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Developing a generalized multiple-step loading damage model to predict rock behaviour during multiple-step loading triaxial compression test

Un modèle multi-séquentiel d'endommagement pour prédire le comportement des roches en compression triaxiale

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ABSTRACT
Multiple-step Loading Triaxial Compression Test (ML-TCT) method is a useful tool to evaluate strength parameters from a single specimen. However, because of accumulated damage in the specimen with repeated cycles of axial loading/unloading, the shear strength is prone to be underestimated. Therefore, considering two models which were previously developed for a siltstone and a mudstone, a generalized Multiple-step Loading Damage (MLD) model was developed to simulate ML-TCTs with various stress paths. Numerical simulations of generalized MLD model indicated that the margin between shear strength parameters determined by Single-step Loading Triaxial Compression Tests (SL-TCT) and ML-TCTs, increase with increasing rock strength. Moreover, the upper bound values for c' and lower bound values for ϕ' could be resulted for ML-TCTs with confining pressure in increasing manner. Whereas, the upper bound values for ϕ' and lower bound values for c' can be obtained for decreasing manner tests.

RÉSUMÉ
L'essai de compression triaxiale à plusieurs étapes de chargement constitue un méthode interessante pour déterminer les paramètres de résistance d'un échantillon. Cependant, le dommage accumulé lors des multiples cycles de charge/décharge conduit généralement à une sous-estimation de la resistance globale au cisaillement. C'est pourquoi, un modèle généralisé d'endommagement multi-séquentiel est proposé, fondé sur deux précédents modèles développés pour des roches sédimentaires de type grès et calcaires. Les simulations numériques indiquent que les differences obtenues sur les paramètres de rigidité selon que des essais triaxiaux simples ou à plusieurs étapes soient réalisés, augmentent avec la rigidité de la roche. Les valeurs limites de cohésion (maximale) et d'angle de frottement (minimale) (c' , ϕ') ont pu être obtenues à partir d'essais triaxiaux à plusieurs étapes en augmentant successivement la pression de confinement, tandis que les valeurs limites de cohésion (minimale) et d'angle de frottement (maximale) ont pu être obtenues à partir d'essais triaxiaux à plusieurs étapes en diminuant successivement la pression de confinement.

Keywords : multiple-step loading, triaxial compresion test, damage model, rock

1 INTRODUCTION

Multiple-step Loading Triaxial Compression Test (ML-TCT), which was proposed by Kovari & Tisa (1975), permits determination of a failure envelope by testing a single rock specimen in a series of consolidation and shearing stages. In appropriate rocks, this technique allows the determination of strength parameters from fewer specimens than does conventional triaxial testing, which requires three or more specimens to determine a failure envelope.

Akai et al. (1981) conducted multiple-step loading triaxial compression tests to study siltstone and tuff and compared the results with those by conventional triaxial compression tests. In addition, based on several experiments on different rocks, similar comparison have been made to study the effect of loading path and determination of peak strength (Kim & Ko 1979, Crawford & Wylie 1987 & Bro 1997).

The above studies showed that, in ML-TCT, the properties measured after the first loading step, are not representative of the "undisturbed" sample, and the shear strengths are prone to be underestimated by ML-TCTs because of the accumulated damage in the specimen with repeated cycles of axial loading/unloading. Therefore, it is necessary to evaluate the applicability of this test method for various stress paths and for various types of rocks. In addition, any kind of modification in ML-TCT method in order to reduce the margin between the results of ML-TCTs and the results of conventional SL-TCTs will be desirable and give more validity to this test method.

As a result, attempt was made to propose a Multiple-step Loading Damage Model, hereafter denoted as MLD model, on a Siltstone (q_u=3-4 MPa) (Tani 2007) and a mudstone (q_u=6-8 MPa) (Taheri & Tani 2009) to simulate ML-TCTs of various stress paths. To verify the proposed models, the shear strengths obtained by the experiments were compared with those calculated using the MLD model. The calculated results were agreeable with the experimental results and the MLD model was found to be justifiable to apply for simulation of ML-TCTs with various stress paths.

In this study, based on the previous achievements and considering the stress-strain behavior of different types of rock from start of shearing up to the post failure, it is tried to propose a generalized MLD model for all types of rock. Verification of the model is carried out using the testing results on siltstone and mudstone and a numerical simulation for all rock types.

2 MULTIPLE - STEP LDADING DAMAGE MODEL, MLD MODEL

Figure 1 schematically demonstrates the results of a ML-TCT conducted to determine the geotechnical parameters to describe the MLD model. The relationships between the deviator stress, q, as well as the excess pore water pressure, Δu, and the axial strain, ε_a, are shown for repeated cycles of axial loading/unloading under the undrained condition and isotropic consolidation under the constant effective stress, σ'_c. The residual axial strain after the isotropic consolidation following the i-th cycle of axial loading/unloading is denoted as the cumulative plastic axial strain, $\varepsilon_a{}^p{}_i$. In the proposed MLD model, this $\varepsilon_a{}^p{}_i$ is assumed to represent the amount of damage

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accumulated in the specimen during the previous cycles of axial loading/unloading.

Figure 2 schematically demonstrates the five relationships to formulate the MLD model. The shear strength and the excess pore water pressure at failure for Single-step Loading Triaxial Compression Test (SL-TCT), $q_{f,SL}$ and $\Delta u_{f,SL}$, can be measured from the first loading step of ML-TCTs. Thus, the $q_{f,SL}$ - σ'_c relation, Figure 2(a), and the $\Delta u_{f,SL}$ - σ'_c relation, Figure 2(b), are evaluated from ML-TCTs conducted under different values of σ'_c. From the second and the following loading steps of ML-TCTs, the shear strength ratios, $q_{f,ML,i}/q_{f,SL}$, and the excess pore water pressure ratios, $\Delta u_{f,MLi}/\Delta u_{f,SL}$, can be evaluated where $q_{f,ML,i}$ and $\Delta u_{f,MLi}$ are the shear strength and the excess pore water pressure at failure for the i-th loading step of ML-TCTs. As shown in Figures 2(c) & 2(d), these two ratios are assumed to be expressed as the functions of the cumulative plastic axial strains at the previous, i.e. (i-1)-th, loading step, $\varepsilon_a{}^p{}_{i-1}$, and σ'_c. Furthermore, the increment of plastic axial strain for the i-th loading step, $\Delta\varepsilon_a{}^p{}_i = \varepsilon_a{}^p{}_i - \varepsilon_a{}^p{}_{i-1}$, is also assumed to be given as the function of $\varepsilon_a{}^p{}_{i-1}$ and σ'_c as shown in Figure 2(e).

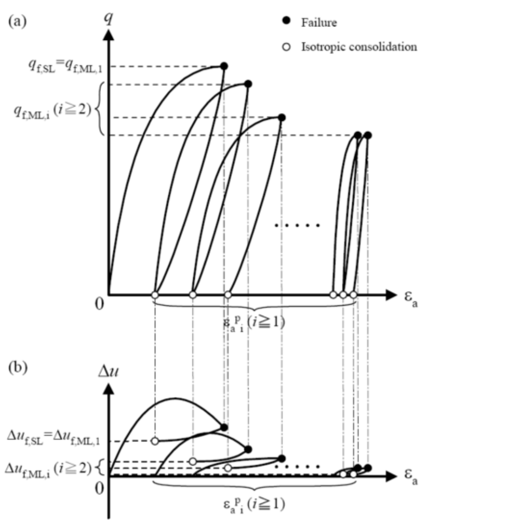


Figure 1. Definition of geotechnical parameters obtained from ML-TCT to describe MLD model

3 MLD MODEL FOR SILTSTONE AND MUDSTONE

To develop MLD model for a siltstone and a mudstone, two series of ML-TCTs were carried out for each rock type with different stress paths (Tani 2007, Taheri & Tani 2009). In the first series of ML-TCTs, repeated steps of axial loading-unloading were made under the constant effective confining pressures to determine all geotechnical parameters to describe the MLD model. In the second series of ML-TCTs, repeated cycles of axial loading-unloading were made under different effective confining pressures to verify the MLD model. Two kinds of stress paths were attempted, increasing effective confining pressures and decreasing effective confining pressures, in a stepwise manner.

Each model is comprised of five relationships as schematically demonstrated in Figure 2, and is prepared on the basis of the results obtained from the first series of ML-TCTs.

4 GENERALIZED MLD MODEL

Having established MLD models for mudstone and siltstone, attempt is made to extend the MLD model to all types of rock.

Kim and Ko (1979) have described the dependency of the effectiveness of ML-TCT on the type of stress-strain curve. They report that the quality of the results strongly depends on the post failure behavior of the rock. Basically, when the axial stress – axial strain and the volumetric strain – axial strain curves are plotted together for different material, as shown in Figure 3; there are three different cases with respect to the onset of the post-peak behaviour.

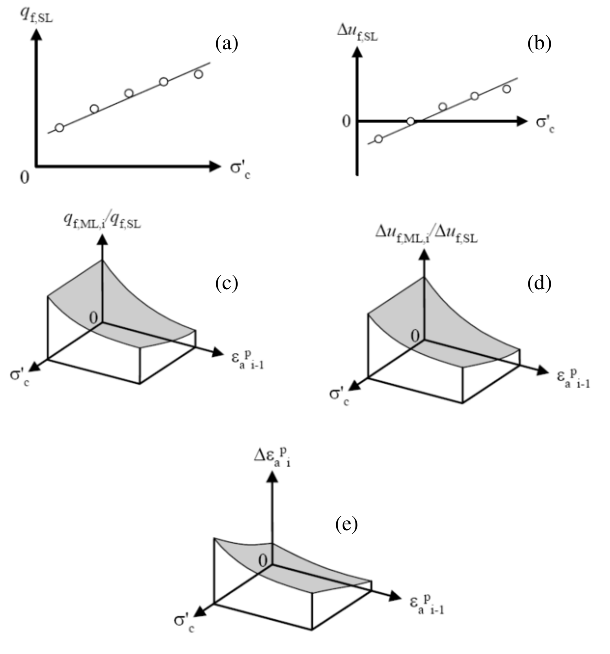


Figure 2. Concept of MLD model

The first case for hard rock (Figure 3(a)) exhibits significant dilation reflected by the volumetric strain – axial strain curve near the peak point. In addition, the post-failure behavior is typically strain-softening with brittle fracture. In this situation, attempting the multiple-step test is very difficult, since the change of the slope of the volumetric strain –axial strain curve would imply losing the integrity of the specimen. In this behaviour, which is the case for most of the hard rocks, at the peak point, failure occurs almost instantaneous without any precursory signs. Therefore, the amount of damage to the specimen is more than other types of material.

The second case for medium-hard rocks (Figure 3(b)) exhibits moderate dilation near the peak point. The post-failure behavior is strain softening with more ductile behavior. This situation which is the case for medium-hard rocks is suitable for multiple-step loading. However, still there is a risk of losing the integrity of rock if the specimen stresses much beyond the elastic region.

In the third case for soft rocks or other rocks tested at high confining pressures (Figure 3(c)), exhibits marginal dilation or contraction with axial compression up to the critical state. The failure is predominantly ductile with little sign of strain softening. In this situation, as compared to the cases of harder rocks with more brittle failure, the less damage in the specimen is expected to occur due to previous loading/unloadings.

Therefore, the amount of underestimation of shear strength parameters depends on the type of failure that the rock is expected to have. Increasing the stiffness/strength or brittleness of rock may result in losing the integrity of the specimen during multiple-step loading test or increasing the amount of damage and consequently increasing the discrepancy between results of single-step and multiple-step tests. As a result, the rock behavior and its brittleness which were described in Figure 3 should be considered in the generalized MLD model.

Moreover, the shapes around the peak point and the subsequent softening portions of the stress-strain curve are known to depend on the confining pressure applied to the rock specimen. The shapes of these portions of the stress-strain curve are more abrupt under lower confining pressures and become milder under higher confining pressures. Consequently, with confining pressure increase the brittleness and thus the amount of damage during multiple-step test reduces.

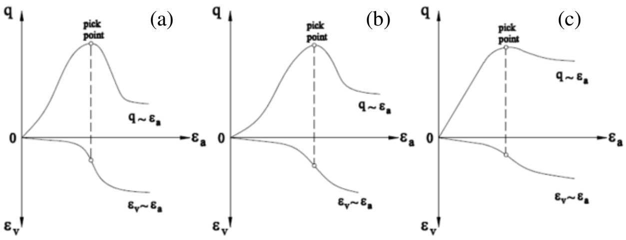


Figure 3. Volumetric strain –axial strain and deviator stress – axial strain relations in different kind of rocks

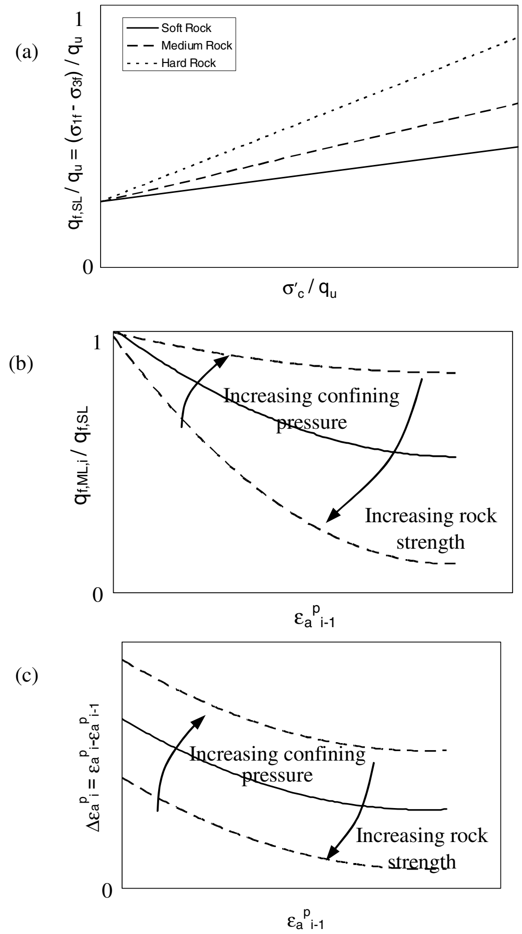


Figure 4. Concept of developing MLD model for all types of rocks

Finally, attention should be paid to the increments of plastic axial strains for the relevant, i.e. the *i*-th, loading step, $\Delta\varepsilon_a{}^p{}_i$, and the cumulative plastic axial strains at the end of the previous, i.e. the (*i*-1)-th, loading step, $\varepsilon_a{}^p{}_{i-1}$, for different rock type. Comparing the MLD models developed for siltstone and mudstone, the amount of change is found to be about three times. Therefore, a specific set of values should be allocated to each type of rock.

Based on the above statements, in Figure 4 the concept of the generalized MLD model is schematically demonstrated. In Figure 4(a), which shows the relation between $q_{f,SL}/q_u$ and σ'_c/q_u, where q_u is the unconfined compressive strength, increasing the strength of rock results in raising the slope of the lines intersected to $q_{f,SL}/q_u$ axis at 1.0. In Figure 4(b) the descending amount of $q_{f,ML,i}/q_{f,SL}$ is increased with increase of rock strength and decreased with increase of confining pressure. In Figure 4(c) the same trend is shown for changing of $\Delta\varepsilon_a{}^p{}_i$ versus variations of rock strength and confining pressure.

Having demonstrated the concept of unified MLD model and based on the model previously constructed for siltstone and mudstone, a generalized MLD model is proposed and presented in Figure 5.

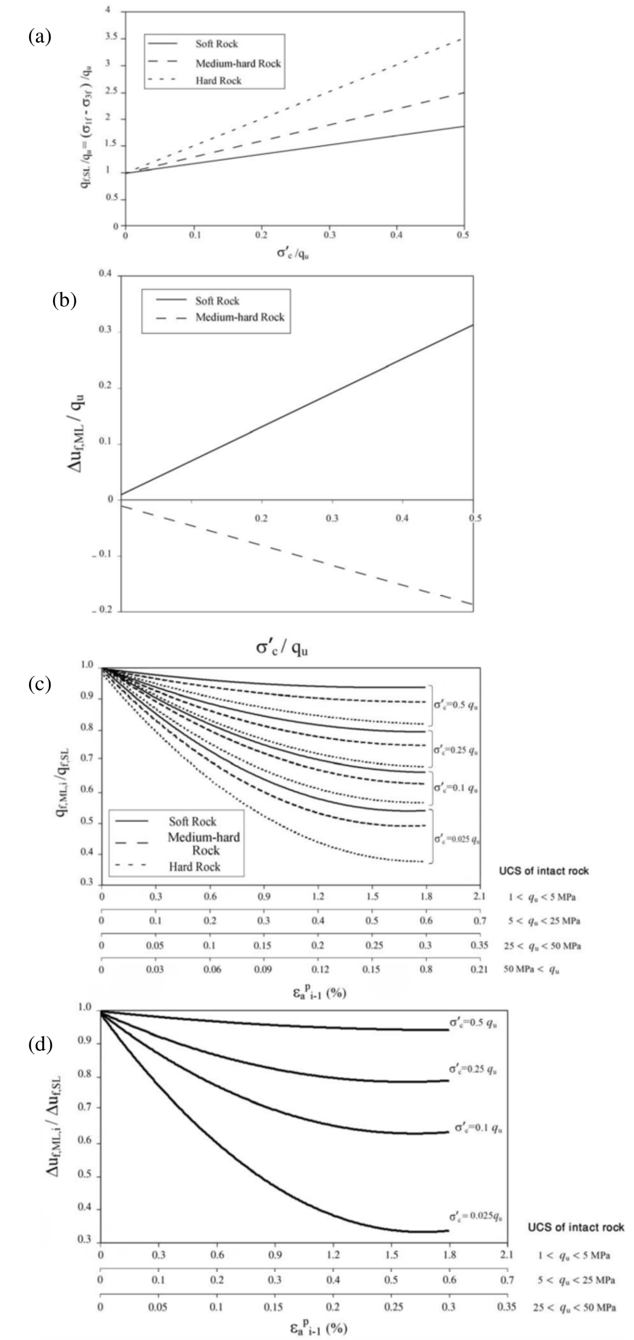
Knowing the unconfined compressive strength, q_u, of a particular rock and with adopting the σ'_c value, $q_{f,SL}$ is obtainable from Figure 5(a) for the relevant rock type. As shown in Figure 5(b), depending on the type of rock and its strength, $\Delta u_{f,SL}$ can be determined as an positive or negative value. The effect of pore pressure in hard rocks is neglected. From Figures 5(c)&(d), $q_{f,ML,i}/q_{f,SL}$ as well as $\Delta u_{f,MLi}/\Delta u_{f,SL}$ can be calculated for each type of rock and chosen confining pressure. Similarly, the relations between $\Delta\varepsilon_a{}^p{}_i$ and $\varepsilon_a{}^p{}_{i-1}$ for different types of rocks and various confining pressures are demonstrated in Figure 5(e).

For the purpose to verify the generalized MLD model, comparison were made between the experimental results of second series of ML-TCTs on siltstone and mudstone, and the calculated results of the ML-TCTs with the identical stress path. The procedure is described in Figure 6. The calculated results were agreeable with the experimental results for both stress paths with increasing and decreasing values of σ'_c. But, verification of the model for other types of rock is also desirable. For this, a numerical simulation is carried out to predict ML-TCT in three typical soft, medium-hard and hard rock. Two ML-TCTs were simulated in increasing and decreasing manner under the different confining pressure, σ'_c/q_u = 0.25, 0.1, 0.15, 0.2, 0.25, 0.3, 0.4, 0.5, where q_u assumed to be 20, 40 & 80 MPa for soft, medium-hard and hard rock respectively. The procedure is shown in Figure 6. The deviator stresses in peak state for the first loading step (SL-TCT) were determined from Figure 5(a) and for the second to the following loading step from Figures 5(c)&(d). To make a straightforward comparison, the effect of excess pore water pressure is neglected in all the cases.

The results are compared in Figure 7 based on the amount of error for Mohr-Coulomb shear strength parameters calculated for each type of rock and loading method. The upper bound values for c' and lower bound values for ϕ' could be resulted from ML-TCT with increasing confining pressure steps, whereas, an opposite trend can be seen for ML-TCTs with decreasing confining pressure steps. As can be seen, in increasing manner test, the amount of error for cohesion, c', increases with rock strength, whereas for friction angle, ϕ', seems to be constant with rock strength increase. Whereas, in decreasing manner test, with rock strength increase, the margin between results of single-step loading test and multiple-step loading test decrease for friction angle, ϕ', and increase for cohesion, c'. Moreover, since, the difference between amounts of effective cohesions obtained by single-step loading test and decreasing multiple-step loading test is significantly large, ML-TCTs with increasing confining pressure steps is preferable to those with decreasing confining pressure steps for all types of rock. It should be noted that, the large amount of underestimation for c' and small amount of overestimation for ϕ' in medium-hard and especially hard rock for ML-TCTs with decreasing confining pressure steps, is attributed to the large plastic strains accumulated in early stages of loading cycles with high confining pressures. It can result in largely underestimated shear strength values.

5 CONCLUSIONS

For the purpose to investigate the applicability of multiple-step loading triaxial compression tests, ML-TCTs, a generalized Multiple-step Loading Damage model, MLD model, was proposed to simulate ML-TCTs with various stress paths. In the MLD model, the damage accumulated in the specimen by the repeated cycles of axial loading/unloading is represented by the plastic axial strains.



Notice: In increasing manner test, $q_{f,ML,i}/q_{f,SL}$ should be reduced by 5%, when $0.25q_u<\sigma'_c$

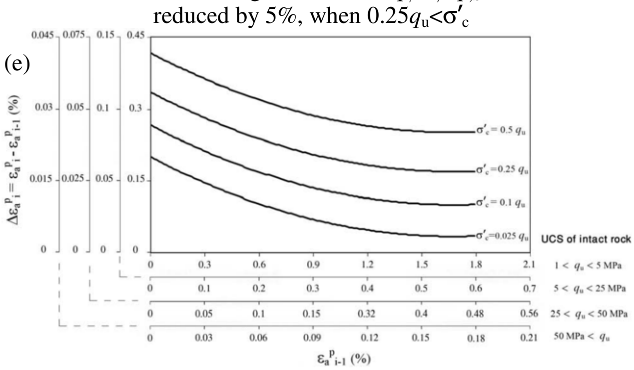


Figure 5. Generalized MLD model for all types of rocks

Numerical simulations of the generalized MLD model indicate that, generally the margins between shear strength parameters determined by SL-TCT and ML-TCT, increase with rock strength increase. Moreover, the upper bound values for c' and lower bound values for ϕ' could be resulted from ML-TCT with increasing confining pressure steps, whereas, an opposite trend can be seen for ML-TCTs with decreasing confining pressure steps. It is also concluded that, ML-TCTs with increasing confining pressure steps is preferable to those with decreasing confining pressure steps for all types of rock.

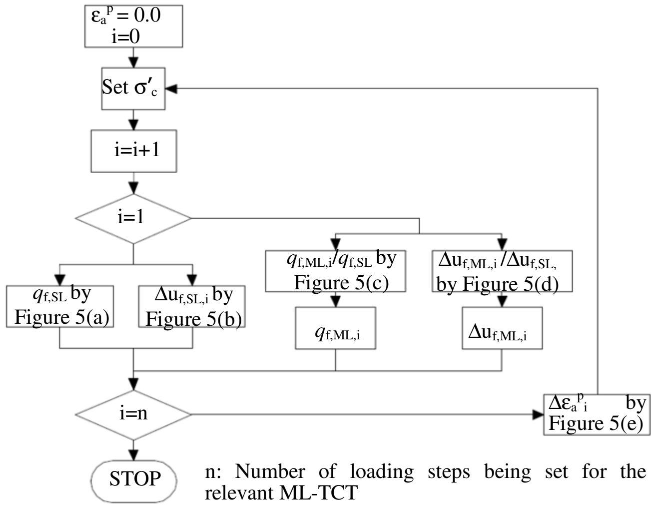


Figure 6. Procedure of simulation of ML-TCTs using MLD model

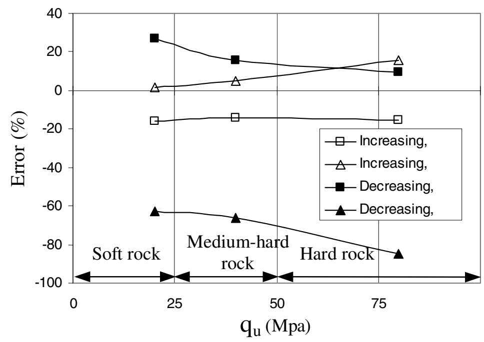


Figure 7. Results of simulation of ML-TCT by MLD model in different types of rock and loading path

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