a perpendicular between the excavation axis and the strike of these persistent joints would be advisable both from stability and conductivity-enhancement considerations. Optimistically this orientation would also place the principal horizontal stress perpendicular to the excavation axes, if as is often the case, the persistent jointing was parallel to the principal stress direction.

Avoidance of large, plane excavation walls parallel to the principal horizontal stress direction is particularly important in the case of high level nuclear waste repositories, due to the likely high levels of virgin stress (depth of burial constraints) and due to the significant stress rise that will occur as thermal loading increases.

Maximum *in situ* stresses as high as 60 MPa, and thermal stress concentrations as high as 150 MPa are currently being considered at one of the planned nuclear waste repositories in the U.S.A. While this sounds very high, the rock concerned also has very high strength. The above-figures can be put in perspective by the mining-induced stress concentrations of up to 1000 MPa occurring in some South African gold mines at depths of 3 km or more [4]. Mining-induced seismicity (rock bursts) resulting from faulting events in previously intact rock have caused numerous instances of slip, but at these very high stress levels dilation is effectively suppressed.

3.1. *Estimating potential changes in conductivity*

The complexity of fully-dimensional seismic loading and response in a jointed rock mass will probably always prevent accurate analysis of the resulting deformation. Nevertheless, the simplified scenarios modelled by Wahi *et al.* [12] and others, provide useful data for improving design decisions.

It will be recalled that the Wahi *et al.* [12] models predicted joint openings of up to 0.6–0.8 mm, maximum slips ranging from 1 to 30 mm, and mean joint aperture changes in the vicinity of the tunnels ranging from 0.16 to 0.4 mm. In all these two-dimensional models the joints strike in the most unfavourable direction, parallel to the tunnel axes.

A recently developed constitutive model for rock joints that couples joint conductivity with joint deformation [9] provides insight into the potential magnitude of conductivity-enhancement with joint slip. The model utilizes the following input data:

JRC_0 joint roughness coefficient;
JCS_0 joint wall compression strength;
σ_c unconfined compression strength;
ϕ_r residual friction angle;
e_0 initial conducting aperture;
L_n/L_0 ratio of natural block size to laboratory sample size.

The model generates full-scale values of JRC_n and JCS_n, calculates shear stress–displacement and dilation–displacement behaviour (at full-scale) and normal stress–closure behaviour (for multiple cycles of loading and unloading). Changes of mechanical aperture (E) caused by closure, opening or shear-induced dilation are converted to changes in conducting aperture (e), using the empirical relationship shown in Fig. 5. As indicated, the smaller the aperture the greater is the divergence of (e) and (E), unless the joint is very smooth (low JRC). Conducting apertures are coverted to conductivities using the well known relationship:

$$k = e^2/12.$$

Examples of stress–closure–conductivity modelling are shown in Fig. 6. The joint chosen for this example is moderately rough ($JRC_0 = 10$) and moderately weathered or altered ($\sigma_c/JCS_0 = 2$). The four cycles of loading effectively consolidate the joint to *in situ,* undisturbed conditions. The apertures and conductivities found on the fourth load cycle are used as initial conditions for hypothetical shearing events under effective normal stress levels of 6 and 24 MPa. (The treatment of reversals during dynamic loading is not considered here. Reversed shear is treated by Barton [13].

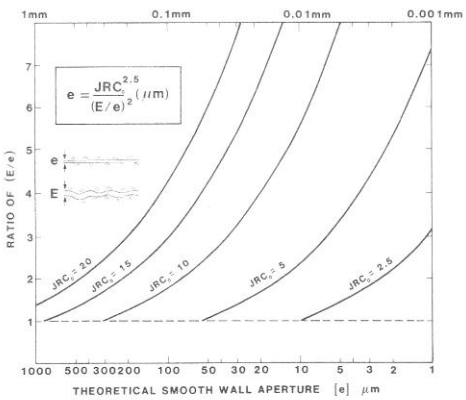

Figure 5. Empirical relationship between mechanical and conducting aperture [9].

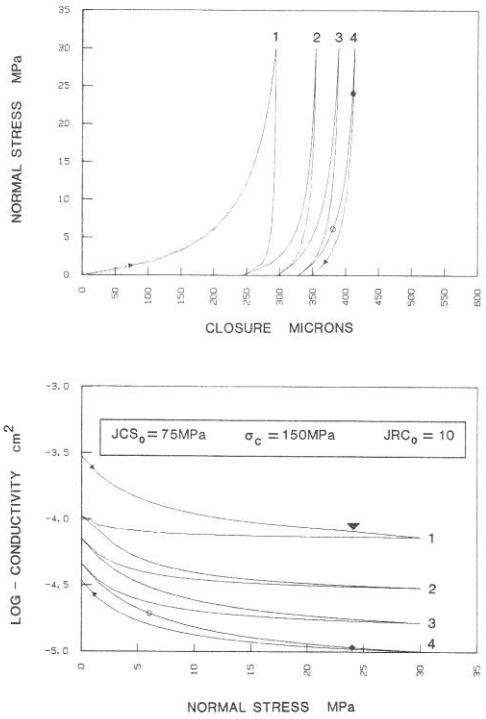

Figure 6. Examples of normal stress-closure-conductivity modelling [9].

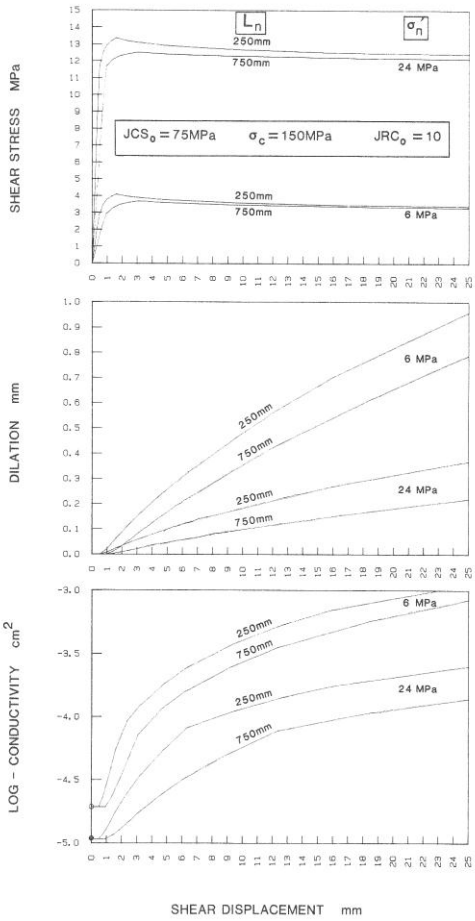

Figure 7. Examples of shear stress-displacement-dilation-conductivity modelling [9].

Figure 7 illustrates the principal results of shearing. Two hypothetical natural block sizes of 250 and 750 mm were assumed, to demonstrate the effect of scale. Several interesting aspects of behaviour are illustrated. Slip magnitudes of only 1 mm will mobilize the majority of available shear strength but will not be sufficient to cause dilation and conductivity changes. In other words a rock reinforcement system that is successful in limiting joint displacements to the range 0–1 mm, will optimize stability and minimize conductivity changes. A flexible lining such as mesh or fibre reinforced shotcrete might also tolerate such displacements without cracking. Leakage or inflow problems would probably not develop at these levels of shear displacement.

Once this threshold of 0–1 mm displacement is passed, shotcrete and concrete linings will tend to crack, shear and dilate, and the conductivity of the fractures behind the cracks may increase dramatically, probably by an order of magnitude in the first 10 mm of shear.

3.2. *General tunnel support strategies*

The natural tendency to use thicker and more rigid linings in poorer ground may actually be contrary to optimum seismic design practice. Since a tunnel lining will experience essentially the same strains as the surrounding rock mass, a tough flexible "skin" such as fibre reinforced shotcrete will tend to be less damaged than a thicker concrete lining, provided that the surrounding ground is adequately reinforced with rock bolts and grout-injected if loose. These measures will help to reduce the mismatch between the modulus of the lining and the modulus of the surrounding rock mass.

Numerous earthquake case records comparing lining performance and thickness have been analysed by Okamoto [14]. It appears from this work that poor ground conditions cannot be overcome by mere increases in lining thickness. The forces on a lining increase as its rigidity increases relative to that of the surrounding ground. Measures to increase the modulus of incompetent, loose rock by gout injection and bolting will therefore be very useful.

The rock reinforcement and sup-

port recommendations provided by the 200 case records coded in the Q-system [15] are generally consistent with good seismic design principles, due to the emphasis on various combinations of shotcrete and rock bolts for many different ground conditions. However, additional reinforcement to that recommended for static design will be required, if seismic design is an important consideration at a particular site.

Dowding and Rozen's [16] review of earthquake damage to tunnels suggests "no damage" below accelerations and velocities of approximately 0.2 g and 20 cm sec^{-1}, "minor damage" up to 0.5 g and 90 cm sec^{-1}, and "damage" above these levels. The work of Glass [17] and others suggests that the ratio of dynamic to static stress for both lined and unlined excavations is unlikely to exceed 1.20 during the passage of seismic waves. Additional reinforcement is required to resist both the positive and negative stress waves, since both may be adverse given either a high stress or a low stress environment.

A simple rule of thumb used by the author in tunnels or excavations requiring seismic design, is to increase the term SRF (stress reduction factor) used in estimating the Q-value of a particular rock mass.

The 200 cases used in developing the Q-system contained some 30 examples in which support pressure (P) was specifically described. As a crude approximation, it is found that in the poorer qualities of rock mass, the log P — log Q curve has a linear negative slope. A 100% increase in SRF, which results in:

$$Q \text{ (seismic)} = 1/2\ Q \text{ (static)}$$

provides for approximately 25% increase in support pressure, as shown in Fig. 8.

When applying the Q-system the above rule-of-thumb for seismic design generally ensures a closer spacing of rock bolts, and may also result in the addition of mesh reinforcement to the shotcrete, if a change in support category occurs as a result of the reduced Q-value. At the extreme end of the scale, where very low rock mass qualities demand concrete linings, the "seismic reduction" in Q may result in the recommendation for steel reinforcement of the concrete, which is likely to be beneficial in controlling crack development in the lining.

3.3. *Specific rock support strategies*

The use of a general rock mass reinforcement and tunnel support method such as the Q-system will not be appropriate in cases where adverse geological features define deep unstable wedges, or where potential motion on a fault has to be tolerated by the reinforcement. Such cases warrant special design and should incorporate appropriate bolt orientation angles, and bolt stiffnesses that are consistent with the strength-deformation properties of the geological feature being secured.

Direct shear tests performed on bolted blocks or jointed rock indicate peak shear resistances when the bolt forms an acute angle of about 35–50° from the plane of the joint, and is under tension rather than compression. The bolt tension contributes an increase in normal stress, thereby enhancing the shear strength. The range of angles (35–50°) corresponds to the mobilized friction angle which may be pre- or post-peak according to the shear displacement reached when the maximum combined shear resistance is reached.

A similar bolt orientation in a tunnel intersected by a persistent set of joints will be successful only if the bolts have high enough capacity to

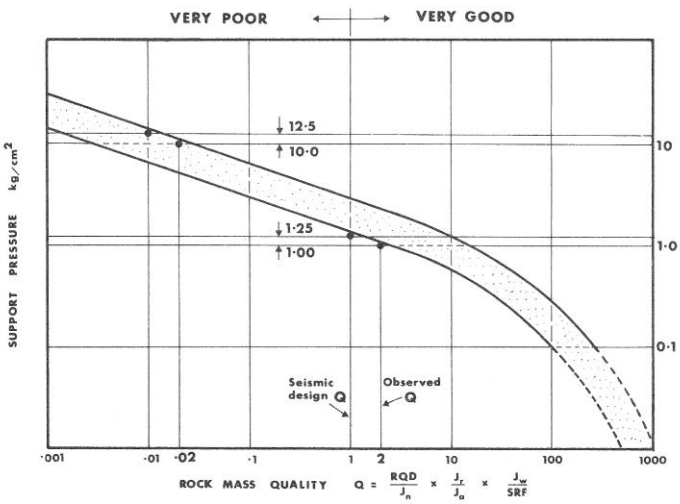

Figure 8. Seismic reduction of Q-value to obtain 25% increase in support pressure.

prevent large post-peak displacements from occurring. As indicated in Fig. 7 (upper diagram) the bolts would need to be stiff enough (fully grouted) and of high enough capacity (large cross-sectional area of high strength steel) to prevent displacements larger than 1–3 mm from occurring.

Small displacements may be difficult to guarantee during a major earthquake, and some flexibility in the bolting may need to be designed; for example by grouting only the ends of the bolts. When such a design is contemplated the appropriate post-peak, pre-residual shear strength — termed the mobilized strength — will need to be estimated to ensure optimum orientation of the bolts.

In the force diagram illustrated in Fig. 9, a major unstable wedge in the right wall of a large excavation is assumed. The wedge has a total weight represented by the force (W), an uplift force (U) from water pressure in the adversely dipping joint, and an outward-directed force (T) from an assumed worse-case water pressure acting in a secondary joint which forms a potential tension crack through the upper part of the wedge. An additional force ($W.fg$) representing the horizontal component of peak seismic acceleration acting on the wedge would be added to the force diagram for the case of seismic design.

Closure of the force diagram and calculation of the bolting capacity needed to ensure an adequate factor of safety is achieved by constructing the appropriate strength envelope using the joint perpendicular (dotted line) as the axis of effective normal stress [18]. The frictional resultants (R_r, R_m and R_p) are oriented according to whether design is based on residual, mobilized or peak strength and displacement respectively. The bolt forces required for equilibrium (B_3, B_2 or B_1) are dimensioned and oriented accordingly. Note that moment equilibrium is satisfied only if the above forces pass through the centroid of the wedge. This is unlikely, but associated errors are acceptably small.

4. Conclusions

(1) Severe seismic shaking has caused increased water flows into several mines, and joint slip, dilation and conductivity-enhancement in physical and numerical models of excavations in jointed rock.

(2) Such occurrences warrant consideration when designing water transport tunnels, beneath building foundations in clay, and nuclear waste repositories below the ground water table. The eventual breaching of a repository will be greatly accelerated if joint conductivities increase due to a seismically induced slip. Persistent, inclined jointing and high values of differential stress are likely to accentuate such problems.

(3) Tunnel support and rock mass reinforcement strategies in seismically active regions should incorporate the use of thin flexible liners of fibre- or mesh-reinforced shotcrete and systematic rock bolting, to increase the modulus of the surrounding ground. A design giving flexibility in the rock bolt system is advisable if extreme shaking is likely to cause joint slips of some millimetres or more. However, joint conductivity may be expected to increase dramatically if joint slips of more than 2–3 mm occur.

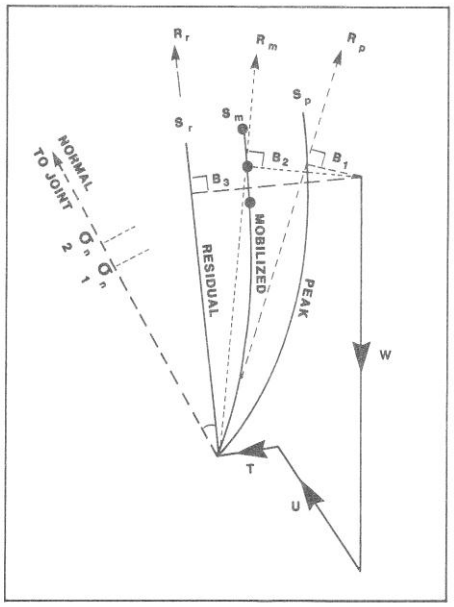

Figure 9. Estimation of bolting requirements based on peak, mobilized or residual strength and displacement [18].

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