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SEISMIC RISK ASSESSMENT AND RETROFIT DESIGN OF TWO EXISTING REINFORCED CONCRETE VIADUCTS

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KEYWORDS: Existing bridge assessment, HDRB isolation device, seismic vulnerability, seismic isolation, response spectrum, retrofit.

ABSTRACT

The paper is mainly focusing on the retrofit interventions designed to bring two reinforced concrete viaducts to acceptable levels of seismic safety. The original drawings used at the time of construction and the results of recent visual inspections and complete characterization of the materials were available for both bridges. The strengthening design was preceded by structural modelling and analysis of the viaducts and seismic risk assessment analysis (carried out by using linear response spectrum analysis). The retrofit is based on the restoration of local structural damages and the seismic isolation of all spans by *HDRB* isolation devices.

INTRODUCTION

The seismic behaviour of existing bridges can be assessed with different tools, ranging from simplified linear elastic calculations to more refined 3D linear or non-linear finite element analyses (see e.g. Priestley *et al.* 1996, FEMA 356 2000). This paper first applies linear dynamic analyses to two existing reinforced concrete (R.C.) viaducts, using EuroCode 8 (2005) and the Italian seismic design guidelines (O.P.C.M. 3274, 2003 and O.P.C.M. 3431, 2005).

The investigated bridges are: (a) a two-lane plan-irregular viaduct, with 6 simply supported spans, located in the neighbourhood of Levanto, in the Liguria region, Italy; (b) a two-lane, regular and very long viaduct (19 spans of 32.0 m each) with short piers, placed in the neighbourhood of Viareggio (Tuscany, Italy). For both structures, accurate visual inspections and characterization of the materials were available.

The classic Response Spectrum Analysis was used to determine the seismic demand computed as maximum response quantities from the inelastic design spectrum provided by the code. Once the seismic vulnerability of the viaducts has been assessed, retrofit interventions have been proposed and designed. Full details of the retrofit, basically consisting in the restoration of local structural damages and the seismic isolation of all spans (by using High Damping Rubber Bearings, *HDRB*), are given in the paper. The seismic isolation, together with the repair of local damages, is generally apt to bring existing bridges – although not designed following capacity design principles – to acceptable levels of seismic safety; some further provisions (with the placement of additional tie-rods) were only required for the abutments of one viaduct, due to the defective arrangement of the original foundation system.

DESCRIPTION OF THE BRIDGES

(1) The "Carrodano" viaduct

The first viaduct (Fig. 1), generally known as "Carrodano" viaduct, is a reinforced concrete bridge over-passing the national highway A16 Genova-Livorno near the town of Levanto (a well-known seaside resort, west of La Spezia in the Liguria region, Italy).

Fig. 1. View of the "Carrodano" viaduct (neighbouring the Levanto town, Liguria, Italy)

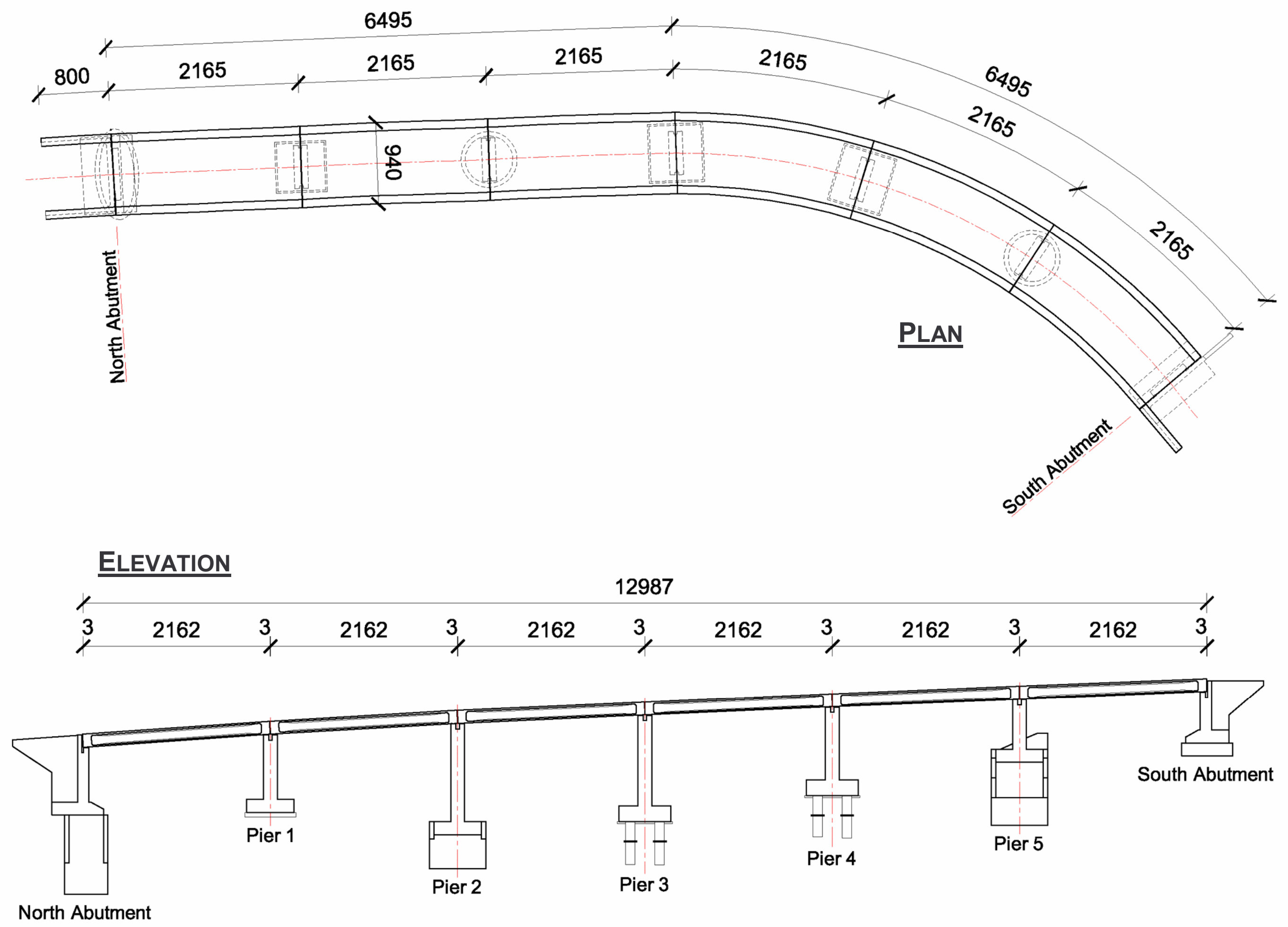

Fig. 2. "Carrodano" viaduct: plan and elevation (dimensions in cm)

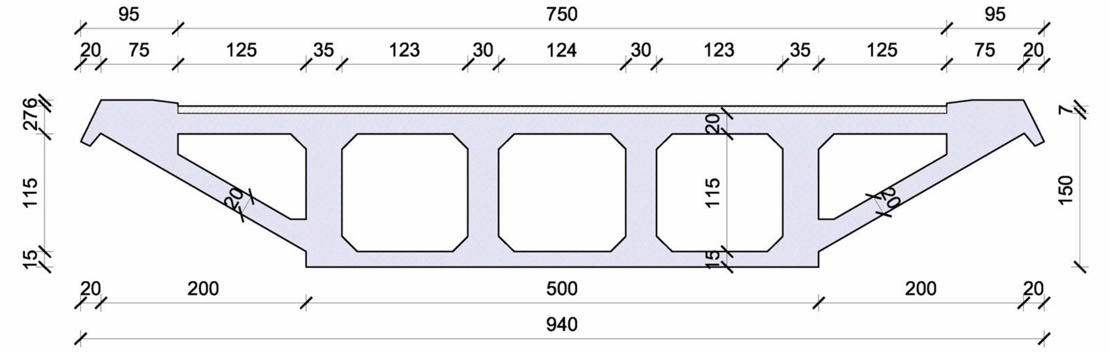

Fig. 3. "Carrodano" viaduct: typical cross-section (dimensions in cm)

Fig. 4. "Carrodano" viaduct: structural details and examples of local damage

Fig. 2 shows plan and elevation views of the 129.90 m long bridge with the basic structural dimensions. The viaduct (Figure 2) dates back to 1976 and consists of 6 simply supported spans, each of 21.62 m length; the first 3 spans (from North abutment) are straight while the subsequent ones curve with a radius of 72.72 m; all spans exhibit a transverse slope of about 4.40%.

Each R.C. deck superstructure is a five-cell box girder, 1.50 m high and 21.62 m long; the total width is 9.40 m, including the traffic lanes (7.50 m) and two pedestrian walkways (Fig. 3). The expansion joints between the different spans are only 3 cm thick.

The H-shaped piers consist of two rectangular elements (5.00×0,60 m^2) connected by a very large web and have different type of foundations (Fig. 2), due to the variability of the ground conditions. The pier longitudinal steel reinforcement consists of 72 18 mm diameter bars. The stirrups have an 8 mm diameter and are spaced at about 15 cm.

The evaluation of the general state of preservation of the structure was carried out by traditional on-site inspection procedures. Diffuse detachment of the coverage, reinforcement corrosion and material washing are the main damage factors affecting the structure (Fig. 4). The decay is especially localised in the following structural elements: (a) the external lateral sides of the piers and of the pulvins, where the coverage is widely detached due to the water washing; (b) the parts of the lateral tie beams close to the drain-off bores and to the expansion joints; (c) the deck areas close to the expansion joints.

The decay seems mainly cause by the concrete carbonation and by the water washing dripping from the deck, due to the ineffective drain-off system and to the pavement joints.

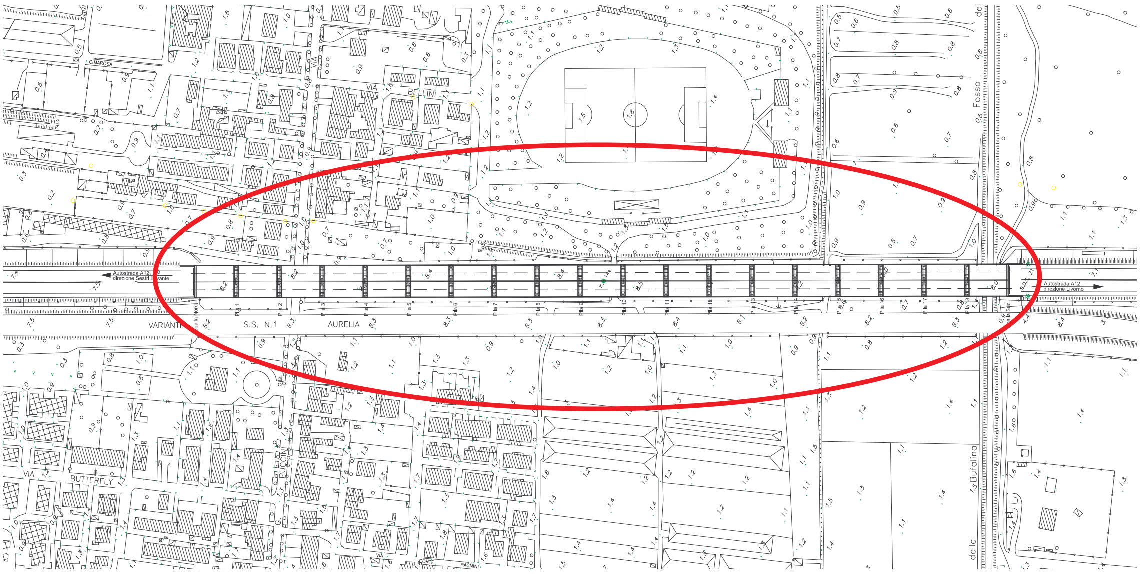

Fig. 5. "Torre del Lago" viaduct: general arrangement

Fig. 6 Partial view of "Torre del Lago" viaduct (neighbouring the Viareggio town, Tuscany, Italy)

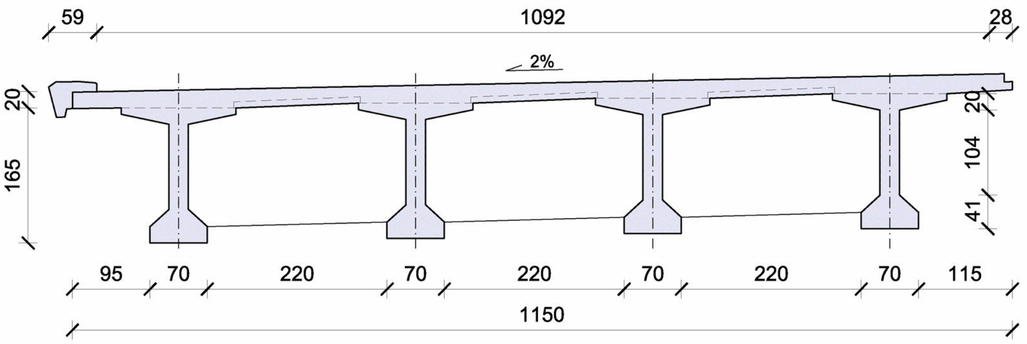

Fig. 7. "Torre del Lago" viaduct: typical cross-section (dimensions in cm)

(2) <u>The "Torre del Lago" viaduct</u>

The second infrastructure, named "Torre del Lago" viaduct, was erected at the end of 60's and belongs to the national highway A12 (Sestri Levante-Livorno); the viaduct includes two ways of 19 simply supported spans, for a total length of 604.00 m (Figs. 5-6).

The deck of each span (Fig. 7) is 11.50 m wide and consists of a cast in place R.C. slab (20 cm high) supported by 4 longitudinal girders and 4 transverse floor beams (Fig. 8). The distance between the axis of bearings (clear span) is 29.00 while the overall span is 32.00 m.

The bearing (fixed and sliding) consists of steel elements characterized by diffuse corrosion. The piers are composed by two short rectangular elements (with dimension of 2.40 × 0.90 m^2, Fig.8) supporting a massive pulvin beam (Fig. 8), about 12.00 m long.

Also in this case, accurate visual inspections leaded to the evaluation of the general state of preservation. As it has to be expected, the main damage factors affecting the structural elements are diffuse detachment of the coverage (especially in pulvin beams, Fig. 8), carbonation of concrete, corrosion of reinforcement and bearings and material washing.

Fig. 8. "Torre del Lago" viaduct: structural details and examples of local damage

For both bridges, it was possible to retrieve the complete set of original drawings used at the time of construction; however it was necessary to verify this information with an accurate geometric survey. A limited number of onsite non destructive tests were carried out to evaluate the concrete compressive strength of the piers. The design concrete cylinder strength, f_{cd} is assumed to be: (a) 10.50 N/mm^2 for the piers of Carrodano viaduct; (b) 15.80 N/mm^2 for the piers of Torre del Lago viaduct. In both cases, the steel was assumed to have average yield strength of 270 MPa (f_{yd}=235 N/mm^2). As indicated by (O.P.C.M. 3431, 2005), the average material strengths are used for the analyses.

SEISMIC RISK ASSESSMENT AND RESULTS

(1) Seismic action

Both structures sit on type C subsoil: deep deposits of dense or medium dense sand, gravel or stiff clay (O.P.C.M. 3431, 2005). Four hazard levels are possible according to the Italian seismic codes (O.P.C.M.

3431, 2005); the reference peak ground acceleration a_g is selected with reference to the return period of the seismic action for the no-collapse (severe damage limit state) requirement (or equivalently the reference probability of exceedance in 50 years). For the sites at study, which are zone 3, the a_g value is 0.15 g.

As suggested by EuroCode 8 (2005) and Italian seismic code (O.P.C.M. 3431, 2005), the earthquake motion is represented by an elastic ground acceleration response spectrum; furthermore, as it is usual, the horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum while the vertical component of seismic action is neglected for seismic zone 3 (O.P.C.M. 3431, 2005).

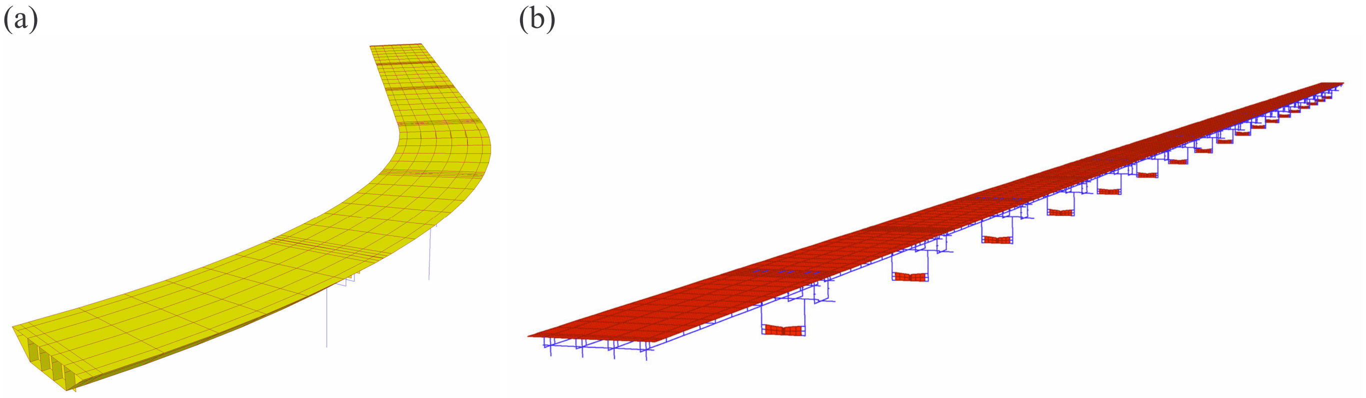

Fig. 9. F.E. models developed for the analysis of: (a) "Carrodano" viaduct; (b) "Torre del Lago" viaduct

(2) Numerical models of the bridges and force reduction factor *q*

The dynamic behaviour of simple bridges can be usually described by a limited number of vibration modes, with the fundamental period and mode shape providing a good indication of the bridge dynamic response. Also, many Italian long bridges, erected as simply supported multiple support girders, can be studied as separate structural subsystems, depending on the support details.

Bridge systems with irregular geometry, such as the "Carrodano" viaduct, exhibit a complex dynamic response which cannot be captured in a separate subsystem analysis. Hence, a global model of the entire bridge was developed, as shown in Fig. 9(a), with the well-known commercial program SAP2000. In the model, the piers and the top deck are idealized with linear beam and shell elements, respectively. The model results in a total of 1420 nodes, 212 beam elements and 1500 shell elements with 8461 active degrees of freedom.

Although the other viaduct at study is indeed more regular, a global model was developed as well. The linear elastic model (Fig. 9(b)), based on the design drawings, was formulated using the following assumptions: (a) four-node shell elements were used to represent the deck slab and the stiffening diaphragms connecting the two bents of piers; (b) girders, floor beams, columns and pulvins were modelled as tapered beam elements; (c) the pier footings were considered as fixed.

The design spectrum is obtained dividing the elastic spectrum by the force reduction factor *q*. For existing bridges, which were not designed following capacity design principles and that do not present appropriate seismic details, the values to adopt for the *q* factor are often uncertain. A possible procedure to estimate *q* can be based on the evaluation of the structural ductility capacity (displacement ductility capacity of each pier). For example, according to (Berrill *et al.* 1981) the force reduction factor can conservatively be related to the structural displacement ductility factor μ_Δ by the following:

$$q = 1 + 0.67\ (\mu_\Delta - 1)\ \frac{T}{T_o} \le \mu_\Delta \tag{1}$$

where T_o is the period at peak elastic spectral response and T is the period of the structure's fundamental mode. Eq. (1) provides a gradual variation from $q=1$ at T=0, which is theoretically correct, through the equal energy equation ($q=(2\mu_\Delta-1)^{0.5}$) at about T=0.8 T_o, to the equal displacement equation ($q=\mu_\Delta$) at

T=1.5 T_o.

Of course (see e.g. Priestley *et al.* 1996, EuroCode 8.2 2005), the evaluation of the displacement ductility capacity of the piers involves the moment-curvature analysis of the base section and the evaluation of equivalent yield curvature ϕ_y, ultimate curvature ϕ_u and a reasonable estimate for the plastic hinge length.

(3) Linear analysis and results

As previously stated, the linear method of analysis used is the classical response spectrum analysis, according to (O.P.C.M. 3431, 2005). Both the investigated viaducts can be classified as class II bridges so that the importance factor value is equal to 1.0. Since the structures have limited ductility and the pier bases are quite squat (see Figs. 1 and 8) and prone to shear failure, the behaviour factor q is set equal to 1.0 in both longitudinal and transverse directions. The vibration modes are combined using the CQC method while the multidirectional combination used is the SRSS rule.

For the design spectrum used, the capacity of both bridges is well below the demand. Specifically, the seismic capacity of both existing viaducts is largely exceeded in:

(a) the bearing systems and expansion joints;
(b) the piers, where the flexural and shear demands are higher than the section capacities;
(c) the pier foundation system, for which elastic behaviour can be no longer guaranteed.

In addition, the abutments of "Torre del Lago" exhibited important global and local problems due to the defective arrangement of the original foundation system (Fig. 12).

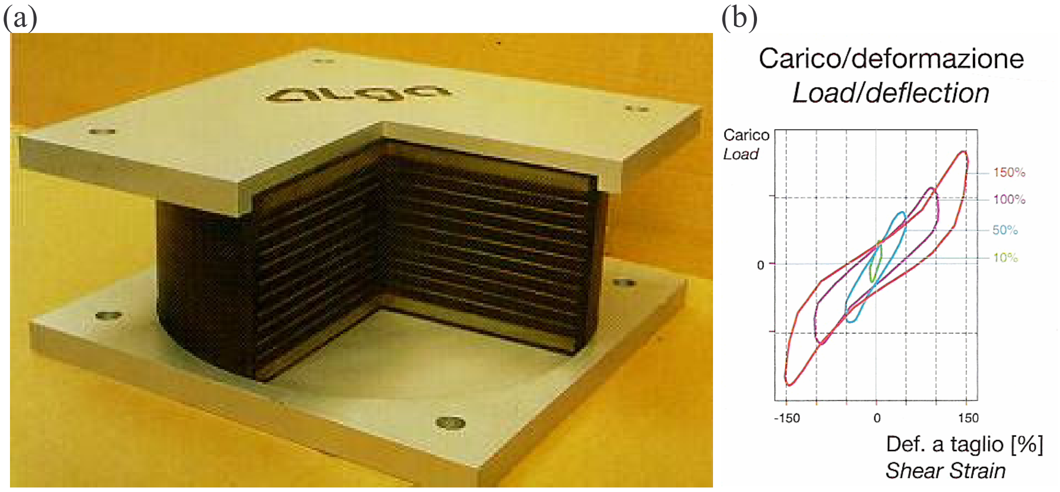

Fig. 10. (a) Typical *HDRB* (High Damping Rubber Bearing) isolation device; (b) Typical load-deflection curves for *HDRB* isolation device

DESCRIPTION OF THE PROPOSED INTERVENTIONS

As previously pointed out, the assessment of the general preservation state of "Carrodano" and "Torre del Lago" viaducts revealed diffuse local damages to the material of both bridges; furthermore, the seismic risk assessment highlighted that the capacity is well below the demand. In particular, since both visual inspection and seismic analysis suggested the need of substituting the bearing devices, it was decided to drastically reduced the seismic vulnerability of the bridges by using an isolation system, with High Damping Rubber Bearings (*HDRB*). These devices (Fig. 10) are capable of significantly improving the seismic resistance through the artificial increase of both the fundamental period and the energy dissipation capacity.

Hence, the proposed strengthening interventions can be summarized in three main steps:

(1) Repair interventions on the materials. The objective is to obtain the maximum durability of the structure by economically sustainable interventions. The main steps of the concrete repair in the damaged area of superstructures, piers and abutment are:
    - removal of damaged concrete and cleaning of corroded reinforcements;
    - re-profiling of the concrete with cement-based tixotropic mortar spraying and manually finished;

- coating of re-profiled surfaces by weather-resistant and carbonation-protective meta-acrylic or epoxy products.

(2) Adequacy of seismic resistance by *HDRB* seismic isolation and proper local interventions. In order to increase the period of vibration of the structures with a high energetic dissipation, after the deck lift by hydraulic jacks, seismic isolators will be positioned above the piers and the abutments.
The characteristics of *HDRB* devices of "Carrodano" viaduct are the following:

Maximum vertical load:	2900 kN
Maximum horizontal load:	385 kN.
Seismic displacement:	±179 mm
Horizontal stiffness:	2.15 kN/mm
Damping:	16%

In such instances, the dynamic behaviour of "Carrodano" bridge s is described only by four vibration modes (with periods ranging from 1.48 s and 1.24 s) and the decrease in seismic demand brings back the section capacities to acceptable safety levels. For example, typical interaction diagrams (N-M_x-M_y and N-M_x) of pier 2 (see Fig. 2) and seismic demand for a_g= 0.15 g are shown in Fig. 11.

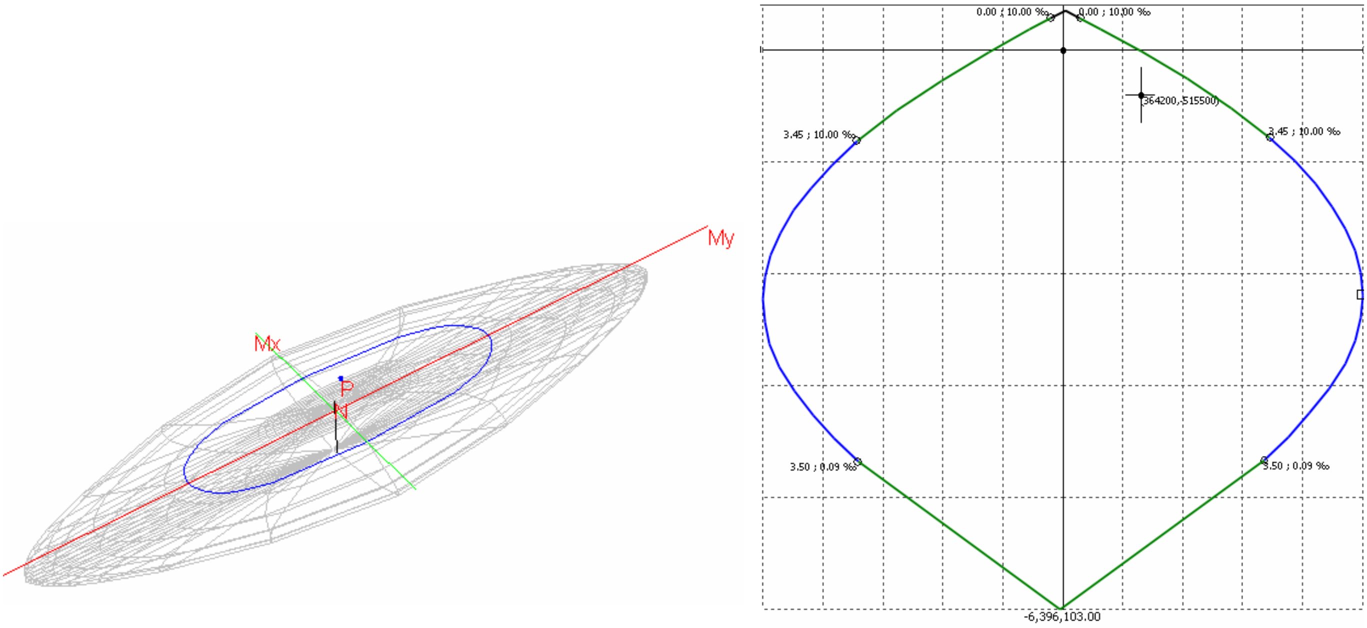

Fig. 11. "Carrodano" viaduct: Typical interaction diagrams of pier 2 and demand for a_g=0.15 g

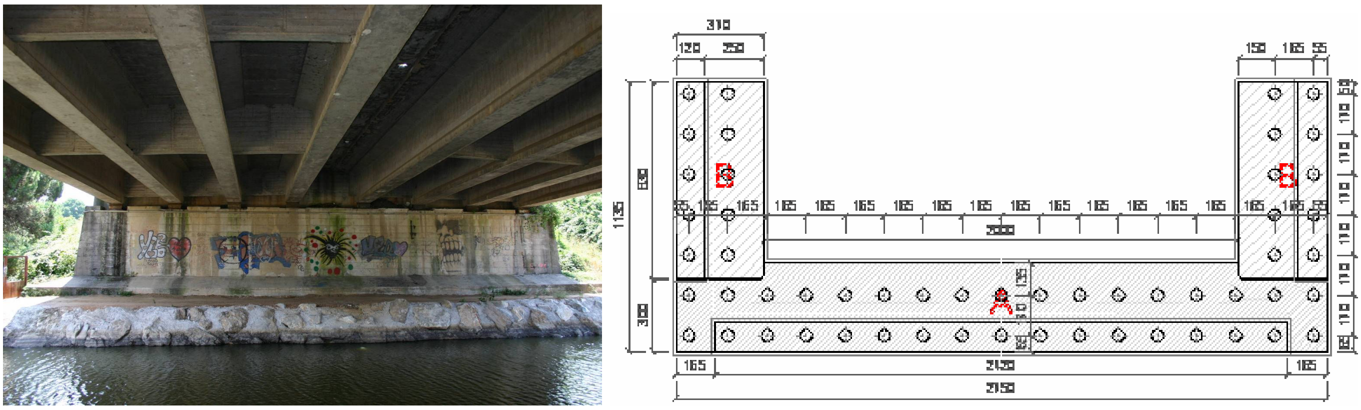

Fig. 12. "Torre del Lago" viaduct: view of the north abutment and foundation system

It should be noticed that due high corrosion generally observed in the piers and abutments, beside the complete removal of the damaged concrete by high-pressure water sandblasting, the seismic retrofit design requires a further reinforcement by steel bars Fe B 44k; the bars are connected to the foundation by chemical anchorages. Furthermore, the intervention on this viaduct is completed by: (a) micro-piles to strengthen the foundations of the piers 1, 2 and 5; (b) the longitudinal connection between the spans (in order to avoid pounding or excessive longitudinal displacements) through a pre-stressed tie rod.

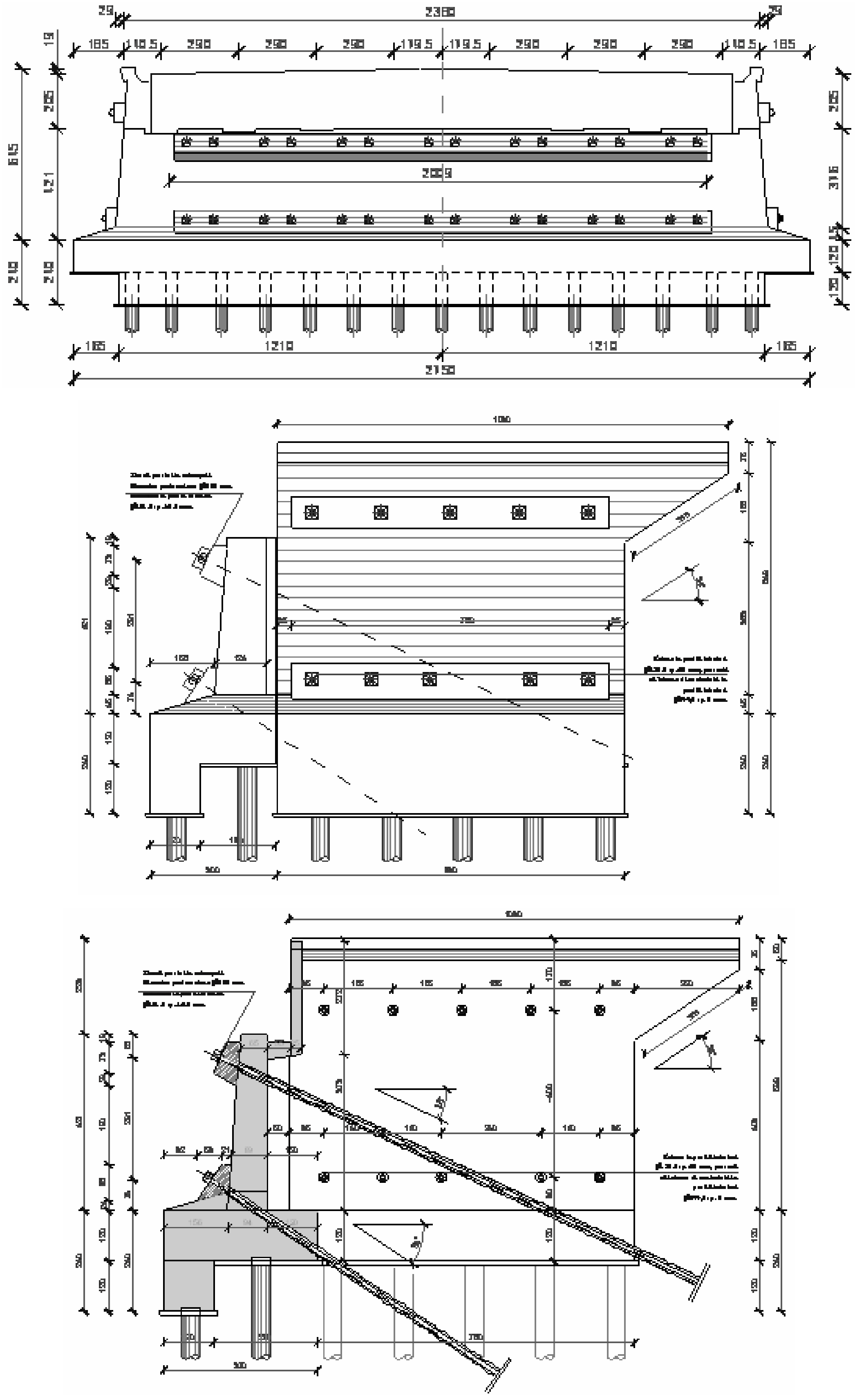

Fig. 13. "Torre del Lago" viaduct: schematic of abutment strengthening

For "Torre del Lago" viaduct, the characteristics of the proposed isolation devices are:

Maximum vertical load: 1850 kN
Maximum horizontal load: 246 kN.
Seismic displacement: ±146 mm
Horizontal stiffness: 1.69 kN/mm
Damping: 16%

Again, the dynamic behaviour of the isolated bridge is described by few vibration modes and the decrease in seismic demand brings back the section capacities of piers to acceptable safety levels. In addition, due to a better state of preservation of the piers, no further steel bars are needed.

On the other hand, the seismic isolation is not sufficient to protect the abutments and the piles of foundation system from excessive flexural demand. As clearly shown in Fig. 12, this behaviour is basically to due to the poor and defective arrangement of the original foundation system. Hence, the intervention has to be completed by the placement of tie-rods in order to protect both the abutment walls and the piles from excessive flexural demand (Fig. 13).

CONCLUSIONS

The paper summarizes the results of on-site investigation of the state of preservation and theoretical assessment of the seismic risk of two R.C. existing viaducts. Since visual inspections revealed diffuse local damages of the material and the seismic analysis clearly indicated that the capacity is well below the demand, a repair and seismic strengthening intervention were designed.

The seismic strengthening is based on isolation concepts, which demonstrated to be especially suitable when dealing with existing structures, which were not designed following capacity design principles and that present inappropriate or poor seismic details.

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