# Reliability Analysis and Deterministic Validation with Experimental Data of a Historical R.C. Bridge.

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**SUMMARY:**
In a project developed in north-east Italy, a R.C. bridge was subject to analytical and dynamic investigations to evaluate static and dynamic characteristics, due to live-cycle damage. It has a typical cross-section with girders bearing a thin slab and characterized by three spans. A reliability analysis has been done using the OpenSees reliability package. At critical cross sections the reliability index has been calculated to assess safety. At a second stage the analysis has been updated with data about material. Severe damage on the edge girders was revealed recently, so lots of destructive tests and ambient-modal-tests were executed. The vulnerability analysis showed consistent deficiency and suggested the installation of a SHM system.

*Keywords: Safety evaluation, R.C. bridge, damage assessment, destructive tests, dynamic identification, SHM.*

## 1. INTRODUCTION

To contain the inconveniences due to a live-cycle damage it is necessary to study in depth the causes of damage of infrastructures and in particular of bridges. Old and historical reinforced concrete bridges currently represent a great proportion of the European road and railway bridge stock; the public network authorities are responsible not only for routine maintenance but also for retrofitting interventions. For many bridges, the intrinsic weakness of some structural elements, the deterioration occurrences and the updating of structural codes, evidenced inadequate structural performance and necessity to be upgraded to the standards of the current seismic codes.

The assessment of the actual structural behaviour by means of experimental and theoretical investigations helps in choosing the proper intervention both in terms of materials and application techniques. In this framework structural characterization via dynamic tests is immediate and cheap for reinforced concrete bridges, in order to help understand the structural behaviour and damage of the structure before taking any decision of drastic intervention on it.

In the context of a large project developed between the University of Padua and the Regional Road Authority of Veneto, in the East-North of Italy, a great number of bridges were subject to structural investigation in order to examine their safety evaluation. The project initiated with the visual inspection and cataloguing of a total of 500 bridges of various structural kinds from masonry to reinforced and steel bridges. The further step was to extend to 80 bridges the investigation by destructive and non-destructive tests in order to increase the material characterization of all kind of structures available in the database. After that a simplified procedure to evaluate the structural behavior of the entire fixed network has been developed by the team of University of Padua. During this step parameterization of the safety levels and indexing of the structures not satisfying the National Code levels was carried out. This latter phase stimulated also the Individuation of the first bridges to execute deeper structural identification and to simulate the response through numerical models.

In particular, the work presented here is concentrated on a reinforced concrete bridge highly damaged.

Due to non-workmanlike details and not scheduled maintenance since constructed in 1945, recently the bridge has revealed severe damage on the edge girders at the middle of the spans. Before taking any provision or deciding to retrofit it, the state of the bridge has been analysed. It includes the description and more importantly the damage of the structure but also the dynamic characterization and sensitivity analysis done in order to best understand the bridge's reserve in resistance terms.

A reliability analysis has been performed by means of OpenSees Reliability package. The reliability index in critical cross sections has been calculated to assess safety of entire bridge. In a second stage data about materials and geometry have been obtained from in-situ testing and the analysis has been updated.

## 2. DESCRIPTION OF THE BRIDGE

The original bridge situated in the centre of Verona in Italy was destroyed in 1945 by the Second World War bombarding and a new bridge was completely rebuilt the following year. The main structure component is a reinforced concrete segmental beam with external stone facing.

The structure bears a four-lane roadway, two for each direction of travel, plus two sidewalks with a cross section of 14.32 m and a slab of 18cm thick. The static schema consists of seven longitudinally main girders and eleven cross beams with stiffening function, connecting all main girders in the transverse direction. There are also some other cross joists at regular intervals between the five central longitudinal beams.

**Figure 1.** – The analysed bridge (photo 2011).

The bridge has a total length of 90.5m; it consists of three spans of segmental tapered beams: the central span is 33.50m long, while the two side span 27.0m each .The height of the sections of the "arched" beams is 1.0m at the key positions and 1.5m at the bases.
The bridge was designed as a continuous beam simply supported at the two piers; at both ends (abutments), the bridge is simply supported and left free to move, as well as at the piers.

A two-day detailed experimental investigation was also carried out in order to provide all the dimensional information about the bridge structural elements and the materials characterization.

## 3. DAMAGE DETECTION AND MATERIAL CHARACTERIZATION

On June 2011 a survey underneath the deck was carried out, in order to obtain a general framework for structural damage and material deterioration. It revealed severe damage on the longitudinal edge girders at the middle of the spans, in particular at the points where the bridge deck drains are located, due to water percolation (Figure 3).

To best evaluate the condition of the structure and use best fitted material properties for the structural analysis, in every element of the structure were executed lots of destructive and non-destructive tests as pull-out, rebound hammer and core sampling technique and pachometer tests.

The main girders and cross beams are highly deteriorated, the concrete is mouldy and carbonated. Moreover, some localized shear cracks have been detected at centreline of cross beams. The reinforcing steel bars are fully visible where water percolates, as the concrete cover has been eroded by water and pollutant chemical elements as antifreeze salts. A strong corrosion of steel bars, in addition to concrete carbonation, determines a reduction of net resistant beam section.

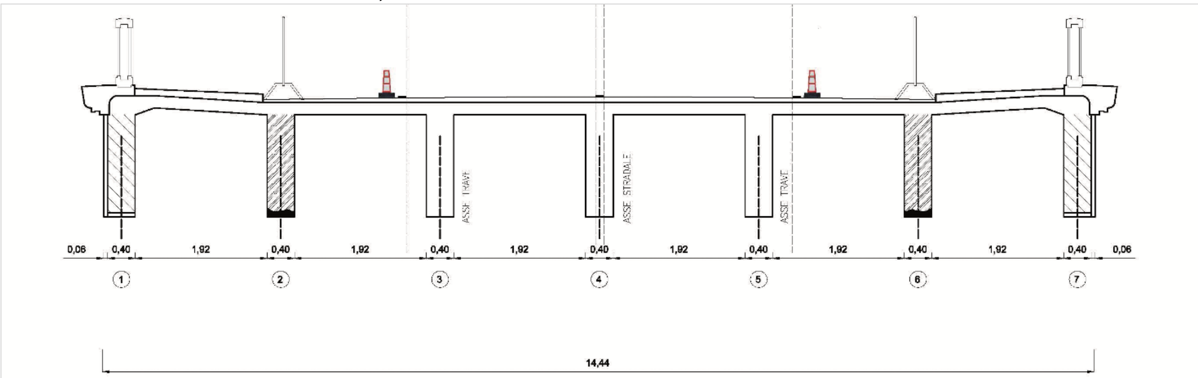


**Figure 2.** – Cross section of bridge.

**Figure 3. -** Damage on the longitudinal edge girders; Core sampling; Laboratory tests.

Four kinds of rebar were extracted and subjected to tensile tests which revealed an actual tensile strength of 360.2MPa and a Young's elastic modulus of 144000MPa. In addition, 10 circular concrete core samples were taken and subjected to compression tests resulting in 21.9MPa for the main girders and 19.1MPa for the cross beams, while the secant elasticity modulus is 33040MPa. All destructive tests were performed at the Laboratory of Structural and Transportation Engineering, University of Padua, Italy.
In Figure 4 it is shown the summary of the levels of deteriorating with five colors, having indicated with red the worst case. It can be observed that the girders number 2 and 6 are the most damaged due to the water percolation from the above lanes.


**Figure 4 -** Outline of the state of degradation

## 4. DYNAMIC TESTING

Dynamic identification is a well-known technique that allows to determine modal parameters (natural frequencies, mode shapes, damping ratios) from experimental data. There are many experimental modal identification methods; among others, an Output-Only technique, or Operational Modal Identification, consists in the characterization of modal parameters measuring the only response in terms of random environmental excitation condition: the excitement (input) is not known; the natural excitation (wind or vibrations induced by traffic) is used as the excitation source in order to capture the response of all possible modal contributions.

During May 2011 two acquisition campaigns were carried out and piezoelectric accelerometers with vertical axes were used to measure the bridge's response. The flow of vehicles over the bridge was normally allowed in a single direction of travel for logistical reasons during the two-day dynamic data acquisition. The response of the bridge was recorded using high-sensitivity piezoelectric acceleration transducers, which can record the vibration and are connected by coaxial cables to a computer equipped with the data acquisition board (Figure 5). The ambient accelerations time series were recorded for nearly 11 minutes with sampling frequency of 100Hz.

### 4.1. Modal identification results

The extraction of modal parameters from ambient vibration data was carried out by using the Frequency Domain Decomposition (FDD) developed by Brincker et al. (2000), and the Stochastic Subspace Identification (SSI) method (Peeters & De Roeck 1999).
The analysis included frequencies corresponding to the first 11 Eigen-modes, but considering more accurate the first 5 values. In Figure 8 the singular values and the mode identification are shown. It can be seen that the two plots (FDD and SSI method) present very similar features and comparable natural frequency values. For both methods identified frequencies and damping ratios are listed in Table 1.

**Table 4.1.** Correlation between numerical and experimental modal behavior (MAC).

| Mode | FDD f | FEM f | Δ | MAC |
|---|---|---|---|---|
| | Hz | Hz | % | Adm. |
| 1 | 4.980 | 4.958 | 0.44 | 0.952 |
| 2 | 6.250 | 5.654 | 10.55 | 0.866 |
| 3 | 6.738 | 5.895 | 14.29 | 0.827 |
| 4 | 7.422 | 6.605 | 12.37 | 0.920 |
| 5 | 8.691 | 7.510 | 15.73 | 0.860 |
| 6 | 8.960 | 7.510 | 19.31 | 0.760 |

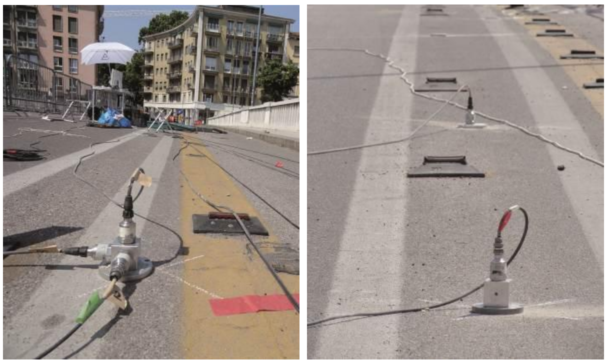

**Figure 5. -** Installation of sensors during the ambient vibration testing.

## 5. FEM CALIBRATION AND COMPARISON WITH EXPERIMENTAL DATA

During the study of the bridge over the river Adige several 3D FE Models were implemented using essentially simple elements. Specifically, the main girders, the cross beams and the cross joists have been modelled as “beam” elements, while the reinforced concrete slab and the piers as 4-node “plate” elements, connected to the deck by “rigid link” elements. The two segmental arched beams have been obtained by considering a variable beam section. Geometric and materials characterization has been performed according to the surveys carried out in situ and the lab tests. A preliminary analysis revealed that the F.E. model initially was underestimating real deformations and the actual stiffness of the bridge.

Therefore the influence of a possible change in the system constraints modeling was evaluated by a sensitivity analysis on the elements and the restraints of the model.
Consequently the FEM model have been updated by fixing horizontal translations at the connections, since no longer exists the possibility of translating there, letting however the rotational DOF be free.

Experimental identified natural frequencies carried out by the two above-mentioned methods have been compared with the finite element model results. Only the frequencies relating to the first 6 vibration modes were considered in the results analysis. As expected the principal natural frequencies for both FDD and SSI method were identified between 0 and 10 Hz.
Considering the obtained results, for the first six main vibration modes, the Modal Assurance Criterion indicates a good correlation between FEM and experimental modes, with MAC values approximately equal to 0.8-0.9 in all six cases, as shown in Table 1.

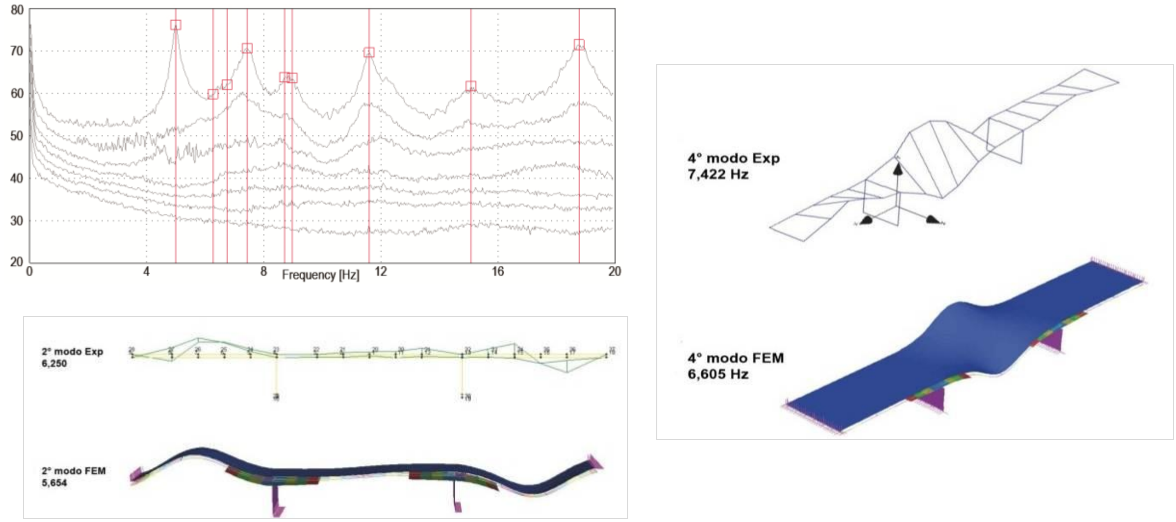


**Figure 6. -** PSD graphs and natural frequencies identification,
Comparison between FEM and experimental identified mode shapes

## 6. SAFETY AND PARAMETRIC ANALYSIS

In order to study in more detail the structural response of the most deteriorated elements some validations have been conducted under the Italian Codes (NTC 2008) considering a reduced section of concrete, in order to simulate the behavior of the degradation, mainly due to the carbonation concrete and corrosion of reinforcing bars. Thus, the resisting moment has been plotted against the variation of the resisting concrete cross section and the number of steel bars present, considering the sections more stressed as is the middle of central span.

It was noted (even through some other statistical approaches) that the section of the longitudinal beam number 6, the most deteriorated, resists, albeit slightly, for the entire section. Considering 3 rows of bars instead of 4, the section does not withstand the loads, surely even with the elimination of amplification factors of the Limit States (Figure 7).

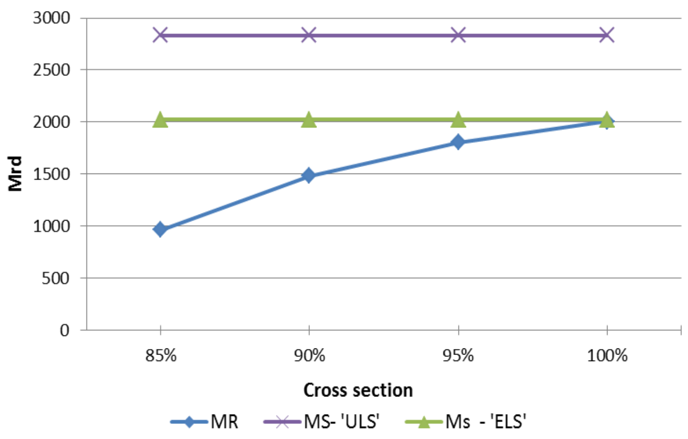


**Figure 7.** Beam nr. 6, section 4. Comparison. Mrd for the different levels of cross section.

### 6.1. Reliability analysis

According to Stewart MG et al. (2001) the reliability analysis can be done considering three different levels.
The level 1 approach accounts uncertainties using partial safety coefficients to be applied to characteristic values of parameters; using level 2 the uncertainties about dimensions, load and resistance are considered through the statistical distribution. In third level a numerical analysis needs to be used to compute the actual reliability index.
The problem of structural reliability is, in general terms:

$$g = R - S \tag{6.1}$$

where $g$ is the performance function, $R$ is the resistance and S the solicitation. The probability of failure is:

$$P_F = prob\ ( R - S < 0 ) = prob\ ( g < 0 ) \tag{6.2}$$

$$P_F = \int_\Omega f_X(X)dx \tag{6.3}$$

$$P_F = F( - \beta ) \tag{6.4}$$

where $f_X\ (X) = f_X\ (x_1,\ x_2,\ \ldots,\ x_n\ ),\ x_i$ is a stochastic variable and Ω is the region where collapse occurs.

To solve Eq. (6.4) Montecarlo method can be used with high computational costs. To reduce computa-tional effort the Latin Hypercube sampling optimiza-tion can be used. Furthermore Rosenblueth point es-timation method can be considered to sample the response function in 2k+1 key points. With this method there is no need to know the input variables stochastic distributions and the total number of sam-plings is less than Montecarlo method and Latin Hypercube sampling. Assuming that resistance and solicitation are sta-tistically independent, reliability index becomes:

$$\beta = \frac{\mu_R - \mu_S}{(\sigma_R^2 - \sigma_S^2)^{1/2}} \tag{6.5}$$

where μ is the mean value of R and S, respectively, and σ is its standard deviation. In this paper the OpenSees (2009) code, along with its reliability package, has been used to perform the analysis. Stochastic parameters have been described with their statistical distribution and values of mean and standard deviation. FORM analysis has been chosen for the analysis in this paper. A planar model has been used to describe the bridge structural behavior. Beams elements have been used to model the piers and beams of the bridge (Figure 8).

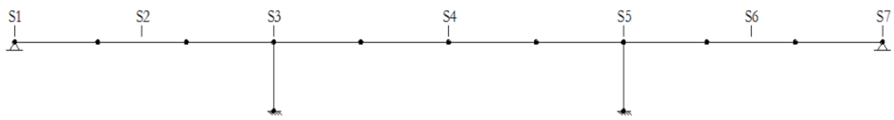


**Figure 8.** Simplified Fem model of bridge. (Sx denotes the section of analysis).

Several load cases were analyzed to maximize response on typical cross section of the bridge. In Figure 9 the load combination 1 for maximum bending moment in central span, and in Figure 10 the load combination 2 for maximum shear at first pier are shown.

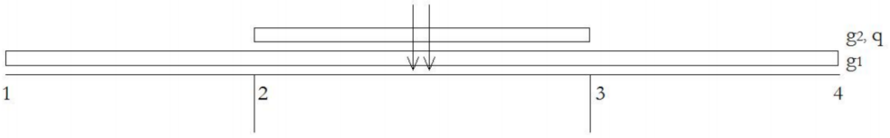


**Figure 9.** Load combination 1: maximum bending moment in second span.

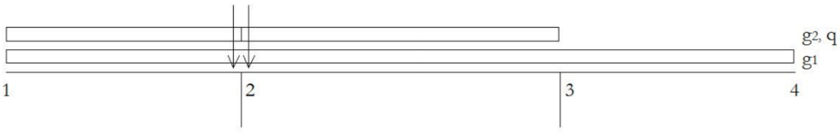


**Figure 10.** Load combination 2: maximum shear at first pier.

Structural uncertainties have been considered in relation to the elastic modulus, inertia moment and area of sections. All parameters have been described with normal distribution. For each analysis a performance function has been used:

$$g_i = 1 - \frac{S}{R} \tag{6.6}$$

where S is solicitation and R resistance. Structural damage has been taken into account us-ing the degradation model for steel corrosion of Du et al. (2003). Data from visual inspection has been used to determine the depth of concrete cover degra-dation and rebar corrosion.

The value of β = 3.8 corresponds to a probability of collapse of 10-3 as stated in Eurocode 0 (2002). Reliability analysis has been performed under various hypotheses about data dispersion. Before performing field-testing on the bridge the following values of the c.o.v. (coefficient of variation) have been assumed: 5%, 10%, 15%, 20%. In Figure 11 the value of β along the bridge is shown.

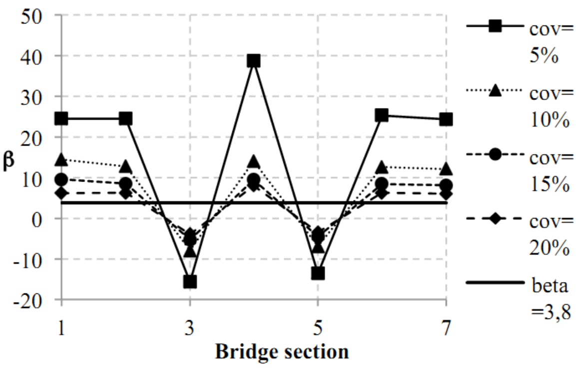


**Figure 11.** Reliability index across bridge span.

When the uncertainty about structural parameters decreases, the bridge reliability index also decreases.

The bridge reliability index calculation can ben im-proved by acquiring more data to reduce uncertain-ties of structural parameters. Bridge reliability index varies with the number of variables related to mate-rials’ characteristics and cross-sections’ geometry used for the analysis (see Figure 12 for combinations 1 and 2). Using the damage model for the rebar corro-sion, the reliability index varies with time. In Figure 13 the trend of the reliability index for a performance function where the bending moment is the accounted parameter, shows that it goes under the level of 3.8 for an age of 42 years.

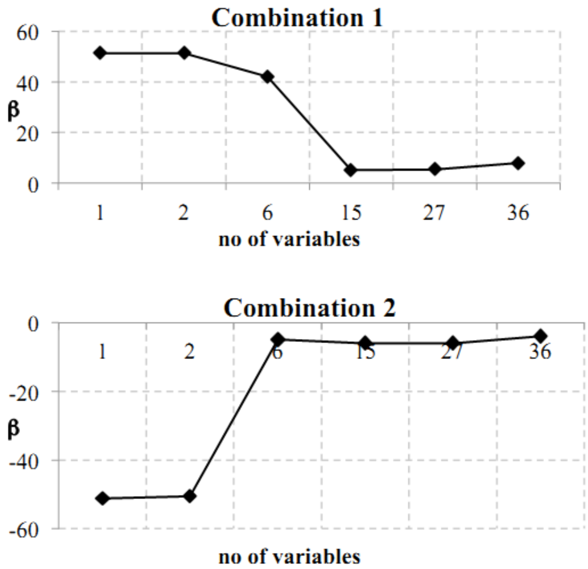


**Figure 12.** Variation of reliability index vs. the number of variables ac-counted in the combinations 1 and 2.

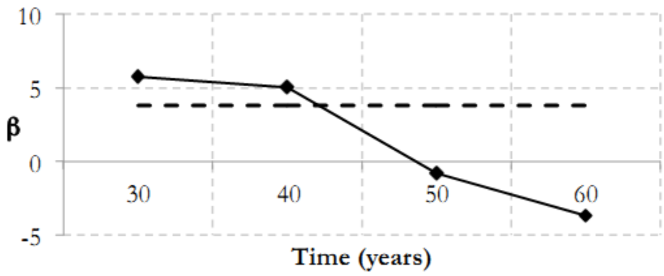


**Figure 13.** Variation of reliability index vs. time.

## 7. STRUCTURAL HEALTH SYSTEM INSTALLATION

After this latter affirmation it was necessary to take some more safe provisions before ensuring the investment necessary for the retrofitting intervention. It was proposed to install a monitoring system to evaluate the general behavior of the structure but also the local displacement at the damaged elements. The system (Figure 14) is composed by displacement potentiometers, strain gages, accelerometers, temperature and humidity sensors. It will help the bridge manager to evaluate the current conditions of the structure and to adopt the best decisions in order to improve the traffic of the city center.

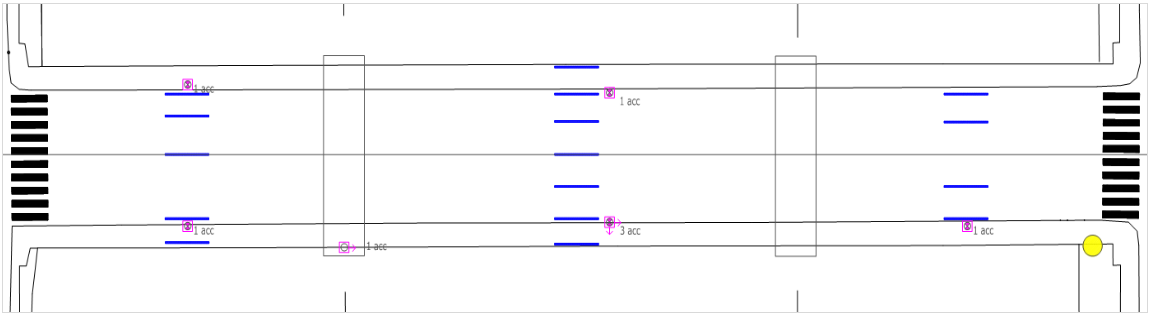


**Figure 14.** Scheme of the SHM system on damaged bridge.

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