Risk-based maintenance of rubble-mound breakwaters

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ABSTRACT: In order to decide on the most adequate time schedule for the maintenance and/or repair works to be carried out on a rubble-mound breakwater, it is of paramount importance to assess the evolution over time of the considered structure as well as to establish its failure probability over a given period of time. A key point here is the evaluation of the operational structure resistance, i.e. after suffering some damage, something which is not yet an issue at the design stage of such structures. To forecast the damage evolution use can be made of the Melby & Kobayashi formula (1999). The paper aims to illustrate the use of a level III probabilistic approach to get better maintenance plans for rubble-mound structures.

1 INTRODUCTION

A good measure of the structure evolution with time can be obtained by surveying the envelope of the armour layer. Although it is common to have surveys of some fixed points of the emerged part of these structures, as a complement to regular visual inspections, the same does not apply to their submerged part. In fact, due to the cost of underwater surveys only a few rubble-mound breakwaters have their submerged part surveyed on a regular basis.

In Portugal, a cheap and efficient solution for this problem is being developed within the scope of the joint industry project code-named MEDIRES that involves the Sines’ Port Authority, the Portuguese Civil Engineering Laboratory, the Institute for Systems and Robotics from the Lisbon Technical University and the Avilés Port Authority. The first objective of this project is to produce a high accuracy measuring device that is made of a mechanically scanned sonar profiler to survey the submerged part of the structure and of a laser range finder to map the emerged part of the structure. The second objective of the MEDIRES project is to develop methods to establish the structures’ diagnosis based upon the wave climate at the structure’s location and on the surveys of the armour layer envelope. The work described here fits within the scope of this second objective.

Although there are several failure modes that can be considered to establish the failure probability of a rubble mound breakwater, for the time being we focus on the failure by hydraulic instability of the armour layer. Melby & Kobayashi (1999) presented a formula that may be adequate for this kind of use. In fact they establish that the damage that may occur at a particular section of a rubble mound breakwater during a given sea-state depends on the damage it suffered before the beginning of that sea-state and on the sea-state parameters, including its duration. This formula allows not only to assess the damage evolution of the armour layer during the structure’s lifetime but also to assess the damage evolution of a structure knowing its present damage only.

In order to evaluate the armour layer failure probability one may use the so-called level III probabilistic approach: a set of series of independent sea-states are randomly generated – according to their distribution probabilities in a selected time interval – the armour layer damage after each of those series is estimated and the number of failures, i.e. the number of times the damage exceeds a preset threshold, in the whole set is counted. The quotient of the number of failures over the number of series gives an estimate of the failure probability.

The objective of this paper is to illustrate the use of this approach in the evaluation of the failure probability of the rubble-mound breakwater of Sines fishing port, in the south-western coast of Portugal. For this, the sea-state parameters distributions are established based upon the sea-state parameters measured at the SINES 1D wave rider directional buoy that are transferred to the study location by means of a numerical model for wave propagation.

After this introduction the paper proceeds with a description of the formulae used to evaluate the damage evolution, namely the Melby & Kobayashi (1999) formula and the critical stability number defined in Smith et al. (1992). Then, the case study is presented and the paper finishes with the conclusions of this exercise.
2 ASSESSMENT OF ARMOUR LAYER DAMAGE PROGRESSION

2.1 Previous formulae

Hudson’s formula (Hudson, 1958) is undoubtedly one of the most commonly used in the design of armour layer elements for rubble-mound breakwaters. The current version of this formula, which has minor differences from the original one, reads as follows:

\[ \frac{H}{D_{n50}} = \left( K_p \cot \alpha \right)^{1/3} \]  
(1)

where \( H \) is a characteristic wave height of the incident sea state, \( \Delta \) is the submerged density of the armour elements, \( D_{n50} \) is the nominal diameter of the same elements, \( K_p \) is the stability number, \( \alpha \) is the angle between the armour layer envelope and the horizontal plane. There is only one load variable (\( H \)) and three variables that characterize the structure (\( \Delta \), \( D_{n50} \), \( \alpha \)). The remaining factors that may influence the relation between the previously mentioned variables – type of armour element used, number of layers and placement of the elements, trunk or head profile, breaking waves – are taken into account in the stability number \( K_p \).

Using previously published results from scale model tests, as well as results from his own experiments, van der Meer (1988) presents a set of equations that relate the armour layer damage with the number of waves, the significant wave height and the average zero upcrossing period during a storm. For rock-armoured breakwaters exposed to waves whose height is not limited by the water depth that relation can be expressed as:

\[ \frac{H}{D_{n50}} = 6.2 \, P^{0.18} \left( \frac{S}{\sqrt{N}} \right)^{0.2} \xi_m^{0.5} \quad \text{for} \quad \xi_m \leq \xi_c \]  
(2)

\[ \frac{H}{D_{n50}} = 1.0 \, P^{-0.13} \left( \frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot \alpha} \xi_m^{P} \quad \text{for} \quad \xi_m > \xi_c \]  
(3)

where \( H \) is the significant wave height, \( P \) is a permeability parameter for the breakwater, \( S \) is the eroded profile area non-dimensionalized by \( D_{n50} \), \( N \) is the number of waves, \( \xi_m \) is the Iribarren parameter evaluated with the average period \( T_m \). The transition between the plunging (\( \xi_m \leq \xi_c \)) and surging breaking (\( \xi_m > \xi_c \)) mode can be estimated by means of the critical Iribarren number,

\[ \xi_{mc} = \left[ 6.2 \, P^{0.31} \tan \alpha \right]^{1/3} \]  
(4)

The parameters of Equations 3 and 4 were fitted using results from scale model tests whose duration equalled 1000 and 3000 average periods (i.e. 2.7 and 8.3 hours, respectively, assuming an average period of 10 s). It must be pointed out that Thompson & Shuttler (1976) presented results from scale model tests with the duration of approximately 5000 waves and in most of their tests it was not possible to conclude that an equilibrium state had been attained.

In addition to those shortcomings, all of the previously mentioned formulae are limited to uniform sea states and undamaged structures. So they cannot be used to forecast damage evolution along the structure’s life.

Although van der Meer’s formulae can not be used to forecast damage evolution, they corresponded to an important improvement of design formulæ. Previous designs formulæ only characterized load by means of the wave height. In van der Meer’s formulæ, load is characterized by means of the wave height and of the wave steepness and sea-state duration.

By introducing the duration of the exposition to the incident sea-state it is clearly assumed that the designed structure should withstand, not a design wave height, but a design “storm”.

Other important improvement in van der Meer’s formulæ: the damage that the armour layer is expected to allow is explicitly assumed. Since the failure of armour layer is progressive (definitive failure of the armour layer is only attained after much damage is accumulated), it is evident that such formulæ should explicitly include a damage related variable.

2.2 Melby & Kobayashi formula

Melby (1999) describes with some detail a set of experiments and a methodology for measuring the armour layer damage in scale model tests. The results from three of these tests, whose total duration reaches 46 hours, were used to fit the parameters of the formulæ presented in the same work. One of those tests was carried out up to the failure of the armour layer, which took place 28 hours after the beginning of that test (i.e. after 60,000 waves hit the structure, for \( T_m \approx 1.7 \) s). The other two tests lasted for approximately 9 hours (which corresponds to 18,500 waves). This means that the formula for the evolution of the armour layer of rubble-mound breakwaters in Melby (1999) is based upon experiments whose duration is much larger than any of the previous formulæ experiments.

Starting with van der Meer formulæ, Equations 2 and 4, Melby (1999) proposes a formula for the damage evolution of an armour layer exposed to sea states that vary with time:

\[ S(t) = S(t_n) + a_n \sum_{i=0}^{n-1} \left( S(t_{n+i}) - S(t_n) \right) \quad \text{for} \quad t_n \leq t \leq t_{n+1} \]  
(5)

where \( S(t) \) is the damage at time \( t \) (an unknown quantity); \( S(t_n) \) is the damage at time \( t = t_n \) (a known
A correction in the model is then due. To reduce the influence of low energy sea states in the simulation of the armour layer behavior under wave action, a wave height threshold below which damage increase can be discarded has to be established. That way, the structure is designed to withstand, not a truly design formula, but a wave height threshold below which damage is unlikely to occur. Since this is one of the objectives of the project MEDIRES, this is the equation used in the remainder of the paper.

Melby & Kobayashi (1999) state that the best fit of Equation 6 to the experimental results were obtained with $a = 0.011$ and $b = 0.5$. It must be pointed out that those experiments were carried out with uniform rock-armoured layer and some of the waves that acted on the slope broke before hitting the slope. As happens with the rest of the formulae for the design of armour layer elements that preceded them, the Melby & Kobayashi (1999) formula is semi-empirical. This implies that different values of the parameters $a$ and $b$ can have to be found if the formula is to be applied to different conditions.

Although Melby formula, and Melby & Kobayashi formula are based on van der Meer formulae, they are not a truly design formulae. They represent a completely new approach to the problem. It is not possible to use those equations to, directly, calculate the dimension of armour layer elements. Only by calculating the armour layer damage progression for the entire structure’s life it is possible to use them for such propose. That way, the structure is designed to withstand, not a single wave event but “storm”, but to “all” the incident sea-states the structure will be exposed to.

It is not difficult to conclude that, even for sea states with low wave heights, Equation 6 predicts an increase in the damage of the structure. Should this sea state last for a longer time interval and the equation produces a cumulative damage that is not likely to occur. A correction in the model is then due.

### 2.3 Critical stability number

To reduce the influence of low energy sea states in the simulation of the armour layer behavior under wave action, a wave height threshold below which damage increase can be discarded has to be established.

Smith et al. (1992) proposed the following critical stability number,

$$ N_c = 0.4 \frac{0.62 \rho^{0.18}}{\sqrt{\sigma_m}} \text{ for } \xi_m \leq \xi_c \quad (7) $$

$$ N_c = 0.4 \xi_m^{b} \frac{1}{\cot \alpha} \text{ for } \xi_m > \xi_c \quad (8) $$

Equations 7 and 8 were obtained by modifying Equations 2 and 3. When the stability number, $N_c$, falls below the critical stability number, $N_c$, the structure is assumed stable and its damage does not increase. Since the instants that matter to compute the damage, $S(t)$, are at the end of each interval where sea wave characteristics can be assumed constant, $S_{n+1}$, Equation 6 can be written as:

$$ \bar{S}_{n+1} = \left( \bar{S}_n^{1/b} + \left(a, N_s^x \right)^{1/b} \frac{T_{n+1} - T_n}{T_m} \right)^{b} \text{ for } N_s > N_c \quad (9) $$

$$ \bar{S}_{n+1} = \bar{S}_n \text{ for } N_s \leq N_c \quad (10) $$

### 3 CASE STUDY

#### 3.1 Description

Since one of the structures being monitored within the scope of the MEDIRES project is the SinesWest breakwater – in fact the surveying device developed in the project is tested in this breakwater – the first test of this level III probabilistic procedure to assess the armour layer evolution should be tested with that structure. However, that armour layer is made of Antifer cubes and these are not the armour elements for which the parameters of Equation 6, or Equations 9 and 10, were fitted – rock.

That is why this test was carried out for the breakwater of Sines fishing port (Fig. 1). The armour layer of this breakwater is made of rocks whose average weight is 45 kN that are randomly placed in two layers in a slope of 1:2 (V:H). Considering a density of 29 kN/m$^3$ the rocks’ $D_{50}$ is 1.16 m. The water depth at the structure’s toe is enough to avoid waves to break before hitting the structure.

#### 3.2 Sea-state parameters transfer

The breakwater of the fishing port is located at a region sheltered by the Sines West breakwater. This implies that the transfer of the sea-wave characteristics measured at the SINES 1D directional buoy, up to the breakwater location, has to be carried out by a numerical model that takes into account the diffraction induced by the West breakwater in addition to the
refraction caused by the bottom depth variation. For this, the numerical model DREAMS (Fortes 2002) is used. This model evaluates the propagation and deformation of monochromatic sea waves in coastal regions. It is based on the elliptic form of the mild slope equation, which describes the combined effects of refraction and diffraction of monochromatic waves that propagate over mild sloping bottoms such as those that occur at ports, harbours and coastal regions.

After a brief analysis of the sea-wave characteristics, measured at the SINES 1D buoy, and of the bottom depth variation in the study area, it was decided to propagate waves with directions between 190° and 330° (with a 10° interval) and periods between 5.5 s and 15.5 s (with a 1 s interval). This means that 165 different waves were propagated.

The computational domain was discretized by two meshes (Fig. 1): grid#1, for periods between 5.5 s and 8.5 s, 158,102 nodes and 313,430 triangular elements; grid#2, for periods between 9.5 s and 15.5 s, 115,431 nodes and 228,372 triangular elements. The annual average tide level +2.0 m (CD) was considered.

For each wave direction, the wave height indexes, the wave directions and the wave crests were obtained in the whole domain as well as the wave heights and wave directions at several selected points near the fishing port (Tables 1 and 2).

By analyzing these tables it is evident that only the waves from South (190° to 220°), specially the longer ones that get amplified, reach the fishing port breakwater without losing most of their energy.

Knowing the wave height indexes, it was possible to transfer the sea-wave characteristics measured at the SINES 1D buoy to the area in front of fishing port breakwater. The sea-wave characteristics corresponding to, approximately, 14.5 years were transferred. The maximum wave obtained, at the area in front of fishing port breakwater, was of 3.53 m. As expected, that wave height is much smaller than the depths of the study domain. This means that, no wave breaking should occur.

### 3.3 Generation of sea-state parameters time series

Most of the armour layer damage will be caused by the sea-states characterized by large wave heights. On the other side, the occurrence of these sea-states is truly exceptional. So, their probability will not be correctly described by a statistical distribution fitted to the entire wave climate. Furthermore, and as described in section 2.3, there is the need to reduce the influence of low energy sea states in the simulation of the armour layer behavior under wave action.

In order to contemplate these facts in the simulation process, only the three-hourly sea-state parameters transferred to the location of the fishing port breakwater for which the stability number, _N_s_, exceeded the critical stability number, _N_c_, were used to define the probabilistic distributions of _H_s_ and _T_m_. It was

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<th>Table 1. Wave height indexes at the breakwater location for wave directions (at SINES 1D buoy) from 190° to 250°.</th>
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<th>Table 2. Wave height indexes at the breakwater location for wave directions (at SINES 1D buoy) from 260° to 330°.</th>
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observed that the critical stability number fell within a quite narrow interval (between 1.09 and 1.39) and so, it may be possible to replace it by a fixed threshold. According to the critical stability number criteria, only during 0.061% of the time is the fishing port breakwater exposed to damaging sea-states.

Using the chi-squared, the Anderson-Darling and the Kolmogorov-Smirnov tests, the fit of several statistical distributions, to the samples of $H_s$ and $T_m$ values, was checked. It was chosen to model: $H_s$ using a Gumbel distribution characterized by $a = 2.97$ m (location parameter) and $b = 0.17$ m (scale parameter); $T_m$ using a logistic distribution characterized by $\alpha = 8.35$ s (location parameter) and $\beta = 0.51$ s (scale parameter).

The software package @Risk was used to get the time series of sea-state parameters in front of the fishing port breakwater. Use was made of the Latin Hipercube Sampling Technique. These time series can be generated disregarding the possible dependence of consecutive values that may be observed in wave records because, the cumulative armour layer damage obtained with the Equations 9 and 10 does not depend on the sequence of sea-states acting on the structure.

3.4 Results

100 simulations of 100 years each were carried out. This implied getting 29,219,400 samples of each of the variables modeled as random ($H_s$ and $T_m$). Equations 9 and 10 were used, after each sampling procedure, to evaluate the damage evolution of the armour layer. Figure 2 presents the damage evolution in 5 of the 100 simulations performed, as well as the curve obtained by averaging those 100 simulations.

Figure 2 shows that damage increments occur at discrete events. It is particularly remarkable that most of the damage is the result of outstanding events. It can be also seen in that figure, particularly in the average curve, that there is a trend for the cumulative damage to slow down as the armour layer becomes damaged. This can be interpreted as approaching an equilibrium state that, however, is never attained.

In none of the simulations the structure failed, according to the threshold suggested by van der Meer (1988) – $S \geq 8$. It must be pointed out that this does not imply that the failure probability of the structure is zero. When the failure probability is very small, the number of simulations has to be increased. Since the objective of this work was not to evaluate the failure probability, the number of simulations was not increased. However, this exercise made evident the small failure probability of the armour layer of the breakwater of Sines fishing port.

To get more insight on the influence of the rock size on the armour layer performance, as well as on the simulation procedure itself, it was decided to repeat the simulations with smaller rock elements (average weight of 22.5 kN and $D_{50} = 0.92$ m). Figure 3, which is similar to Figure 2, summarizes the obtained results. Since the armour rock elements are smaller it is expected that more sea-states will be included. In fact, some of those that were not sampled before, because it wasn’t expected that they would cause any damage to the previous armour layer (45 kN), will certainly cause damage to the armour layer now considered. So, it is required to analyze the transferred sea-state parameters, as described in the first two paragraphs of section 3.3.

According to the critical stability number criteria, during 0.134% of the time the fishing port breakwater is exposed to damaging sea-states. $H_s$ was modeled using a Pearson type V distribution, right-shifted by 0.19 m, and characterized by $\alpha = 32.46$ (shape parameter) and $\beta = 77.36$ m (scale parameter). $T_m$ was modeled using a logistic distribution characterized by $\alpha = 8.01$ s (location parameter) and $\beta = 0.51$ s (scale parameter). Figure 3, similar to Figure 2, shows the obtained results.
The results show that, should the armour layer be made of rock elements whose average weight is 22.5 kN, its failure probability at the end of 100 years would be 78%. Failure probabilities were evaluated also for smaller time spans: \( P_{f_{50}} = 35\% \), \( P_{f_{25}} = 20\% \) and \( P_{f_{10}} = 4\% \). Again, it was assumed that the armour layer fails when \( \Sigma \geq 8 \), according to van der Meer (1988). It must be pointed out that Melby (1999) states that, in some of the experiments that led to the formula and coefficients of Equation 5, damage values of 12 were obtained and no failure of the armour layer (i.e. no exposure of the filter) occurred.

Bearing in mind the objective of getting more insight on the influence of the criteria chosen to limit the influence of low energy sea states in the simulation of the armour layer behavior under wave action – the critical stability number – its value was changed. Firstly, it was increased 10%, being its value calculated through the use of these equations:

\[
N_c = 0.44 \left( \frac{6.2P^{0.18}}{\sqrt{\xi_m}} \right) \text{ for } \xi_m \leq \xi_c
\]

\[
N_c = 0.44 \xi_m^{p} \frac{\cot \alpha}{\beta^{0.13}} \text{ for } \xi_m > \xi_c
\]

After a new analysis of the transferred sea-state parameters, it was verified that, according to the increased critical stability number criteria (Eqs. 11, 12), during only 0.101% of the time the fishing port breakwater is exposed to damaging sea-states. In this group of one hundred simulations, \( H_s \) was modeled using a normal distribution with 2.82 m mean value and 0.37 m standard deviation and \( T_m \) was modeled using also a normal distribution with 8.16 s mean value and 0.91 s standard deviation.

After running one hundred 100-year simulations, failure was never attained. So, according to increased critical stability number criteria, the failure probability is 0%.

By decreasing 10% the critical stability number value, one gets:

\[
N_c = 0.36 \left( \frac{6.2P^{0.18}}{\sqrt{\xi_m}} \right) \text{ for } \xi_m \leq \xi_c
\]

\[
N_c = 0.36 \xi_m^{p} \frac{\cot \alpha}{\beta^{0.13}} \text{ for } \xi_m > \xi_c
\]

According to this decreased critical stability number criteria (Eqs. 13, 14), during 0.235% of the time the fishing port breakwater is exposed to damaging sea-states. This time: \( H_s \) was modeled using a Pearson type V distribution, right-shifted by 1.46 m, and characterized by \( \alpha = 1.52 \) (shape parameter) and \( \beta = 0.37 \) m (scale parameter); \( T_m \) was modeled using also a Wald distribution, right-shifted by 0.2 s, and characterized by \( \mu = 7.54 \) s (location parameter) and \( \lambda = 548.15 \) s (scale parameter). Using the decreased critical stability number criteria, failure was attained in all the simulations, \( P_f = 100\% \).

Another test consisted of replacing the critical stability number by a fixed significant wave height threshold. In order to determine that threshold value, the Hudson’s formula (Eq. 1), and the rock armour layer design recommendations of the Shore Protection Manual (1984), were used. If a 22,5 kN median weight rock armour layer was designed that way, the design sea-state would have been characterized by \( \frac{H_s}{10} = 3.47 \) m and \( H_s = 2.73 \) m. Analyzing again the transferred sea-state parameters it was verified that, during 0.061% of the time, \( H_s > 2.73 \) m. This means that, the new criteria used to limit the influence of low energy sea states in the simulation of the armour layer behavior, is more restrictive than the previous one. So, the failure probability calculated by the simulation process should be smaller than the previous one.

In the simulations whose some results are shown in Figure 4: \( H_s \) was modeled using a Weibull distribution, right-shifted by 2.73 m, and characterized by \( \alpha = 1.52 \) (shape parameter) and \( \beta = 0.37 \) m (scale parameter); \( T_m \) was modeled using using also a normal distribution with 8.34 s mean value and 0.90 s standard deviation. In only 3 of the 100 simulations the armour layer reached failure. Again, it is noticeable that the damage increment occurs at discrete events and it is particularly remarkable that most of the damage is the result of outstanding events.

Five-year simulations of previously damaged armour layers were also run. This kind of application, of the procedure described herein, should be of great assistance on the decision of the most adequate time schedule for the maintenance and/or repair works of rubble-mound breakwaters armour layer.
Figure 5. Evolution of the cumulative damage in five 5-year simulations and average of the 100 simulations performed with a previously damaged armour layer made of 22.5 kN rock elements.

Figure 6. Different time spans failure probability of previously damaged rock armour layers vs. initial damage value.

The damage progression of a 22.5 kN median weight rock armour layer was simulated, using the critical stability number defined by Smith et al., and by modeling $H_t$ with a Pearson type V distribution, right-shifted by 0.19 m, and characterized by $\alpha = 32.46$ (shape parameter) and $\beta = 77.36$ m (scale parameter) and $T_m$ with a logistic distribution characterized by $\alpha = 8.01$ s (location parameter) and $\beta = 0.51$ s (scale parameter). In only 3 of the 100 simulations the armour layer reached failure. The results presented in Figure 5 correspond to the 5-year simulations of an armour layer whose damage at the beginning of that period was $S_0 = 6$.

Due to the fact that Figure 5 concerns only a 5-year period, it is even more noticeable that the damage increment occurs at discrete events and that most of the damage is the result of a few, outstanding, events. In only two, of the one hundred simulations carried out, failure was attained, $P_f = 2\%$.

Other values of initial damage were also simulated: $S_0 = 3.0$ to 7.5. Figure 6 summarizes the obtained failure probabilities. One should note the fact that, since just one hundred simulations were performed the failure probability value resolution equals one percent.

4 CONCLUSIONS

This paper aimed at illustrating the use of a set of formulae for the evaluation of the damage evolution of the armour layer of a rubble-mound breakwater.

This procedure was applied to the breakwater of Sines fishing port and its evolution along 100-year periods was simulated.

The model used to predict the damage evolution, which results from coupling the formula of Melby & Kobayashi (1999) with the concept of critical stability number (Smith et al. 1992), is able to simulate the episodic character of the cumulative damage increase and the trend for the same cumulative damage to slow down as the armour layer becomes damaged.

The proposed model can be used to assess the damage progression of rock armour layers exposed to variable load conditions, although such a load has to be decomposable in constant sea states.

When combined with a sampling technique, the procedure described in the paper can be used to design the armour layer elements and to evaluate the probability of a given solution meeting the design criteria. Moreover, this procedure can be of great assistance to schedule the maintenance or repair works of an armour layer since, as demonstrated, it enables the simulation of the damage evolution of structures with some initial damage.

It should be borne in mind that the damage evolution model is only valid for structures similar to those of the example described here. Should there be the need to use it for structures whose armour layer is not made of rock, it is necessary to check the applicability of the model and possibly to adjust the parameters of Equation 9 with a new set of experimental results.

In addition, and especially because it became evident that the results are highly influenced by the stability criteria, the validity of the critical stability number should be checked and the possibility of replacing the literature formulae by a structure-related threshold investigated.

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