EVALUATION OF MOVEMENT DATA AND GROUND CONDITIONS FOR THE ÅKNES ROCK SLIDE

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ABSTRACT

Catastrophic rock slope failures have caused destructive tsunamis in Norwegian fjords. At the Åknes rock slope the tsunami generating potential is large due to the potential large volume involved in a possible catastrophic failure. Widening of the upper crack has been recorded since 1986, and in recent years, a quite extensive investigation and monitoring campaign has been conducted. Data from some of these investigations are presented and analysed with respect to a preliminary evaluation of the stability of the slope.

1 INTRODUCTION

Large rock slides represent one of the most serious natural hazards in Norway, as exemplified by the Tafjord disaster of 1934 when 2 – 3 million m$^3$ rock mass and scree material dropped into the fjord (Jørstad, F. 1968). The tsunami generated by the slide reached a maximum of 62 m above sea level, and several villages were destroyed. 41 people were killed by the tsunami. In the 20$^{th}$ century 175 people lost their lives in three such events in the region of northern West Norway (Tafjord 1934 and Loen 1905 and 1936, Figure 2).

Figure 1. Index map showing the location of the area in Figure 2.

Figure 2. Locations of historical rock slides in the county of Møre og Romsdal. Skafjellet (year 1731), Tjellefjellet (1756), Loen (1905 and 1936) and Tafjord (1934) generated destructive tsunamis killing 224 people. At Åknes, a tsunami generating slide is feared.
Data available on the historical rock slides and rock slides in general since deglaciation of Norway used to be sparse. In the 90’s a systematic study on rock slides and their hazard started by the Geological Survey of Norway, and later also by the International Centre for Geohazards. These studies and investigations in fjords and onshore have shown numerous rock slide deposits, in addition to a series of large-scale unstable mountains slopes along valleys and fjords. (Blikra, L. H. et. al. 2004, 2005a and 2005b, Braathen, A. et. al. 2004 ). Some of these unstable rock slopes present a threat to people, buildings and infrastructure.

The most detailed investigations have been conducted at the Åknes rock slope (Figure 2). The first investigations started in the late 80’s (NGI 1987 and 1989) after local authorities had been informed that a well known crack in the rock slope was widening. The first reports were followed up by installation of some bolts for monitoring movements over the crack (NGI 1987 and 1996). The Åknes/Tafjord project was initiated in 2004 aiming at investigations, monitoring and early warning of the unstable slope at Åknes, and also of some slopes along Tafjorden. The responsible for the project is the municipalities of Stranda and Norddal, with the Geological Survey of Norway as the geo-scientific coordinator. The investigations have till now been focusing on detailed lidar survey, geological field investigations, geophysical surveys, core drilling, and measurements of movements. The investigations also include initial studies of the tsunami generating potential of a 35 mill. m³ rock slide at Åknes. In this study the maximum water surface elevation estimated to 90m, and maximum run-up heights are estimated to more than 100m across the fjord. Maximum run-up heights are roughly estimated to 25 – 35m in Hellesylt (Figure 2) and 2 – 40m in the other settlements along the fjord. The tsunami will strike Hellesylt five minutes after its generation (NGI 2005). It is preliminary estimated that 600 to 1200 people may stay in the tsunami hazard zones as an average over a year (Åknes/Tafjord project, unpublished). During the tourist season the number of people at risk can be several thousands.

The Åknes landslide area indicated in Figure 3 is estimated to approximately 800,000m². The slope is dipping towards SSE with dip angle of 35 – 40°. Just below sea level the slope flattens to about 20°. Single open cracks and areas with several open cracks, indicating movements, are found many places in the slope. Three historical rock slides are known in the Åknes rock slope, all of them from the western flank. The approximate dating of these slides is as follows: 1850 – 1900, 1940 and 1960.

Figure 4 shows details of the western part of the upper crack. The upper western flank is separated from the back wall by about 20 – 30m. To the east of the upper western flank the minimum horizontal crack width of the upper crack is typically around 1m.

The present paper aims primarily on describing data on displacements and ground conditions as a basis for future stability analyses of the Åknes rock slope.
2 DISPLACEMENTS

2.1 Across the upper crack

The first three extensometers for automatic reading were installed in 1993. Each extensometer is fixed in solid rock at both sides of the upper crack and it measures the distance between these fixed points. Measurement takes place once a day through 40 readings in a short period of time of which the mean value is stored in a computer as the recorded value. The extensometer monitoring programme was extended with two more extensometers in 2004. The locations of the extensometers are shown in Figure 5. The monitoring results are shown in Figure 6 and 7. Figure 8 shows the results of manual measurements which started in 1986. The manual measurements were carried out by measuring manually the distance between fixed bolts on each side of the upper crack with a measuring rod.
Figure 5. Ortophoto of the Åknes Landslide area. The extensometers along the upper crack are marked with circles and numbers (Nos. 1 – 5). A detail of Ext 5 is shown. Locations of core borings are marked with triangles. (U = upper, one boring, M = middle, two borings and L = lower, one boring).
Figure 6. Displacements at the upper crack. Extensometer readings from 1993-08-28 to 2005-11-25. Ext 3b is a replacement of Ext 3a which was destroyed. Ext 3b is aligned more parallel to the slope movement than Ext 3a, which means that Ext 3b picks up a larger portion of the movement.

Figure 7. Displacements at the upper crack. Extensometer readings from 2004-11-25 to 2005-11-25.
Figure 8. Displacements at the upper crack. Manual measurements from 1986-08-26 to 2004-06-22.

With reference to Figure 5 Mp 1 is placed 10m east of Ext 1, Mp 3 is placed a few metres east of Ext 1, and Mp 2, Mp 4a and Mp 4b are placed a few metres west of Ext 4.

Figure 9 sums up the displacement in terms of mean displacement per year for all the measuring locations at the upper crack. The recorded values have been adjusted to an assumed direction of slope movement. This adjustment should not be regarded as a "correct" adjustment, but is believed to give a better comparison of the results than the measured values since some of the measuring directions are quite oblique to the assumed direction of movement, meaning that they only pick up a fragment of the real displacement. The mean values of the adjusted values for all measuring locations are 21.7mm/year, in the westernmost part 25mm/year and in the eastern part 17.8mm/year. Ext 5 in particular draws down the mean value of the eastern area. Ext 5 is located about 30m west of the point where upper crack dies out as a clearly visible open crack. From Figures 6 – 8 it is clear that the displacements at the upper crack go on with an overall steady pace. Some periods with faster movements and some periods with slower movements can be identified, but there is no general tendency of acceleration or deceleration. It may be noted that some places, near the upper crack, narrow cracks sub-parallel to the upper crack, exist. Up to now, these cracks have not been monitored, but their existence shows that the total displacement in the upper part of the slide area is somewhat larger than shown in Figures 6 – 9.
2.2 At the slope surface

2.2.1 Measuring methods

Several methods are used for measuring movements in the slope: GPS, total station, radar and photogrammetry. This paper presents some main results of the photogrammetry. The photogrammetry covers a period of 43 years.

2.2.2 The photogrammetric method

Photogrammetric studies have been conducted for the periods 1961 – 1983 and 1983 – 2004. Aerial photographs of the scale 1:15000 of 1961 and 1983 were used to make elevation models (Digital Terrain Model) and orthophotos of pixel size 20cm by use of software from ZI-Imaging. For 2004 an orthophoto produced by FUGRO was used. On the orthophotos points that appeared identical have been located, i.e. mainly rock blocks. The coordinates of the apparently identical points have been used to calculate possible displacement vectors. For the 1961 and 1983 aerial photographs the coordinate system was identical. The 2004 orthophoto refers to a different coordinate system, which made it necessary to establish a transformation between apparently identical points in the 1983 and 2004 orthophotos. This transformation routine was established by use of points in the border area of the area covered by the orthophotos. At these points the displacements were presumed equal to or close to zero.

Since points at the slope surface are compared, the photogrammetric method does not distinguish between movements that take place just below the slope surface, e.g. solifluction, and surface movements that are a caused by movements at deeper levels in the slope. The accuracy of the method is estimated to be 0.5m.
2.2.3 Results

93 points were analysed in the period 1961 – 1983, of which 62 points showed displacement larger than 0.5m. 122 points were analysed in the period 1983 – 2004, of which 73 points showed displacement larger than 0.5m.

Table 1 sums up the results for the two periods based on points that showed displacement larger than 0.5m.

Table 1. Displacements derived from photogrammetric studies. Values are given as cm/year, that is the total displacement over the whole period divided by the number of years.

<table>
<thead>
<tr>
<th>Period</th>
<th>Mean</th>
<th>Median</th>
<th>Maximum</th>
<th>Variation coefficient (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1961 – 1983</td>
<td>Magnitude: 6.4 cm/year</td>
<td>4.6 cm/year</td>
<td>17.0 cm/year</td>
<td>63</td>
</tr>
<tr>
<td>1983 – 2004</td>
<td>Magnitude: 5.9 cm/year</td>
<td>5.8 cm/year</td>
<td>13.6 cm/year</td>
<td>46</td>
</tr>
<tr>
<td>1983 – 2004</td>
<td>Dip direction: N192º</td>
<td>N188º</td>
<td>45</td>
<td></td>
</tr>
</tbody>
</table>

Table 1 indicates that the displacement rates on average have been quite stable from 1961 to 2004. It should be noted that displacement measured over the upper crack (Figure 9) are smaller or near the accuracy of the photogrammetric method. In other words; the numbers in Table 1 are almost entirely derived from points that have moved more than upper crack has widened. The results of the photogrammetric studies reflect mostly (but not only) displacements that have taken place in the upper western part of the landslide area, with the largest displacements taking place in the western flank. The other measuring methods (GPS, total station and radar) all demonstrate that the largest movements take place in the western flank, and that the movements are in the order of 10cm per year.

2.3 Evaluation of stability based on displacement rates

Catastrophic failure of creeping slopes is associated with an acceleration phase before the catastrophic failure (e.g. Petley, D. N. et. al. 2002, Kilburn, C. R. J. and Petley, D. N. 2003, Crosta, G. B. and Agliardi, F. 2003). According to idealized creep behaviour the tertiary, accelerating, creep phase, is preceded by a primary, or strain hardening phase, and a secondary steady phase. Use of this idealized creep behaviour on the Åknes rock slope indicates that the rock slope in general must be in the steady (secondary) phase, or perhaps in the primary phase. This implies that there is still some time before a possible catastrophic collapse of the slope. If, or when, the slope will start accelerating is the big question. An extended and improved monitoring programme will be implemented at Åknes for the purpose of an early warning system, and treshold values for different parts of the slope have to be established.

3 GROUND CONDITIONS

3.1 Rock types

The general picture from mapping the rock outcrops in the area is that three gneiss variants exist, namely granitic gneiss (pink with dark minerals), dioritic gneiss (light
grey with dark minerals) and biotitic gneiss (dark). The granitic gneiss may appear both as massive rock and as quite dense jointed along the foliation. The dioritic gneiss appears as massive, and the biotitic gneiss appears as weak layers with dense jointing along the foliation. A quite typical picture of the rock outcrops is illustrated in Figure 10 which shows general massive rock, and layers with dense jointing along the foliation. The foliation joint spacing may be less than 10cm in some places. The granitic gneiss is dominating in three of the four diamond drilled boreholes; the exception is Borehole L1 (Figure 5) where the dioritic gneiss makes up the largest portion of the rock core. A summary of the rock type distribution is given in Table 2. It should be noted that the rock type classification in Table 2 has been simplified with respect to the original core log where the rock type is described by meter: One single meter rock core may in some cases include three sections of rock, for instance two sections with granitic gneiss separated by a section of biotitic gneiss. In such cases the rock type has been classified as granitic gneiss.

![Figure 10. Rock outcrops. Left: dense jointing along the foliation. Right: dense jointing along the foliation with more massive rock above.]

Table 2. Distribution of rock types in the boreholes

<table>
<thead>
<tr>
<th>Borehole No.</th>
<th>Length (m)</th>
<th>Inclination</th>
<th>Granitic gneiss (GG) (%)</th>
<th>Dioritic gneiss (DG) (%)</th>
<th>Biotitic gneiss (BG) (%)</th>
<th>BG/GG and BG/DG</th>
</tr>
</thead>
<tbody>
<tr>
<td>U1</td>
<td>162</td>
<td>Vertical</td>
<td>49.4</td>
<td>22.8</td>
<td>19.8</td>
<td>6.8</td>
</tr>
<tr>
<td>M1</td>
<td>149</td>
<td>60°</td>
<td>57.7</td>
<td>2.7</td>
<td>29.5</td>
<td>8.7</td>
</tr>
<tr>
<td>M2</td>
<td>151</td>
<td>Vertical</td>
<td>43.0</td>
<td>13.9</td>
<td>27.8</td>
<td>13.2</td>
</tr>
<tr>
<td>L1</td>
<td>150</td>
<td>Vertical</td>
<td>16.2</td>
<td>42.6</td>
<td>33.8</td>
<td>5.4</td>
</tr>
</tbody>
</table>

1) M1 and M2 are spaced apart only a few metres.
2) Drilled nearly perpendicular to the slope.
3) Biotitic gneiss in combination with granitic gneiss or dioritic gneiss.

In addition to the rock types listed in Table 2, there are about 1 – 2 % dioritic gneiss in combination with granitic gneiss in the four boreholes. The biotite content in the three rock types has been estimated by visual judgement during logging. The mean biotite content based on all estimates are as follows: 35 % for granitic gneiss, 41 % for dioritic gneiss and 62 % for biotitic gneiss. Probably, this variation of the weak mineral biotite, can explain some of the variation in uniaxial compressive strength (UCS) for the three rock types which have the following mean values: 162MPa for granitic gneiss (23 tests), 134MPa for dioritic gneiss (9 tests) and 113MPa for biotitic gneiss (16 tests).
3.2 Discontinuities

3.2.1 Data collection

Mapping of discontinuities has been carried out by the following methods: measurement of the orientation and spacing of discontinuities in the rock outcrops, core logging and by structural analysis of the Digital Elevation Model (DEM). The structural analysis by DEM is performed by Derron, M.-H., et. al. (2005) for the upper and middle part of the assumed landslide area (the lower part is covered with vegetation), with most data from the upper part.

The field mapping has focused on the typical conditions at each location, which particularly for the foliation and foliation parallel joints means that some data are left out; namely the quite large variation that exist at some locations due to small scale folding.

The diamond drilled boreholes have mainly intersected the foliation. Joints that are not parallel to the foliation are generally quite steep as registered by the field mapping, and these joints have been intersected only to a small extent, due to steep inclination of the boreholes. Since the attempt to orientate the cores during drilling was unsuccessful, orientation data could only be measured in the three vertical boreholes in form of dip angles of the foliation. The foliation dip angles were measured metre by metre in the three boreholes by the following procedure: If it was concluded by visual inspection that the foliation was overall consistent over the metre, one measurement was done. If variations were detected by the visual inspection two measurements were done trying to capture the minimum and maximum values.

3.2.2 Orientation

All orientation data are given as dip direction and dip angle in degrees unless otherwise is assigned.

Derron, M.-H., et.al. (2005) identified three joint sets in the upper part of the landslide area by structural analysis of the DEM (Figure 11). The mean orientations of the joint sets are: J1-N180/45 (foliation parallel joints), J2-260/70 and J3-050/50. Derron, M.-H., et. al. (2005) compared this to field mapping of joints along and near the upper crack and found a fairly good agreement (Figure 12).
Figure 11. A) Orthophoto of the upper part of the Åknes landslide. The white line is the open upper crack. B) Detection of the cells of the DEM which have orientations that correspond to the joint sets J1 (foliation joints, white), J2 (black) and J3 (grey) respectively. From Derron, M.-H., et.al. (2005).

Figure 12. Measurements of the joint orientations of the upper part of the Åknes landslide. Left: DEM analysis. Right: Field measurements (courtesy of Braathen, A.). Lower hemisphere stereographic projections. From Derron, M.-H., et.al. (2005).

246 orientations have been measured by field mapping in the whole landslide area, distributed as 142 foliation parallel joints and 104 joints that are not parallel to the foliation (Figure 13). Figure 13 compared with Figure 12 shows that the joint orientations are much more scattered when measurements from the whole landslide area are included. Figure 13 shows also that the joints that are not parallel with the foliation are generally sub-vertical.
Figure 13. Structural measurements in the landslide area. Top: Pole plot of all the joints. Bottom: Contour plot.

Only foliation parallel joints have been plotted in Figure 14. It is clear from the plot that the global mean vector of the foliation joints (N155°/23°) is nearly parallel with the slope orientation (N157°/34°).
Figure 14. Foliation joints. Top: Contour plot with the global mean vector and the slope orientation. Bottom: Rosette plot.

Foliation joints from different areas are shown in Figure 15. The figure shows that the foliation generally dips quite parallel to the slope in Zones 1 – 3 whereas the foliation dips more easterly and non-parallel to the slope in the upper part along the upper crack (Zone 4).
Table 3 shows the dip angles of the foliation from the field mapping and the core logging. For the calculations reported in Table 3, the borehole data have been treated as follows: Where minimum and maximum values have been measured over 1m core (see Section 3.2.1), three values have been recorded: the minimum value, the maximum value and the mean of the two values. Where only one measurement has been taken, three values have also been recorded such that the each metre of the core has been represented consistently by three values: the measured value and the measured value ±2°.

Table 3. Dip angle of the foliation

<table>
<thead>
<tr>
<th>Dataset</th>
<th>Mean (°)</th>
<th>Median (°)</th>
<th>Variation coefficient (%)</th>
<th>Minimum (°)</th>
<th>Maximum (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole U1</td>
<td>27.5</td>
<td>28</td>
<td>28.8</td>
<td>4</td>
<td>53</td>
</tr>
<tr>
<td>Borehole M2</td>
<td>33.5</td>
<td>33</td>
<td>27.2</td>
<td>8</td>
<td>57</td>
</tr>
<tr>
<td>Borehole L1</td>
<td>34.2</td>
<td>34</td>
<td>32.5</td>
<td>3</td>
<td>70</td>
</tr>
<tr>
<td>U1+M2+L1</td>
<td>31.4</td>
<td>31</td>
<td>31.3</td>
<td>3</td>
<td>70</td>
</tr>
<tr>
<td>Field mapping, all data</td>
<td>39.9</td>
<td>37</td>
<td>38.8</td>
<td>10</td>
<td>89</td>
</tr>
<tr>
<td>(142 measurements)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Field mapping, selected data</td>
<td>35.6</td>
<td>35</td>
<td>32.6</td>
<td>10</td>
<td>70</td>
</tr>
<tr>
<td>(114 measurements)¹)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

¹One series of the field mapping consist only of data collected along the upper crack between 30m west of Ext 1 and some tens of metres east of Ext 3, and these data are not included in the
“selected data”. The reason is that the foliation dips steeper in this area than generally elsewhere in the slope, such that the “selected data” appear more comparable with the borehole data.

Table 3 shows that there is a considerable difference between minimum and maximum dip angles of the foliation for all five data sets. The mean values of the five data sets are quite similar, and they suggest that the dip angle of the foliation is generally of the same magnitude as the dip angle of the slope (35 – 40°).

3.2.3 Jointing in boreholes

The core logging included counting of number of natural joints, length of core loss and length of crushed core, as described below.

Fractures/breakage caused obviously by the drilling itself was not included in the joint count. For the foliation parallel fractures/breakage in the rock cores, it was to some extent difficult to distinguish between a natural joint and a fracture/breakage that was caused by the drilling. Some foliation parallel joints included in the joint count may therefore represent weaknesses along the foliation broken apart by the drilling, rather than natural joints. It should be noted that the majority of joints intersected by the boreholes (and counted during the core logging) are parallel to the foliation. This is certainly due to the drilling direction which favours intersection of the foliation rather than the more vertically inclined joints, but probably also for the reason that the frequency of the foliation parallel joints is higher than for other joints.

Core loss is sections of the borehole where the length of collected material is less than the drilling length. Core loss is assumed to represent weak material (e.g. fine grained material) or even voids (e.g. intersection of an open crack) in the rock mass. The term “crushed core” is used for sections where the collected material appears as rock fragments and/or fines. Crushed core is assumed to represent poor rock mass quality (dense jointing and/or low strength of the intact rock).

The jointing and core loss / crushed core in the boreholes are summarized in Figure 16.
Figure 16 shows that the major part of the core loss / crushed core has occurred from 0m to 50 – 60m depth, and that the joint frequency decreases at about 50m in all four boreholes. It is reasonable to assume that some, perhaps the major part, of the poor rock mass quality leading to core loss / core crushing during drilling is associated with ongoing movements in the slope. It follows from this assumption, that ongoing movements in the slope may be restricted to depths of about 60m.

During drilling loss of water was such a problem in the upper part that all the holes except for Borehole L1 had to be lined. Steel casing was used down to 40m in Borehole U1, 20m in M1 and 30m in M2. L1 was only lined through a few metres of soil. These experience show that the rock mass at these shallow depths is very permeable, which may be interpreted as a broken and disturbed rock mass.

Figure 17 shows the rock type distribution, joint frequency and core loss / crushed core for the full lengths of the four boreholes. Figure 18 shows the same information, but restricted to the upper 60m of all four boreholes. Figure 17 does not reveal any specific trends with respect to core loss / crushed core and joint frequency versus rock type. Figure 18, however, shows that the biotitic gneiss and biotitic gneiss in combination with granitic or dioritic gneiss has more core loss / crushed core than the other rock types, and also; it is more jointed. Figure 19 shows that the dioritic gneiss has most core loss / crushed core when one looks at the total without considering the distribution in the various boreholes. However, this is caused only by the large portion of core loss / crushed core of dioritic gneiss in Borehole U1.
Figure 17. Rock type distribution, joint frequency and core loss / crushed core for the full lengths of Boreholes U1, M1, M2 and L1. Core loss / crushed core is given as percentage of the total length of the rock type in the borehole.

Figure 18. Rock type distribution, joint frequency and core loss / crushed core for the upper 60m of Boreholes U1, M1, M2 and L1. Core loss / crushed core is given as percentage of the total length of the rock type in the upper 60m of the borehole.
3.3 Ground water

The depth to the ground water in the four boreholes has been measured a few times during the autumn 2005 (Figure 20). In this period the depth is around 50 – 60 m in Borehole U1, and around 40 – 45 m in Boreholes M2 and L1. Continuous monitoring of the ground water level in Borehole M2 started in December 2005, and in the period December 2005 – February 2006 the depth to the ground water has fluctuated between 38 m and 40 m. The period of measurement is too short to draw firm conclusions about the ground water, but pretty large depths to the ground water in the slope are certainly indicated.

Figure 20. Depth to ground water in the vertical boreholes U1, M2 and L1.
3.4 Evaluation of ground conditions with respect to stability
The instability of the Åknes rock slope appears to be caused mainly by unfavourable orientation of the foliation in relation to the orientation of the slope. The presence of gneiss rich in biotite may play an important role due to the relative weakness of this rock type. The instability may be restricted to depths of about 60m below the ground. It is suggested that shear movements along the foliation take place at several levels in the rock mass from depths of about 60m and upwards. The possible maximum depth of about 60m may be governed both by the presence of ground water at these depths as well as the slope inclination of 35 – 40°. A possible lower failure plane is indicated in Figure 21.

![Figure 21. Profile trough the central part of the landslide area. Possible lowest level of shear movements are indicated (modified after geophysical survey conducted by the Geological Survey of Norway).](image)
4 CONCLUSIONS

The Åknes rock slope is located in a fjord system where several rock slides have occurred since deglaciation. Three slide events from the western flank are known to have occurred in historical times, the latest event occurred around 1960. These facts combined with the documented ongoing movements define the Åknes rock slope as a hazardous object. Because of the possible large volume involved in a possible catastrophic failure, the tsunami generating potential is large, meaning that people and infrastructure are at risk. The rather steady displacements rates that have been recorded over the years, indicate that the slope is in a secondary, or steady, creep phase, which means that one would expect an accelerating phase prior to a possible catastrophic failure.

The instability of the Åknes rock slope appears to be caused mainly by unfavourable orientation of the foliation compared to the orientation of the slope. The presence of gneiss rich in biotite may play an important role due the relative weakness of this rock type. The instability may be restricted to maximum depths of about 60m below the ground. Displacements along the foliation may take place at several levels above about 60m. Monitoring of displacements in boreholes is needed to verify or reject this hypothesis.

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