SEISMIC ANALYSIS AND RETROFITTING OF THE HIGHWAY BRIDGE OVER THE RIVERS TANARO AND BORMIDA

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ABSTRACT

Seismic retrofitting of existing bridges, aimed at enhancing their safety level under earthquake conditions according to current design standards, is often a challenging task. When designing the strategy for seismic retrofitting, in fact, the practitioner have often to face remarkable limitations to the possible interventions, mostly due to the geometry of the original structures and to construction site accessibility issues. A meaningful example of this task is represented by the concrete bridge over the Tanaro and Bormida rivers of the A21 Italian highway between the cities of Turin, Alessandria and Piacenza, managed by Satap S.p.A in Turin. The overall bridge is characterized, in fact, by three different structural types consisting of: (a) a box girder deck, (b) a deck supported by regular r.c. beams and (c) a deck supported by p.c. beams.

The analysis of the seismic vulnerability of the bridge in its ‘as-built’ configuration has been carried out according to the current Italian code DM 14/01/2008. The resulting seismic shear forces on the bridge piers and foundation piles are larger than their shear capacity, showing the need for seismic retrofitting. Instead of using conventional strengthening techniques which increase the capacity of the structure to meet the expected demand, in this case, the strategy of seismic demand reduction was adopted, which consists in minimizing the demand caused by the given earthquake to such an extent that it becomes less than the capacity of the existing structure. This strategy was pursued by adopting a seismic isolation system made of friction pendulum bearings, that are particularly suitable as energy dissipating elements and allow the reduction of seismic forces and, at the same time, of the displacements (this leading to lower dimensions of the expansion joints). The present paper shows and discusses the results of the vulnerability analysis and the design of the retrofitting intervention.

INTRODUCTION

The bridge on the Tanaro and Bormida rivers belongs to the A21 highway, connecting the cities of Turin, Alessandria and Piacenza. The bridge consists of three different structural systems: (a) a series of statically independent 20 m-spans with decks made of simply supported r.c. beams linked in the transverse direction by means of secondary stiffening elements connected at the top by means of a cast-in-situ slab, (b) statically independent 33 m-spans with decks made of simply supported p.c. beams linked in the transverse direction by means of secondary stiffening elements connected at the

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top by means of a cast-in-situ slab, and (c) a continuous 186 m-long box girder deck, having both longitudinal and transverse stiffening ribs. The 3 different structural configurations are shown in Fig.1.

A seismic vulnerability analysis was carried out on the whole bridge by means of numerical Finite Element models of the three different structural systems. In order to correctly design the retrofitting intervention, it was necessary to calculate the relative displacements of the different parts of the bridge, so that suitable seismic gaps could be designed. If the creation of new gaps could not be pursued because of the geometrical configuration of the bridge, a reduction of the seismic displacements could be achieved, in order to avoid pounding between two adjacent deck segments. Since the conceptual approach and the executive design were the same for the three structural systems, in the present paper only the box girder part of the bridge will be dealt with.

VULNERABILITY ANALYSIS

Before the actual numerical structural analysis of the bridge was implemented, a preliminary analysis was carried out, in order to choose the correct behaviour factor (q) and the effective stiffness of the piers. Following the preliminary study, a Finite Element structural analysis of the bridge in the 'as-built' configuration was carried out, implementing the numerical models into the commercial structural analysis software "PRO_SAP, Professional Structural Analysis Program" – 2 S.I. Software per l'ingegneria S.R.L.

Preliminary analysis

In order to compute the base shear for each pier, a q factor equal to 1 for both the vertical and horizontal seismic input was chosen.

Figure 1. Bridge over the Tanaro and Bormida rivers: r.c. beam deck (top-left), p.c. beam deck (top-right) and box girder deck (bottom).

The low value of the q factor was determined on the basis of the moment-curvature plots of the base sections of the piers and of the displacement ductility of the same sections. The shear capacity of the foundations was evaluated as well. In the case at hand, the global shear capacity of the piers-foundations system is limited by the shear resistance of the foundation piles, which are transversely reinforced by a Φ10/25 spiral made of steel type Aq 50 (R.D. 16/11/1939 N.2229, 1949). The calculations on the strength capacity for the different failure mechanisms allowed to conclude that, under seismic load, the piers have a fragile behavior, since the shear failure is reached before the collapse due to bending action. Hence, the q factor was assumed equal to 1.

The seismic base shear evaluated by means of the numerical analysis has been later compared with that evaluated via simplified hand calculations. The total weight of each roadway of the box girder deck is approximately equal to 51118 kN, and the fraction that is supported by each central pier is equal to 24156 kN. At first, it was assumed that the fundamental period of the structure for both longitudinal and transverse earthquakes was within the range $T_B - T_C$.

The corresponding value of the design response spectrum, according to NTC 2008, is equal to:

$$a_g \cdot S \cdot \eta \cdot F_0 = 0.33\ g \tag{1}$$

The shear action at the top of each pier is, therefore, $0.33 \cdot M \cdot g$, where M is the total mass of the bridge when the longitudinal seismic action is considered, since all the horizontal action has to be counteracted by the only fixed pier ($0.33 \cdot M \cdot g = 16869$ kN). When the lateral seismic action is considered, the shear is approximately one half, since the action is divided between the two central piers, and, only to a minor extent, supported by the lateral abutments ($0.33 \cdot M \cdot g / 2 = 8434$ kN).

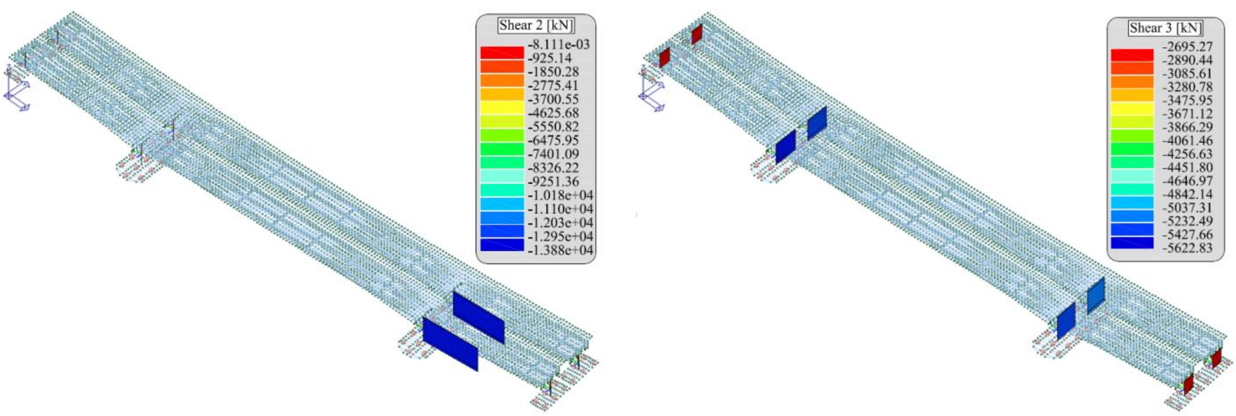
The values of the shear demand are largely greater than the shear capacity of the piers-foundations system, which is equal to:

$$F_{Rd} = (19 \cdot 204\ \text{kN}) / 2 = 1938\ \text{kN} \tag{2}$$

The above displayed values, obtained by hand calculations, were thus validated by a Finite Element analysis, developed with the software PRO_SAP.

For the seismic analysis, the box girder deck was modeled together with the two adjacent kinematic chains of r.c. beams. The geometric features of the structure were simulated accurately, including internal transverse beams, variable cross sections and planimetric curvature.

The foundation slab and the piles were modeled as well. The box girder deck was modeled with bi-dimensional shell elements. For piers and foundation piles, beam elements were used. For the soil-structure interaction, a Winkler model was adopted. In the case at hand, the numerical analysis via FE software proved to be essential for understanding the effective behavior of the structure, taking properly into account the abovementioned geometric features, the non-negligible flexibility of the piles and the axial deformability of the deck (Fig.2).



(a) (b)

Figure 2. Shear loads in the piles in: (a) x-direction (seismic action in x-direction), and (b) y-direction (seismic action in y-direction).

The most severe shear values obtained from the FE analysis are equal to 13888 and 5622 kN in longitudinal and transverse direction, respectively (corresponding to 82% and 67% of those calculated by hand), as shown in Figs.2a,b.

The preliminary analysis of the structure as it is, has, therefore, confirmed the need of an intervention for seismic retrofitting in order to withstand the design seismic actions established by the actual code, NTC 2008. It may be noticed that the lateral shear actions are larger than the shear resistance of the foundation piles (equal to 1938 kN per each central pier). Consequently, it would not have been possible to redistribute the seismic action between central and lateral piers.

In the case in object, a seismic retrofitting of the bridge was thus designed, with the goal of increasing the global capacity through massive strengthening, or, alternatively, reducing the demand adopting, for example, the base isolation strategy.

SEISMIC RETROFITTING: OBJECTIVES AND RESULTS

As above described, the bridge as such proved to be inadequate from the seismic vulnerability point of view for the design earthquake defined accordingly to NTC 2008. Different possibilities of retrofitting have been investigated with the aim of fulfilling the code provisions.

The complexity of existing structure retrofitting comes from the various economical and geometrical restraints that the designer has to take into account when defining the different interventions, which have to be, at the same time, effective and convenient from the executive point of view. The main aspects to be taken into account are: (a) geometrical restraints related to the structural configuration, (b) feasibility, compatibility and the necessity of minimizing the downtime, and (c) the optimization of the overall cost-to-benefit ratio of the intervention (Pinto et al., 2009).

In the case of the bridge over the Tanaro and Bormida rivers, the distance between the box girder deck and adjacent r.c. beam decks was rather high, leaving wide margins in terms of relative displacements. This suggested the possibility of adopting an isolation system characterized by a rather low stiffness, able to significantly reduce the shear forces on piles and foundation (Dolce, 1994).

On the contrary, any intervention of foundation strengthening seemed hardly applicable due to feasibility restraints (mainly for riverbed piles). Moreover, since the replacement of the supports was a prerequisite for the design (which was also aimed at improving the durability of the bridge), the choice of introducing an isolation system by means of elastomeric isolators or friction pendulum bearings seemed the most convenient strategy. Consequently, it has been decided to reduce the seismic actions on piles and, mainly, on the foundations, by adopting friction pendulum bearings (six and four on each central and external pile, respectively).

Such an intervention is based on two pillars: to distribute the shear forces accordingly to the stiffness of the isolators and to keep the box girder deck separate in terms of horizontal displacement from the adjacent decks, thanks to properly designed seismic joints. Obviously, to correctly design the seismic joints, the numerical modeling of all the parts of the viaduct has been performed, with the aim of defining the joint width necessary to arrange all the displacements, as discussed in the following Sections.

The well known basic principle of seismic isolation is to uncouple the motion of the ground and of the structure, introducing a disconnection between deck and piles (the latter, however, remains rigidly connected to the ground). As for the transmission of vertical loads, the structural continuity is guaranteed by the seismic isolators thanks to their high vertical stiffness (despite their limited horizontal stiffness).

The increase in global deformability brought in by the isolation system leads to increasing values of the natural period of the structural system (substructure – isolation – superstructure), for which the associated accelerations given by the spectrum decrease. Consequently, the seismic acceleration experienced by the isolated structure is dramatically lower with respect to that in the case of non-isolated structure, for the same level of peak ground acceleration – PGA (Dolce et al., 2010).

On the other hand, the increase of the natural period has some drawbacks, first of all the increase of the displacements of the isolation devices (i.e. the elements where most of the earthquake energy is absorbed and dissipated).To avoid unacceptably high values of displacements in the isolation system - leading to the need of very wide seismic joints (which in turns would be less durable and more expensive to repair) - the isolation system should be characterized by high dissipative capability.

In the following, the preliminary design and the subsequent numerical modeling via a FE Software will be described, in the specific case of the box girder deck (bridge over the Bormida River).

Preliminary Design of the Seismic Isolation System

The isolation system of the structure in object has been designed with three main goals:

- to bear the vertical loads;
- to limit the seismic actions;
- to increase the natural period of the structure.

The preliminary design of the seismic retrofitting consisted on the definition of stiffness and damping capacity of the isolation system (on which modal natural periods and damping of the structure depend), together with the fundamental features of the dynamic response (seismic loads and shear forces at the basis of the structural members).

In particular: firstly, the values of natural period and damping capacity needed to sufficiently reduce the seismic effects on the decks have been evaluated and, secondly, the stiffness of the global isolation system and of each isolation device have been determined. In this way, the global performance target of the isolation system could be clearly defined.

With reference to the procedures described by NTC 2008 (Section 7.10.5.3.1 – Linear Static Analysis), shear loads and displacements of the structure have been computed by using handily simplified models (characterized by low computational cost, even though some approximations are introduced) for different isolation systems, as mentioned in the previous Sections. Note that, if friction pendulum bearings are used, the overall behavior can be assumed as linear elastic, by adopting an “effective” equivalent value for the stiffness (as defined by NTC 2008, Section 7.4).

Figs.3a,b show the acceleration and the displacement elastic spectra for different values of damping (5, 10-15 and 30%, namely: average value for a structure without isolation, average value for one with an elastomeric isolation system and limit value provided by the Codes for linear analysis – the latter easily guaranteed by adopting friction pendulum bearings). Starting from the spectrum given by NTC 2008 (Section 4.2.1), Fig.3a clearly shows how an increase of period and/or damping leads to lower values of the spectral acceleration and of the ensuing seismic actions.

The displacement spectrum is obtained for different values of damping by multiplying the spectral ordinates for $(T/2\pi)^2$. From Figs.3a,b, it is worth noting that an increase in period leads to opposite results for acceleration and displacements (the former decreases while the latter increases); on the other hand, an increase in damping leads to a reduction of both acceleration (hence, of shear forces) and displacements. This consideration justifies the choice of friction pendulum bearings, which provide higher values of damping with respect to elastomeric devices.

In the preliminary design of the isolation system, the Acceleration-Displacement Response Spectrum (ADRS) proves to be a useful tool, representing at the same time the evolution of (pseudo-) acceleration and displacements for different values of damping, as reported in Fig.4.

The angle θ, slope of the straight lines passing through the origin of the graph, is linked to the period through the following equation:

$$tg\theta = S_e/S_{De} = 4\pi^2/T^2 \tag{3}$$

ADRS allows to identify all the main characteristics (period and damping) of the isolated structure.

For any given values of target displacement and acceleration, the natural period and damping necessary to obtain seismic actions lower than the bearing capacity of the structural members can be easily evaluated. Assuming a natural period of the isolated structure higher than T_D, the displacement is constant (equal, for instance, to 56.3 mm for ξ = 30%; see Fig.3b), while the acceleration sharply decreases for increasing periods.

Then, the global shear forces have been computed in the case of ξ = 30%, for different values of natural periods of the isolated structure (IS). In Table.1, these values are compared with those obtained in the case of the non-isolated structure (NS).

Table 1.Evaluation of the seismic shear force applied in the different configurations.

C	**C_f**	**T[s]**	**M[t]**	**ξ [%]**	**η[-]**	**$S_e(T)$[g]**	**F_h[kN]**	**% NS [%]**
1	NS	0.65	5211	5	1.000	0.330	16870	100
2	IS	1.50	5211	30	0.535	0.080	4090	24
3	IS	2.00	5211	30	0.535	0.057	2914	17
4	IS	2.50	5211	30	0.535	0.036	1840	11

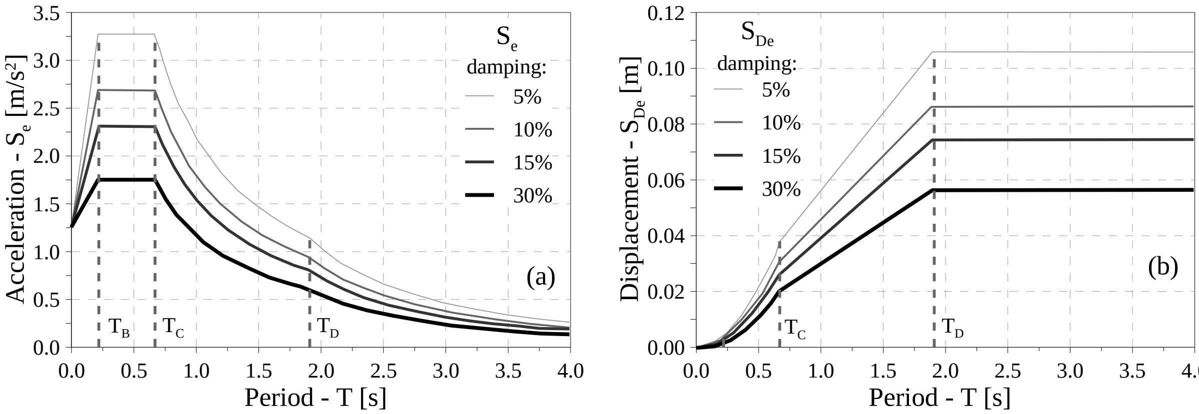


Figure 3. Plots of: (a) acceleration elastic spectrum and (b) displacement elastic spectrum, for different values of damping (LSL).

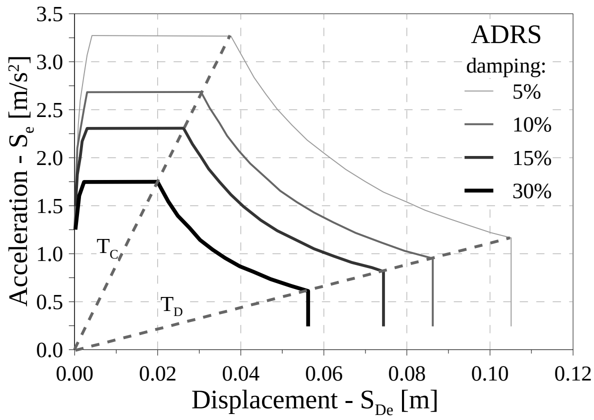


Figure 4. Acceleration-Displacement Response Spectra (ADRS) for different values of damping (LSL).

The global shear force related to one roadway of the box girder deck has been evaluated as follows:

$$F_h = M \cdot S_e(T) \tag{4}$$

where $M = M_{iso}$ = 51118 kN = 5211 t = total mass of the deck.

The natural period of the isolated structure is:

$$T_{is} = 2\pi\sqrt{(M/k_{esi})} \tag{5}$$

In Eq.(5), the total stiffness of the isolation system has been introduced:

$$k_{esi} = \sum_i k_{e,i} \tag{6}$$

being $k_{e,I}$ the horizontal stiffness of each isolation device.

For given values of the natural period, the stiffness of the isolation system has been defined in order to figure out the stiffness of the single isolation device.

For a target natural period of 2 s, the seismic actions are reduced up to 17% of the value obtained without isolation (see Table.1); for such a value of natural period, the global horizontal stiffness of the isolation system is:

$$k_{esi} = (2\pi/T_{is})^2 \rightarrow \quad k_{esi} \approx 51.4 \text{ kN/mm} \tag{7}$$

If the target natural period of the isolated structure became 2.5 s, the seismic action would be reduced up to 11% of the value without isolation (see Table.1); in this case, the horizontal stiffness of the isolation system would be:

$$k_{esi} = (2\pi/T_{is})^2 \rightarrow \quad k_{esi} \approx 32.9 \text{ kN/mm} \tag{8}$$

On the basis of the aforementioned considerations, it has been decided to define an isolation system able to increase the natural period of the structure up to 2-2.5 s, with a global horizontal stiffness in the range of 32.9-51.4 kN/mm, in order to obtain a seismic action equal to 11-17% of that in the case of non-isolated structure, this value being compatible with the bearing capacity of piles and foundation.

The corresponding displacements are expected to be lower than 60 mm, as reported in Fig.3b, with reference to a damping coefficient of 30%.

The global stiffness, equal to the sum of the stiffnesses of the 20 isolation devices, represents the starting point to choose the friction pendulum bearings. Note that, since the effective stiffness of the friction pendulum bearings depends on the applied vertical load, their calculation is an iterative procedure. In the following Section, the design of the isolation system is finalized.

Design of the Isolation System

The dynamic behavior of friction pendulum bearings can be described in a simplified way through the scheme reported in Fig.5. One of the advantages of these devices is that their natural oscillation period is independent from the vertical applied loads, this making the uncertainties related to mass distribution and eccentricities totally negligible. Mass and stiffness centers coincide.

The natural period of the isolation system is defined starting from the two main properties of the friction pendulum bearing: curvature radius R and dynamic friction coefficient μ (see Fig.5).

The damping coefficient also depends on the features of the device only; hence, the exact values of natural period and damping coefficient can be worked out. Note that Codes allow linear analyses to be performed only if the damping coefficient is assumed smaller than 30%.

In order to carry out a linear modal analysis by using a Finite Element Software, the mean effective stiffness has been evaluated depending on the seismic mass of the system.

As for the box girder deck, the isolation devices on the central piles (six per each pile) are subjected to a vertical load equal to 4200 kN (corresponding to the seismic mass), while the isolation devices on the external piles (four per each pile) are subjected to a much lower load, equal to 111 kN.

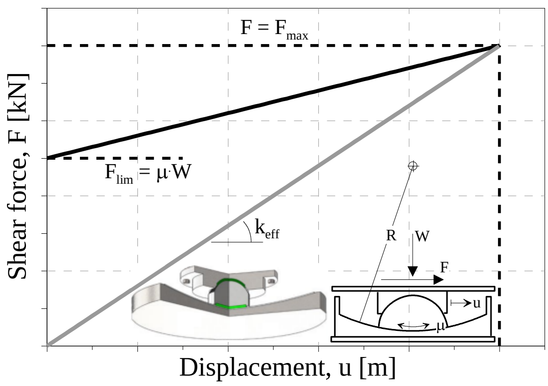


Figure 5. Plot of the Shear force-Displacement relation (F-u) for fiction pendulum bearings.

Because of the remarkable difference in terms of vertical loads, a separate evaluation of the horizontal stiffness for central and external piles is needed, according to the scheme reported in Fig.5, on the basis of the data provided by the producer.

In particular, the maximum load in static conditions at the ULS for each device is 8768 kN on the central pile and 1637 kN on the external piles, while the effective stiffnesses are 3.23 and 0.09 kN/mm, respectively.

Numerical Analyses of the Isolated Structure

The structural analysis of the isolated system has been performed on the same FE model previously implemented in the FE Software PRO_SAP for the as-built structure (without isolation). 'Isolator' Elements have been introduced in the model of the isolated structure to properly take into account the vertical and horizontal stiffnesses, and the damping coefficient.

Starting from the features of the isolation devices (curvature radius and friction coefficient equal to 4000 mm and 3%, respectively), the following values have been worked out:

- central piles: vertical load per each support due to permanent load equal to 4200 kN and horizontal stiffness of 3.23 kN/mm;
- external piles: vertical load per each support due to permanent load equal to 111 kN and horizontal stiffness of 0.09 kN/mm;

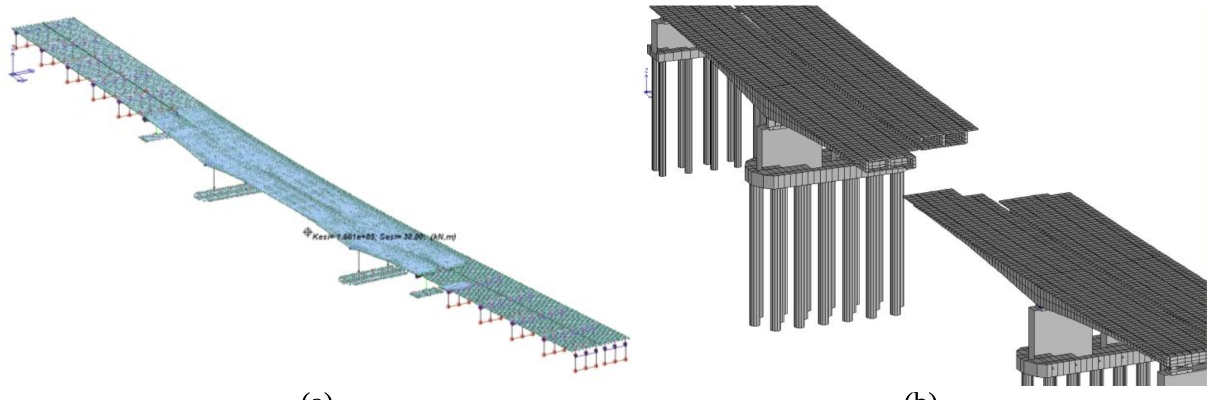
The vertical stiffness has been assumed to be 1000 times the horizontal one.

In Fig.6 the axonometric view and a 3D perspective of the structural model are reported. It is worth noting that the introduction of the isolation system is only part of a more comprehensive retrofitting intervention, which targeted not only seismic vulnerability but also capacity and durability. For this reason, in the FE model of the isolated system, the geometric characteristics of the structure have been updated on the basis of the interventions needed for the retrofitting of the deck and of the piers and for the replacement of the bearings

These interventions can be summarized as follows:

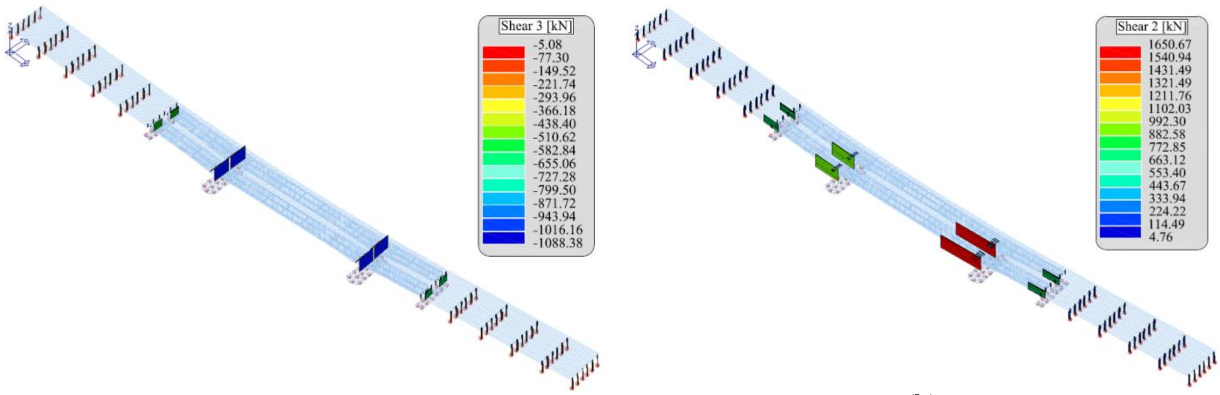
- an increase of the thickness of the upper slab of the deck from 240 to 270 mm, to remove the original concrete cover, insert additional rebar and cast a new layer of high strength concrete
- an increase of the thickness of the road paving, up to 120 mm, in order to make it uniform all along the length of the bridge;

Linear modal analyses have been performed on the updated FE model to compute the seismic loads. In order to take into account the damping of the isolation system, the reference elastic spectrum is reduced for all the periods $\geq 0.8\ T_{is}$ via the coefficient η, which is evaluated for $\xi = \xi_{esi}$, according to the provisions given by NTC 2008 (Section 7.10.5.3.2). The value of ξ_{esi} has been assumed equal to 30% (i.e, the limit value for linear dynamic analysis given by NTC 2008). Some results in terms of shear loads on the piers and piles and of maximum displacements are reported in Figs.7,8. The loads are automatically computed by the FE Software by means of the CQC approach, considering all the modes needed to exceed the 85% of the total mass.



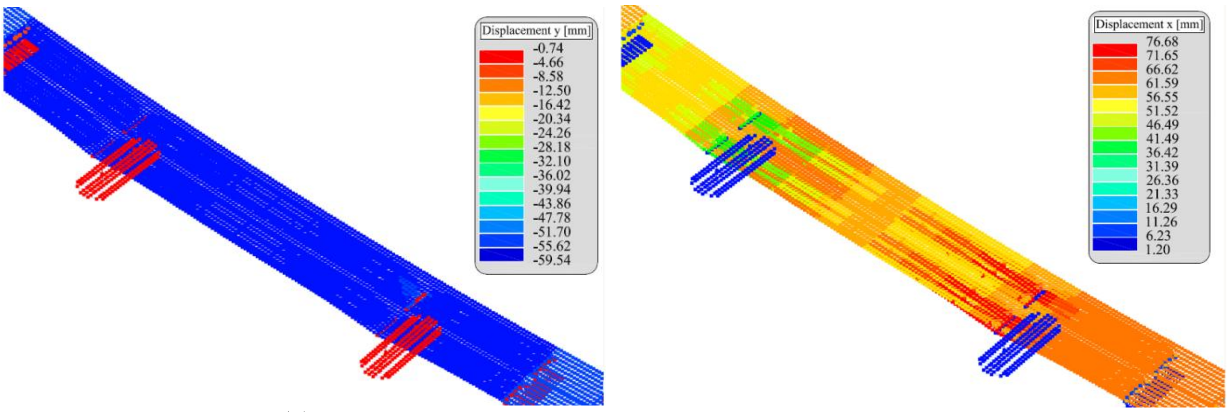
(a) (b)

Figure 6. (a) Axonometric view of the FE model provided with the isolation system, and (b) axonometric view and detail of deck, piles and foundation.



(a) (b)

Figure 7. Shear loads in: (a) y-direction (seismic action in y-direction), and (b) x-direction (seismic action in x-direction).



(a) (b)

Figure 8. Displacement in: (a) y-direction (seismic action in y-direction), and (b) x-direction (seismic action in x-direction).

STRUCTURAL ASSESSMENT OF THE SEISMICALLY ISOLATED BRIDGE

Structural members

All the structural members in the new retrofitted configuration, in particular piers and foundations, were successfully checked for shear, bending and punching shear actions resulting from the design earthquake input.

It is worth noting that the maximum shear loads in the central piles of the isolated structure were reduced, respectively, to 12 and 19% of the values of the as-built structure. The values of the displacements and of the modal properties of the system are listed in the followings.

Displacements and Modal Properties

The maximum displacements of the decks are 77 and 60 mm in longitudinal and transversal directions, respectively. The first natural period of the structure (torsional mode) is 2.52 s, the second mode (longitudinal shift) is 2.40 s and the third (transversal shift) is 2.36 s.

As for the insulation devices on the central piles:

- longitudinal maximum displacement: 71 mm
- transversal maximum displacement: 55 mm
- maximum absolute displacement: 73 mm

As for the insulations devices on the external piles:

- transversal maximum displacement: 59 mm
- maximum absolute displacement: 63 mm

Design of the Deck Joints

For the box girder deck and the r.c. beam deck, the following values of displacement in longitudinal direction have been derived from the dynamic modal analysis:

- box girder deck: D_{max} = 77 mm;
- r.c. beam deck: D_{max} = 49 mm.

Assuming these displacements to be in counter-phase, the maximum displacement between the two decks becomes D = 126 mm. In addition to this value, the thermal dilation needs to be considered. According to the provisions by NTC 2008 (Section 5.2.2.5.2), the effects of a thermal gradient $\Delta T = \pm 15°C$ must be taken into account, increased by 50% for joints and supports.

The final values thus become $\Delta T = \pm 22{,}5°C$, inducing the following displacements:

- box girder deck: $\Delta L_{t1}=10^{-5}\,°C^{-1} \cdot 186\ m \cdot 22.5\ °C = 42.0mm$;
- r.c. beam deck: $\Delta L_{t1}=10^{-5}\,°C^{-1} \cdot 100\ m \cdot 22.5\ °C = 22.5\ mm$.

Such displacements have to be multiplied by 0.5 (i.e. the value of Ψ_2 in the seismic combination as defined in NTC 2008) and furthermore by 0.5, two being the longitudinal directions of dilation; the following values are finally obtained:

- box girder deck: $\Delta L_t = 0.5 \cdot 0.5 \cdot 42.0\ mm = 105.0\ mm$;
- r.c. beam deck: $\Delta L_t = 0.5 \cdot 0.5 \cdot 22.5\ mm = 5.6\ mm$.

Hence, the displacement value for the design of the joint has been assumed equal to 150 mm.

Loads and Displacements in the Isolation Devices

The results in terms of loads and displacements on the isolation devices are summarized in Table.2,3.

Table 2. Seismic displacements on the isolation devices on the central and external piers (cp and ep, respectively).

	$S_{x,sism,max}$[mm]	$S_{y,sism,max}$[mm]	$S_{max,sism}$[mm]
cp	71	55	73
ep	62	59	63

Table 3. Vertical static and seismic loads on the central and external piles (cp and ep, respectively). Sign minus in tension.

	$V_{max,SLU}$[kN]	$V_{SLE,q.perm}$[kN]	$V_{sism,max}$[kN]	$V_{sism,min}$[kN]	$V_{sism,med}$[kN]
cp	8768.0	4195	5044	3346	4193
ep	1636.5	111	172	38	105

CONCLUDING REMARKS

The retrofitting intervention designed for the bridge over the Rivers Tanaro and Bormida was aimed at increasing both service life performance and durability, and at improving the seismic performance.

The seismic vulnerability assessment proved some inadequacy of the piers and foundations, since the bridge had been designed well before the new seismicity maps for Italy were defined in NTC 2008. Consequently, the seismic shear actions induced by the currently defined design earthquake for the site where the bridge is located are higher than the bearing capacity of the lateral load resisting members.

For this reason, different strategies for the seismic retrofitting intervention have been investigated:

- structural strengthening of the foundation;
- isolation of the deck.

The first choice is by far more expensive and time consuming; hence, the second strategy has been pursued, namely the replacement of the existing supports by friction pendulum bearings, whose design, analysis and verification procedures have been discussed in the present work.

Considering isolation devices with a curvature radius of 4000 mm and a friction coefficient of 3%, the designed isolation system has the following properties:

- the vertical load due to permanent load in each support of the central piles is 4195 kN, with a corresponding horizontal stiffness equal to 3.23 kN/mm;
- the vertical load due to permanent load in each support of the external piles is 111 kN, with a corresponding horizontal stiffness equal to 0.09 kN/mm.

The vertical stiffness of the isolating devices has been assumed equal to 1000 times the horizontal one, while the damping coefficient has been assumed equal to 30% (i.e. the limit value allowed by NTC 2008 to perform a linear modal analysis).

The sectional safety of all the structural members in the retrofitted configuration was evaluated by means of a linear response spectrum analysis implemented in a FE model of the retrofitted bridge, from which the shear, bending and punching shear actions for each member were derived by means of the CQC approach.

The beneficial effects of the isolation system are two-fold: (a) significant reduction of the shear forces on all the structural members (by more than 80% in the present work), and (b) accommodation of the displacements induced by the design earthquake, thanks to the proper design of the deck joints.

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