Seismic Rehabilitation of RC Bridges by Using FRP and SRP: Case Study of a Bridge in the South of Italy

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Synopsis: The paper deals with a rehabilitation case study on a pre-stressed concrete (PC) bridge (named "Torrente Casale"), located in the south of Italy (on the Salerno-Reggio Calabria highway). The bridge, built in the '70s, was enlarged in 2001 in order to satisfy the new traffic demand. A seismic assessment of the bridge resulted necessary in order to verify its capacity to sustain both gravity and seismic loads. Both destructive and non-destructive tests have been performed in order to evaluate concrete and steel reinforcement mechanical properties. A theoretical analysis was performed, showing that the bridge piers existing cross section and internal reinforcement were not adequate to satisfy the seismic actions. Thus, two rehabilitation systems were investigated: a) an innovative technique based on the combined use of Fibre Reinforced Polymer laminates (FRP) and Steel Reinforced Polymer spikes (SRP), b) a traditional rehabilitation technique (i.e. RC jacketing). The design assumptions and calculations for the rehabilitation as well as the comparison between the effectiveness of the two investigated strategies are presented and discussed in the paper. The main construction phases of the strengthening technique, executed by following the first outlined strategy are also presented and illustrated.

Keywords: bridge, bridge belt, FRP, PC, RC, seismic rehabilitation, SRP

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INTRODUCTION

Most of western industrialized countries are characterized by highways that are more than 25-30 years old; in particular, a large part of the existing bridges were built after the second world war period or during the construction of the highway network (i.e. 1940 – 1970). They are mostly made by reinforced concrete (RC) or pre-stressed concrete (PC) members; durability problems as well as the poor maintenance, lead to a high demand of restoration and strengthening in order to ensure acceptable safety levels. In some cases rehabilitation systems are necessary as a result of increasing knowledge about the regions seismicity and, consequently, the design seismic actions. Furthermore, in some other cases it results necessary to enlarge the bridge dimensions due to the increased traffic demand. The present paper focuses on an existing bridge built in the '70s in the south of Italy (Salerno-Reggio Calabria highway); a bridge view is reported in Figure 1. In 2001 a new geometrical configuration was designed to supply the increased traffic demand; consequently an enlargement of the carriageway was planned. However, only some parts of the design project were effectively realized on the bridge. In particular, it was foreseen to enlarge the bridge belt by building two new external piers and strengthening the existing piers by RC jacketing. The existing six PC girders were removed and replaced with fourteen new ones; moreover two new external piers, an added part of footing connected to the existing one and two new foundation piles were also added. On the contrary nothing has been done to increase the structural capacity of the existing piers. Thus a seismic assessment of the bridge resulted necessary in order to verify its capacity to sustain both gravity and seismic loads. Since the theoretical analysis showed that the bridge capacity was insufficient with respect to the actual demand, a rehabilitation strategy was investigated. Traditional retrofit techniques (i.e. RC jacketing) were largely used in the past to strengthen bridge structural members, with particular attention to piers bending capacity [1, 2, 3]. Nowadays the use of innovative strengthening techniques based on the use of Fibre Reinforced Polymer (FRP) are strongly grown [4, 5, 6, 7] due to their easy and fast installation procedure and durability performances. The present paper deals with the rehabilitation intervention design and execution on the "Torrente Casale" bridge; both a traditional and innovative strengthening strategy was investigated. The assessment of the bridge capacity and the rehabilitation systems were performed by following different steps: 1) analysis of existing documentation related to the bridge (provided by "Autonomous National Company of Roads", ANAS); 2) materials characterization through destructive and non-destructive in situ tests; 3) bridge capacity assessment by using linear static analysis and structural deficiency evaluation; 4) design of two different rehabilitation techniques; based on a traditional RC jacketing or on an innovative technique based on the use of Carbon Fibre Reinforced Polymer laminates (CFRP) and Steel Reinforced Polymer spikes (SRP); 5) selection and execution of the structural rehabilitation.

DESCRIPTION OF THE STRUCTURE

The "Torrente Casale" bridge was built in the early 70's; it is located at km 318 +175 of the Salerno-Reggio Calabria highway in the south of Italy. It consists of a double lane for each traffic direction. The "north direction" is the carriageway connecting Reggio Calabria to Salerno and the "south direction" is the opposite (see Figure 2).

The original bridge belt configuration is reported in Figure 3. The "north direction" and the "south direction" carriageways have been realized by two unconnected decks. Each deck was sustained by three girders (height H=1400 mm [55.12 in.]) 2500 mm [98.43 in.] spaced). The deck was about 17100 mm [673.23 in.] large. The girders length varied according to the bridge spans dimension; the external spans length were almost 15900 mm [625.98 in.] while the internal span length was about 13400 mm [527.56 in.] (see Figure 2). Bridge belt was characterized by the top beam (17100 mm [673.23 in.] wide), three circular piers (diameter D=1000 mm [39.37 in.] and height H=5550 mm [218.50 in.]) and a deep foundation system (foundation beam characterized by height H=1200 mm [47.24 in.] and width B=1000 mm [39.37 in.]; piles characterized by a circular cross section, diameter D=1000 mm [39.37 in.]). The piers and foundation piles were spaced at 6000 mm [236.22 in.] each other. The piles total length was equal to 12000 mm [472.44 in.] in order to reach the rock belt. An enlargement intervention was realized in the 2001 to satisfy the increased traffic demand; as a consequence, the geometrical and structural configuration of the bridge changed significantly. In Figure 4 the actual bridge configuration is reported; the original bridge belt configuration is pitted by hatching. In particular, seven PC girders have been placed for each deck on renovated setoffs realized on the top beam. The girder height was H=1700 mm [66.93 in.] and they were spaced at 1850 mm [72.83 in.]. The cap beam length changed from 17100 mm [673.23 in.] to 26500 mm [1043.31 in.] of the actual configuration. Furthermore, two external piers (diameter D=1000 mm [39.37 in.]) were added in order to ensure the geometrical dimension increase. The new piers were placed on the extension of the foundation beam; two piles (diameter D=1000 mm [39.37 in.]) were also added.

PRELIMINARY INVESTIGATION: MATERIALS CHARACTERIZATION

Destructive and non-destructive tests were performed on existing structures to increase the level of knowledge of the structural material mechanical characteristics and to properly model the structure. Non-destructive tests were performed on the bridge belt in order to assess the longitudinal and transversal steel reinforcement layout as well as the bars diameter. By using a pacometer, it resulted that transverse reinforcement on existing piers was made by 8 mm [0.32 in.] spiral bars spaced at 200 mm [7.87 in.], while 16 mm [0.63 in.] diameter bars were used as longitudinal reinforcement. The external added piers are characterized by 16 mm [0.63 in.] longitudinal bars diameter and transversal 12 mm [0.47 in.] spiral bars spaced about 150 mm [5.91 in.]. Bars of 28 mm [1.10 in.] diameter were found as longitudinal reinforcement of existing cap beam, while bars 26 mm [1.02 in.] diameter were used as longitudinal reinforcement during the construction of the cap beam enlargement. Two separate non-destructive tests were planned to evaluate the concrete mechanical properties on both existing and added piers by using sclerometric test (type Schmidt N/NR) and sonic test. The compressive strength, f_c, obtained by such tests are summarized in Table 1 and Table 2. The SONREB method was also adopted to take into account both rebound index (RI) for sclerometric tests and sonic waves propagation (V_{dir}) for sonic tests. In particular, three expressions were adopted:

- $f_c = 9{,}27 \cdot 10^{-11} \cdot V_{dir}^{\,2{,}6} \cdot RI^{1{,}4}$ (RILEM formula) (1)
- $f_c = 8{,}06 \cdot 10^{-8} \cdot V_{dir}^{\,1{,}85} \cdot RI^{1{,}246}$ (GASPARIK formula) (2)
- $f_c = 1{,}2 \cdot 10^{-9} \cdot V_{dir}^{\,2{,}446} \cdot RI^{1{,}058}$ (DI LEO – PASCALE formula) (3)

The compressive strength obtained by using such expressions are summarized in Table 3. The compressive strength provided by non destructive tests were, especially for existing piers, very different from those expected by available design information (i.e. compressive strength equal to 20.75 MPa [296.4 psi] and 29.05 MPa [415 psi] for existing and added piers, respectively). This is probably due to the carbonation phenomenon, the absence of binder and higher maximum diameter of the rubble as showed in Figure 5. Thus the high values of compressive strength computed by using the non destructive tests lead destructive tests to became of primary importance. Hence, a program of destructive tests was planned on the bridge structural elements: existing and added piers, and on existing and added cap beam. Cylindrical samples were obtained from coring with a diameter – height ratio (D/H) equal to 1.00. The cores were extracted from the added and existing piers at different position and from the added portion of cap beam. Compression tests on cylindrical concrete specimens were performed according to the procedures outlined in UNI 6131:2002 [8] and UNI EN 12504-1:2002 [9]. The average concrete compressive strength, f_{cm}, was computed starting from experimental results on each core, by using the following expressions:

$$f_c = \frac{f_{test} \cdot D}{1{,}5 + \frac{1}{\lambda}} \quad \text{and} \quad f_{cm} = \frac{\sum_1^n R_c}{n} \tag{4}$$

where f_{test} is the coring compression strength, D (about 94 mm [3.70 in.]) is the coring diameter, λ is the ratio between the coring geometrical properties (D/H) and n is the number of specimens. The results are summarized in Table 4 and Table 5. The average values of cylindrical compressive strength related to added and existing piers and to the new part of the cap beam are reported in Table 6. In the theoretical assessment of bridge capacity, the concrete compressive strength and steel yielding strength introduced into the structural model must be modified taking into account the construction knowledge level (KL). The level of knowledge (KL) is related to the Confidential Factor (CF). According to Eurocode 8 provisions and available data the KL2 was considered for the analysis of the bridge and consequently CF=1,2 was assumed in the calculations. Based on such assumptions, design concrete and steel strength, f_{cd} and f_{yd}, were computed as follows:

$$f_{cd} = f_{cm} \cdot \frac{1}{CF} \text{ (concrete)} \quad \text{and} \quad f_{yd} = f_y \cdot \frac{1}{CF} \text{ (steel)} \qquad \text{(5-A: ductile structural element behavior)}$$

$$f_{cd} = \frac{f_{cm}}{\gamma_c} \cdot \frac{1}{CF} \text{ (concrete)} \quad \text{and} \quad f_{yd} = \frac{f_y}{\gamma_s} \cdot \frac{1}{CF} \text{ (steel)} \qquad \text{(5-B: brittle structural element behavior)}$$

where f_{cm} is the concrete strength according to destructive test results, γ_i are the material factors derived from Eurocode 2 , CF is the confidential factor and f_y is the yielding stress based on the steel type. Design values adopted in the structural modelling are reported in the last columns of the Table 6 and Table 7.

BRIDGE CAPACITY ASSESSMENT

According to the reached knowledge level a numerical analysis was implemented to verify the bridge capacity; in this particular case the linear static analysis is adequate for the aim. The theoretical bridge belt capacity was assessed by using the finite element analysis program SAP2000 [10], very commonly used by structural engineering practitioners. Thus, after defining the material mechanical properties, the bridge structural model and the acting loads were defined. The horizontal structural elements (i.e. PC girders) were replaced during the bridge enlargement and they were designed in order to stand the new traffic loads and structural demand increasing. They were made according to the design and thus their bending capacity assessment is out of the scope of the present paper. On the other hand, the rehabilitation technique of existing piers was not conforming to design provisions; therefore, the analysis has been focused on the bridge belt structural performances. Linear static analysis was implemented to analyze the bridge belt because of its structural scheme (cantilever scheme when longitudinal seismic actions are considered and frame scheme when the transverse seismic actions are analyzed); indeed the girder are not embedded on the cap beam and thus it is possible to model the vertical structural element only by applying due to horizontal structural members and no-bearing elements. The bridge belt geometrical model is reported in Figure 6; the structure-soil interaction has been modelled applying elastic spring one meter spaced along the foundation piles. The elastic constant of the spring into the model is placed equal to the soil Winkler constant k, generally accepted as a main soil characteristics, equal to 20000 t/m^3 [13449685.73 tons/in.^{3}]. The loads acting on the structure have been assessed according to European design rules by means of Eurocode 2 [11] and Eurocode 8 [12]. Sixteen load combinations were computed in order to have the worst condition in terms of stresses and displacements of the bridge structural model. In order to evaluate the belt capacity under longitudinal and transversal seismic action (direction of vehicular flow and orthogonal to vehicular flow) two different models were adopted as reported in Figure 7 and Figure 8. For longitudinal seismic actions the bridge belt was modeled as a one degree of freedom system for which a mass m_1 was assigned; the mass m_1 (equal to 50.30 t/g [55.10 tons/g]) has been evaluated considering the masses of PC girders and decks connected to the bridge belt by fixed constrains; the structural elements linked to the bridge belt by sliding constrains were not taken into account. In the case of transversal earthquake actions, the mass m_2 (equal to 47.30 t/g [51,82 tons/g]) is computed by applying on the bridge belt both half masses of PC girders and decks on the left and on the right of the vertical structure, without considering any kinematics changing between sliding and the fixed constrain (Figure 8). Indeed, for transversal seismic direction the sliding constrains have been hypothesized equal to fixed constrains in terms of cinematic characteristics. In order to compute the seismic loads acting on the structure, an associated spectral acceleration must be taken into account by evaluating the design response spectra, using the structural factor, q. The structural factor, q, was computed by evaluating local and global ductility. The structural factor q of the existing structure was evaluated through the structural ductility calculated in terms of displacement. To compute the q factor a structural model was adopted considering the bridge belt loaded by the structural elements self-weight only. In particular, the local ductility is computed with reference to the base piers cross section, where the plastic hinge formation is expected.

The local ductility, μ_χ, is given by the ratio between the ultimate curvature, $\chi_u=\varepsilon_{cu}/x_{cu}$ and the yield curvature, $\chi_y=\varepsilon_{sy}/d\text{-}x_c$ of the pier cross section (ε_{sy} is steel yield strain, d is the effective cross section height and x_c and x_{cu} are the neutral axis at yielding and ultimate respectively and ε_{cu} is ultimate concrete strain equal to 0.0035). According to Eurocode 2 [11], the global ductility can be applied equal to q factor, if the vibration period of the main structure is higher than T_c (that is the period of the elastic response spectra at the end of the constant branch), as occurred on the present case. The global ductility is a function of local ductility, μ_χ, by using the following expression:

$$\mu_\delta = 1+3(\mu_\chi - 1)\cdot\frac{l_{pl}}{L}\cdot\left(1-0{,}5\cdot\frac{l_{pl}}{L}\right) = 1+3(4{,}63-1)\cdot\frac{0{,}96}{6{,}00}\cdot\left(1-0{,}5\cdot\frac{0{,}96}{6{,}00}\right) = 2{,}60 \qquad (6)$$

where μ_δ is the global ductility in terms of displacements, μ_χ is the ductility in terms of curvature, l_{pl} is the plastic hinge length taken equal to d and L is the column length. Thus, in order to consider the worst condition in terms of stresses and displacements on the structure for seismic load combinations, in a conservative manner the q factor value was assumed equal to 2.50. The design response spectra computed by using the q factor allowed the associated seismic accelerations for both longitudinal (a_l =0.98 m/sec^2) and transversal (a_t =2.19 m/sec^2) directions to be computed. Then multiplying the masses and the accelerations, the seismic actions were computed ($F_i = a_i\ x\ m_i$). Thus, at the end of the computations, two different seismic actions for longitudinal and transversal directions were assumed equal to F_l = 49.20 t [54.22 tons] and F_t=110.10 t [121.33 tons], respectively.

EXISTING STRUCTURE

The structural analyses of the existing bridge belt indicate that a rehabilitation technique was necessary in order to make the structure safe. Several deficiencies both in terms of bending and shear capacity of piers and cap beam were detected respectively. In particular the results showed that for gravity load combinations the bridge reached an acceptable safety level according to Eurocode 8[12]. On the contrary, the main structural problems are related to seismic load combinations in both longitudinal and transversal directions. The interaction domains point out the flexural capacity deficiency at the ends of the existing piers on both bottom and top cross sections. For instance, the modelling results have shown the need to increase the strength capacity by a factor of 25% and 18% at the bottom section and at the top section of the existing piers, respectively. The shear demand was higher than capacity on the added part of the cap beam (i.e. 2001) due to gravity loads combined to vehicular flow as showed in Figure 11 (the strength capacity need to be increased by a factor of about 11%).

REHABILITATION STRATEGIES

Once the structural deficiencies were assessed, two different rehabilitation strategies were investigated in order to reach a good safety level for the whole structure: a) an innovative technique based on the combined use of Carbon FRP (CFRP) laminates and SRP spikes, b) a traditional rehabilitation system (RC jacketing).

Innovative rehabilitation system

The innovative rehabilitation technique is based on the combined use of CFRP laminates and SRP spikes to increase the structural performances under both gravity and seismic loads; this rehabilitation technique does not require the changing of the structural model in terms of geometry, elastic stiffness and seismic actions. The design was performed according to CNR DT 200/2004 [13]. In particular, SRP spikes are used to increase the bending capacity of RC piers at the ends, without enlarging their cross section, while CFRP wrapping is used to avoid existing steel vertical reinforcement and SRP spikes buckling as well as to improve the concrete compressive strength owing to the passive confinement generated by the FRP jacket as the concrete dilates under axial, shear and moment load combination. Z-3x2-Zinc SRP spikes were used; the 3X2 symbol indicates the number of wires making up the single "cord". The cord is composed by three wires wrapped by other two wires of the same diameter equal to 0.35 mm [0.014 in.]. The asymmetrical shape of the Z-3x2-Zinc SRP allows to reach excellent fatigue strength due to tension and high bending stresses. According to the manufacturer data [14], the mechanical properties of the individual cord are characterized by modulus of elasticity of 184000 MPa [26686936.8 psi], tensile strength of 3070 MPa [445265.8 psi] and ultimate strain equal to 1.70% (see Table 8).

To assess the SRP spikes contribution on the cross section strength, the following expressions have been used. An equivalent steel area has been computed starting from the cords area; the latter was assumed equal to 0.48mm^2 [0.019 in.^{2}], as listed in the manufacturer data. 8.7 cords are present in one centimetre of strip:

$$Area_{(cords)} \cdot n°_{cord} = \frac{Area_{(SRP)}}{cm} \qquad \left[Area_{(cords)} \cdot n°_{cord} = \frac{Area_{(SRP)}}{in.} \right]$$
$$0.48mm^2 \times 8.7 = 4.176\,\frac{mm^2}{cm} \qquad \left[0.000744\,in^2 \times 8.7 = 0.00647\,\frac{in^2}{in}\right] \tag{7}$$

As design hypothesis, an SRP wide strip around 70 mm [2.76 in.] was chosen corresponding to an equivalent steel area of:

$$L_{strip} \cdot \frac{Area_{(SRP)}}{cm} = Area_{equivalent} \qquad \left[L_{strip} \cdot \frac{Area_{(SRP)}}{in.} = Area_{equivalent} \right]$$
$$4.176 \times 7 = 29.23mm^2 \cong 1\phi 6 = 0.28cm^2 \qquad \left[0.00647 \times 2.76 = 0.0179 in.^2 \cong 1\phi 6 = 0.0434 in.^2\right] \tag{8}$$

The SRP spikes were located in appropriate holes realized drilled the beam foundation (pier bottom cross-section) and the cap beam (pier top cross-section) ensuring a bond length equal to 300 mm [11.81 in.]. The bond length was computed according to the following equation (extracted from EC2[11]):

$$L_{bd} = \frac{\phi}{4} \cdot \frac{f_{fd}}{f_{u,res}}$$
$$L_{bd} = \frac{\phi}{4} \cdot \frac{550{,}0MPa}{3{,}0MPa} = 45.83 \cdot \phi = 275mm \quad L_{bd} = \frac{\phi}{4} \cdot \frac{79770.74\,psi}{435.12\,psi} = 45.83 \cdot \phi = 275mm[10.83 in.] \tag{9}$$

where ϕ is the SRP diameter, f_{fd} is the SRP delamination strength and $f_{u,res}$ is the epoxy resin tensile strength. Sixteen spikes were used, placed around the piers cross section inside the holes before mentioned (see Figure 12). SRP spikes were supposed to be applied on the pier bottom cross-sections starting from the foundation beam to 770 mm [30.32 in.]. Such a length was computed in order to fully cover the critical zone, where demand is higher than capacity, equal to a height of 450mm [17.72 in.] from the foundations. Adopting a bond length equal to 300 mm [11.81 in.], a total length was assumed equal to 1070 mm [42.12 in.]. The same configuration was adopted for pier top cross-sections, starting from the cap beam. CFRP sheets were applied in order to prevent SRP spikes buckling due to cyclical loads. Considering that the bridge is exposed to aggressive environment, durability issue is the driving design criterion and thus, carbon fibres have been selected and preferred to glass fibres. In particular, uniaxial CFRP laminates characterized by 900 gr/m^2 density, 0.13 mm/ply [0.005 in./ply] dry fibres thickness, modulus of elasticity about 230000 MPa [3285714 psi] and tensile strength equal to 4830 MPa [69000 psi] were used. Two CFRP layers were adopted to wrap the piers ends; each layer was applied by using two sheets (height of 400 mm [15.74 in.] each) with an overlap length of 30 mm [1.18 in.], see Figure 13. They were designed in order to avoid the existing steel vertical reinforcement and SRP buckling, improve the concrete behavior and to avoid the slip of existing steel reinforcement in RC piers at the locations of lap splice (note that the overlap between the foundation beam reinforced bars and the pier reinforced bars was equal to 650mm [25.59 in.]. The number of CFRP layers used to confine the member cross sections was computed taking into account the buckling of longitudinal bars, by applying the CNR[13] formula, reported in the following:

$$t_f \cong \frac{10 \cdot n \cdot d}{E_f} = \frac{10 \cdot 5 \cdot 1000}{230000} = 0{,}22mm[0.009 inc.] \tag{8}$$

where *n* represents the total number of steel vertical reinforcement subjected to buckling (highlighted in the fig.13), d represent the size of cross section parallel to the bending plane and E_f is the CFRP Young's modulus. The use of SRP spikes wrapped by CFRP laminate lead to an increase of piers bending capacity at the ends equal to 40%, almost 10% higher than the existing cross-section capacity gap (see Figure 14). CFRP sheets were also used to increase the cap beam shear capacity; they were applied by using four U-shaped sheets characterized by 300 mm [12.00 in.] width and a total length equal to 3700 mm [145.67 in.] (see Figure 15). Three layers of CFRP U-wrap were installed on the top beam (Figure 15). This strengthening configuration provides 12% shear strength increase, reaching an acceptable safety level if compared to the shear capacity increase target equal to 11%.

Traditional strengthening system

The aim of the traditional rehabilitation strategy was to increase the bending strength of both exiting and added piers of the existing structure. Such a goal can be achieved by means of RC jacketing system preserving the original piers cross section (circular cross section); the original cross-section piers pass from diameter of 1000 mm [39.37 in.] to 1120 mm [44.09 in.]. A concrete design strength equal to f_c = 29.05 MPa [415 psi] was assumed in the calculations. The longitudinal reinforcement of the jacketed piers was designed as 16 bars 20 mm [0.79 in.] diameter and 10 mm [0.39 in.] stirrups spaced at 250 mm [9.85 in.]. Steel bars and stirrups design strength were f_y = 430 MPa [6143 psi]. The added bars were anchored in the cap beam and foundation beam in order to get the existing part and the added safe collaborating one. The thickness of the cross section added part was computed in order to insert the longitudinal steel bars and spiral bars in the correct position thus preserving their integrity from the outside environment (Figure 16). The added structural elements (piers with RC jacketing) assessment in terms of strength and ductility was performed considering three main hypotheses: a) the jacketed elements should behave as a monolithic material, providing full adhesion between the existing concrete and added one (Figure 16), b) the axial load is applied to overall jacketed section and c) the concrete mechanical properties for the whole structure is assumed equal to the lowest between the added and existing one. To model the RC jacketed structure it was necessary to consider that, as a consequence of the piers enlargement, the bridge masses change (masses increase of about 25%); thus higher values of lateral forces have to be taken into account. The design rehabilitation based on the jacketed technique led to a bending strength increase of about 37%, which is higher than the target increase equal to 25% (see Figure 17). The shear strengthening of the top beam was achieved by increasing the cross-section, assuring the structural collaboration between the added element and existing one by using 10 mm [0.39 in.] diameter steel bars mechanical connectors (see Figure 18). The cross-section enlargement leads to shear strength increase of about 21%.

REHABILITATION SYSTEMS COMPARISON

The structural analysis results confirmed the effectiveness of the two rehabilitation methods investigated. In particular, the rehabilitation strategy based on the use of composite material increases the global bridge strength without affecting the structural stiffness and the masses involved. Thus, the seismic actions did not change compared to those of unreinforced structure. For that reason, this strengthening does not involve the foundation of the bridge, being subjected to the actions computed for the "as built" structure. In addition, the installation procedure is fast and easy and can be adopted also when time or space restrictions are the design driving criteria.

The traditional rehabilitation system provides a significant structural mass increase and, consequently, the seismic actions to be computed are higher. As a drawback, such technique may results much more invasive and difficult from constructability standpoint with a lengthy disruption of the structure function. As summary, the innovative rehabilitation system performs advantages as reported in the following: a) it provides a faster application and less invasive intervention on the structure, leading to costs reduction; b) closure of the vehicular traffic is not required; c) CFRP choice ensures a good strengthening durability also taking into account the aggressive environment conditions, typical of bridge structures. Based on these reasons it was decided to strengthen the bridge by using innovative rehabilitation strategy.

REHABILITATION INTERVENTION BY USING INNOVATIVE MATERIALS

The innovative solution was selected for this particular case study; the main steps installation procedures are reported in the following. Prior to laminates installation, unsound concrete was removed in all zones of the elements where crushing was detected; then the original cross-sections were restored using a non-shrinking mortar. In addition, all the holes caused by destructive tests realized on the bridge were epoxy-injected. Then, according to the design provisions, the holes were made at the ends of the piers by using a suitable hammer, reaching an appropriate hole deep (see Figure 19-a). Once primer was applied on the pier surfaces (see Figure 19-b) and the pier holes were filled by using epoxy mortar (see Figure 19-c). The SRP strips total length is equal to 1150 mm [45.28 in.]; 380 mm [14.96 in.] of the SRP strips total length were rolled at the end in order to realize the spikes that were put into the holes. The free parts of the SRP strips were rested on the ground (see Figure 19-c). An epoxy mortar layer was applied on the piers surface and the free parts of the SRP were placed on them, ensuring the adhesion between the mortar and the SRP strips; moreover the direct interaction between the SRP and CFRP was avoided in order to prevent corrosion phenomena (see Figure 19-d). Then, an epoxy resin layer was applied on the piers (Figure 19-e) and the first CFRP ply was wrapped on the piers surface (Figure 19-f). This procedure was repeated for the second layer. Another layer of mortar was applied on the last laminate coat in order to protect the rehabilitation system from environmental effects. Typical FRP installation procedure was made to realize the shear strengthening system, by rounding the corner to avoid stresses concentration. The procedure was repeated to install three FRP sheet layers.

CONCLUSIONS

Most of the bridges built in Italy between the '60s and '70s were designed considering gravity loads only. Current design codes have introduced new rules to evaluate the seismic actions combined with other loads. As a consequence of the new design rule requirements, several existing bridges need to be strengthened in order to reach an acceptable safety level. The paper deals with an RC bridge seismic rehabilitation for which two different strategies based on the use of innovative materials or on a traditional techniques have been evaluated. A structural model of the existing bridge was developed and two rehabilitation solutions were analysed.
Both investigated strengthening systems resulted effective to ensure a global acceptable safety level on the bridge. In particular, in the case study presented:

- RC jacketing could increase the piers bending capacity of about 37% (25% requested by the structural analysis) and the cap beam shear strength of about 21% ;
- The combined use of SRP spikes and FRP laminates could increase the piers bending capacity of about 40% and the cap beam shear strength of about 12%.

The first rehabilitation technique is realized by increasing the cross-section of the structural element, leading to a stiffness increase and changing the masses involved into the calculation of the bridge. Hence, the seismic actions acting on the structure are higher than those computed for the existing structure and the foundation system could be subjected to higher actions. The execution of the traditional rehabilitation technique appears difficult because it demands invasive methods to strengthen the foundation system and the structural element, supposing to close the vehicular traffic on the bridge and using particular machinery. All these conditions lead to increase the time to realise it and thus, the costs. On the other hand, the latter technique, based on innovative materials, does not change the stiffness of the structure and structural element masses. The innovative strengthening technique could be done without closing the traffic on the bridge. The rehabilitation case study presented herein was completed within two weeks. About calculations, the same finite element model was used to analyse the existing and the strengthening structures. Thus, the same actions by means of gravity and seismic combination loads were applied on the structural model, leading to perform the strengthening system, reducing the time. The costs-benefits analysis highlighted that the strengthening system based on SRP spikes and CFRP laminates ensure a performances level higher than those offered by the traditional one.

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Table 1—Rebound Index and sclerometric test results

Pier	Rebound Index - RI	f_{ck} – Mpa [psi] test result
# 1	37,8	27.40 [391.44]
# 2	58,5	>49.00 [700.00]
# 3	57,8	>49.00 [700.00]
# 4	56,8	>49.00 [700.00]
# 5	38	27.40 [391.44]

Table 2—Sonic test results

Pier	d – cm [in.]	V – km/sec [miles/sec]
# 1	40.0 [15.5]	4.18 [2.60]
# 2	40.0 [15.5]	3.80 [3.36]
# 3	40.0 [15.5]	3.88 [2.41]
# 4	40.0 [15.5]	3.85 [2.39]
# 5	40.0 [15.5]	4.13 [2.57]

V – km/sec [miles/sec]	Concrete Quality
>4,5 [2.80]	Very Good
3,5 – 4,5 [2.18 - 2.80]	Good
3,0 – 3,5 [1.86 - 2.18]	Uncertain
2,0 – 3,0 [1.24 - 1.86]	Bad
< 2,0 [<1.24]	Very Bad

Table 3—Non destructive test results

Pier	Rilem Formula f_{ck} – Mpa [psi] (1)	Gasparik Formula f_{ck} – Mpa [psi] (2)	Di Leo – Pascale Formula f_{ck} – Mpa [psi] (3)
# 1	25.97 [16.14]	25.48 [15.83]	28.80 [17.90]
# 2	46.48 [28.88]	44.65 [27.75]	42.08 [26.15]
# 3	48.39 [30.07]	45.82 [28.47]	43.82 [27.23]
# 4	46.23 [28.73]	44.16 [27.44]	42.16 [26.20]
# 5	31.54 [19.60]	30.46 [18.93]	32.70 [20.32]

Table 4—Concrete compressive strength of existing piers

Pier	Height of coring	Cores dimension (mm) [in.]		f_c MPa [psi]	f_{cm} N/mm^2 [psi]
		D	H		
# 2	h=5000 mm [196.85 in.]	93.45 [3.68]	93.08 [2.93]	22.10 [315.70]	29.52 [421.70]
# 3	h=1000 mm [39.37 in.]	93.23 [3.67]	93.04 [3.66]	38.00 [542.90]	
# 4	h=3000 mm [118.11 in.]	93.54 [3.68]	93.50 [3.68]	34.00 [485.70]	
# 7	h=5000 mm [196.85 in.]	93.54 [3.68]	93.55 [3.68]	24.50 [350.00]	
# 8	h=1000 mm [39.37 in.]	93.15 [3.67]	93.08 [3.66]	29.70 [424.30]	
# 9	h=3000 mm [118.11 in.]	93.69 [3.68]	93.03 [3.66]	28.80 [411.50]	

Table 5—Concrete compressive strength of added piers

Pier	Height of coring	Cores dimension (mm) [in.]		f_c MPa [psi]	f_{cm} N/mm^2 [psi]
		D	H		
# 1	h=5000 mm [196.85 in.]	94.10 [3.70]	98.80 [3.88]	20.00 [285.70]	36.08 [525.70]
# 1	h=1000 mm [39.37 in.]	94.20 [3.70]	94.20 [3.70]	41.00 [585.70]	
# 1	h=3000 mm [118.11 in.]	94.20 [3.70]	93.10 [3.66]	38.50 [550.00]	
# 6	h=5000 mm [196.85 in.]	94.20 [3.70]	91.50 [3.60]	40.50 [578.60]	
# 6	h=1000 mm [39.37 in.]	94.20 [3.70]	93.70 [3.68]	36.50 [521.50]	
# 6	h=3000 mm [118.11 in.]	94.20 [3.70]	93.60 [3.68]	40.00 [571.40]	