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Damage analysis and seismic retrofitting of a continuous prestressed reinforced concrete bridge

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SUMMARY

The seismic analysis and retrofit of prestressed reinforced concrete bridge is discussed by considering a real case of a viaduct still in use. The unique features of this bridge make this type of bridge particularly interesting, either structurally or architecturally. The paper begins with the analysis of certain particular structural deficiencies that emerged during the viaduct operation. The results of the analysis indicate that the structural performance can be enhanced by only modifying the support devices. The primary structural components are not required to be involved in the retrofitting process. Using the modern seismic code, the upgrading of the viaduct performance is obtained by replacing the old bearing devices on the piers and existing viscous dampers connected abutments to the deck with new modernised ones.

Introduction

Due to the particular typology, lifetime loading and environmental condition of bridges and viaducts, enhancing the structural performance of existing bridges and viaducts involves special issues. In certain cases, the requirement of strengthening arises not only from deficiencies of the primary elements but also from the insufficiency of the structural details. Indeed, the viaduct presented in this paper faces these challenges. The design of the prestressed reinforced concrete deck of more than one kilometre in length was made to be structurally continuous above the supports given by piers and abutments, with the sequence of three different curvatures in the plan being required to address and solve special problems related to

(i) The global behaviour of the structure;
(ii) The thermal effects on such a long monolithic deck;
(iii) The seismic response of the viaduct due to the complex geometry.

In the following, after the reference structure is described in detail, the detected failures are introduced. Next, the possible causes are discussed on the basis of the design documents, as well as the structural analyses performed.

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Description of the bridge structure

The viaduct is made of 32 continuous spans. The total length of the prestressed reinforced concrete deck is 1127 m. According to the design specifications, the structure has been constructed in subsequent cantilever stages. The first and last spans are approximately 27 m long, while the intermediate ones have a length of approximately 36 m. The cross-section is variable along the length, gradually passing from a given mid-span shape to a higher one in correspondence to the supports (Fig. 1). Due to the scheme of the continuous beam, the deck is quite slim, with the height of its cross section varying from 106 cm (at the mid-length of each span) to 156 cm (at the supports).

In plan, the structure is substantially characterised by a curved shape. The first part has a moderate curvature with a radius of approximately 1632 m. Next, the radius of curvature decreases to approximately 743 m. The end part has an almost straight shape. From the altimetric perspective, the viaduct is built with a slope from the upstream abutment (SP1) to the downstream one (SP2), except for a small initial portion. A number of 31 piers in the central part (with a circular cross section having a diameter of 2.60 m and a height that varies between 4 m and 9 m) and two end abutments represent the supports for the deck. Two bearing devices are placed on each pier and each abutment, leaving free the relative longitudinal displacement of the connected parts.

Bearings

The viaduct kinematics associated with thermal effects is characterised by homothetic deformation with respect to the deck axis. Such displacements are allowed because of the presence of a two unidirectional sliding bearings for each pier, with each being characterised by a stroke of ±300 mm.

Regarding the response under seismic actions, for the design, the deck has been assumed to behave as a rigid body oscillating in the horizontal direction and not constrained on the piers but constrained only at the two abutments. The connection between the two ends of the deck and the abutments has been realised by means of four viscous dampers (two for each abutment). According to the original design, the seismic horizontal displacement of the deck is mainly represented by a rigid rotation around a vertical axis defined by the intersection of the planes containing the two end cross sections of the deck (Fig. 2). However, because of the particular geometry, the axes of the unidirectional sliding bearings do not coincide at a unique point. Therefore, the rigid deck rotation around the vertical axis is not allowable. To allow such rotation, the designer established that the unidirectional constraint devices were assembled on elastomeric reinforced pads. Moreover, due to the adoption of a metallic carpentry, only displacements transverse to the axis of the viaduct are allowed, thereby preventing the activation of those longitudinal displacements.

Dampers

Two viscoelastic dampers have been originally designed and placed at each end of the bridge. Each device is equivalent to a viscous damper and a spring connected in series. The purpose of these devices is to dissipate part of the input energy coming from seismic action and to link in a non-rigid manner the viaduct to both abutments. The innovativeness of such design is acknowledged considering the age of construction. Nevertheless, such link has no re-centring capability regarding the deck.

Fig. 1. A picture of the viaduct.

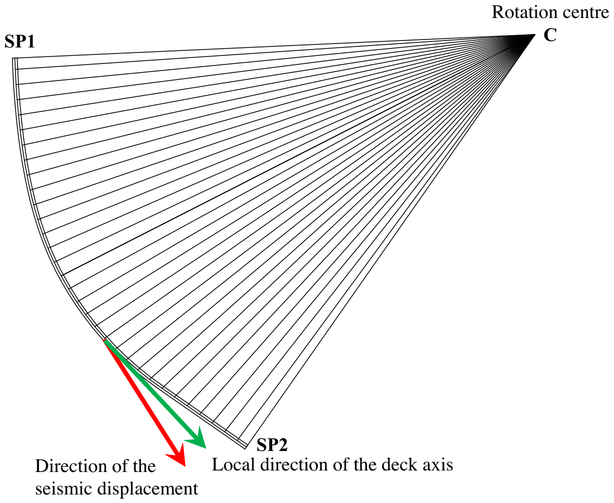

Fig. 2. Kinematics in plan of the viaduct.

Damages and causes

The damaged state the viaduct exhibits is described in the following, primarily involving abutments and supports on the piers.

At the abutment SP1 (upstream):

(i) the expansion joint is extended beyond its capacity and its modular elements are broken;
(ii) the dampers are roughly at the end of the stroke in extension (Fig. 3);
(iii) the support devices are almost at the end of the stroke in the downstream direction.

At the SP2 abutment (downstream):

(i) the expansion joint is fully closed due to excessive compression;
(ii) the dampers are in the end-stop position in contraction;
(iii) the support devices are at the end of the stroke in the downstream direction.

At the bearings on piers:

(i) all the bearings are significantly shifted in the downstream direction, although with a displacement magnitude that varies (from 1 to 29 cm) for each pier (Fig. 4);
(ii) in many cases, the screws joining the support devices plates to the deck are pulled out because of the excessive rotation around the transverse horizontal axis of such plates.

Fig. 3. Upstream abutments: dampers in extension.

Fig. 4. Sliding displacement of a bearing device.

From observation of the damage, it is evident that the entire deck of the viaduct is shifted in the downstream direction, following a sub-horizontal rigid motion, which corresponds to a rotation around the centre previously identified. This displacement perfectly corresponds to the constraint system designed and built for the viaduct, which was once practically hypostatic. The action that has activated such a degree of displacement is the inclined component of the gravity force associated with viaduct weight. This component is due to a non-perfect horizontality of the support devices and the lack of re-centring devices placed between the deck and the abutments. Note that the pre-stressed reinforced concrete deck and the piers are in satisfactory condition.

Structural analysis for gravity loads

The viaduct has been designed according to the code in force during that period [1]. The current code [2] provides different ways for both the definition of the structural demand (moving loads) and for the evaluation of the capacity (limits for the tensions and deformations). Comparing the envelope diagrams of internal forces derived by these two codes, considering the most unfavourable positions of the moving loads, with reference to the characteristic value of the actions, the results indicate that the bending moment assumes almost the same negative values in both cases. The maximum values of positive bending moment evaluated with the current standard are larger (25 ÷ 30% increase). The shear forces also results to be greater when evaluated with the new code (30 ÷ 35% increase).

Nevertheless, these differences are strongly reduced when both dead and variable loads (in the following G_k and Q_k, respectively) are considered. In fact, the increase of the positive bending moment and the shear given by the most recent code assumes values smaller than 10%. The differences, although not so large, also required the evaluation of the structural capacity of the bridge according to the Italian code [2] for the required verifications. All the verification work, which were performed both at the serviceability and the ultimate limit states, gave positive results.

The piers having circular cross section satisfy all the verifications conducted with the new standard without any problems [3]. However, the support devices placed on piers are solicited by stresses that sometimes exceed the capacity in terms of the axial load declared in the original design (9000 kN), even for a value of 10%. Therefore, the devices must be slightly undersized.

Structural analysis for seismic action

To analyse the dynamic behaviour of the structure and the response under the seismic actions of design, a Finite Element Model (FEM) of the viaduct was realised using the software SAP2000 [4]. Particular attention has been paid in modelling the complex system of pier-deck links, as discussed above. The modal analysis of the viaduct in its original state was performed. The first four periods of vibration resulted to be 153 s, 1.99 s, 1.83 s and 1.79 s. Most of the mass (88%) participates with these first modes. Nevertheless, all of the analyses used to evaluate seismic effects were performed with the inclusion of the first fifty modes. The high value associated to the first period of vibration once again highlights the condition of structural hypostaticity due to the substantially unconstrained rotational degree of freedom. To be more precise, referring to the criteria used for the seismic design of the viaduct and the supporting devices (See Bearings), from a dynamic point of view, the structure can be represented as a rigid body with a rotational degree of freedom. This type of movement is partially constrained only by the elastic lateral stiffness of the elastomeric bearings. Because this stiffness is clearly inadequate to resist the inertial forces associated with the structural and non-structural total mass, the global dynamic characteristic of the entire deck in its first mode of vibration is equivalent to a structure that is virtually hypostatic and characterised by a very large first elastic period of vibration. Hence, the viaduct is not capable to withstand the seismic design force as prescribed in the relevant code. In particular, the absence of elastic elements having re-centring capability for the deck causes the structure to be unable to vibrate around the equilibrium configuration. Consequently, this flaw prevented the damping devices from operating properly to reduce seismic response of the viaduct. In summary, both the survey result and the numerical analysis confirmed that the structure of studied viaduct did not have appropriate lateral capacity regarding the requisite demand; as a result, the structure is exposed to risk and categorised as a vulnerable structure.

Seismic upgrade of the bridge

The seismic protection technique originally designed and realised for the viaduct is a hybrid solution, which is innovative considering the age of construction. It can be described as follows:

- Along the curvilinear direction corresponding to the axis of the deck, a system similar to a modern seismic isolation scheme is installed. The system is based on the adoption of sliding support and supplemental damping devices at the top of each pier and of four viscoelastic dampers at the two abutments.
- Seismic actions in the transversal direction are instead resisted by a traditional seismic system based on the shear and flexural response of the piers.

In the design documents, the seismic protection system is not analysed, but its main features can be derived by the drawings related to the supports, joints and dampers. The examination of such documentation proved that any system and/or device addressed to the re-centring of the viaduct in terms of longitudinal displacements was not properly designed. The result is a structural system unsuitable to withstand to the seismic actions.

In summary, the complex system of seismic protection as designed and built was found to be inadequate and therefore must be corrected.

Design of the structural retrofit

Once the critical examination of various design alternatives for retrofit was performed, the following conclusions were reached. Given the unchangeable boundary conditions due to the way the structure was originally conceived, the optimal strategy for seismic retrofitting of the bridge is the reuse of the current protection system with a suitable revision and correction of the parts that were deficient, even aiming at fulfilling all the Italian code [2] requirements for structural safety.

A simplified elastic damped single degree of freedom dynamic model was assumed for the retrofit, with the main objective being to fix the design to make the fundamental vibration period as close as possible to four seconds. This value is assumed as the best compromise, taking into account the magnitude and the cost of the re-centring devices, the space actually available inside of existing abutments to install them, as well as the analytical possibility to study the dynamics of such type of structure under seismic actions. In fact, both the response spectra and the accelerograms, typically used for design and verification of structures subjected to earthquakes, suffer the limitations of significance when adopted to analyse dynamic systems having a period longer than 4 s.

The mass of the deck is approximately 31,200 tons, including both the structural and non-structural components. Therefore, the elastic stiffness K required to reach a period T equal to 4 s is $K = 4\pi^2 M/T^2 = 77$ kN/mm. Considering the gamma of elastic re-centring devices available on the market and that the viaduct and its abutments were designed to accommodate 4 damping elements, the value $k = 20$ kN/mm was chosen, so that the global re-centring stiffness is $K = 4k = 80$ kN/mm. For this stiffness value, the undamped vibration period of the vibration is $T = 2\pi(M/K)^{0.5} = 3.92$ s, very close to the value (4 s) set as the objective.

Again, aiming to find a compromise between costs and effectiveness of the intervention, the target damping ratio ξ, to be achieved through four dampers, is set as 20%. Recent scientific studies on the subject reported that damping ratios higher than 20% correspond to a marginal reduction of the displacements, but with significant increases of the accelerations [5]. Therefore, first assuming linear viscous behaviour for dampers, the global damping constant C is calculated as follows:

$$C = 2 \cdot 0,2 \cdot \sqrt{K \cdot M} = 19,984\ \frac{\text{kN}}{\text{m/s}} \tag{1}$$

corresponding to a constant $c = 4.996$ kN s/m for each of the four devices.

From the elastic response spectra in acceleration $S_a(T,\xi)$, velocity $S_v(T,\xi)$ and displacement $S_d(T,\xi)$ to be assumed at the Collapse Limit State for the structure and the site objects of the present study, the values 0.029 g, 0.179 m/s and 111 mm, respectively, have been derived for $T = 3.92$ s. Note that this approach of the design is based on the implicit assumption that the dissipation of energy during the seismic action is concentrated only in the above special devices, considering a linear, non-dissipative behaviour of the deck and the piers. For this reason, the behaviour factor is set equal to 1.

The spectral displacement of 111 mm is compatible with the available capacity of the dampers, joints and bearings, even if composed of those induced by thermal variations. The maximum elastic force F_e for each damper is determined by Eq. (2). The demand in acceleration is very low, as expected for the present case, while the pseudo-spectral velocity of 0.179 m/s leads to a maximum viscous force in each damper equal to $F_v = 894$ kN/m (Eq. (2)).

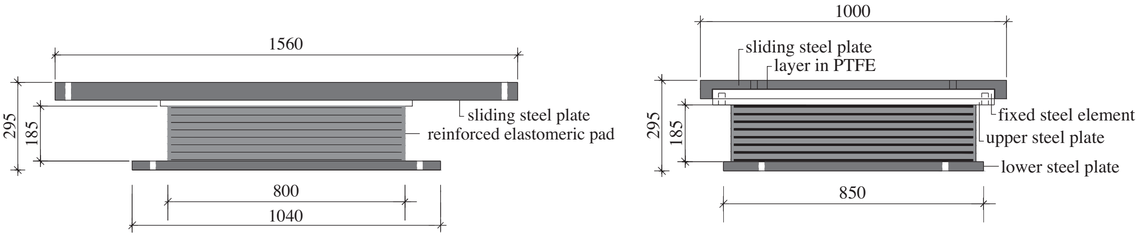

Fig. 5. Purposely designed support devices. Longitudinal (left) and transversal views. Dimensions in mm.

$$F_e = k \cdot D = 20\ \frac{\text{kN}}{\text{mm}} \cdot 111\ \text{mm} = 2222\ \text{kN}; \qquad F_v = c \cdot v = 4996\ \frac{\text{kN}}{\text{m/s}} \cdot 0,179\ \frac{\text{m}}{\text{s}} = 894\ \text{kN} \tag{2}$$

The viscous devices available on the market are typically characterised by a non-linear behaviour such as the one described as $F_v = c{\cdot}v^{\alpha}$, where c is a dissipation coefficient (having dimension of a force divided by a velocity to the power of α) that measures the magnitude, in terms of reactive force, of the device, and α is a dimensionless parameter much less than 1, equal to 0.15 in the most cases. This type of device has the advantage of being able to damp vibrations (reacting with a force whose maximum value is less dependent from the actual exciting velocity) and to exhibit large dissipative capabilities even in the field of small velocities (e.g., induced by earthquakes of lower intensity).

To determine the optimal value of the damping constant c, given α equal to 0.15, the following procedure was followed: (1) the force-displacement cycle relative to the above linear viscous device under an harmonic displacement law of amplitude of 111 mm and of period of 3.92 s was plotted; (2) the area inside this loop (dissipated energy) was evaluated (306 kJ); (3) the optimal value of the damping coefficient c was calculated as the one allowing a dissipating capability equal to that of the linear device (936 kN/(m/s)$^{0.15}$ for $\alpha = 0.15$); (4) the final value for c was selected among those corresponding to devices available on the Italian market as the one closest to the above desired value (1000 kN/(m/s)$^{0.15}$).

Therefore, the objectives of the retrofit design can be achieved through dissipating and re-centring devices the reacting force F of each is related to relative displacements x and relative velocity v by means of the following law:

$$F = k \cdot x + c \cdot v^{\alpha} + k_0 \tag{3}$$

where the values of the elastic stiffness k and of the damping constant c have been previously defined, and k_0 is the elastic preload, whose value depends on the executive technology that the particular production company adopts.

As mentioned above, the global kinematics of the bridge, even in the post-intervention configuration, requires that at each pier, a transverse displacement of the deck is allowed. This displacement must be elastically contrasted, and its magnitude can be estimated through the product between the longitudinal seismic displacement and the sine of the maximum angle, in the horizontal plane, between the tangent to the viaduct axis and the straight line orthogonal to the one passing through the pier axis and the centre of rotation (Fig. 2). A careful choice of the bearing devices was performed to meet both the requirements of allowing significant longitudinal displacements for sliding and not so small transversal displacements for deformations. Among the supports defined in the code EN 1337:1 [6], the type 1.5 was selected as the one able to provide the performance requested in the case under examination. Because this type of device is not currently produced by any of the largest Italian producers, it was necessary to proceed to the design of the 64 supports according to the cited code EN 1337:1. Fig. 5 shows the final result of this design from two different perspectives.

Conclusions

The vulnerability assessment and the seismic upgrading of an existing pre-stressed reinforced concrete viaduct was undertaken. The motivation of this study arose because a sort of instability was noted in the viaduct structural system A detailed analysis of the support devices and viscous dampers placed between the deck and the abutments allowed for the individuation of the most likely causes of the observed damage and the design of a suitable retrofitting intervention. The field observation and numerical investigation led to the conclusion that the mechanism responsible of the unsafe structural conditions is certainly due to a pathological phenomenon that began most likely at the earliest stages of the work, and subsequently worsened over time. This phenomenon is essentially related to some special characteristic of the viaduct structure, such as non-horizontal inclined deck, which is connected to the piers by sliding supports and the abutments that are linked with the insufficient viscous dampers. The structural analysis of the viaduct according to the recent criteria of the seismic code indicated that the structure is not fully capable to resist the applied loads; as a result, upgrading the structural system is inevitable. The seismic upgrading was followed by certain approaches, such as the replacement of the old bearing devices by modern ones designed based on the current code requirements and the replacement for dampers at each end of the viaduct by new devices with a higher technological content, such as the re-centring capability required to eliminate the pseudo-lability of the deck.

For comparison, the authors are currently investigating how the final performance of the bridge would change if, as an alternative to the technology proposed in the original design, innovative retrofit techniques are adopted based on the use of magnetorheological dampers [7] for the semi-active or smart passive control [8,9] of the seismic induced vibrations.

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