Seismic assessment of old existing masonry arch bridges: an application to a case study

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ABSTRACT
Masonry arch bridges are widely spread in Europe and they still serve nowadays many roads characterized by an important vehicular traffic. Moreover, most of them result inadequate since they have been designed mainly for vertical traffic loads by neglecting the lateral earthquake action. For this reason the seismic assessment of old masonry arch bridges becomes a complex matter of structural engineering in conjunction with the fact that, very often, the materials deteriorate and worn away because of cyclic vertical loads and weathering conditions.
This paper presents the seismic assessment of an Italian multispan masonry arch bridge built before the Second World War. The bridge is actually in service and crosses the "Cavone River" (in the Southern Italy), from which takes the name. A series of nonlinear analyses are performed in order to investigate the nonlinear response of the bridge and for evaluating its seismic performance.

1 INTRODUCTION

One of the actual engineering challenge is the conservation of old masonry arch bridges built in the last centuries. Most of them nowadays are located along important roads characterized by transit loads heavier and more frequent than the ones of past. Furthermore, these masonry arch bridges were designed without considering any lateral force and, consequently, to date they result very vulnerable in areas with high earthquake intensity zones. Thus, the seismic response prediction through numerical simulations of old existing arch bridges becomes a very important issue of the structural engineering.

It is known that the response of masonry arch bridges under seismic loads depends by many and different factors, that furthermore differ from case to case: texture, integrity, and quality of masonry, difference in and deterioration of materials, structural geometry, connections among the bridge elements. It is evident, hence, that for properly assessing the seismic response of these structures is necessary to define clear modeling criteria that allow us of simulating the actual response of the bridge, without any loss of objectivity of the numerical solution.

To date, many and different modern modeling strategies have been proposed Among them, without any doubt, it is worth noting the refined two or three dimensional models taking into account the material nonlinearities. They have the disadvantage of requiring a high computational cost but, at the same time, the advantage of giving detailed informations on local response as well. Very often, however, one may be interested in evaluating only the nonlinear global response of the bridge, considering all local mechanisms inhibited with specific interventions. In doing so, the numerical model may be simplified with the aim of facilitating its implementation and control, and of performing nonlinear analyses numerically acceptable from a computational standpoint.

At this scope the displacement-based design methods, proposed for example by ATC-40 (1996), NTC-08 (2008) and FEMA 273 (1997), represent an attractive tool for seismic assessment of a structure. They combine the static nonlinear pushover analysis with the response spectrum approach.

Figure 1. An overview of Cavone Bridge.

Figure 2. Particular of a main brick masonry arch.

This paper shows the seismic assessment of "*Cavone Bridge*", an italian multi-span arch bridge still in service crossing the Cavone river (into the province of Matera), from which takes the name. A series of nonlinear pushover analyses are performed in order to investigate the nonlinear response of the bridge and for evaluating the seismic performance in according to the ATC-40 (1996) method. The bridge is analyzed by referring to the current state, and considering some interventions designed for the conservation of the masonry structures. In another paper (Laterza et al. 2015) the results of fatigue assessment of the bridge are presented.

2 CASE STUDY DESCRIPTION

The "Cavone bridge" is a multispan masonry arch bridge located in the province of Matera (Figure 1). It is actually in service and was built before the Second World War (approximately during the decade of 1930s). The bridge takes the name from the crossing river, and consists of seven brick masonry arches (Figure 2) covering an overall length of 140 m, with a width of 5.6 m.

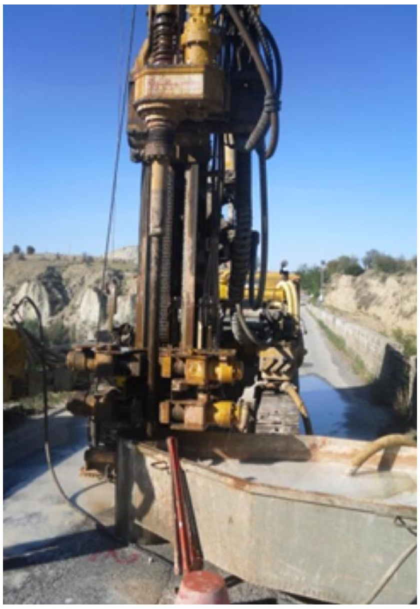

Figure 3. Core sampling from the bridge deck.

Figure 4. Spandrel wall of regular stone blocks.

More in detail, the bridge has three main arches of 22 m span length and four secondary arches of 10 m span length. The three main arches are supported by two piers sunk into the riverbed, having a total height from the foundation plane of about 24 m, of which 14 m above the river bed.

The bridge has been interested by some in situ tests mainly addressed to identify the materials typology and the elements thickness. In particular, several core samplings in different sampling points have been carried from the bridge deck to below, reaching in correspondence of the piers 30 m in depth (Figure 3). The in-situ tests have highlighted that the piers are made by an external layer of regular stone blocks containing a core of cohesive backfill. Whereas, the traffic loads on the roadway spread with depth to the arches through an incoherent backfill, confined by spandrel walls in regular stone blocks (Figure 4).

To date, no in-situ test for determining the materials mechanical properties has been performed. Therefore, the material properties assigned to the bridge elements are default values in accordance with standards of the construction time indicated in commentary to NTC-08 (Circ. 2009, n.617) for existing masonry. On the basis of geometrical, details and material informations collected in this work a limited knowledge level (KL1) is achieved. Therefore, the confidence factor CF = 1,35 is applied in reducing the materials strength. In Table 1 are summarized the materials properties assigned to the bridge elements.

3 NUMERICAL MODEL

Nonlinear analyses have been conducted with a finite elements model of the Cavone Bridge (Figure 5) implemented in CSI BRIDGE v. 15.0. Linear brick elements have been used for modelling the backfill of arches and piers/abutments. Whereas, nonlinear layered shells elements have been used for modelling the stone blocks external leaf of piers/abutments and for the spandrel walls. Figure 6 shows the uniaxial stress-strain relationships in compression assigned to the arches and piers/abutments masonry. It has been assumed an uniaxial linear elastic stress-strain relationship with an ultimate strain assumed equal to 0,35%, as suggested in CNR-DT 200 R1(2013). In order to avoid solution convergence problems, after a descending branch with a slope equal to the initial stiffness, a residual compressive strength equal to 1/10 of the strength peak has been assigned to the stress-strain relationships. In Table 2 are reported the strength in tension and compression assigned to the masonry.

In Figure 6 is also reported the stress-strain in compression if one considers a deep repointing of arches mortar joints. This intervention has been considered for the bridge arches conservation and, as indicated into the Instructions of NTC-08 (Circ. 2 Febbraio 2009, n. 617), it results in an increasing of 1.5 times the current compressive strength.

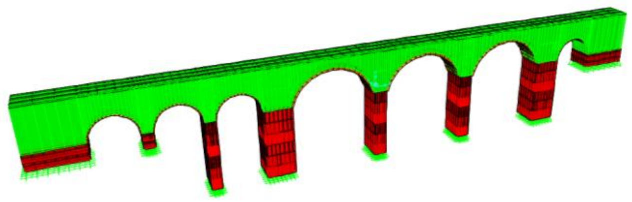

Figure 5. Finite elements model adopted for the Cavone bridge.

Table 1. Materials properties assigned: Young modulus (E), material density (γ) and Poisson ratio (ν).

Component	γ (kN/m^3)	E (MPa)	ν
Arches masonry	18	1500	0,2
Arches backfill	18	150	0,2
Piers/Abutments masonry	22	2800	0,2
Piers/Abutments backfill	18	280	0,2

Table 2. Strength assigned to the masonry.

Element/Material	fc (MPa)	ft (MPa)	Φ (°)	c (MPa)
Brick masonry Arches	2,40	0,09	68	0,022
Stone blocks Piers/ Abutments	6,00	0,135	73	0,034

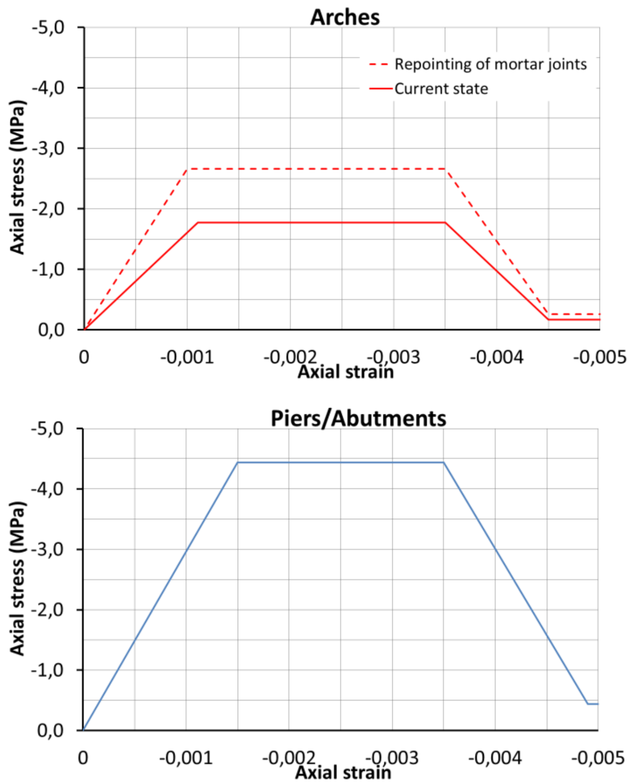

Figure 6. Uniaxial stress-strain relationships in compression assigned to the masonry of arches and piers/abutments.

In performing the numerical simulations also the nonlinear shear behaviour has been taken into account through a frictional stress-strain relationship. In particular, the shear strength is calculated in according to the criterion adopted in CSI Bridge (2015). The friction angle ϕ is derived starting from the uniaxial tensile strength *ft* and the compressive strength *fc* as proposed by the Drucker–Prager failure criterion (Chen and Han, 2007):

$$\sin\phi = \frac{f_c - f_t}{f_c + f_t} \tag{1}$$

The Drucker-Prager parameters obtained for the analyzed case are reported in Table 2.

4 NONLINEAR ANALYSES AND SEISMIC ASSESSMENT

In this work only the transverse seismic response of the Cavone bridge is discussed. The assessment procedure is carried out using the Capacity Spectrum method (NTC-08, ATC-40, FEMA 273), combining the nonlinear static analysis (pushover) with the response spectrum approach. In according to the performance based design philosophy, the adopted method assesses the displacement capacity of an equivalent single DOF system comparing it with the seismic demand.

For determining the pushover curve capacity an incremental lateral forces system proportional to the displacement pattern of the first vibration mode has been applied. In Figure 7 is reported the displacement pattern of the bridge first mode, and the control node. In this study it has been chosen corresponding to the central main arch keystone.

The bridge nonlinear pushover curves are reported in Figure 8 and Figure 9. The Figure 8 shows the influence on the transverse response of the nonlinear shear behavior. At this aim three different curves are compared, obtained as follows: considering both axial and shear stress-strain relationships as linear elastic laws (elastic global model - S11 L, S22 L and S12 L curve); by assigning the nonlinear stress-strain law only along the axial directions (S11 NL, S22 NL, and S12 L curve); and by adding as well the nonlinear shear stress-strain relationship (S11 NL, S22 NL, and S12 NL curve). It is easy to understand that the nonlinearities due to the shear behavior are significant into the global transverse response of the bridge. Moreover, the displacement of the control node reaches a maximum displacement at failure d_{max} of 23 cm, equal to a central pier drift of 0.95 % approximately.

Instead, in Figure 9 the influence on the global response of the arches and piers is investigated. The plotted pushover curves are obtained by assigning alternatively to the arches (Arches NL – Piers/Abutments L) and to the piers (Arches L – Piers/Abutments NL) an axial and shear nonlinear behavior. It is evident that the transverse nonlinear response of the bridge is quite entirely due to the nonlinear response of the piers/abutments, while the arches nonlinear behavior is negligible.

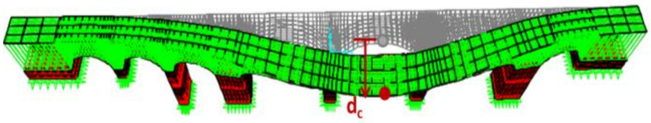

Figure 7. Displacement pattern of the first vibration mode and the control node chosen (T_1 = 1,45 sec).

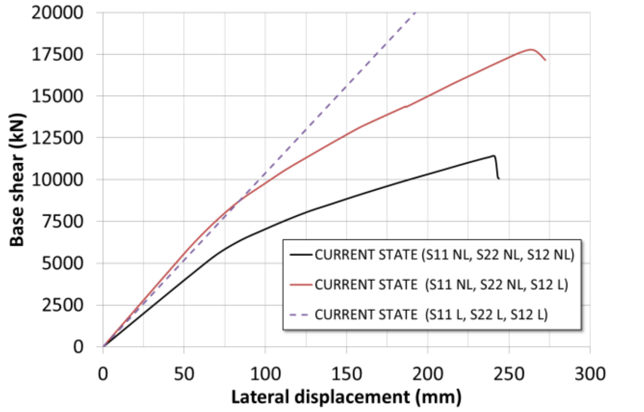

Figure 8. Bridge pushover curves in the current state: influence on the response of the nonlinear shear behavior.

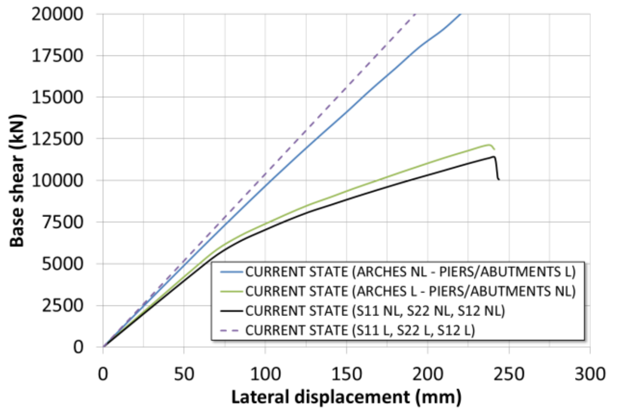

Figure 9. Bridge pushover curves in the current state: influence on the response of arches and of piers elements.

More in detail, the nonlinear capacity curve depends mainly by the two piers sunk into the riverbed with respect to the abutments. This may be demonstrated starting from the analysis of the transverse response of one pier, whose finite element model is reported in Figure 10. The related nonlinear pushover curve is shown in Figure 11, where is also plotted the transverse response of the two piers sunk into the riverbed. It has been derived by supposing that the two piers act in parallel against lateral forces and, therefore, the resulting nonlinear response is obtained just doubling the base shear of one pier for each displacement amplitude. In Figure 11 is possible to realize that the piers pushover curve is quite similar to the global one. In particular: the global initial stiffness is very close to the one of the two piers; the peak strength of the two piers is about 60% of the global strength peak of the bridge.

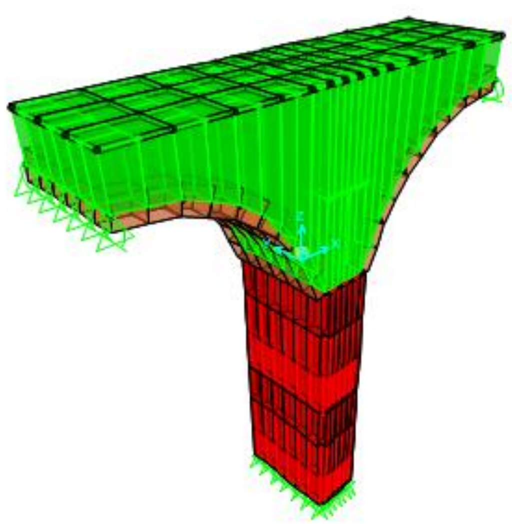

Figure 10. Model of a single pier sunk the riverbed.

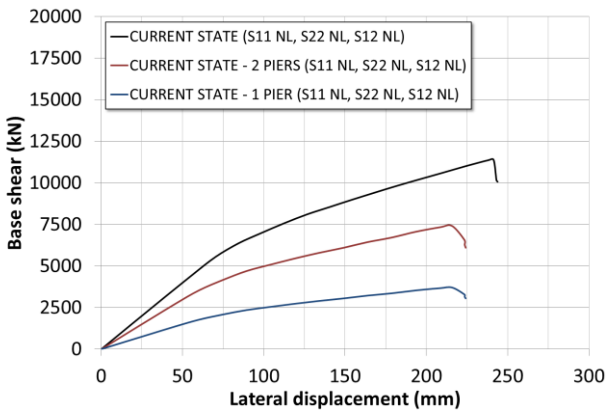

Figure 11. Current state: comparison with the pushover curve of the two piers sunk in the riverbed.

In this study the seismic assessment of the Cavone bridge is performed in accordance with the ATC-40 Capacity Spectrum Method. With this method the bridge global response is converted into the response of an equivalent nonlinear single degree of freedom (SDOF) system, permitting a direct comparison between capacity and seismic demand in terms of the response spectrum. In Figure 12 is shown in the ADRS format the bridge capacity spectrum and the reduced seismic demand, the latter derived from the elastic response spectra in according to NTC-08. The performance point is sought for two different ultimate limit states of the seismic action: life safety limit state (LS) and collapse prevention limit state (CP), considering a return period of 75 years.

In Table 3 are reported the resulting performances of the bridge for the two limit states considered, by adopting the following symbols:

- *V*: base reaction shear of the capacity curve corresponding to the performance point;

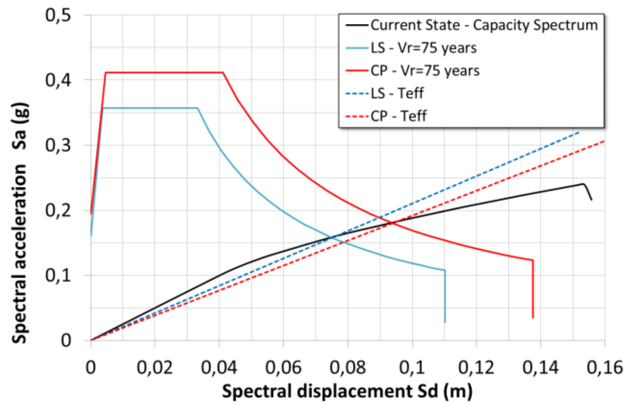

Figure 12. Current state: performance point evaluation.

Table 3. Performance point evaluation in the current state for 75 years returning period.

State limit	V (kN)	D (m)	S_a (g)	S_d (m)	T_{eff} (sec)	B_{eff} (%)
LS	7866	0.120	0.160	0.076	1.384	9.90
NC	8916	0.152	0.184	0.096	1.452	11.80

- *D*: displacement of the control point in the pushover curve (capacity curve) corresponding to the performance point;
- S_a: spectral acceleration of the equivalent elementary oscillator corresponding to the performance point in the ADSR plane (capacity spectrum);
- S_d: spectral dispacement of the equivalent elementary oscillator corresponding to the performance point the ADSR plane (capacity spectrum);
- T_{eff}: effective period of the equivalent elementary oscillator corresponding to the performance point;
- B_{eff}: equivalent damping ratio of the equivalent elementary oscillator corresponding to the performance point;

The seismic assessment performed shows that, in according to the ATC-40 method, the bridge reaches the displacement demand at both life safety and collapse prevention limit states and, thus, results seismically adequate. It is important to remark that the global performance obtained may be attained only if any local collapse mechanism is inhibited. To this end a series of local interventions have been designed, also for the conservation of the bridge elements.

First of all, in order to improve the spreading of traffic loads a remaking of the road pavement with a lightweight concrete slab has been considered. The slab has a thickness of 50 cm and is just placed on the backfill with no linking to the arches and piers core. It is also considered of restoring the masonry texture of bridge elements (where necessary), and a deep repointing of all arches mortar joints with epoxy resin. Furthermore, uniaxial C-FRP wraps to the three main arches have been applied at intrados as local strengthening mainly addressed to improving the resistance under vertical cyclic loadings (Figure 13).

In Figure 14 is reported the global response obtained by considering all of the interventions aforementioned and the comparison with the current state (no interventions) curve. Moreover, in the same figure are also plotted the pushover curves referred only to some of the designed interventions. In comparing all the interventions it is evident to note that the repointing of the arches mortar joints is the only intervention among all that slightly modifies the global response is terms of stiffness and strength. The others, on the contrary, have no influence on the transverse response of the bridge.

In Figure 15 is reported the seismic performance evaluation in according to ATC-40 method of the Cavone bridge by considering all the interventions (design state). The performance point evaluation (Table 4) is quite similar to the one obtained for the current state (d_{max} at failure of 24 cm, related drift of 1 %.), demonstrating the non-invasiveness of the chosen interventions. They, in summary, do not alter the pre-existing bridge response and are aimed to the conservation of the masonry elements.

Finally, in Figure 16 the ratios α of the maximum displacement ($D_{capacity}$) to the required displacement (D_{demand}) are calculated. It is possible to observe that the values are substantially equal in the two structural conditions (before and after the interventions), remarking, again, that the global response is unchanged.

Figure 13. C-FRP wraps applied to the intrados of the main arches.

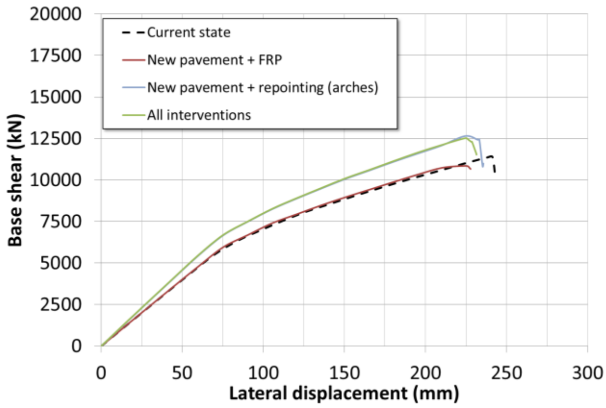

Figure 14. Comparisons between current state capacity curve and design state.

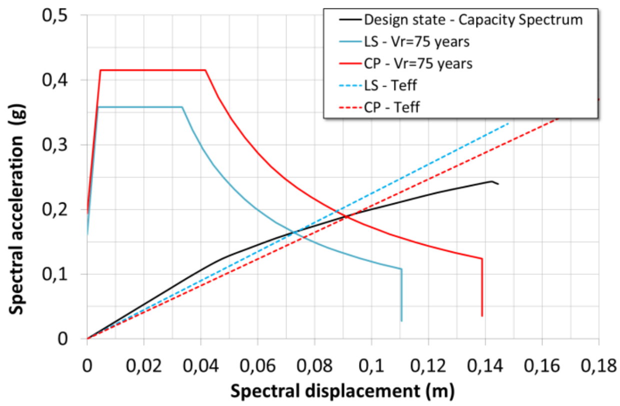

Figure 15. Design state: performance point evaluation.

Table 4. Performance point evaluation in the design state for 75 years returning period.

State limit	V (kN)	D (m)	S_a (g)	S_d (m)	T_{eff} (sec)	B_{eff} (%)
LS	8737	0.116	0.166	0.074	1.336	9.80
NC	10006	0.148	0.193	0.094	1.339	11.50

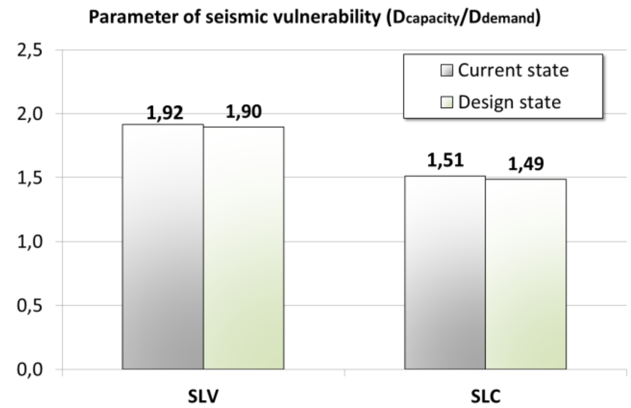

Figure 10. Seismic vulnerability parameters.

5 CONCLUSIONS

In this paper, static nonlinear analyses have been performed in order to evaluate the seismic performance of an existing masonry arch bridge. The bridge response has been simulated with nonlinear elements modeling the masonry behavior of arches and piers.

The obtained results have shown the importance of simulating the shear nonlinear behavior of the critical masonry regions that, in the case analyzed, arise along the central piers supporting the three main arches. As demonstrated, the global response mainly depends by these elements representing the most vulnerable elements of the bridge.

The seismic performance evaluation, performed in according to the ATC-40 method, has shown that in the current state the bridge satisfies for both life-safety and near collapse limit states the displacement seismic demand. The designed local interventions do not modify the pre-existing global response, providing an improvement of the bridge elements conservation state and preventing the local collapse mechanisms.

6 ACKNOWLEDGEMENTS

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