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Structural assessment of a stock of motorway viaducts located in north Italy: results, statistics, and intervention.

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Nicola Longarini[a], Angelo Ciciriello[b], Pietro Crespi[a], Marco Zucca[c], Giuseppe Pasqualato[d]

[a] Politecnico di Milano-Dipartimento di Architettura, Ingegneria delle Costruzioni e Ambiente Costruito, via G.Ponzio 31, Milano, Italy
[b]E-Copla engineering, via Piranesi 44, Milano, Italy
[c]Università degli Studi di Cagliari, Department of Civil and Envronmental Engineering, via Marengo 2, Cagliari, Italia
[d]SINA SpA, viale Isonzo 14/1, Milano, Italy

Abstract

This paper provides indications on the maintenance status of 53 viaducts located in the main motorway sections of northern Italy. The viaducts are characterized by reinforced concrete piers and prestressed reinforced concrete or steel decks. For each viaduct, structural analyses are performed to assess the safety according to the rules defined in the Italian Guideline about the risk management, safety evaluation and existing bridges monitoring. The results of the analyses are expressed in terms of appropriate status (operability, practicability) of the viaducts, following the Italian National Code requirements. The results are statistically analyzed considering the viaducts' age, the geometrical and material characteristics of the piers, and the features of the decks. Recurring structural problems linked to the maintenance are also identified and considered in terms of viaducts' age and periodic tests performed for the status identification. The statistical analyses here performed also provide the intervention's time to upgrade the viaducts in order to fit the appropriate structural status prescribed by the Code. Moreover, for the studied viaducts, the analyses give traffic and loads limitations to be respected during the time window in which the bridges are interested by the retrofitting works.

Keywords: Existing bridges; Reinforced Concrete viaducts;traffic loads; structural retrofitting

* Corresponding author Nicola Longarini Tel.: +393939511216
E-mail address: nicola.longarini@polimi.it

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1. Introduction

In the last decades, the strategic role of existing viaducts in the European economic and engineering fields is increased a lot. The appropriate structural conservation represents an important issue to guarantee the static and seismic performances, and unidentified structural deficiencies could cause tragic collapses under traffic or seismic actions, Peng et al. (2020), Fan (2015), Sun et al. (2020). In Italy, many of the motorway bridges were designed and built between 1960's and 1970's by considering different seismic and traffic loads with respect to the current design criteria. Therefore, strength and ductility verifications performed according to current national codes often are not positively satisfied due to the increased traffic loads', the original construction details, and the mechanical properties of the original materials, Bossio et al. (2019).Moreover, inadequate maintenance and severe environmental condition sometimes has caused the degradation of the materials (especially concrete and steel rebars) with a corresponding reduction of load-bearing capacity. About concrete, different mix designs were recently studied to improve its durability, Tang et al. (2012). Differently, corrosion of the steel reinforcement bars is very common, Zhou et al. (2014), Almusallan et al. (2001) and it represents an important reason of safety level reduction of motorway viaducts, Bossio et al. (2019), Zhou et al. (2020).Thus, existing bridges cannot satisfy the current Italian codes' requirements very often even if they were designed and built according to the standards of the design time. A large-scale management policy seems required in order to identify the possible structural lacks of the existing viaducts and to evaluate the priority of the implementation of the retrofitting interventions, the required structural reinforcements, and the temporary traffic variations. In this paper, 53 viaducts have been analysed in terms of structural capacity under traffic loads, considering or not the degradation status detected in the preliminary stage having the aim to lead to a suitable knowledge of the construction. For each viaduct, material tests identifying the mechanical properties of the concrete and steel bars (for both piers and decks) have been performed. The structural analyses of the viaducts have been carried out by adopting specific finite element models (FEMs) where the geometry is represented according to the construction drawings (blueprints) and surveys, while dead loads and conventional live loads (including traffic loads) are introduced following the Italian Building Code, NTC (2018). For each viaduct, the structural status is defined according to the New Guidelines for the classification and management of the risk, safety, and monitoring of the existing bridges - Cultrone et al. (2023) - in terms of the following conditions: Adequacy, Operability, Practicability 1 (with the restriction of the bridge's use) and Practicability 2 (with the limitation of the allowed loads). This evaluation has been performed in presence and absence of the degradation, and the viaduct members causing the worst identified status are reported as well. For 31 out of 53 viaducts (overall stock of viaducts investigated under traffic loads) the effect of the seismic action without traffic is considered too. For this subset of manufacts, an important comparison in terms of risk indexes (namely IR for the traffic loads and IS under the seismic load) is presented to highlight the worst safety conditions for these existing viaducts.

2. Preliminary activities for the Safety Evaluation

For each viaduct, the safety evaluation is generally performed by adopting three different finite element models, with different levels of details, specifically developed for the verifications of piers and decks against traffic and seismic loads. For the safety verification of the deck under traffic loads, single or multiple span models FEMs of the deck only are developed. In this case, beam elements are used to represent longitudinal and transversal beams while the deck concrete slab is included in the cross section of the longitudinal beams. At the ends of the deck's beams, simple supports are generally applied to reproduce the translational constraints of the pier caps, whereas the connection between the deck longitudinal beams and the pier caps is implemented by means of a system of rigid links, to guarantee a simply supported static scheme and to account for the vertical eccentricity between the longitudinal axis of the beam and the bearing device. For the safety verification of the piers under traffic loads, a complete 3D finite element model of the bridge is adopted (Fig. 1). In this case, the piers are represented through beam elements with a fully restrained node at their intersection with the foundation plinth. In this finite element model, also the deck is modelled with Timoshenko's beam elements while the bearings are implemented through elastic links having their translational and rotational stiffness calculated according to the formulas reported in EN 1337-3 (2005). The connections between the elastic links representing the elastomeric bearings and the beam elements of the deck, on top, and the pier caps, on bottom, are modelled by means of rigid links. The abutments are considered in the FEM as perfect restraints applied at the base nodes of the bearings located in correspondence to the deck-abutment interface. This kind of mode is also used for the seismic analyses but nonlinear static analyses are executed considering nonlinear constitutive laws for both steel and concrete (Kent et al. (1969), Park (1975). Moreover, under the seismic action, two failure mechanisms are considered for the piers: ductile collapse mechanism and brittle collapse one as explained in Crespi et al. (2020), Zucca et al. (2023), FEMA 440. Also, the reduction of the piers bending stiffness is considered too, EN1337-3 (2005), Eurocode 8 (2005).

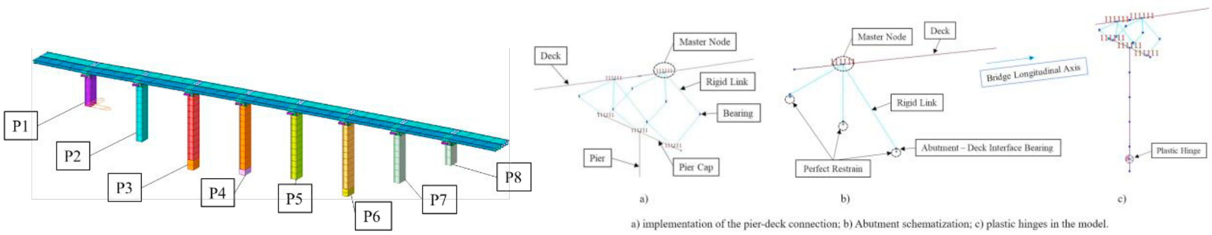


Fig. 1. FEM with details for the modelling of the viaduct for traffic and seismic analyses

The materials are considered as linear elastic and linear analyses are performed under traffic loads, wheares nonlinear properties for the materials are considered in the seismic analyses. For the safety verifications, material strengths can be drawn from the historical design documents or from specific in-situ tests, Circolare (2019), Linee Guida (2022), Imperatore et al. (2017). Once the finite element models are completed and the loads are applied, Piers, pier-caps, longitudinal and cross beams of the decks, slabs, and Gerber half-joints are verified in terms of shear, bending moment, compression, and bending-compression according to NTC (2018), Circolare (2019), and Linee Guida (2022). In the evaluation of the resistance of the different viaduct members, both undamaged and degraded configurations are considered, Berto et al. (2009), Zanini et al. (2013), Li et al. (2014), Zucca et al. (2023). For the beams of the deck, the safety verifications should be performed at the support and the midspan sections, but also in other positions along the longitudinal axis where a particular degradation status has been found and reported during the inspections. Moreover, the safety verifications should be carried out in those sections of the beams characterized by a strong variation of the number of steel reinforcement bars.

3. Safety Evaluation

In the present work, NTC (2018) rules are adopted for the safety verification of each one of the critical sections of the bridge selected according to the criteria defined in Section 2. Sections are checked in both undamaged or damaged (e.g. spalling, corrosion, etc.) configurations in view of a possible evaluation of retrofitting interventions. For the ultimate limit state combinations, the partial factor for the gravity (dead) loads γ_G can be assumed equal to 1.25 when an accurate statistical control of the construction's geometry has been performed and the tests on the materials have given a small deviation. The partial factor for the traffic load is set equal to $\gamma_Q = 1.35$, as it is prescribed in chapter 5 of NTC (2018), while the partial factor for wind action is considered as $\gamma_{Qw} = 1.50$. Therefore, for each control section and for each damage configuration (with or without degradation), the safety factor FS defined as the ratio between the member resistance R_d and the effect of the applied loads E_d can be evaluated (generally evaluated both in terms of bending moment and shear action). Then, only for the control sections where the traffic loads are significantly higher than the others loads, verification coefficient $\xi_{V,i}$ should be evaluated considering both the shear and the bending moment acting on the section. This coefficient is defined as the ratio between the maximum effect of the vertical variable loads withstood by the section and the corresponding effect caused by the variable loads prescribed by the code for the design of new bridges. Formulas for the evaluation of $\xi_{V,i}$ are listed in the following Equations (1) where: M_{Rd} and V_{Rd} represent the resistant bending moment and shear, respectively, M_{perm} and V_{perm} are the bending moment and shear caused by the dead loads, respectively, M_{acc} and V_{acc} are the bending moment and shear due to the live loads, and M_{wind} and V_{wind} are the bending moment and shear caused by wind.

$$\xi_{V,i} = \frac{M_{Rd} - \gamma_G \cdot M_{perm} - \gamma_{Qv} \cdot \psi_0 \cdot M_{wind}}{\gamma_Q \cdot M_{acc}} \; ; \; \xi_{V,i} = \frac{V_{Rd} - \gamma_G \cdot V_{perm} - \gamma_{Qv} \cdot \psi_0 \cdot V_{wind}}{\gamma_Q \cdot V_{acc}} \quad (1)$$

If one of the two $\xi_{V,i}$ coefficients or one of the two safety factors FS is lower than one, it is mandatory to perform Operability or Practicability verifications (if both are greater than one, Adequacy condition is satisfied). The Operability verifications can be performed by reducing the values of the loads' partial factors: $\gamma_G = 1.16$, $\gamma_Q = 1.20$, and $\gamma_{Qw} = 1.50$. Furthermore, the wind action should be referred to return period equal to 30 years. Also the partial safety factors of the materials should be lowered, assuming $\gamma_C = 1.20$ for the concrete, $\gamma_S = 1.10$ for the steel rebars, and $\gamma_{Sc} = 1.05$ for the steel carpentry. If the Operability verification is positively satisfied , restrictive measures of vehicular traffic are not required and it is possible to classify the viaduct as in Operability condition.Otherwise, Practicability verifications should be carried out considering two different approaches. The first one is here defined Practicability 1 and includes: (i) traffic limitation with the closure of the emergency lane (it implies a reduction of the number of conventional lanes as in NTC 2018; if there are three or more lanes for direction, an extensive closure of the only emergency lane one can be considered, allowing at least the transit of two lanes in each direction); (ii) the weight of the safety barriers evaluated in 0.4 kN/m must be introduced in the structural model; (iii) wind' coefficient decreases to $\gamma_{QW} = 1.26$, with the same $\gamma_Q = 1.20$ and $\gamma_G =1.16$; the partial safety factors for the materials still be the same of the Operability condition. If the emergency lane closure is not enough, in order to achieve a positive safety verification, it is possible to apply a further limitation of the considered loads moving to the so called Practicability 2 scenario. In this case, a distributed load of 9 kN/m^2 is considered for the lane interested by heavy vehicles traffic, together with a set of concentrated loads simulating the presence of a 440 kN and 11 m length located as it is defined in Chapter 6 of Linee Guida (2022). The distributed load and the concentrated ones are not overlapped themselves. Moreover, another limitation regards the fast lane where overtaking is forbidden for vehicles with more than 35 kN weight, and a distributed load of 2.5 kN/m^2 without further concentrated loads is assumed for the verifications. Since the width of the lanes is set to 3.00 m, the remaining part of the carriageway (total width of the bridge minus the width of the considered lanes) should be interested by a 2.5 kN/m^2 distributed load as well. Sometimes, given the width of the carriageway, it is not possible to close the emergency lane, this one is loaded by the 440 kN heavy vehicle together with a 75 kN vehicle (having 6 m total length, two axes of 50 kN and 35 kN respectively, spaced by 4 m); these two vehicles are considered at rest so the dynamic coefficient is set to one (for the local verification the dynamic coefficient must be considered as described in the following). For Practicability 2 safety verifications, the breaking and centrifugal loads are applied considering the actual traffic condition. The partial factors are the same of Practicability 1 scenario, except for the traffic coefficient (here named γ_{cds}) that can be considered 1.6, 1.35 or 1.1. by the load's control condition as defined in Linee Guida (2022). If the safety verifications are not satisfied with two traffic lanes, firstly overtaking's ban for 35 kN vehicles must be considered. In the limit case where the emergency lane is closed and the overtaking's ban is applied and the verification still be not satisfied, the Practicability 2 verifications must be repeated by considering a single traffic lane with the heavy vehicles, eventually positioned in the middle of the carriageway. In addition, local safety verification must be carried out by a finite element model representing the deck by means of Timoshenko's beam elements (as illustrated in Section 2). For Adequacy, Operativity, and Practicability 1 conditions, the load values and their partial factors of the load combinations are the ones reported in this Section, whereas in Practicability 2 the following issues are considered. Generally, for deck (especially if characterized by a length less than 16m), on the roadway only one single vehicle can be considered by its two tandem axes amplificated by dynamic coefficient to represent a congestion traffic case. To calculate the dynamic coefficient Φ, can be used Equation (2) where L represents the calculation length, Linee Guida (2022) Moreover, for the dynamic coefficient there is the following limitation: $\Phi = 1,4$ if $L \leq 10$ m, $\Phi = 1$ if $L \geq 70$ m.

$$\Phi = 1.4 - ((L-10)/150) \tag{2}$$

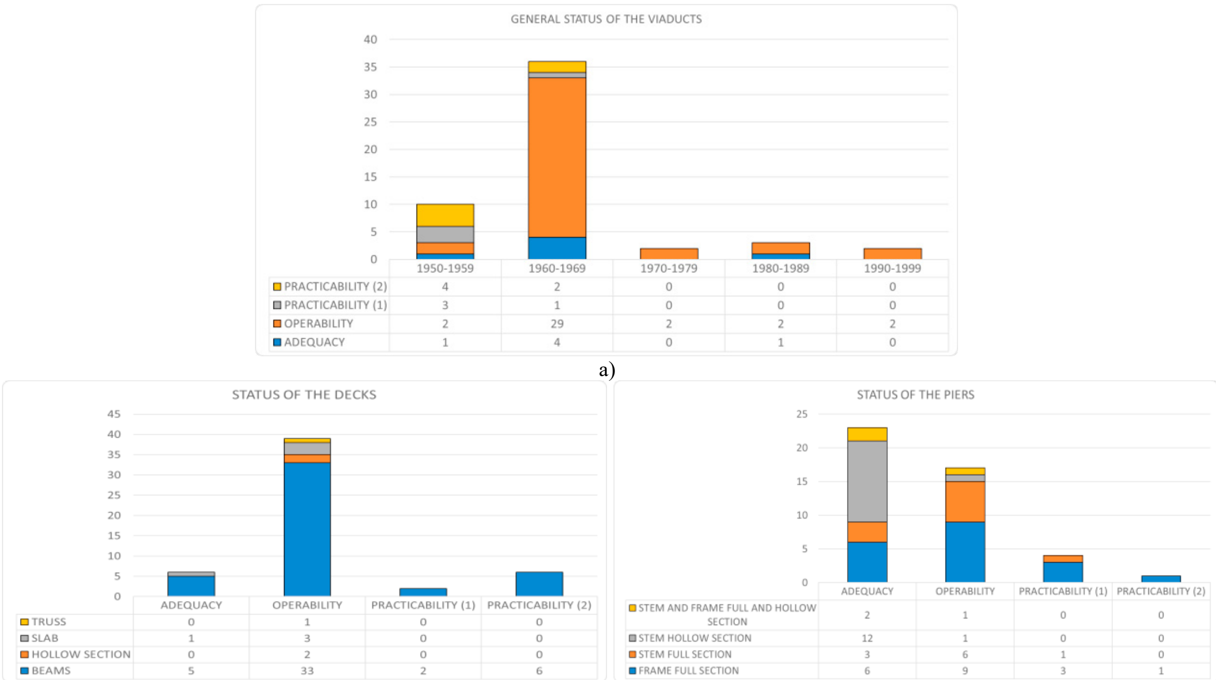
For the local verification of the deck's slab, only one 120 kN axis load is considered (two wheels with 60kN each); it is increased by dynamic coefficient Φ; the safety verifications are performed by adopting the partial coefficient γ_{cds} already discussed in the present Section 3.

4. Case Studies

The 53 viaducts here investigated are located in the northern Italian motorways named A6, A15 and A21. They were built in a wide time window; therefore, the present statistical analysis considers different material performances and degradation conditions. The 23 viaducts located in A6 were designed in different decades: 10 in 1950-1960, 8 in 1960-1970, 3 in 1980-1990, and 2 were recently built (1997). All the 24 viaducts located in A15 were designed between 1968-1970, whereas the 6 viaducts located in A21 were all designed in 1968.The design period is taken from the analyzed historical documents, while the year of the completed execution is assumed from the structural static test date. A total of 21 of these viaducts were interested by maintenance or, in limited cases, retrofitting interventions during their service life. These operations were performed 35 years after the date of the static tests, on average. Among the maintenance, the most frequent operations are the passivation of the exposed steel bars, the renovation of the concrete cover, the rainwater collection and deviation to avoid percolations on reinforced concrete surfaces. The parts of the viaduct mostly interested by the interventions usually are the decks, the piers jacketing - Zucca et al. (2023). The viaducts are different themselves for geometries, piers or deck features, and materials. They have different number of spans (ranging from 1 to 23) and they are characterized by the presence of concrete abutments (only 2 viaducts have gravity walls, while the others have a thin concrete wall typology), and by reinforced concrete piers having a cantilever or frame configuration. For the decks, different solutions are present: in 46 cases there are longitudinal prestressed beams with ordinary concrete transverse ones, 4 consists of a single plate, 2 have a box girder hollow section, and only one deck made by a steel truss. In 19 cases, when the piers are frames, full sections are always present. In 23 cases of stem piers: 13 has hollow sections and 10 has full sections. Only 3 viaducts have mixed piers characterized by stem and frame (with hallow and full sections). The remaining part of 8 viaducts has only one span. Generally (40 cases) they have RC piers, RC frames and RC decks with PRC longitudinal beams. Only 2 viaducts have deck with mixed structure: steel trusses and RC slab. The remaining viaducts have all the elements in RC. The material design values are determined on the basis of the compression and tension tests on samples taken from the structure during mandatory test campaign for construction monitoring. For 3 viaducts tests on the concrete of piers and decks have given higher values with respect to the original ones, thus, the safety verifications have been performed with the original design values. The degradation status is described in the technical documents about the actual condition of the viaducts. The degradation effects affect mostly the beams of the decks, especially the external ones. The percentage of exposed bars and corrosion present at the same time on the piers and beams of the decks is 15%, and the run-off is globally present for 16% (in the half of the cases it contributes to exposed bars and corrosion). Among the viaducts here analyzed, degradation is not present in the 17% of the cases (some of them have been interested by maintenance/modifications over time). The 19% of the viaducts is exposed bars and corrosion of the beams of the deck, whereas a minimum percentage (2-4%) of the viaducts is afflicted by exposed bars and corrosion on abutments, slabs, pier-caps and piers.

5. Statistical Analyses: Results and Discussion

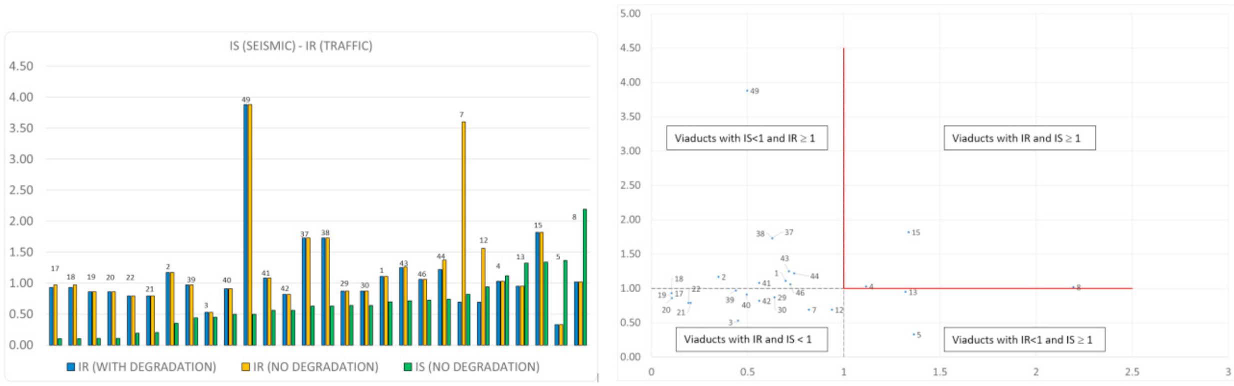
The safety analyses presented in Sections 3 are applied to the viaducts described in Section 4. The results are statistically analyzed and illustrated in the present Section 5. The first investigation concerns the general status of the viaducts. The status is grouped by decades considering the design years (Fig. 2a). The histogram clearly shows that most of the viaducts are in the Operability condition, regardless of the decades, starting from about 1960. On the contrary, viaducts designed (and realized) in the 1950-1960 decade show a higher percentage in Practicability (2) condition. This means that the degradation effects are most evident for this last group of viaducts. It could also be noticed that, for each decade, the Adequacy condition is the minimum part of the histogram. The conditions of each viaduct are expressed in terms of status of the decks in Fig.2b and status of the piers in Fig.2c, where the conditions are grouped together by different typologies of elements. As previously mentioned, most of the considered viaducts have decks composed by longitudinal and transversal beams. These elements are seriously interested by the degradation effects acting on the reinforced concrete, therefore, the beams are very often in Operability condition. About 17% of the decks with beams is interested by strong degradation, leading to Practicability conditions. In the bridge stock here analyzed, there are only four slab decks, but also this typology has been resulted in Operability condition. About the piers, Adequacy and Operability conditions are the most frequent situations, regardless of the piers' typologies. Stem piers (full and hollow sections) show better results in terms of condition with respect to frame ones. Frame piers are characterized by safety conditions with greater variation than other types. The safety analyses, considering the viaducts with degradation and Adequacy condition, have shown several failure mechanisms occurring in the decks and piers with negative safety verifications. About the decks, the longitudinal external beams are the elements mostly affected by degradation. They represent the elements of the deck where the adequacy condition is not satisfied for bending or shear mechanisms with approximately the same percentage (22% and 26%, respectively). Vice-versa, the inner beams can be considered not really exposed to the degradation effects because only 4% of the viaducts do not satisfy the Adequacy condition in these elements. Moreover, the degradation effect is strongly present in the Gerber half-joint (if present). In 9% of the viaducts here considered, shear in Gerber half-joint represents the mechanism leading to negative safety verifications. About the piers' mechanisms, the shear one is the worst one for different types of piers. The percentage of about 51% of the analyzed cases is not in Adequacy condition having verification index minor than one, with 18% for frames with full sections, 9% for stems with hollow sections, and 24% represented by the pier-caps. Therefore, the pier caps represent a part of the viaducts strongly affected by the degradation effects.



b) c)

Fig. 2. a) General status of the 53 viaducts under the traffic loads (including the degradation status) b) Condition due to the status of the decks c) due to the status of the piers

The analyses under the different traffic conditions, with the relevant partial factors in the load combinations, have been performed by considering or not the degradation effects. When degradation is included in the analyses, a reduction of rebars diameter or the elimination of a reinforcement level have been considered (as reported in Section 2). Moreover, for some viaducts of the considered stock (28 out of 53 viaducts), seismic analyses have been performed too, without considering any degradation. A comparison among the safety indexes for traffic loads (IR) and seismic loads (IS) is shown in Figure 3a (both in Fig. 3a and in the following Fig. 3b the labels with the identification number of the 53 viaducts here analyzed are shown). The comparison shows that the degradation effects are generally not significant on the safety index related to the traffic loads. In fact, most cases are characterized by the same value of the traffic safety indexes by considering (or not) the degradation. Only viaduct n.° 7 shows a very important difference between the traffic indexes (IR is five times higher if degradation is not considered). This is caused by the presence of exposed bars with corrosion on both piers and beams of the deck. Also, viaduct n.° 12 shows an important difference between IR with or without degradation. In this case, the reason is the run-off with exposed bars and corrosion on the beams of the deck. Moreover, by comparing IR due to the traffic without degradation (yellow columns in Fig.3a) to IS due to the seismic action without degradation (green columns in Fig.3a), only 14% of the viaducts shows IS higher than IR. These viaducts are all designed between 1957 and 1962, and they are located very close themselves along the same motorway, but they are characterized by different features in terms of pier configurations (nn. 4, 5, 8 and 13 have piers respectively with frame with full section, frame with full section, stem with hollow section, and frame-stem with full and hollow section). Also, the material characteristics are in line with the other viaducts, therefore, local seismicity could be a possible cause of their vulnerability. Generally, a limited number of viaducts positively satisfies the safety verifications due to both earthquakes and traffic conditions simultaneously. In Figure 3b, this situation is represented in the top-right quadrant. The lower-right quadrant shows the viaducts that are safe only against seismic loads. Similarly, in the upper-left quadrant there are the viaducts satisfying only the traffic conditions. Finally, the lower-left quadrant represents the most dangerous situation containing the viaducts not verified for both seismic and traffic loads.



a) b)

Fig. 3. a)Verification indexes for the traffic loads (IR, with and without degradation) in comparison to the Verification index under seismic action (without degradation) b) Representation of the status of the viaducts in terms of IR and IS

6. Conclusions

This paper shows a statistical analysis on a stock of Italian existing viaducts in terms of safety verifications under traffic and seismic loads. The degradation effects on the structural behavior is included in the safety verifications under traffic loads. The viaducts here considered are very different themselves for geometries, configurations of the piers (stem, frame, stem and frame), geometrical characteristics of the piers (full or hollow sections), and also for the deck typologies (with beams, slab or truss). The condition of each viaduct (namely: Adequacy, Operability, Practicability) is identified in relation to the typologies of the piers and the decks. The procedure followed in the analyses for the evaluation of the verification indexes under traffic loads or seismic action is described in terms of numerical approaches (for the piers and the local verifications on the decks), identification of the materials' resistance from the on-site tests, safety coefficients considered in each of the verification scenario. The results have shown: (i) most of the viaducts designed before 1960 are in Practicability (2) condition, especially if they have not been yet interested by reinforced concrete retrofitting; (ii) starting from 1960, most of the viaducts are in Operability condition; (iii) about the decks of the viaducts, the degradation (if present) afflicts especially the longitudinal beams; the status of these elements often leads to Practicability conditions; when the deck is characterized by a slab, Operability condition is the most frequent situation; (iv) about the piers, materials, geometries, and configuration (in terms of steel rebars quantity and position) resulted in Adequacy and Operability conditions most of the time. Piers with stem and hollow section are characterized by higher safety verification indexes than the kind of piers; (v) considering the Adequacy condition under the degradation effects, many viaducts do not satisfy the safety verifications for several mechanisms occurring both in the decks or in the piers. About the decks, the longitudinal external beams are the elements mostly affected by the degradation; about the piers, the degradation hits especially the top of the pier stems, pier-caps and Gerber half-joints (if present); (vi) when the Adequacy condition is not positively satisfied, frequent reasons are pure shear failure (for piers and decks) or shear-bending mechanisms (for decks); (vii) comparing the safety indexes for traffic loads (IR), with or without degradation, basically the results are very similar themselves; it means that the predominant cause in the negative safety verifications is represented by the increase of the traffic loads over time. In the viaducts where the IR without degradation is higher than the corresponding one with degradation, run-off, exposed bars and corrosion are present on significative parts of the longitudinal beams of the deck; (viii) comparing IR due to the traffic to IS due to the seismic action (both without degradation), only 14% of the viaducts shows IS higher than IR; (ix) in the actual conditions, the safety verifications under traffic and seismic actions are positively satisfied by (about) 9.5% of the analyzed viaducts, therefore, the identification of the structural deficiencies in a large-scale management policy appears mandatory for the safe use of existing viaducts over time.

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