PSEUDO-DYNAMIC TESTS ON REPAIRED AND RETROFITTED BRIDGE COLUMNS

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ABSTRACT:

This research aims to study the seismic performance of existing r.c. bridge piers specimens heavily damaged after previous pseudodynamic tests and actually repaired and upgraded by using self compacting concrete, stainless steel rebars and CFRP wrapping. Pier specimens are representative of tall and squat circular r.c. piers designed according to Eurocode 8 and Italian Code before 1986. The study comprehends preliminary tests carried out on self compacting concrete and stainless steel bars, as well as pseudodynamic tests in order to evaluate the effectiveness of adopted repairing and upgrading techniques to increase both ductility and shear strength.

KEYWORDS: Self compacting concrete, stainless steel, FRP, bridge piers, pseudodynamic test.

1. INTRODUCTION

This paper focuses on current experimental tests at the *Laboratory of experiments on materials and structures* of the University of Roma Tre on column specimens representative of tall and squat circular r.c. piers designed according to Eurocode8 (1998-2) and Italian Code (D.M. LL.PP. 24.01.86). In previous research (De Sortis & Nuti 1996) some columns were tested until collapse by pseudodynamic tests but others are still entire. An accurate study to detect the level of degradation in materials, as the case of real structures after an earthquake, is now performed. Then, based on the evaluation done, EC8 columns are repaired and the Italian ones retrofitted by mean of FRP jacket, longitudinal stainless steel and self compacting concrete. Actual tests aim to evaluate the effectiveness of this technique. Repairing and retrofitting operations, test equipment and planned tests are presented.

2. PIER SPECIMENS

Eight 1:6 scaled column specimens representative of tall and squat 2.500 m diameter circular r.c. piers of regular and irregular bridges (Figure 1) designed according to Eurocode8 Part 2 (1998-2) and Italian Seismic Code (D.M. LL.PP. 24.01.86) are considered.

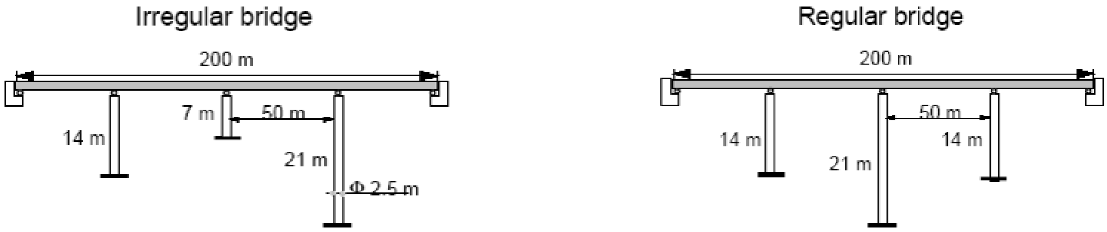

Figure 1. Layout of designed bridges

Reinforcement scaling criteria, based on similitude fulfillment of global quantities (flexural and shear strength, confinement effect, post-elastic buckling and pullout of rebars) were applied (De Sortis et al. 1994) allowing the use of ordinary concrete mixing and commercial reinforcing bars. An improved anchorage detail of the longitudinal bars to improve bond-slip scaling accuracy was also investigated. Geometry and reinforcement configurations of pier specimens together with relative design criteria are summarized in Table 1.

Table 1. Pier specimens characteristics (*D*=diameter [mm], *H*=height [mm], *C*=cover [mm]) and design criteria (*hoop spacing lager than EC8 prescriptions).

pier	#	design	*D*	*H*	*C*	bars	spiral
	1	DM	420	2340	30	24∅10	∅5/80
	2	DM	420	2340	30	24∅10	∅5/100
tall	3	DM	420	2340	30	24∅10	∅5/100
	4	DM	420	2340	30	24∅10	∅5/80
	5	EC8	420	2340	30	24∅10	∅6/40
	6	DM	420	1170	30	12∅12	∅6/120
squat	7	EC8*	420	1170	30	42∅12	∅6/120
	8	DM	420	1170	30	12∅12	∅6/120

Longitudinal and transverse reinforcement configurations and concrete cover thickness have some differences with respect to design drawings as sometimes it happens in practice. Concrete with R_{ck}=25 MPa and FeB44k steel were fixed in design. Tested concrete strength was about the same of the design one whilst steel yielding varied from 550 MPa to 600 MPa, therefore exceeding the design one.

3. CURRENT SPECIMENS STATE

Piers 2 (tall DM), 4 (tall DM), 5 (tall EC8), 6 (squat DM) and 8 (squat DM) were tested until collapse in previous pseudodynamic tests (De Sortis & Nuti, 1996) thus they are damaged while piers 1 (tall DM), 3 (tall DM) and 7 (squat EC8) have not been subjected to tests and are intact (Figure 2). Detected damages include cover spalling, longitudinal bar buckling and rupture, yielding in transverse reinforcement and concrete core crushing.

Figure 2. Current specimens state: tall (1, 2, 3, 4, 5) and squat (6, 7, 8) specimens

4. REPAIR AND RETROFITTING

Foreseen operations on seriously damaged piers 2, 4, 5, 6, 8 are (for example piers 6 and 8 in Table 2): mechanical removal of damaged concrete cover and cleaning of substrate from residue particles (phase 1), concrete core repair with resin injections (phase 2), substitution of damaged (yielded, buckled or broken) bars using stainless steel ones (phase 3), restoration of damaged concrete cover with self compacting concrete (SCC) (phase 4) and C-FRP strips application (phase 5).

Table 2. Repair and retrofitting of pier.

phase 1	phase 2	phase 3	phase 4	phase 5
				

4.1. Damaged concrete removal (phase 1)

Cracked and nearly detached part of concrete cover and core have been completely removed over an height of 550 mm (plastic hinge length plus overlap splice of restored longitudinal bar) from the pier base. About 20 mm of concrete around reinforcing bars have been removed and the latter accurately cleaned in order to guarantee optimal bond with the repairing material.

The quality of the interface between the existing concrete and the repairing material is essential for durability and effectiveness of restoration. In particular, lack of surface roughness makes the interface a preferential plane for rupture: mechanical removal followed by cleaning of substrate from residual particles was used to provide good bond (Table 2: phase 1).

4.2. Concrete core consolidation (phase 2)

Injection of concrete core with bicomponent expoxy resin (EPOJET LV with 70 MPa and 20 MPa compressive and flexural strength respectively at 7 days and adhesion to concrete higher than 3.5 MPa) with very low viscosity (140 mPa.s Brookfield viscosity with 1 hour workability time at 20°C) have been used. About 20-40 mm deep perforations have been performed to place small plastic tubes. Cracks were closed by spreading with thixotropic bicomponent expoxy plaster (MAPEWRAP12 with 30 MPa tensile strength after 7 days - ASTM D 638, adhesion to concrete higher than 3 MPa and setting time at 23°C of about 5 hours) in order to obtain a sort of pipe system inside the core ending with the tubes. ∅2-5 mm sand obtained by crushing was fixed on the fresh plaster to make its surface rough. Compressed air was insufflated in order to remove powder and to check the communicability between inner pipes. The injections were preformed from the bottom towards the top until inner cracks complete filling (Table 2: phase 2).

4.3. Reinforcing bar restoration (phase 3)

Austenitic stainless steels combine very good corrosion behavior with excellent mechanical properties (strength and toughness). Although initially more expensive, it offers cost savings in the long term because of reduced maintenance and protection operations and increasing life of the structure. These characteristics make it suitable for restoration of existing damaged reinforcing bars and for seismic retrofitting. Stainless steel (AISI304) ∅12 bars were used for damaged reinforcing bar substitution. First, the spiral reinforcement was cut and removed; the ends of embedded hoop were welded to remaining longitudinal bars. Yielded and buckled longitudinal rebars were cut and removed too. Remaining rebars were cleaned, by mean of a metallic brush, to eliminate rust layers and residual concrete, plaster and resin. Single new bars were placed aside the existing ones trying to keep inner lever arm and cover thickness equal to the original ones: it was impossible to apply them in pairs to avoid asymmetries because of the congested reinforcement of the foundation. Welding was performed away from the potential plastic hinge (Figure 3): since the portion of damaged bar often extends into the foundation, it was necessary to dig it (at least 70-80 mm deep) around the bar before welding.

Welding was the only practicable solution since the congested reinforcement of the foundation did not allow to perform holes deep enough and in the right position to guarantee effective bond even using resin.

Practical difficulties in welding due to reduced space around the bars made it necessary to realize two 50 mm weld puddles (the design length of a single puddle was 35 mm only). Removed spiral reinforcement was replaced with circular stainless steel ∅5 stirrups; spacing was calculated to assure the original design ratio $f_t\ A_s/s$ (A_s=hoop cross section, s=spacing, f_t=peak strength). Stirrups were fixed with intercrossed metallic fastenings to avoid welding that could produce dangerous local deformation in bars with such a small diameter. Anyway, 20 mm length welding was necessary to close stirrups (Table 2: phase 3).

Figure 3. Welded joints (a) top (b) bottom (c) detail.

Stainless steel reinforcing ribbed bars have been tested and their mechanical properties determined both for monotonic and cyclic behavior including post-elastic buckling (Albanesi et al. 2006). Monotonic tests in tension and compression on ∅12 bars (Figure 4) used in restoration have been performed according to the European Code UNI EN 10002-1 (2004).

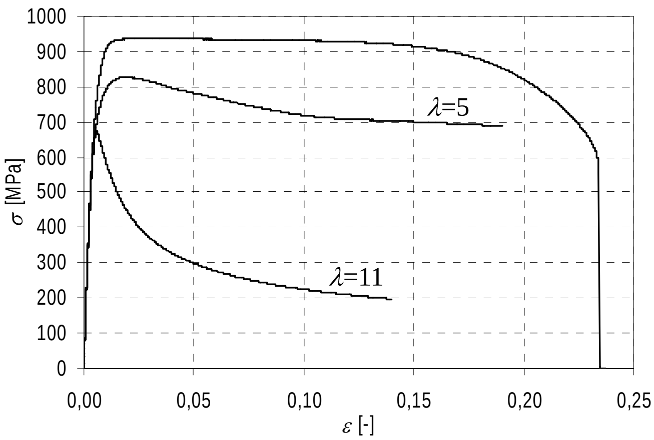

Figure 4. Monotonic tests in tension and compression ($\lambda=L/\varnothing$=5, 11) for ∅12 stainless steel bars.

Yielding strength f_{sy} (stress corresponding to 0.2% residual strain), peak strength f_{sm}, f_{sm}/f_{sm} ratio, uniform elongation at maximum stress A_{gt} and at rupture A_{su} are summarized in Table 3. Notice that mechanical characteristics for Italian stainless steel will be changed.

Table 3. Mechanical properties of stainless steel bars (mean values of strength and elongation).

∅ [mm]	f_{sy} [MPa]	f_{sm} [MPa]	f_{sm}/f_{sy} [-]	A_{gt} [%]	A_{su} [%]
5	829	927	1.19	5	22.6
12	790	941	1.19	5	22.0

4.4. Concrete Restoration (phase 4)

Good workability and resistance to segregation and remarkable filling and passing ability make Self Compacting Concrete (SCC) an optimal material to restore damaged concrete parts thus restoring element continuity and homogeneity (crossing thick new and existing reinforcing bars without causing vacuums into the element and discontinuity at contact surface). A restoring material should also have low shrinkage, high tensile strength and similar elastic modulus to that of the substrate in order to reduce tensile stresses due to shrinkage causing disjunction at interface and to guarantee continuity of the structural element after

restoration. SCC with addition of an expansion anti-shrinkage agent is suitable to this purpose.
Cast-in-place SCC seemed the most adequate solution for the piers under consideration; it includes, in addition to substrate preparation, formwork construction, concrete production (Figure 5a), its application and ageing. Formwork originally used to build the specimens was opportunely modified to include a metallic groove (Figure 5b) allowing SCC pouring without interruptions from a single hole in the formwork (Figure 5c). The final result was generally good although superficial imperfections were present after formwork stripping (Table 2: phase 4). SCC filled up all available spaces even the upper part, opposite to pouring position. Concrete restoration results in a SCC jacket 50-90 mm thick and 550 mm high.

Figure 5. (a) SCC production (b) formwork (c) SCC jet process.

A 35 MPa mean cubic strength is required as the most similar value to that of the original piers achievable with SCC. The main aims of the SCC mix design were: high fluidity, small diameter (<12 mm) aggregate (to easily fill up the space between reinforcement and concrete core), shrinkage offset adding conveniently measured out expansive agent. The latter was essential to guarantee complete filling and thus continuity at the interface between existing and new material. Resulting SCC mix was made of grit, sand, calcareous filler, cement expansive agent, superplasticizing and water with water-cement content equal to 0.48.

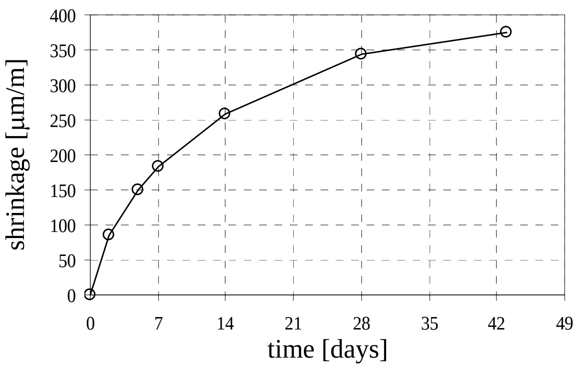

Figure 6. Hydraulic shrinkage of SCC.

In case of pier 6, SCC was formed with the same mix design but required more water and superplasticizing to get acceptable slumps due to possible filler humidity now under investigation. SCC compressive cubic mean strength at failure, measured over six standard cubes, resulted in 48.19 MPa (1.60 MPa standard deviation). Tests were carried out on fresh material to verify Code UNI 11040 (2003) prescriptions. Shrinkage behavior with time (Figure 6) was measured at the Laboratory of the Research Center BUZZI UNICEM in Guidonia (Rome).

4.5. Retrofitting (phase 5)

By analyzing pier characteristics (Table 1), piers 2 and 6 seem to be the most representative for retrofitting design of slender and squat piers respectively. Detected damage state clearly shows that tested squat and tall piers suffered shear and flexural failure respectively. These failure mechanisms were also analytically proved by comparing shear strength with shear at resisting bending moment.

Retrofitting designs for pier 2 and 6 aim to increase ductility capacity and shear strength respectively in order to match EC8 design prescriptions. The thickness of the FRP jacket to provide the target capacity (EC8) was determined according to CNR (CNR-DT 200 2004) guidelines. No partial coefficient was considered in the adopted capacity models. Commercial 0.169 mm thick C-FRP (Tenax HTS 300/10 with ultimate strain ε_{fu}=1.0% and elastic modulus E_f=240 GPa) was wrapped in 100 mm wide unidirectional fiber strips with interval of 20 mm within plastic hinge zone. FRP strip configurations for tall and squat piers are shown in Figure 7. The area of FRP strips application was brushed with a metallic brush to remove the very superficial concrete substrate. The reinforcement was fabricated with a wet layup process: first, resin (EPR320+EPH550) was directly applied to the concrete surface, then layers of fabric (1.700 m strips to guarantee 300-350 mm lap splice) were hand-placed with fibers direction perpendicular to pier axis and impregnated in place using ribbed rolls and squeegees to avoid entrapment of air. In case of more layers, distinct overlapped FRP strips were applied.

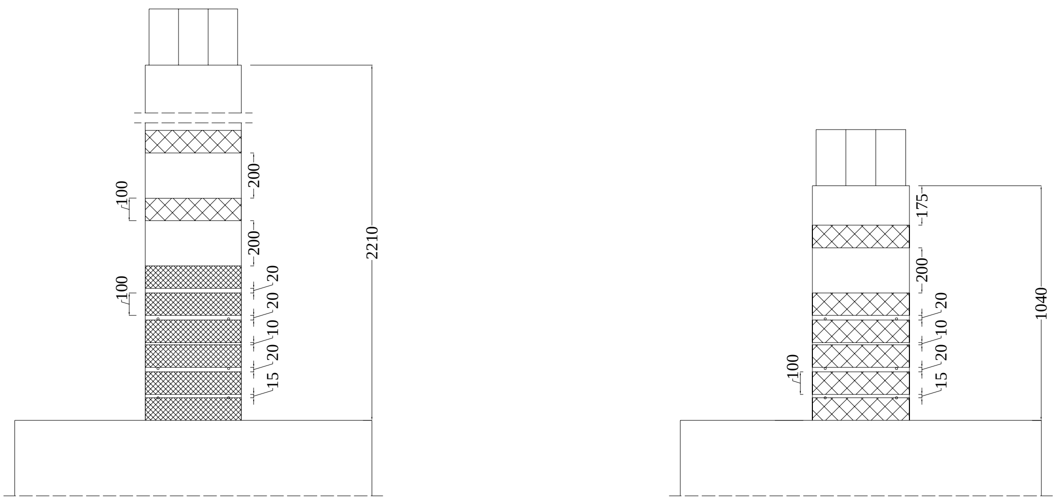

Figure 7. FRP strips configuration designed for (a) tall piers 2, 4, 5 and (b) squat piers 6, 8
=3 layers, =1 layer

5. TEST EQUIPMENT

The test equipment, schematically shown in Figure 8, is composed by:
#1: system for the application of vertical loads at the top of the specimen: the specimen is placed within a testing frame realized using a transverse spreader beam which is linked to the ground by means of two ∅47 mm Dywidag rods, one on each side of the specimen, with hinged connections. The axial load is applied by prestressing the two rods by mean of two 1000 kN Hollow Plunger Cylinders, which are inserted in a cage bordered by four ∅26.5mm Dywidag rods and two hollows steel plates, the upper reaction one and the plate connected to the hinge.
#2: system for the application of horizontal loads and displacements: a 500 KN electromechanical actuator (stroke ±200mm), connected on one side to the top of the pier and on other side to a reaction wall, is used to impose displacements or loads to pier;
#3: ground anchorage system: the specimen footing is restrained to the laboratory strong floor using two transverse beams fixed to ground with steel tendons to avoid base horizontal displacements and rotations;
#4: control system and data acquisition: a closed loop control system governs the movement of the actuator by means of LVDT (stroke ±200 mm), data acquisition system and control LabVIEW virtual instrument. The hardware of the data acquisition system is composed by the following elements: NATIONAL INSTRUMENTS NI PCI 6281, High-Accuracy M series Multifunction DAQ 18-Bit, up to

625 kS/s, 3 modules NI SCXI 1520 Universal Strain Gauge Input Module with 8 Channels, and 2 modules NI SCXI 1540, 8 Channels LVDT Input Module, with software LabVIEW Full System. In particular, the horizontal displacements of three point of the pier, placed respectively at the height 230 mm, 450 mm and 1170 mm or 2340 mm for squat and tall piers respectively from the pier base section, are recorded using 3 linear potentiometers with stroke ±50 mm (channels 19, 18, 1), whereas other 12 linear potentiometers are used for the acquisition of vertical displacements of other three different pier sections (channels from 2 to 13) in order to estimate their curvature along the height. Two linear potentiometers (channels 16, 17) are placed on pier foundation in order to check incidental rigid rotations. Two 1000 kN hollow load cells, between the axial load jacks and the upper steel plate of the cage, measure the axial load in tendons and one 1000 kN pancake load cell at the top of the pier specimen measures the horizontal reaction.

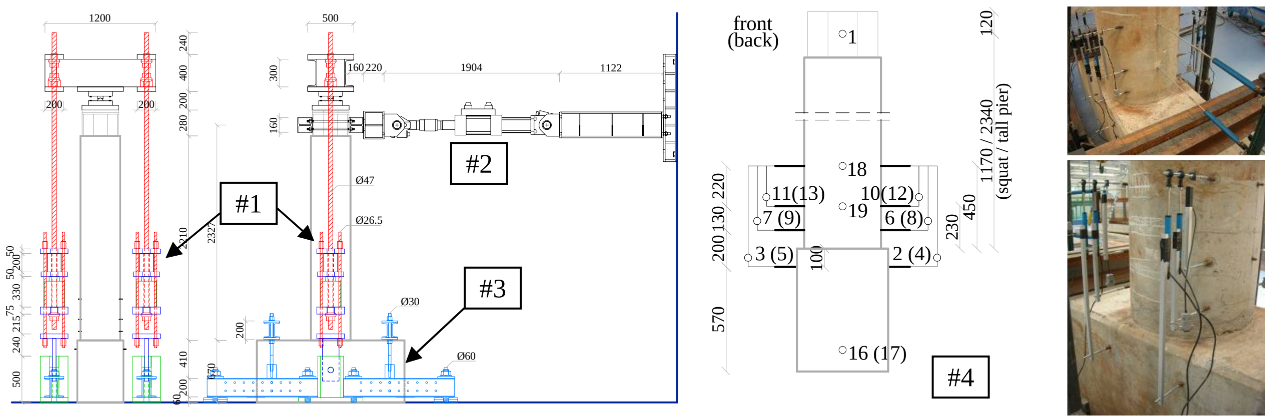

Figure 8. Test equipment: #1 vertical load application system, #2 MTS hydraulic actuator, #3 ground anchorage, #4 displacement acquisition system.

7 CONCLUSIONS AND FUTURE DEVELOPMENTS

This paper focuses on current experimental tests at the *Laboratory of experiments on materials and structures* of the University of Roma Tre on column specimens representative of tall and squat circular r.c. bridge piers designed according to EC8 and Italian Code before 1986. Damages in pier specimens previously tested until collapse by pseudodynamic tests are accurately detected. Damaged EC8 columns are just repaired whilst the Italian ones are first repaired using stainless steel bars and self compacting concrete and then retrofitted by mean of C-FRP layers designed to upgrade ductility and shear strength to match EC8 design requirements.
Elastic and pseudodynamic tests are foreseen in the next future: the former aims to characterize the piers, the latter to evaluate their non linear behavior until collapse and to assess the efficiency of the adopted repairing and retrofitting techniques. Comparisons with the results of previous pseudodynamic tests on the original piers will be carried out.

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