Seismic assessment of Maillart-arch-type Bridge: the case study of Viadotto Olivieri in Salerno

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ABSTRACT
This paper deals with the seismic assessment of a Maillart-arch-type bridge, also known as concrete deck stiffened arch bridge. The effectiveness of this structural solution and its possible deficits will be valued for the arch's worse load conditions. Analyzing an existing case study, Viadotto Olivieri in Salerno (1960's, South Italy), structural behavior will be defined, in particular looking at the effects of longitudinal and out of plane horizontal forces (estimated as 10% of the overall bridge dead load) on the main structural elements, as the lower ribbed arch, the upper concrete deck and the intermediate cross walls. FEM modelling has been required. Discretizing the structure by using frame and shell elements, on the base of the results of a 3D-scanner relief, six different options are compared. Variable aspects in modelling have concerned: upper deck characterization, passing from three deformable deck model to single deformable or un-deformable deck system, and constraint conditions at the bottom of the arch and of cross walls, valuing the case of fixed or hinged joints. The resulting stress distributions, defined as percentage of the overall applied forces, will underline the critical aspects of a structural system and the probable advantages in changing current bridge configuration. The consequence of the possible arch instability due to out of plane horizontal forces and of cross walls buckling as consequence of deck sliding for longitudinal horizontal actions will be argued, while modal analysis outputs will guarantee to describe dynamic response of this complex system.

1 INTRODUCTION: MAILLART'S SOLUTION FOR CONCRETE DECK STIFFENED ARCH BRIDGE

Maillart's innovative use of concrete, especially in the design of thin arch structures, and his introduction of a wide range of new engineering forms, make him a seminal figure in the history of modern engineering. He rejected the complex mathematical analysis of loads and stress that was adopted by most of his contemporaries. His method was a form of "creative intuition": he had a knack for conceiving new shape to solve classic engineering problems (Billington, 1973) (Billington, 1983) (Billington, 1997) (Billington, 2000)) (Bruun, 2014).

Maillart's design approach, with its emphasis on new possibilities for arched bridge forms, represents, still now, the heart of creative engineering thinking, creativity not only with respect to the appearance of form, but also with regard to its engineering substance.

Starting form 1912, Maillart's firms proposed several design innovations, as the hollow box solution or the deck stiffened arch system. This last one, also called arch without rigidity or Maillart-type arch, is characterized by a thin ribbed polygonal arch which works combined to a relatively stiff deck, connected each other by transversal walls. The purpose for an aesthetically pleasant shape leads Maillart to design an arch as thin as the bridge, but still able to carry all traffic loads safety. A concrete arch can carry permanent loads when it is designed with the proper shape: for a load uniformly distributed over the horizontal bridge deck this shape would be a parabola, the so-called funicular curve, Figure 1 (left). The difficulty comes when traffic loads only a part of the span length; then the arch will try to bend into a new shape.

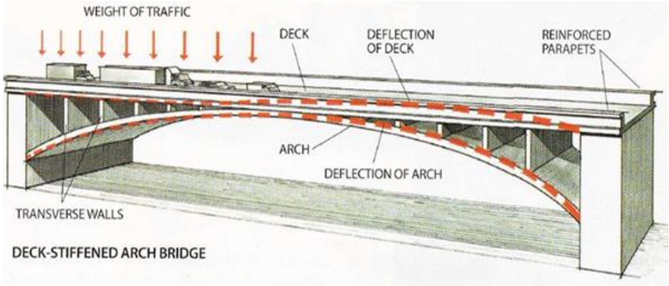

Figure 1. Maillart-arch-type bridge characterization. Structural scheme under live loads © Billington)

Maillart reacted against massive concrete as a musician to tone deaf singers (Billington, 2003).

The deck-stiffened arch, Figure 1 (right), works because the arch and deck are connected firmly together by a series of cross walls.

Then as the arch tends to bend when loaded by traffic over one half of the span, the cross walls make the deck bend to the same new shape as the arch. The bending effect is now shared between arch and deck and, as Maillart further reasoned, that effect will load each part in proportion to its stiffness. The arch-and-deck cooperation suggested to Maillart a different view of those elements, of their relative proportion, especially for the arch appearance.

Perhaps the most beautiful of Maillart's bridges of this type is the Bridge at Schwandbach, Figure 2. This curved reinforced concrete masterpiece is a typical example of Maillart-type no-rigid arch: this structure consists in reducing arch rigidity by increasing the deck one. The arch is polygonal rather than curved, and is only 200 mm thick. It supports the bridge deck via 160 mm thick reinforced concrete cross walls. The deck is thicker than the arch, and is stiff enough to prevent the slender arch from buckling. The highway deck is curved in plan. In this particular solution, the arch varies in width from 4.2 m to 6 m, with one edge forming a straight line between river banks, and the other following the curve of the road. This arrangement helps to resist centrifugal forces from the traffic loads and from the curved deck tendency to twist.

The effectiveness of Maillart solution is readable in benchmark recent solution, as the Infant Dom Henrique Bridge by A. Adão da Fonseca, Figure 3. The bridge is composed of two mutually interacting fundamental elements: a very stiff (slenderness ratio 1/62.2) prestressed concrete box beam, 4.50 m in height, supported on a very flexible (slenderness ratio 1/186.6) reinforced concrete arch, 1.50 m thick. The span between abutments of the arch (L_{tot}) is 280 m, the arch span (L) is 70m long and the rise (r) until the crown of the arch is 25 m, reaching a static coefficient (L^2/r) of 1/3136, the lowest never achieved. This system, characterized by a flexible arch combined to a rigid deck, guarantees: the absence of relevant bending moments in the arch except at the piers; moderate variation of axial forces are led by the arch; deck behaviour is assimilated to a continuous beam on elastic supports, provided by transversal cross walls spaced 35m apart.

2 CASE STUDY CHARACTERIZATION: CURRENT STATE OF VIADOTTO OLIVIERI

In order to prove the structural efficiency of Maillart-arch type bridge, eventually underlining any possible deficits, especially due to horizontal loads, an existing structure has been analysed.

Figure 2. Deck stiffened arch system applications Schwandbach Bridge by Robert Maillart, 1933

Figure 3. Deck stiffened arch system applications : Infant Dom Henrique Bridge by A. Adão da Fonseca, 2002

The case study is Viadotto Olivieri in Salerno, along A3 Highway Pompei – Salerno. It makes part of an infrastructural system, Figure 4, that was included in the renewal plans of Cassa per il Mezzogiorno (Grassini et al., 1962) proposed in 1950's to finance industrial initiatives to raise South of Italy. This plan had the aim of relaunching South Italy economy and reducing the existing gap with Northern Italy. About 40% of the whole budget was spent to build a new track in one of the most pleasant tourist place, Pompei-Salerno.

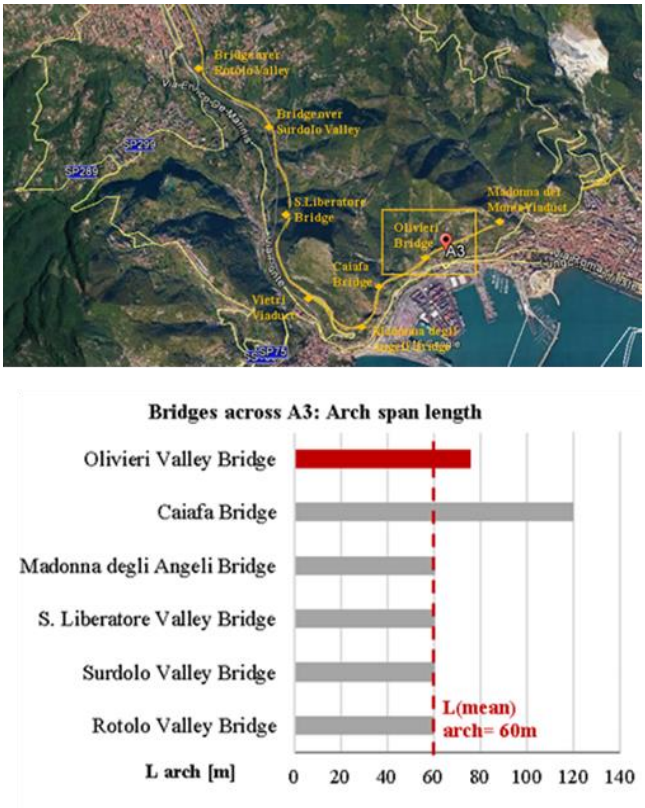

Figure 4. Eight bridges along A3 Pompei – Salerno: location and bridge spans.

Except for Rotolo Valley Bridge, all the other infrastructures are characterized by two staggered carriageways, one for travelling direction, each one made of two lanes (7.50m-wide carriageways). This solution led to significant advantages: (1) it avoids discomfort or disability caused by passing cars lamps; (2) deck width is reduced, as the central wall-beam, which links the upper portion to the lower one, is less bulky than the common bollards. Considering deck static behavior, "Z-shaped"section is capable to carry stress due to bending moment reversal, Figure 5.

For bridges along Pompei- Salerno Highway, arch shape was defined considering funicular polygon due to dead loads: considering that Maillart arch type system has been adopted, it could be assumed that all the effects of live load are carried by rigid deck, so there are no additional bending moment in the arch. According to designers' assumptions, arch-to girder transferring system has been verified, considering second order effects.

Apart from a visual and compositional uniformity, the choice of building 6 (among 8) bridges using Maillart ach-type has been greatly advantageous from an economical point of view: Considering that arch spans were quiet similar (mean value: 60m), bridge constructions proceeded step-by-step in series, adopting quiet similar centrings. Accomplishing to technical, structural and architectural requirements, the choice of Maillart arch type bridge, with a stiffen girder and thin ribbed vault, appeared the most congenial one to cover long span in a so impressive contest. Slender arch is stiffened with a rigid deck, capable to carry bending stress due to accidental loads, while arch supports only compression strengths.

Using a "Z-shaped" cross section for bridge deck, arch-to-girder transferring system is guaranteed by a wall-beam, connecting two staged carriageways; at the same time, vertical cross walls, as pendulums, make the arch following deck deformed shape, improving cooperation between load bearing structural elements. Thanks to this typology, many advantages could be obtained:

Figure 5. Olivieri Bridge general layout: Maillart arch bridge with staggered carriageways ("Z-shaped" deck)

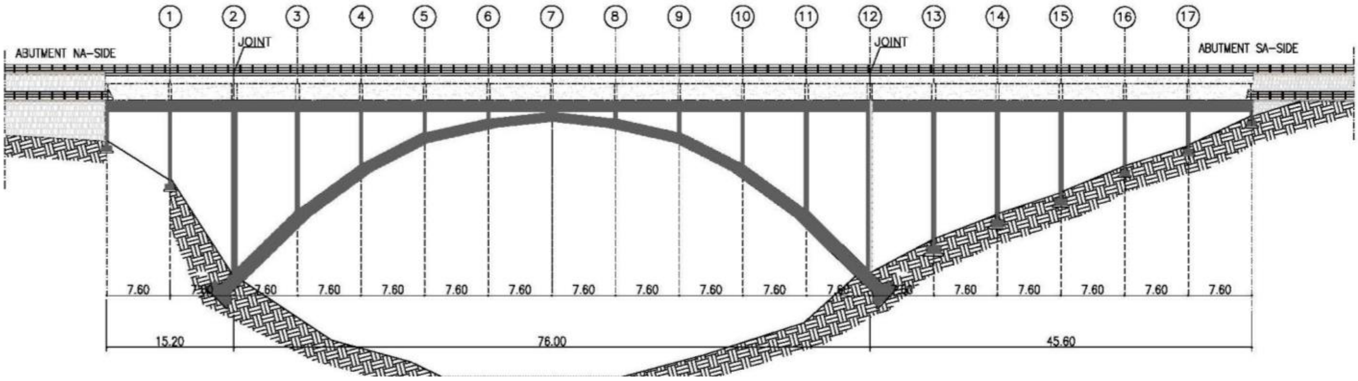

Figure 6. Olivieri Bridge: longitudinal layout

(1) economy in using materials: strict interaction between arch and girder allows to better use *strength coming from different structural* elements;

(2) effect of concrete shrinkage and settings are negligible, adopting a low-thickness vault; (3) reduction of centring cost, having to support the weight of a slender vault; (4) aesthetic value of a no redundant structural solution.

2.1 General layout of Olivieri Bridge

Olivieri Bridge is characterized by a reinforced concrete deck- stiffened parabolic arch, whose access ramps consist of 2-span reinforced concrete beam bridge on Naples side15.20m long, and 6-span beam bridge on Salerno side, 45.60m long, Figure 6. Bridge has an overall length of 136,80m and is separated in three portions by two 6 cm-large joints, one in correspondence of pier n.2 and the other of pier n. 12. Bridges roadways settle at different grades above the ground, being 4m-staggered one from the other.

Bridge main structure is the central Maillart arch-type bridge, having a thin ribbed vault with an upper stiffer girder. The arch spans 76.00m and reaches the rise of 19.30m at the crown: a rise-to-span ratio (r/L) of 1\4 (0.25), i.e. a static coefficient (L^2/r) of 299 can be estimated. Connection between arch and longitudinal girder is guaranteed by slender cross walls, working as pendulums. Each cross wall is made of a thin concrete slab stiffened by 5 columns, Figure 7, whose cross section size grows passing from the middle to the edge of the cross wall.

2.2 Load analysis

Dead loads estimation starts from the geometrical configuration of structural elements. Arch vault is made of a 0.20m-thick concrete slab, stiffened by five ribs, with variable cross section along longitudinal axis.

The polygonal arch consists of 10 straight portions, whose section size grows passing from the arch crown to the springing sections.

The arch is characterized by a central rib, which changes dimensions, form (100cm x 40cm) at the crown to (157cm x 40cm) at the springing sections. The arch slab is made stiffer by two additional intermediate ribs, whose cross section varies form (100cm x 30cm) at the middle span to (157cm x 30cm) at the abutments, and other two external ribs that wide form (100cm x 45cm)at the crown section to (157cm x 45cm) at the springing ones.

Bridge deck consists of two staggered carriageways (Figure 8), each one made of two lanes, connected by a continuous beam-wall (Δz= 4.00m), 25cm deep. Road-deck is a thin slab, whose depth varies from 24cm in the central portion to 49 cm in cantilevers. Each staggered portion is supported by two longitudinal girders.

Deck is connected to the arch by vertical cross-walls, 7.60m spaced. These ones area characterized by a concrete membrane, 12cm deep, ribbed each one by five pillars. The central pillars have dimension (30cm x 30cm), the two intermediate ones (40cm x 30cm), the external ones (45cm x 30cm). About superimposed-dead loads, a flexible paving has been assumed. Upon the two 7m wide carriageways three different layers have been considered.

Figure 7. Olivieri Bridge details: cross wall layout between the upper deck and the lower arch

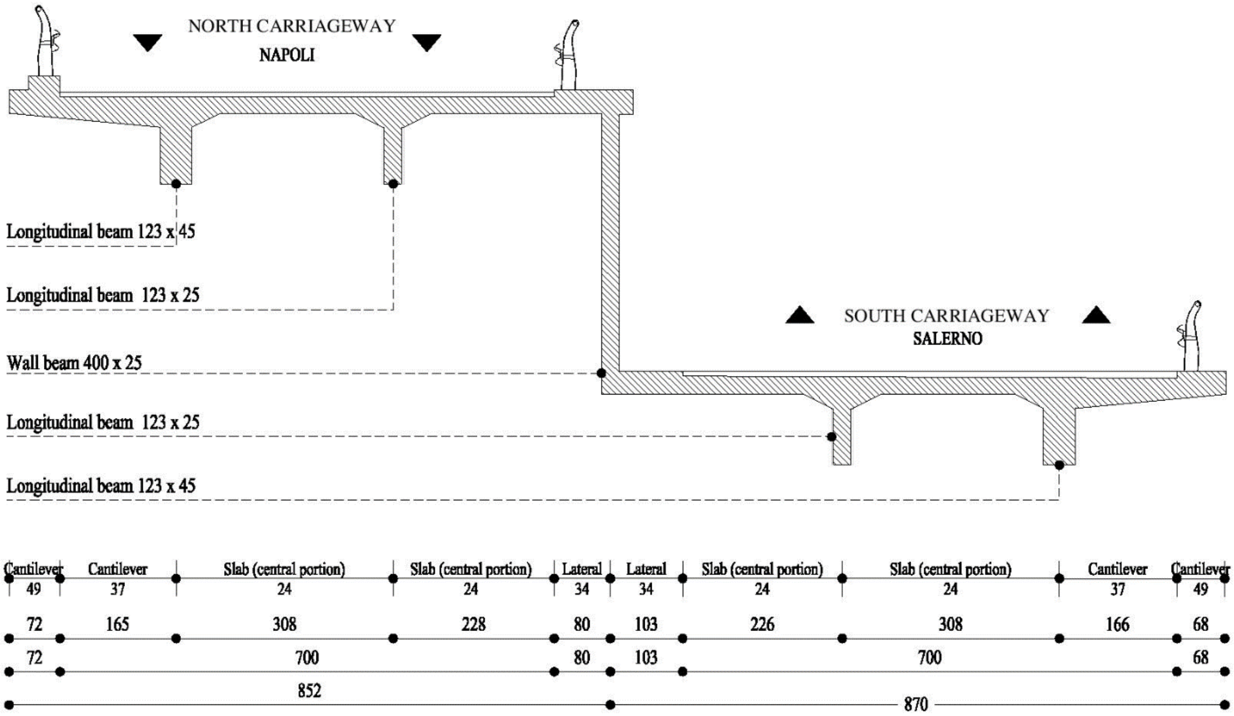

Figure 8. Olivieri Bridge deck cross section

They are: finishing asphalt surface, 4cm thick; (2%) slope increment, with a maximum thick of 7cm; binder layer, 10cm deep.

Traffic loads have been estimated in accordance to Italian Building Code (NTC 2018) that foresees conventional lanes wider than 3.00m. Olivieri Bridge consists of two independent staggered carriageways, 7.00m wide, each one including two 3.50m large lanes. Lanes have been numerated in order to induce the worst effect. For each carriageway, the loading lane which causes the most unfavourable effect is defined as “Lane 1”, the second one is named “Lane 2”. For a bridge of “1st Category” (i.e. bridge which carries whole traffic load __ no reductions are assumed), variable live loads, comprehensive of dynamic effects, consist of concentrated force acting along two tandem axis, upon squared pneumatic tracks (0.40 m x 0.40 m), and a uniformly distributed loads. For “Lane 1”double concentrated forces of 300 kN and an uniformly distributed load of 9 kN/m² are considered; Lane 2 concerns double concentrated forces of 200 kN and an uniformly distributed load of 2.500 kN/m². Upon “Lane 3”double concentrated forces of 100 kN and an uniformly distributed load of 2.50 kN/m² act; Lane 4 bears an uniformly distributed load of 2.500 kN/m². Loads estimation, Table 1, underlines that permanent loads (dead + superimposed-dead) represent about the 85% on the total. Live loads correspond to a reduced percentage (less than 15%): live-to-permanent loads ratio is nearly 1:6.

Table 1. Olivieri Bridge Loads (overall length L=137,10m; deck loaded width w= 15.83m)

Type	Portion	FL^{-2}] (t/m²)	% Sub.total	% Tot
Dead	Arch	0.45	20.27%	15.25%
	Deck	1.23	55.43%	41.73%
	Cross walls	0.54	24.30%	18.29%
	Dead tot	2.22	-	75.27%
Super-imposed dead	Asphalt	0.08	25.00%	2.72%
	(2%) slope	0.02	6.30%	0.68%
	Binder	0.19	59.25%	6.44%
	Barriers	0.03	9.45%	1.03%
	Super.dead tot	0.32	-	10.87%
Live	Lane 1	0.23	59.09%	7.77%
	Lane 2	0.07	17.10%	2.37%
	Lane 3	0.06	14.64%	2.03%
	Lane 4	0.05	12.17%	1.69%
	Live tot	0.41	-	13.86%
Total	-	2.95	-	-

Table 2. Olivieri Bridge Loads distribution for each portion

Portion	Section 1 (NA)	Section 2 (arch)	Section 3 (SA)
L [m]	14.90	76.56	45.70
Dead [t]	523.62	2688.41	1606.01
Super-dead [t]	74.20	380.90	227.58
Live [t]	96.70	496.51	296.60
Tot [t]	698.52	3538.82	2130.19
Tot [%]	11.34%	60.54%	28.12%

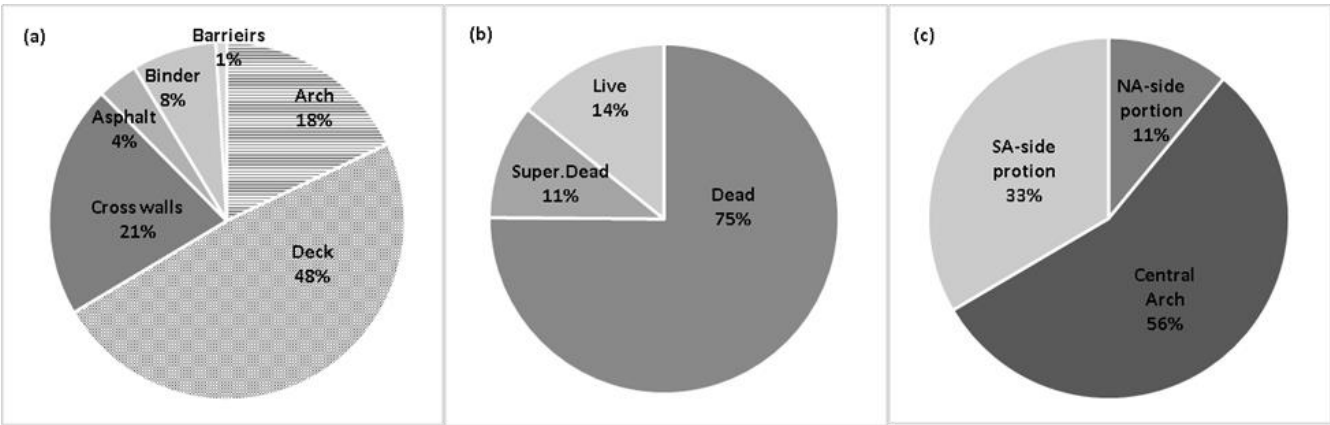

Figure 9. Olivieri Bridge Loads. (a) Permanent loads; (b) overall loads distribution; (c) load distribution for each portion

Table 2 and Figure 9 summarize loads acting upon each of the three portions: access ramp at Naples side, as Section 1, the arch central portion, as Section 2, the access ramp at Salerno side, as Section 3.

3 SEISMIC CHARACTERIZATION OF OLIVIERI BRIDGE

To characterize bridge static and dynamic behaviour, FEM modelling is required. Matching the model with the real structure has been no so easy, both for lack of information about this bridge, and for its geometrical and technical complexity: a cloud of points, coming from 3d-scanner relief, has been adopted as guide to modelling the structure. Bridge has been discretized using frame and shell elements, by software SAP2000: arches and beams, making the "skeleton" of this structure, have been modelled as frame elements, corresponding to their barycentre axis; wall and slab have been defined as shell elements. Six different cases have been analysed, changing bottom constraint conditions for the arch and the cross walls, and deck characterization. The effect of horizontal forces (FOX = FOY = 10% overall weight, W) has been valued, in order to define the most vulnerable elements. At the same time, dynamic characterization has been argued, as result of modal analysis.

3.1 Fem analysis: comparison between model

Bridge seismic assessment has been tested valuing six different options, defined as follows. (1) Three deformable deck model with fixed joints: it is quiet similar to the real structure, characterized by three joined portions, having deformable deck. The overall horizontal force has been distributed to each portion, proportionally to its weight, as seen before: NA-side: 11%, central arch: 56%; SA-side: 33%. For each single portion, the corresponding force has been equally distributed between upper and lower decks, applied at their barycentre. Considering that, for each cross walls, 5 ribs attach to foundation plinths, each point that connect structure to foundations has been modelled as (elastically yielding) fixed joints. (2) Three deformable deck model with hinged joints: differently from the previous model, in this case, for the arch and for each cross wall, the five points of connection to foundation are modelled as multi-directional hinge, but the presence of concrete membrane elements, which make the cross walls stiffer, reduces the possibility for the structure to rotate out of plane. Hinged joints effects is particularly noticeable only along longitudinal direction. (3) Single deformable deck with fixed joints: bridge deck is modelled by using the scheme of kinematic chain, that consists in assume an elastic connection between different deformable deck portions, both in longitudinal and in transverse direction. In this way all deformations occurred along deck are concentrated in a single section, either at the midspan or at the abutments. This arrangement reaps the benefits coming from continuous deck system. The overall force in applied at the midspan of central arch portion, uniformly divided between two deck levels. Connections of the cross walls to foundation plinths are defined as (elastically yielding) fixed joints. (4) Single deformable deck with hinged joints: in comparison to model (c), base connections are modelled as (elastically yielding) hinged joints. (5) Single un-deformable deck with fixed joints: differently form model (c), bridge deck is modelled by assuming that, for each staggered plane, the whole carriageway moves rigidly. (f) Single un-deformable deck with hinged joints: differently form model (6), in this case, bridge deck is modelled by assuming the carriageway moving rigidly in their own plane.

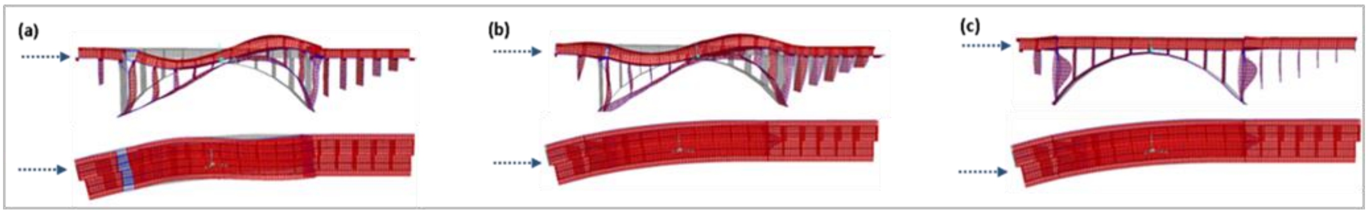

Figure 10. Olivieri Bridge deformed shapes due to longitudinal horizontal force (Fox = 10%W): (a) three deformable decks model; (b) single deformable deck model; (c) single un-deformable deck model

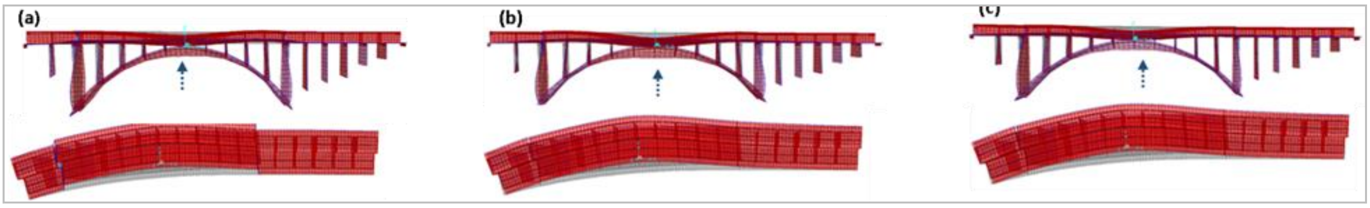

Figure 11. Olivieri Bridge deformed shapes due to out of plane horizontal force (Foy = 10%W): (1) three deformable decks model; (2) single deformable deck model; (3) single un-deformable deck model.

3.2 *Effects due to longitudinal and out of plane horizontal forces: most critical aspects*

Linear static analysis help to analyse, in a more simple way, the vulnerability of the main structural elements, when horizontal forces act in longitudinal and in transversal directions. The total force, properly distributed among different portions, is approximately 10% of the overall bridge weight. The corresponding effects have been read at the bottom of the arch and of the cross walls, in terms of shear (V) and moment (M) distribution, axial force variation (ΔN). A comparison between the six analysed models underlines the most critical aspects of the bridge at the current state, suggesting any possible improvements, also to counter seismic actions.

When horizontal force acts in longitudinal direction, Figure 9, upon the bridge at the current, similar to model (1), considerable sliding effects occur to arch and slender cross walls. In this case, the three portions act independently one from another. Similar results are obtained from model (2), where hinged joints replace fixed ones.

This last restraint condition has no many influences on bridge behavior in longitudinal direction: at most, considering cross walls' characterization, hinged solution ensures an out of plane rotation capacity higher than the fixed base one. Acting longitudinal horizontal forces, the shortest and stiffest side spans show little sliding motions, while the arch, as the most vulnerable portion, follows deck deformed shape.

Using a kinematic chain scheme that joints the three portions, Figure 10 (b), the resulting continuous rigid deck makes the overall structure stiffer. If in the case of single deformable deck all cross walls are involved in sliding motion, the hypothesis of un-deformable deck, Figure 9 (c) makes the arch practically unloaded, concentrating the worst effects on the abutments. On the contrary, current configuration, with three deformable decks, ensures the structure to have a great deformability: this characteristic could be assumed as a great potential for the bridge to mitigate any possible effects due to seismic actions.

Looking at bridge configuration, transversal forces appear the worst load condition. The corresponding effects have been valued, comparing aforementioned six models. In this case, varying deck layout, bridge behavior changes clearly.

Out of plane horizontal forces, Figure 11, highlight not negligible overturning problems for the thin arch. At the current state, each of three deformable portions acts independently form the others. So, the most loaded central arch records the worst uplift effects, that become insignificant for the lateral portions. When a single deck is considered, the whole structure is involved, so a reduction of arch out-of-plane displacement is visible. Looking at bridge configuration, transversal forces appear the worst load condition.

The corresponding effects have been valued, comparing aforementioned six models. In this case, varying deck layout, bridge behavior changes clearly. In terms of cross wall shear (V) distribution, Figure 13, at the current state the most stressed component is the central arch, both in the case of hinged and of fixed joints solution.

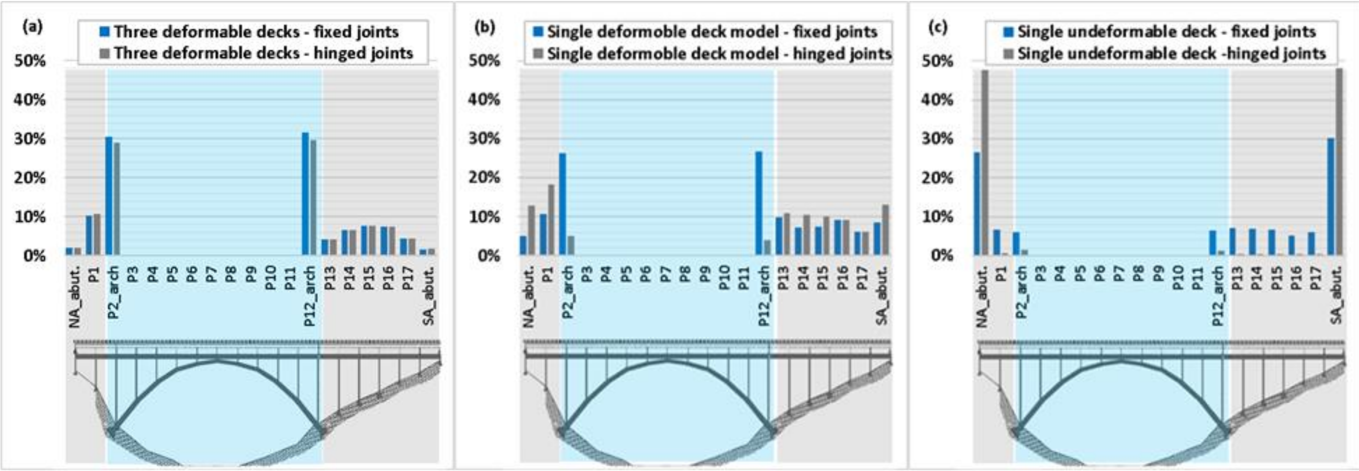

Figure 12. Cross walls Shear distribution (V) due to out of plane horizontal force (Foy = 10%W): (a) three deformable decks model: fixed (v.s.) hinged joints; (b) single deformable deck model: fixed (v.s) hinged joints; (c) single un-deformable deck model: fixed (v.s.) hinged joints.

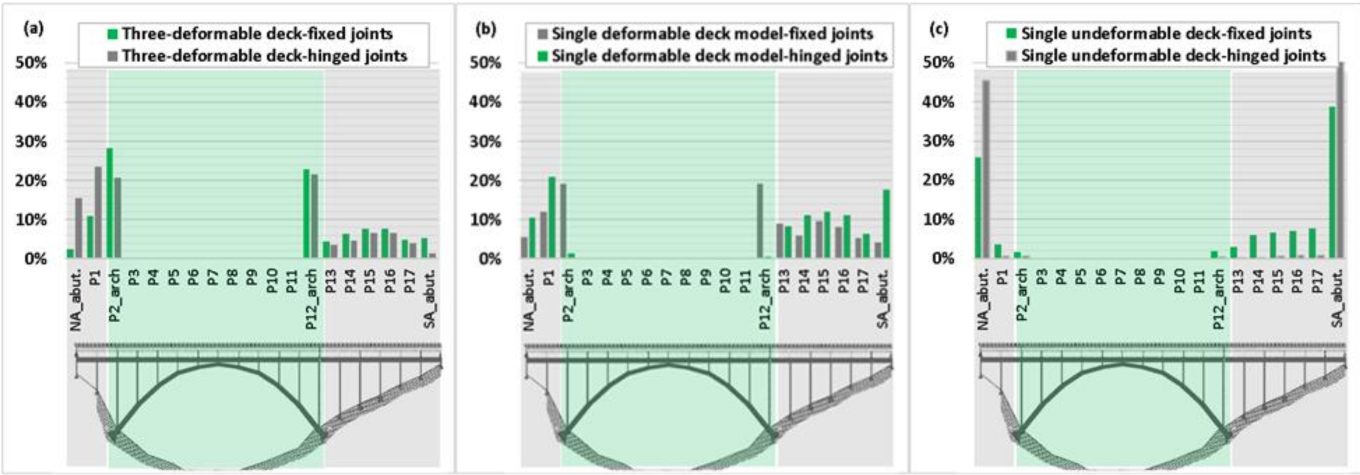

Figure 13. Cross walls Moment distribution (M) due to out of plane horizontal force (Foy = 10%W): (a) three deformable decks model: fixed (v.s.) hinged joints; (b) single deformable deck model: fixed (v.s.) hinged joints; (c) single un-deformable deck model: fixed (v.s.) hinged joints.

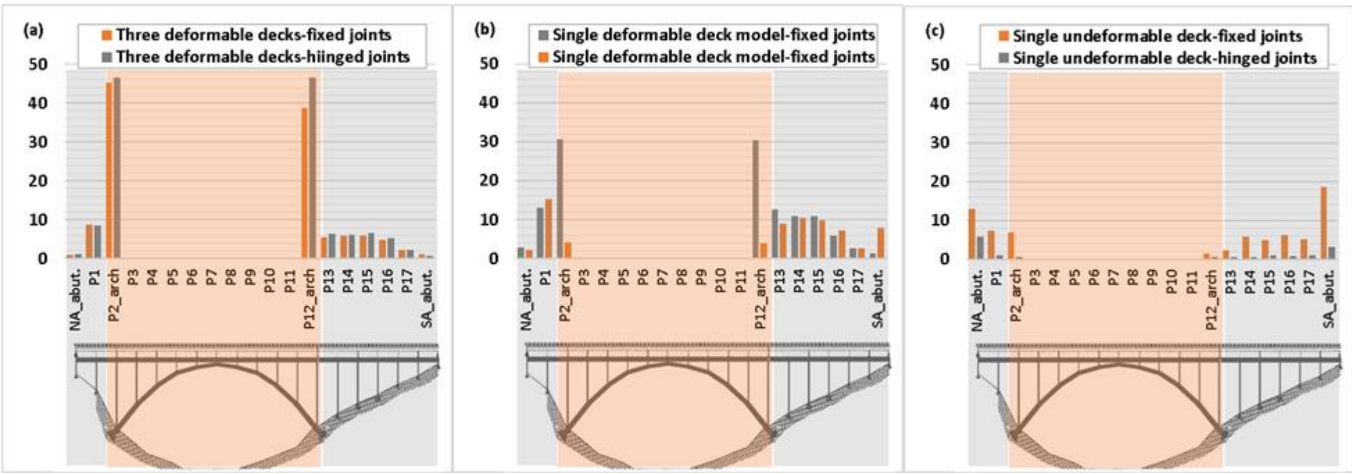

Figure 14. (below) Cross walls Axial force variation (ΔN) due to out of plane horizontal force (Foy = 10%W): (a) three deformable decks model: fixed (v.s.) hinged joints; (b) single deformable deck model: fixed (v.s.) hinged joints; (c) single un-deformable deck model: fixed (v.s.) hinged joints.

The Maillart-arch- portion carries about 60% of the overall shear force, while the remaining is distributed among the intermediate piers; abutments are the less loaded sections. A different deck configuration and added to a change in bottom restraint conditions lead to a redistribution of forces among structural elements. Considering a single deck, all the cross walls are involved in deck sliding. In the case of fixed joint solution, central arch continues to be the most loaded segment, carrying about 50% of the overall shear force, while among the other cross walls a mean value of 8% is readable, even at the abutments. In the alternative case with hinged restraints at the base, shear force distribution is inverted: central arch contribution reaches 10%, both abutments carry about 25%, while intermediate cross walls get close to 10% each one. Finally, the single un-deformable deck option guarantees to completely upset the load transferring system: in this case, the most loaded portion are the abutments, carrying about 60% of the whole shear force, while arch contribution reaches 10% and intermediate cross walls not exceed 7% each one.

(M_V) and Moment due to axial force variation ($M_{\Delta N}$). In the case of three-deformable-decks solution with fixed joints, Maillart-arch portion remains the most loaded one; it carries about 85% of M_{tot}, while 4% of the overall corresponds to the abutments and a mean 7% is withstood by each intermediate cross wall. Changing restraints at the base, a little variation occurs. In the case of hinged scheme, a reduction of arch bending contribution (corresponding to about 40%) and an increase of abutments aliquot to 20% are readable. When the three-deformable-decks models are analyzed, the worst aliquot to the overall bending effects id due to axial force variation ($M_{\Delta N}$ about 75%; M_V about 25%). In the case of single deformable deck model, the kinematic chain adopted for the upper deck leads to an inverted bending moment distribution. In this last case, abutments withstand about 27% of the overall bending effects, while arch contribution comes down to 2% and the intermediate piers carry about 10% each one. The greatest aliquot to the overall bending effects is due to shear-induced moment ($M_{\Delta N}$ about 35%; M_V about 65%). Also in the hypothesis of single un-deformable deck solution, arch contribution is practically nihil, while abutments support about 65% of M_{tot} in the case of fixed joints option and 95% of the total moment. In this last case, the worst contribution to the overall bending effects is due axial force distribution ($M_{\Delta N}$ about 70%; M_V about 30%). Similar considerations concern axial force distribution at the base of cross walls, Figure 14. Three deformable deck model confirms the arch as the most loaded portion, while the introduction of kinematic chain solution guarantees to greatly reduce axial forcce variation at the base of each cross wall.

3.3 Dynamic characterization by modal analysis

A complementary way to characterize seismic behaviour of Olivieri Bridge is the evaluation of modal analysis outputs. Looking at bridge geometrical characterization, as well as the inhomogeneous nature of its material (reinforced concrete cast in space about 60 years ago), it's supposed that this Viaduct will have a dynamic behaviour considerably different from regular buildings. Considering that this bridge was designed (during the Second War World reconstruction) above all to resist to vertical loads, without evaluating the effect of possible horizontal forces, it's necessary to estimate bridge seismic attitude through response spectrum analysis, comparing FEM analysis outputs for each aforementioned cases. Seismic design parameters, are assumed in accordance with Italian building code (NTC2018) [10]. Bridge stands in Vietri sul Mare (Salerno, Italy) upon a carbonate bedrock, partially slatted, classified as Category B. A reference period of 100 years is assumed for this strategic significant strucutre. Table 3 summarizes seismic parameters used to define design response spectrum, for each limit state specified in Italian building code. A precautionary approach leads to assume a behaviour factor (q) equal to 1.0.

Table 3. Seismic parameters for acceleration design response spectrum (NTC 2018) – site: Vietri sul Mare (SA)

Limit state	Tr (year)	ag/g	Fo	T*c
SLO	30	0.038	2.372	0.280
SLD	50	0.048	2.369	0.329
SLV	475	0.150	2.580	0.439
SLC	975	0.127	2.684	0.459

Although to use this kind of analysis is questionable, since it implies an elastic approach, it permits to take into account all the vibration modes, without involving the computational effort of no linear dynamic analyses. The response spectrum analysis has been carried out considering one hundred vibration modes: each one having participating mass ratio greater than 5% has been considered. In this case, the distribution of couples [T, Se(T)] within design response spectrum can be seen an easy way to understand the natural attitude of Viadotto Olivieri to counteract seismic input. Meanwhile, looking at deformed shapes the identification of most vulnerable bridge structural elements appears really simple. Considering the reduced influence of restraint condition at the base of cross walls and the difficulty in creating effectively hinged connection at the base, lead to compare models with fixed joints.

Considering modal analysis outputs for three-deformable-decks model, Table 4, Table 5 and Figure 15, it is visible that modes with relatively high effects concern translations in longitudinal and in transverse direction. Apart from R(Z) component, rotational contribution can be neglected. Despite this bridge is a complex multi-degree of freedom system, 100 modes are sufficient to involve about 100% of participating mass; period associated to main modes are lower than 1sec, characterizing a really rigid structure; modes fall within the first two spectrum section, above all in the highest amplification one.

Rotational contributions don't prevail on translational displacements.

Considering modal analysis outputs for single-deformable-deck model, Table 6, Table 7 and Figure 16, it is visible that modes with relatively high effects refer to translations in both directions. Under the assumption of continuous deformable deck, 100 modes are quiet sufficient to involve all mass. Period referring to interesting modes fall within the first two spectrum section, above all in the highest amplification one.

Table 4. Modal analysis outputs for three deformable deck s with fixed joints: main translational modes

Mode	T[s]	UX (%)	UY (%)	UZ (%)
1	0.674	31.376	0.157	0.013
7	0.472	0.242	49.726	0.238
42	0.215	11.704	0.094	7.1E-05
53	0.1757	0.095	7.733	0.171
55	0.172	0.248	0.693	5.648
56	0.1721	0.026	1.143	41.374
58	0.1621	0.036	5.845	3.97
77	0.099	12.709	0.012	0.072
87	0.057	0.375	0.584	8.607
89	0.0521	5.532	0.1	0.151
90	0.046	1.317	4.632	0.726
91	0.044	0.316	0.00688	9.605
94	0.0328	0.021	0.813	7.321
95	0.0310	0.251	5.029	0.553
Tot	-	98.79	98.63	97.77

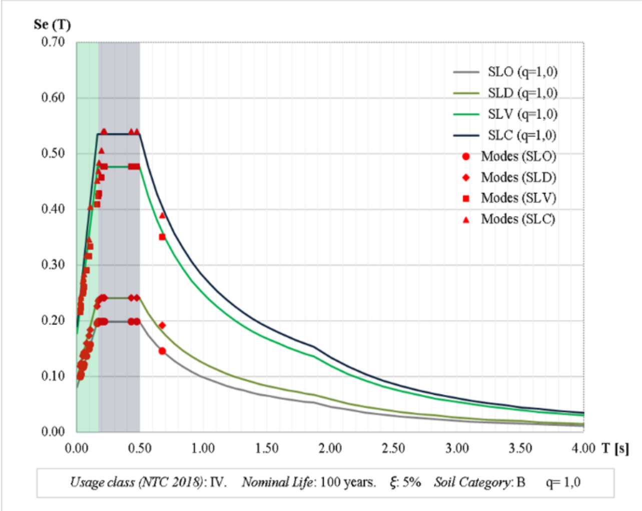

Figure 15. Modal analysis output for three-deformable-decks model.

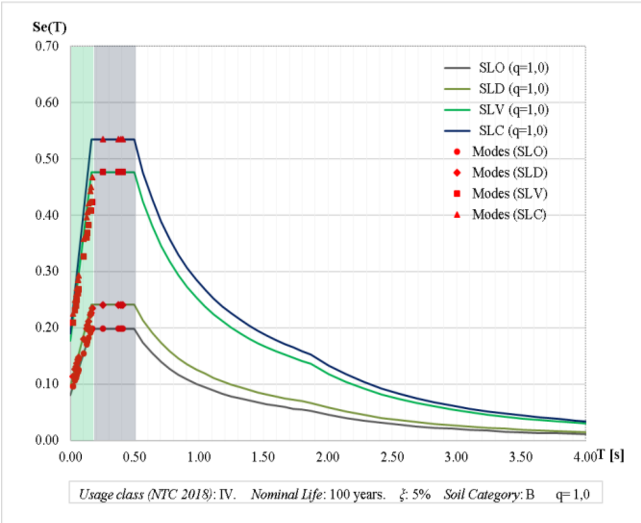

Figure 16. Modal analysis output for single deformable deck model.

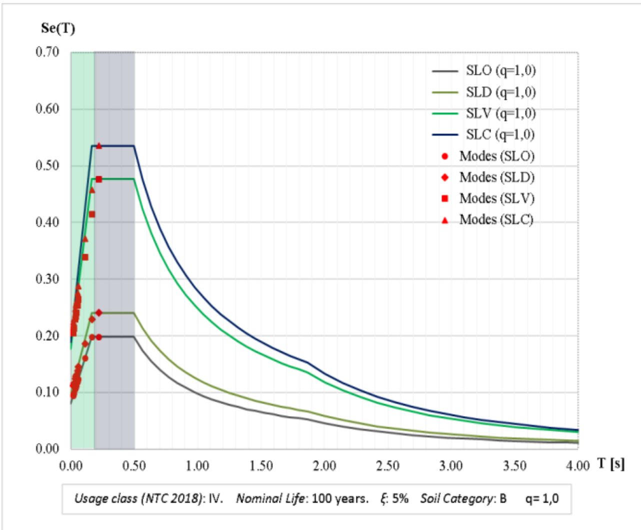

Figure 17. Modal analysis output for single un-deformable deck model.

Table 5. Modal analysis outputs for three deformable deck s with fixed joints: rotational modes

Mode	T[s]	RX(%)	RY (%)	RZ (%)
1	0.674	1.307	9.009	0.565
7	0.472	8.339	0.056	5.499
51	1.249	1.249	6.736	0.181
53	3.734	3.734	0.00394	9.045
58	9.076	9.076	0.488	0.16
77	0.0003	0.0003	0.17	0.273
82	0.645	0.645	1.123	6.981
86	9.561	9.561	0.725	0.16
96	0.385	0.385	0.137	13.043
98	6.111	6.111	0.072	0.015
Tot	68.02	68.02	50.83	97.53

Table 6. Modal analysis outputs for single deformable deck model with fixed joints: main translational modes

Mode	T[s]	UX (%)	UY (%)	UZ (%)
10	0.408	8.483	1.46	0.029
13	0.394	5.508	9.525	0.11
14	0.373	1.041	41.414	0.35
53	0.172	0.1	0.376	45.781
64	0.142	10.65	0.464	0.14
67	0.133	16.47	0.565	0.115
68	0.128	15.23	0.343	0.026
87	0.057	0.076	1.002	6.108
89	0.050	5.019	0.042	0.497
90	0.045	0.338	1.180	9.075
94	0.032	0.059	0.069	6.751
95	0.030	0.109	5.450	0.245
tot	-	98.85	98.72	97.83

Table 7. Modal analysis outputs for single deformable deck model with fixed joints: main rotational modes

Mode	T[s]	RX (%)	RY (%)	RZ (%)
10	0.408	2.121	4.90	0.002
13	0.394	4.741	4.09	0.883
33	0.252	0.11	0.02	22.668
55	0.162	9.263	0.761	1.185
74	0.103	0.0064	0.78	5.122
81	0.080	2.8420	1E-06	6.63
83	0.069	5.9470	0.027	0.0086
90	0.045	0.0710	4.906	0.492
98	0.016	0.3190	0.032	6.249
tot	-	65.833	51.67	97.49

Finally, considering modal analysis output for single un-deformable deck model, Table 8, Table 9 and Figure 17, it can be seen that, having a continuous rigid deck, structure static redundancy increases, so 100 modes are no sufficient to involve all participating mass. Modes with relatively high participating mass ration concern local translations; periods associated to main modes are lower than 1sec, characterizing a really rigid structure. Main modes fall within the first two spectrum section, above all in the previous one.

Table 8. Modal analysis outputs for single un-deformable deck model with fixed joints: main translational modes

Mode	T[s]	Ux (%)	UY (%)	UZ (%)
31	0.222	6.0290	0.0440	0.054
80	0.064	0.0540	5.0330	1.908
81	0.064	8.7810	0.0350	0.036
82	0.061	0.0940	0.5630	4.164
84	0.056	0.0002	7.9210	1.238
85	0.054	2.1150	0.0160	4.611
89	0.041	0.4020	0.8200	8.046
92	0.031	7.0890	1.2700	0.248
93	0.031	7.6590	0.2560	2.374
94	0.030	15.347	0.0420	0.938
96	0.025	0.0330	8.5630	0.002
99	0.013	5.0230	0.0096	0.047
100	0.011	0.0100	8.6410	0.001
tot	-	72.45	72.719	98.376

Table 9. Modal analysis outputs for single un-deformable deck model with fixed joints: main rotational modes

Mode	T[s]	RX (%)	RY (%)	RZ (%)
31	0.222	2.426	17.840	0.014
63	0.113	23.091	0.000	0.185
80	0.064	6.062	3.663	0.148
90	0.032	0.149	0.026	29.147
95	0.028	0.001	0.171	6.794
98	0.018	0.057	0.012	7.839
tot	-	55.386	55.1	66.56

4 CONCLUSIVE REMARKS

Viadotto Olivieri is the result of a careful design by Benini and Schmidt: 60 years from its construction, the current structure, apart from necessary maintenance works, shows an appropriate structural response, though being a little flawed when horizontal forces act. Its original isostatic scheme, almost inevitably if it's considered the lack of sophisticated calculating machines during the Second Post War reconstruction, has guaranteed a certain simplicity in approaching to design calculations, also giving a great contribution to reduce problems related to temperature gradient (Δt) or concrete shrinkage, ensuring bridge serviceability. Its historical value and the structural peculiarity make Viadotto Olivieri a valuable Italian example of Miallart arch-type bridge: each maintenance or retrofitting work should be led through a more conservative approach, preserving the existing viaduct.

At the current state, this bridge consists of three separated portions, a layout that guarantees it to have a great flexibility; this characteristic could be assumed as a great potential for the bridge to mitigate any negative effects addicted by acting loads. In particular, the central arch, as a Maillart-arch-type bridge, appears very deformable, above all when out of plane forces act: at expense of no negligible deck displacements, which could involve cross walls buckling, stress addicted to concrete vault and to the upper deck are very low. Stiffer deck is connected to the lower slender vault through a series of thin cross walls, as pendulums: the arch bends as the upper deck makes. Despite of this bridge thrust regime is governed by a really thin vault, whose shape is funicular of the applied dead loads, no problems occur when vertical forces stand upon it: considering that dead loads prevail on live ones (L/D=6.50%), when only vertical forces act, the effect of accidental loads is quite negligible. Instead, having also horizontal forces, some critical features can be seen for the arch, in terms of overturning effects, and for thin cross walls, in terms of sliding-induced buckling. In addiction, modal analysis output reveals a rigid enough structure, whose main vibration periods do not exceed 1.0s and correspond to modal deform shapes into which few macro elements are involved each time.

A comparison between the aforementioned models (Guidi, 2018) (Caglayan et al., 2012) underlines that an appropriate an efficient solution to improve bridge seismic response could be that of having a single deck. The scheme of kinematic chain would make the arch the less loaded portion, while the abutments will carry about 80% of the overall shear force, leading also to a reduction of ΔN of about 35%. In this case uplift effects will be negligible, except for abutments.

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