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Relationships between the internal erosion parameters and the mechanical properties of granular materials.

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Abstract: Granular soil in construction of earth works can be solicited by hydrodynamic and mechanical forces. Fine particles constituting these soils may be eroded by the flow leading to a modification of their mechanical behaviour. Classically, the susceptibility of granular soils to internal erosion is assessed by internal stability criteria. In this paper, numerical modelling using discrete elements method is performed to study the mechanical behaviour of eroded soils on triaxial paths. Soils samples were generated according to known particle size distribution and void ratio. Triaxial test have been simulated on samples with the same initial void ratio and same confining stress. Relation has been established between soil mechanical behaviour and its internal stability. Finally, taking into account the relationship between the internal erosion parameters and the mechanical properties of granular materials, a new approach to assess the soil stability has been presented.

Key words: Suffusion, triaxial paths, mechanical behaviour, internal stability criteria, DEM.

1. Introduction

Non-cohesive soils subjected to fluid flow may have fine particles migrate through the interstices of the solid skeleton formed by large particles. This phenomenon shows the internal instability of the soil as its susceptibility to lose its fine particles (Terzaghi, 1939; Kézdi, 1979; Kenney and Lau, 1985, 1986; Lafleur et al., 1989; Chapuis et al., 1996; Foster and Fell, 2000). It occurs in natural soil deposits and in embankment structures (Smith and Bhatia, 2010) such as earth dams; dikes; highway embankment. In granular soils, suffusion may occur only when these three following conditions are satisfied: the fine particles are smaller than the size of constrictions formed by the coarse particles; fine particles do not fill the constrictions and are free to move; the hydraulic gradient is high enough. This process causes erosion of fine particles or clogging in granular filters rendering them ineffective and modifies the soil void ratio, affecting its mechanical behaviour leading to the collapse of earth works. Suffusion can induce local changes in permeability, and it can also lead to fines particles migration which have as consequence, the appearance of cracks and potential dam failure. Uniform soils usually do not suffer from suffusion, unlike well-graded or gap-graded granular soils (Kenney and Lau, 1985; Skempton and Brogan, 1994).

To assess the potential of internal stability, several criteria have been proposed such as the geometric (Kézdi, 1979; Kenney and Lau, 1985), hydraulic or hydro-mechanical (Skempton and Brogan, 1994; Moffat et al., 2011).

Terzaghi (1939) defined a retention coefficient of a filter design and proposed $D_{15}/\ d_{85} < 4$ for stable soils. D_{15}, is the maximum diameter of the 15% finest particles of the coarse fraction, and d_{85} the maximum diameter of the 85% finest particles of the fine fraction. Based on this Terzaghi's filter rule, Kézdi (1979) and De Mello (1975) proposed a method by splitting the soil's particles size distributions (PSD) into its fine and coarse fraction, at a point characterized by its diameter d_n. The soil is considered as stable if for any d_n, the Terzaghi's classical rule is verified.

The way to choose $\mathrm{d_n}$ is different according to the author. For exemple, De Mello (1975) divided the curve at a point of discontinuity and limited the use of his criterion to discontinuous gradings, whereas Kezdi (1979) divided the curve at an arbitrary point, then repeated this operation for all the points of the curve to derive a global stability criterion.

The US Army Corps of Engineers (USACE, 1953) studied possibilities of making a granular filter from a mixture of sand and gravel instead of a double-layer system of the same materials. They experimentally evaluated the role of soil composition in controlling the internal instability potential. This method of investigation has been reassessed by Kenney and Lau (1985) and the role of PSD in the internal stability of a soil has been shown in more detail. They proposed a geometrical criterion to assess the internal stability of soil. The criterion supposed that each class of grain, defined by its diameter d, must be prevented to move by the grains whose diameter is comprised between d and 4d. This criterion consists in plotting the PSD curve of a soil in a diagram with the mass fraction F of particles whose diameter is smaller than d on the X-axis and the mass fraction H of the particles whose diameter is comprised between d and 4d on the Y-axis. The soil is considered as stable if, for F < 20 or 30%, the curve obtained is located below the line H = F. Otherwise, if some portion of this curve passes above the line H = F, the soil is considered as unstable. According to Kenney and Lau (1985), F values of 20% apply to the soils with widely graded primary fabric (Cu > 3, Cu is the uniformity coefficient) and the value of 30% to the soils with narrowly graded primary fabric (Cu < 3).

To assess internal stability of soil, Indraratna and Vafai (1997) conducted hydraulic tests on fine sand as well as sand-gravel mixture. The coefficient of uniformity (Cu) of these samples varied between 12 to 40 and relative density between 5 to 90%. Combining the two criteria described above, Indraratna et al., (2015) proposed an internal stability criterion with more accurately boundary. Stables soils were characterized by $d^{f}_{85,SA} > D^{C}_{C35}$. Where $d^{f}_{85,SA}$ is the representative size for finer fraction from PSD by surface area technique and controlling constriction for coarser fraction from CSD

(constriction size distributions) by surface area technique. D_{C35}^{C}: Controlling constriction for coarser fraction from CSD by surface area technique

Other authors developed several criteria which introduced the concept of free or mobile particles (Burenkova, 1993; Skempton and Brogan, 1994; Salehi Sadaghiani and Witt 2012; Salehi Sadaghiani et al., 2012) by focusing on quantity of particles that are free to move in constrictions or free fines "fines particles which are free to move" (Andrianatrehina et al., 2015) by focusing on maximum diameter of these particles. These criteria are based on the free volume available for the finest particles to move. According the free particles concept, Burenkova (1993) proposed a method based on results of laboratory tests on granular materials of maximum particle size up to 100 mm, and coefficient of uniformity (Cu) up to 200. For the tests, the soils were divided into various size fractions, and then dry-mixed in a container. The basic assumption is that a small size fraction does not form part of the basic soil skeleton if it does not cause a volume increase when mixed with a coarse size fraction. Based on the three representative fractions d_{15} , d_{60}, and d_{90} , the heterogeneity of the soils was described by two ratios called conditional factors of uniformity:

$h' = d_{90} / d_{60}$; $h'' = d_{90} / d_{15}$

Based on these two ratios, Burenkova (1993) presented boundaries separating the suffusive soils from non-suffusive. According the concept of free fines, methods which allowed determining the maximum diameter of free fines and the maximum fines content for which the fines fill the voids between the coarse grains without changing their structure were proposed (Andrianatrehina et al., 2015; Ghafghazi and Azhari, 2012).

Skempton and Brogan (1994) carried out laboratory flow tests on uniform and discontinuous PSD in order to determine the hydraulic gradient from which the migration of particles begins and therefore the internal instability in the material. They established relationship between hydraulic gradient and the stability ratio (H / F) defined by Kenney and Lau (Kenney and Lau, 1985, 1986). They noticed an increasing in hydraulic gradient from a small value to a high value for $(H / F)_{min} = 1$. They also point out that the relationship between the hydraulic gradient (i_c) and the stability index (H / F) is strongly influenced by the density of the sample and the proportion of sand in gravel.

Concerning the hydro-mechanical criteria, Moffat et al. (2011) developed a hydro-mechanical relation governing internal stability of cohesionless soil. The onset of internal instability is triggered either by an increase in hydraulic gradient or by a decrease in effective stress. They used a permeameter device (Fannin and Moffat, 2006) that enables the relation between local hydraulic gradient and effective stress to be characterized.

Besides the hydraulic susceptibility of soils, mechanical behaviour was also considered in the study of suffusion's initiation. However, there were only few studies on the mechanical consequences of soil subjected to internal erosion from the fabric point of view. Most of studies that mention the mechanics have focused on the consequences of the suffusion on the mechanical behaviour of soils. The soil PSD changes induced by this phenomenon causes the modification of the critical state line's (CSL) position (Muir Wood and Maeda, 2008; Muir Wood et al., 2010), peak strength (Scholtes et al., 2010) or the angle of internal friction (Hicher, 2012; Ke and Takahashi 2012; Chang et al., 2014; Abdoulaye Hama et al., 2016). At present, there is no method that can predict for example if a dam, in which suffusion occurred will resist or not. In this context, the aim of this paper is to establish a relationship between the internal stability parameters and the failure criterion of materials. To achieve this, a Discrete Elements Method (DEM) is used to model soil sample and triaxial paths are simulated. A new criterion of internal stability taking into account the mechanical behaviour of soils is proposed in this study. It classified as stable materials which have high shear stress ($(q/p)_{max}$ >1.5 in this study) and lower maximum contractancy (ε_{vmax} <0.3% in this study) when subjected to suffusion.

2. Numerical Modelling

This numerical study was performed with Particles Flow Code (PFC 3D) that uses the discrete element method (DEM) originally developed by Cundall and Strack (1979a, 1979b). Materials are discretized in elements of simple shape (spheres) that interact with neighbouring contact elements, giving the discrete elements the ability to easily create complex environments with voids, imperfections and heterogeneities.

To achieve this aim, a material is generated with a target granular distribution and void ratio and then subjected to a triaxial test. Three stages were followed for the numerical specimen tests: (1) specimen preparation; (2) consolidation and (3) specimen shearing. Appropriate care which will be developed in more detail in section 2.1 and 2.2 was taken to select numerical parameters that were consistent with real triaxial tests.

2.1. Numerical specimen preparation

To generate a numerical specimen, a set of confining boundaries are needed to constrain the assembly and restrict the movement of particles across the boundaries. In this work, the generation procedure of granular sample with a given grain size distribution curve and a given void ratio developed by Abdoulaye Hama et al. (2016) was used. With a void ratio of 0.66, the cylinder diameter was 3 cm, and the height was 6 cm. The vertical boundary conditions are made up of a very flexible watertight cylindrical membrane to simulate the latex membranes commonly used in triaxial tests in laboratory soils. This membrane has a normal stiffness Kn = $10^7\ N/m$ which is ten times lower than that of the solid particles constituting the soil. It simulates the confining pressure experienced by the sample. The top and the bottom walls simulate loading platens. After creating the cylindrical cell, the particles were

put in it, but with a reduced radius (respecting the PSD). Using a minimization method, a provisional specimen is generated which is described by the following steps:

- The sample is defined by its PSD which diameters d_j ($1 < j < N+1$, N is the number of interval) and its void ratio e.
- Using the bisection method (Burden et al. 1985), a provisional sample is randomly generated with diameters $(d^i)_j$ $1 < j < N+1$ and void ratio e^i, $e^i > e$. (d^i and e^i are respectively intermediate diameters and intermediate void ratio).
- To reach the real particles size and create a more elastic packing the radius expansion technique is used (Potyondy & Cundall 2004): $d = \alpha . d^i$, where α is a multiplier coefficient.
- Real void ratio is reached using the relationship:

$$e + 1 = \frac{e^i + 1}{\alpha^3} \quad (1)$$

The friction angle is temporary reduced during the generation and radius expansion procedures. Static equilibrium state is reached after applying several damping cycles and when the ratio of the mean unbalanced force to the mean contact force is lower than 10^{-4}.

2.2. Triaxial test Modelling

Triaxial simulation presented here is performed using the PFC3D code (Itasca (2008), Achmus & Abdel-Rahman 2004, Zhao & Li 2013, Abdoulaye Hama et al., 2016). This method involves confining a cylindrical sample of a granular material inside three walls. The two walls (at the top and bottom of the sample) are rigid plates that simulate the load and the side wall simulates the confining pressure laterally applied to the sample. (Figure1).

The axial strain can be calculated by the following equation:

$$\varepsilon = \frac{H_1 - H_0}{\frac{1}{2}(H_1 + H_0)} \quad (2)$$

Where H_0 is the initial height of the sample and H_1 is the current height during the test

During the consolidation stage the isotropic stress of a particle assembly is defined as the mean of the direct stresses. The direct stresses and strains are measured with measurement spheres (at each single sphere). A numerical servo mechanism is applied iteratively to arrive at the sample's radius expansion necessary to achieve the required mean stress.

The implemented servo-control mechanism is a function that is integrated in PFC3D to maintain a constant stress (axial and confining) throughout the simulation. This function is called on in every calculation cycle to determine the current wall stresses and it adjusts the wall velocities in such a

way as to reduce the difference between measured stress and required stress. The calculation algorithm for the servo-control mechanism is given by:

$$\dot{u}^{w} = G\left(\sigma^{W} - \sigma^{t}\right) = G\Delta\sigma \qquad (3)$$

Where $\dot{u}^{w}$ is the wall velocity, σ^{W} is the current wall stress, σ^{t} is the target wall stress and G is the "gain" parameter estimated using the following reasoning. The maximum increment in wall force ΔF^{w}arising from wall movement in one time step is given by:

$$\Delta F^{w} = k_{n}^{w} N_{c} \dot{u}^{w} \Delta t \qquad (4)$$

Where N_c is the number of contacts on the wall and k_n^w is the average stiffness of these contacts. Hence the change in mean wall stress is given by

$$\Delta\sigma^{w} = \frac{k_{n}^{w} N_{c} \dot{u}^{w} \Delta t}{A} \qquad (5)$$

Where A is the wall area.

For stability, the absolute value of the change in wall stress must be less than that of the difference between the measured and target stresses. This would lead to an oscillation of the target stress that would grow in an unbound fashion and lead to instability. Instead, the wall stress should approach the target stress smoothly from either above or below. In practice a relaxation factor (α, set to 0.5) is used so that the stability requirement becomes (Zhao et al., 2011).

$$|\Delta\sigma^{w}| < \alpha|\Delta\sigma| \qquad (6)$$

Substituting Eqs. (3) and (5) into Eq. (6) yields:

$$\frac{k_n^w N_c G|\Delta\sigma|\Delta t}{A} < \alpha|\Delta\sigma| \qquad (7)$$

And the stability condition for the gain is determined via:

$$G = \frac{\alpha A}{k_n^w N_c \Delta t} \qquad (8)$$

The wall velocity is set before each cycle to satisfy Eq. (3), and the gain is set to satisfy Eq. (8).

3. Materials

When the phenomenon of suffusion occurs in a soil sample, its PSD changes and this affects its internal stability. To study the impacts of suffusion on the mechanical behaviour of granular soils, five PSD identified by S1; S2; S3; S4 and S5 were defined from an alluvial deposit from the Rhône River in France (Souli et al., 2011) The maximum grain size is 8 mm (Figure2). Samples S1; S2; S3; S4 and S5 have their number of particles respectively 4021; 5052; 3738; 4263 and 5168 and their aspect ratio between the biggest and the smallest particles are respectively 9.3; 11.4; 11.4; 9.3 and 9.3.

In spite of a small ratio between sample diameter 3cm and maximum particle diameter 8 mm, the samples are relatively homogeneous. This is due to the small percentage of large particles on each sample.

The most important differences among these samples are those related to their particle size distribution and especially the internal stability parameters like the minimum ratio of stability $(H/F)_{min}$ and $(D_{15}/d_{85})_{max}$. These parameters are summarized in the Table 1.

The classification of these samples following Kenney and Lau (1985) and Kezdi (1979)'s criteria shows that S2, S3 and S4 were stable. The two criteria diverge on the internal stability of S1 and S5. Each sample was subjected to a consolidated and drained triaxial test simulation, at same initial void ratio of e=0.66 and constant confining stress σ_3=200 kPa.

4. Results and Discussion

4.1. Triaxial results.

These results are presented in the deviatoric (ε_1; q/p) and the volume change (ε_1 ; ε_v) planes (Figure 3). They are used to calculate the stress ratio $(q/p)_{max}$ and the maximum contractancy (ε_{vmax}) which is the maximum absolute value of volumetric strain. Each sample is characterized by its minimum ratio of stability $(H/F)_{min}$ and its maximum retention index $(D_{15}/d_{85})_{max}$ (Table 1).

4.2. Evolution of mechanical and internal stability parameters.

The relationship between mechanical parameters ($(q/p)_{max}$, ε_{vmax}) and internal stability parameters ($(H/F)_{min}$, $(D_{15}/d_{85})_{max}$) was graphically established and showed that:

- An increasing trend of shear strength and a decreasing tendency of maximum contractancy when the ratio of stability $(H/F)_{min}$ increases. (Figure 4).
- A decreasing tendency of shear strength and an increasing trend of maximum contractancy when the maximum coefficient of retention $(D_{15}/d_{85})_{max}$ increases for the values lower than 4 (Figure 5).

Figure 4 and 5 illustrate also the classification of samples S1, S2, S3, S4 and S5 by Kenney& Lau and Kezdi criteria. The samples S2, S3, S4 which were classified as stable by these two criteria exhibit high shear strength ($(q/p)_{max}$ >1.5 in this study) and low maximum contractancy (ε_{vmax} <0.3% in this study). Samples S1, S5 are classified as unstable.

One can note that a triaxial test carried out on samples from the same soil and same conditions of initial void ratio and isotropic confinement may give different results if geometrical parameters of internal stability $(H/F)_{min}$ and $(D_{15}/d_{85})_{max}$ are different. Therefore the internal stability of a granular material strongly influences its mechanical behaviour. This analysis shows also that the stability parameters (H/F) and (D_{15}/d_{85}) have conflicting effects on the mechanical behaviour of a soil sample. To study these effects, evolution of these previous stability parameters versus particles diameter was presented on Figure 6.

In our previous studies, effects of internal erosion on mechanical behaviour of granular soil were analysed based on the evolution of internal friction angle (Abdoulaye Hama et al., 2016). Changes in this angle in relation to medium diameter D50 and the initial void ratio n provided unexpected results. In addition, literature's results are contradictory about the evolution the friction angle after suffusion. Hicher (2012) showed that it decreases after suffusion, while for Chang et al., (2014) it increases. We concluded that the friction angle taken alone does not seem to be a relevant parameter for the analysis of mechanical stability in granular media. However, if one takes into account also the point of phase change in the (ε_1; ε_v) plane corresponding to the maximal contractancy, relationship suffusion-mechanical behaviour assumes a meaning as shown below.

4.3. Relative position of H / F and D_{15}/d_{85} of a soil

The fact that the two stability parameters act differently on mechanical behaviour of soil samples led us to analyze for each sample the evolution of H/F and the D_{15}/d_{85} parameters depending on the particle diameter (Figure 6). The two curves H/F and D_{15}/d_{85} are defined for particle diameters corresponding to the F values between 0 and 30%. The aim of this representation is to analyze the monotony especially the relative positions of these internal stability parameters. The results showed that for the samples S1 and S5 which suffered most from contractancy and presenting lower shear stresses, the curves intersect and their relative position is not at all clear. However for the samples S2, S3, S4 which have low values of contractancy and higher values of shear strength, the curve H/F is always above the curve D_{15}/d_{85}.

4.4. Proposed criterion

The study of the suffusion by flow tests and by its numerical simulation showed the dependence of this phenomenon to PSD of the material, its density and to the hydraulic gradient (Abdoulaye Hama, 2016). These results were consistent with previous studies (Kezdi 1969, Kenney & Lau 1985, Indraratna 2015). The internal instability criteria resulting from these previous studies consider that a stable material is not liable to lose its fine particles under the effect of a flow. Moreover, in a backfill, a loss of fine particles can modify the density of material causing its collapse in the long term. However, Pavchich (2009a: 2009b) and Abdoulaye Hama et al. (2016) consider that a material can lose up to a certain threshold of its fine particles (5% in Pavchich (2009a: 2009b)) and remain stable. Hence, the need to take mechanical behaviour into account in the criteria of instability is very important. The variation of the parameters D_{15}/d_{85} and H / F corresponds to a level of loss of fine particles and therefore to a level of suffusion that material has suffered. Our results (Figures 4 and 5) showed the impact of these parameters on the mechanical behaviour (contraction / stress ratio) of the samples. The proposed criterion is based on taking into account the effect of these internal erosion parameters on the mechanical behaviour.

According to Kenney & Lau criterion, a material is stable when the ratio of stability is greater than 1. For Kezdi criterion, stable soil is characterised by its retention coefficient lower than 4. In an attempt to link the internal stability of granular materials to their mechanical behaviour, our results show that, sample with a value of maximum contractancy less than 0.3% and with a maximum stress ratio greater than 1.5 are those with H/F values greater than their D_{15}/d_{85} values. So, these samples have a common feature which is materialized by the following relationship $\frac{H}{F} \geq \frac{D_{15}}{d_{85}}$. This equation classifies stable samples in this new proposed internal stability criterion. The limit of stability is the bisecting line in the $((D_{15}/d_{85})_{max}$; $(H/F)_{min})$ plane (Figure 7). The peculiarity of this criterion is to assess the susceptibility of granular soil to internal erosion while taking into account their mechanical behaviour. This criterion propose that sample with $\frac{H}{F} \geq \frac{D_{15}}{d_{85}}$ has a good mechanical properties (ε_{vmax} <0.3% and $(q/p)_{max}$ >1.5 in this study) and is classified as stabe.

Figure 7 presents the state of stability of samples S1 to S5 according to the proposed criterion. It classifies S2, S3and S4 as stable and S1, S5 as unstable.

4.5. Assessing of Proposed criterion with published results.

A set of data of 16 samples from literature is used to assess the proposed criterion in this study. It contains 4 samples collected from Kenney and Lau (1985), 8 from Andrianatrehina et al., (2012) and 4 from Indraratna et al., (2015). The choice of these samples is motivated by the diversity of their density and that of the analysis method of the authors.

Kenney and Lau (1985) carried out laboratory tests by submitting soil samples with known relative densities to a downward flow. After the tests, the samples were removed by three layers (a top transition layer, which was coarsest in its top surface, a central homogenous layer and a bottom layer) and tested for PSD. Authors assume that any coarsening observed relative to the initial grading indicated the existence of loose, movable particles throughout the specimen. They described unstable samples by changes in grading deducted in changes in its diameter d_5.

Samples characterized by their fine particles levels (maximum diameter <1 mm) were subjected to vibrations and undrained consolidated triaxial tests by Andrianatrehina et al. (2012). In order to estimate the migration of the fine particles caused by all solicitations (vibrations, stresses), each sample was divided into three parts (lower part, median and upper part) and their PSD were compared after test. Potential for instability was assessed with a parameter ΔP (%) representing the difference between the upper and lower PSD. These authors proposed a criterion called "direct measurement", which considers material as stable when ΔP<5% and as unstable when ΔP>10%.

Indraratna et al. (2015) conducted laboratory tests by submitting samples to an upward flow. The density of the samples varied from 6.57 to 95.55%. The hydraulic gradient was varied from 0.92 to

1.18. During the post-test examination, all the tested samples were carefully retrieved in five identically compacted layers for which the PSDs of the central layer-3 were then determined. The study consisted of examining whether the PSD of the core layer changed after the test and thus classify the sample as stable. Large visible channels joined together to form a composite failure mode (rising + piping) were observed as the hydraulic gradient approached the unit. More importantly, the erosion of its finer fraction was significantly contained as the degree of compaction increased.

Finally, criteria on which the authors (Kenney & Lau (1985), Andrianatrehina et al., (2012) Indraratna et al., (2015)) classified their samples are all based on fine particles moved across soils sample leading to the change of its PSD. In this work, the proposed criterion is based on the impact of the variation of internal stability ($\frac{H}{F}$ *and* $\frac{D_{15}}{d_{85}}$) parameters on the mechanical behaviour to classify the samples with the smallest value of contractancy as stable.

These 16 samples are classified using the proposed criterion. The Table 2 summarizes for each sample, the minimum ratio of stability $(H/F)_{min}$, the maximum coefficient of retention $(D_{15}/d_{85})_{max}$, its classification according to Kenney and Lau criterion, its classification according to the proposed criterion and observed experimental results. Figure 8 describes the location of samples relative to the proposed limit of stability in $((D_{15}/d_{85})_{max}, (H/F)_{min},)$ plane.

5. Conclusion

Using the discrete elements method, sand samples were modelled and submitted to a consolidated and drained triaxial test simulation at same initial void ratio and confinement stress. The maximum values of contractancy in (ε_1; ε_v) plane and the maximum deviatoric stress in the (ε_1; q/p) plane have been determined for each sample. Results showed that mechanical behaviour of granular material is influenced by the stability parameters $(H/F)_{min}$ and $(D_{15}/d_{85})_{max}$. These parameters act on mechanical behaviour by two opposite effects. For example, the maximum contractancy decreases when $(H/F)_{min}$ increases and it increases when $(D_{15}/d_{85})_{max}$ increases. Classical criteria of internal stability are based solely on the PSD of the material and do not include its mechanical behaviour. From this, a new internal stability criterion $\frac{H}{F} \geq \frac{D_{15}}{d_{85}}$ has been proposed. It classifies as stable materials which have high shear stress ($(q/p)_{max}$ >1.5 in this study) and lower maximum contractancy (ε_{vmax} <0.3% in this study)

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