High strain-rate testing of concrete and steel for the assessment of the Tenza Bridge under blast loading

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ABSTRACT: Nowadays structural engineers, facing critical infrastructure design, must consider severe dynamic load conditions, which reproduce natural actions like strong earthquakes or man-made actions like blast or impact. This research project is centered on the study of a highway reinforced concrete arch bridge, located in southern Italy. The objective is to carry out a complete assessment of the structure under high dynamic loads, validated by in situ test. For this reason a dynamic characterization of materials was performed, through high strain-rate failure tests. A wide range of strain-rate was investigated (from 10^{-4} s^{-1} to 10 s^{-1} for the concrete and from 10^{-4} s^{-1} to 600 s^{-1} for the reinforcing steel) and the result will be used to build dynamic models of the bridge that will be analyzed under different extreme loads.

1 INTRODUCTION

Today structural design of some critical infrastructures cannot avoid to consider particular load condition like blast, impact or strong earthquakes which induce on the structures high dynamic actions. Furthermore recent events have unfortunately centered the attention on vulnerability of some infrastructures to blast events.

The objective of this research project is to study the effect of high dynamic loads on a reinforced concrete (r.c.) arch bridge, called Tenza, located in Southern Italy, part of the no more used path of the Salerno-Reggio Calabria highway. It was built in the sixties and retrofitted with a pier r.c. jacketing in the nineties; the constituent materials are the old and the new concrete, the lightly deformed steel and the typical deformed steel bars used as reinforcement of the old and new concrete, respectively.

In the first phase of the project the characterization of the structure was carried out, through a static analysis under gravity and live loads, and a complete seismic assessment. The FEM model used to perform these analyses was validated comparing the numerical vibration modes with the results of a vibrodyne test (Asprone et al, 2006).

Then the objective of the second phase of the project is to perform an assessment of the structure under severe dynamic loads, through numerical analysis and in situ tests. To this aim a complete dynamic characterization of the materials was performed. High strain-rate failure tests were conducted in DynaMat Laboratory of the University of Applied of Southern Switzerland using three modified Hopkinson bars. Stress – strain relationships under different strain-rate have been evaluated under tensile loads for the materials of the bridge. The strain-rate ranged between 10^{-4} and 10 s^{-1} for the concrete and between 10^{-4} and 600 s^{-1} for reinforcing steel. The results obtained will be used to define the influence of dynamic loads on the constitutive behavior of materials aged in a real structure and they will provide reference laws to be used in structural analyses accounting for strain-rate effects.

For these reasons this paper covers only a part of a larger project, whose objectives are to perform a complete high dynamic assessment of a critical infrastructure and to develop effective mitigation techniques.

1.1 *Tenza Viaduct*

The Tenza bridge (see Fig. 1), located in southern Italy, was built in the sixties as part of Salerno-Reggio Calabria highway and was open to traffic till few years ago.

Figure 1. Tenza bridge view

Recently ANAS, the highway owner society, planned to change the geometry of the route, since it does not respect anymore the current safety standards. Therefore the bridge was closed to traffic as it belonged to a substituted portion of the highway.

The bridge structure of the Tenza Viaduct consists of 3 different structures, one main and two approach spans. The main span is an open spandrel arch structure that is 120 m long and 40 m deep.

The bridge deck and its wall piers are supported by a ribbed, solid slab, and fixed-fixed arch. Each approach span is 30 m long and is supported by multiple wall piers of different heights. Each individual pier is made of two r.c. columns; those external are connected over their entire height by a r.c. wall.

2 BRIDGE ASSESSMENT AND CHARACTERIZATION

The objective of the first phase of the project was to obtain a detailed knowledge of the bridge properties and structural behavior.

First of all a static characterization of materials has been conducted; 4 types of material can be identified in the Tenza bridge structure:

- the original concrete, used in the sixties when the bridge was built; it is the prevailing amount of concrete in the structure;

- the concrete, used for piers and arches strengthening;

- the original steel, lightly ribbed, used for the reinforcement of the original concrete;

- the strengthening steel, ribbed, used for the reinforcement of the more recent concrete.

For each of these materials several specimens have been taken from the real structure. Tensile tests of the steel and compression and ultrasonic tests of the concrete were performed.

The processed data show that the old concrete has an average compressive cylindrical strength of 34.11 MPa in the arch and 46.39 MPa in the deck and an average elastic modulus of 32.34 GPa, while the new concrete has an average cylindrical strength of 31.13 MPa in the arch.

Moreover the old steel has a yielding strength of 400 MPa and an ultimate strength of 593 MPa, while the new steel has a yielding strength of 490 MPa and an ultimate strength of 768 MPa.

Then a modal analysis has been performed and has been validated through a vibrodyne test. Static analysis under gravity and seismic loads has been conducted on the FEM model of the structure.

3 FAILURE MODELS OF THE BRIDGE

The aim of the preliminary study was to obtain the necessary tools to perform a complete assessment of the bridge under high dynamic loads. For this reason the knowledge of the dynamic behavior of the structure and materials is fundamental.

Under impact or blast loading two different types of failure can be distinguished:

- Local failure;
- Global failure.

The first one is due to an explosion which occurs close to structural element; the characteristics of failure depend on:

- The dynamic local properties of the element;
- The ductility local properties of the element;
- The high-strain-rate behavior of materials.

The second one occurs after a local failure and it is related to the attitude of the structure to withstand the loss of elements without activate progressive collapse. It depends on the global ductility properties of the structure and on the quality and the frequency of connections between elements of the structure.

Obviously if local failure is more severe, then global failure becomes more probable.

3.1 *Local failure*

Local failure can be reached by 2 ways:

- Local failure of the material;
- Local failure of the structural element.

The first mode of failure occurs when the shock wave in air produced by the explosion impacts on the surface of the element and determines a field of compression and tensile wave propagating inside the material. This tension can cause the cracking of the concrete and the consequent projection of debris.

The phenomenon can be well modeled using Hopkinson theory in one-dimensional case. The use of more complicated models would be unnecessary, because of the high uncertainty about the pressure function acting on the structure.

At the same time also the use of a complicated stress-strain relationship for concrete could be useless. For this reason an homogeneous elasto-plastic behavior, accounting for the percentage of steel, will be sufficient for modeling r.c.. Anyway the introduction of a strain-rate dependence is fundamental to determine the stress level able to crack the concrete. To this aim the results of the dynamic characterization of materials will be used to define Dynamic Incremental Factor (DIF)–strain rate curves, for the failure tension of concrete and for the yielding tension of reinforcement. These curves will be used to update the stress-strain relationship at different strain-rate levels.

The second failure mode can occur when the action of blast determines the failure of one or more sections within the element. Based on these issues it is planned to study this failure mode performing a dynamic non-linear analysis of the element. To consider strain-rate effect, non-linear moment-curvature relationships will be defined for each homogeneous portion of the element at different strain-rate. These different relationships will be the results of the material constitutive law updating due to strain-rate effect.

The occurrence of a local failure will be consider for 3 different structural elements: the deck, the piers and the arch.

For each of them material failure analysis and element failure analysis will be performed.

3.2 *Global failure*

Global failure occurs after a severe damage of one or more structural elements; the loss of these elements in fact can determine a progressive collapse of the structure. The possibility of this failure mechanism is linked to the capacity of the structure to redistribute loads on other structural elements. It depends on:

- Redundancy of elements;
- Ductility of connections.

The first characteristic means that the static scheme of the structure is far from an isostatic configuration. The loss of some elements is then compensated by the presence of other elements which anyway can carry acting loads.

The second characteristic is also important because after the loss of some elements, the new equilibrium configurations are reached with high local deformations, which must be tolerated by the structure. High ductility of connections is then necessary to allow these static configurations.

For these reasons it is planned to perform a global failure analysis of the bridge, considering a non-linear FEM model in which one ore more elements will be removed. Then a non-linear dynamic analysis will be carried out to evaluate the new equilibrium configuration and to verify that this is compatible with deformation and resistance capacity of the structure. The choice of the elements to remove depends on the criticality of the elements role.

Surely, in the case of the bridge, the piers and the arch are fundamental to guarantee the equilibrium of the whole structure. Anyway the arch, with its massive section, has a very low probability of failure under a blast event, even if it were particularly severe.

For this reason the progressive collapse analysis will be focused on the loss of the piers. Obviously this consideration will influence also the local failure analysis which will be more detailed for the piers too.

4 STRAIN-RATE TESTS ON CONCRETE AND STEEL

The mechanical properties of the concretes and the steel of the Tenza bridge, in tension under high loading rates, have been obtained by means of three Modified Hopkinson Bars (MHB) installed in the DynaMat laboratory of the University of Applied Sciences of Southern Switzerland (SUPSI) of Lugano (Cadoni, 2005b). In particular, the MHB with a diameter of 10 mm was used for dynamic testing of steel, other two set-up were used to load representative plain concrete specimens with diameter of 20 and 60 mm, respectively.

4.1 *Set up for high strain-rate tests of concrete*

The system consists of two circular aluminium bars, having a length of 3 m and a diameter of 60 mm, to which the concrete test specimen is glued using an epoxy resin. The input bar is connected with a pretension bar used as pulse generator. In this way, concrete specimens with same diameter of 60 mm are tested and by instrumenting each bar with semiconductor strain gauges, measurements are obtained of the incident, reflected and transmitted pulses acting on the cross section of the specimen.

A test with the MHB is performed as follows:

- first a hydraulic actuator, of maximum loading capacity of 1 MN, is pulling the pretension Maraging high strength steel bar having a length of 3 m and a diameter of 35.8 mm; the pretension stored in this bar is resisted by the blocking device (see Fig. 2)
- second operation is the rupture of the fragile bolt in the blocking device which gives rise to a tensile mechanical pulse of 1.2 ms duration with linear loading rate during the rise time, propagating along the input and output bars bringing to fracture the plain concrete specimen.

Figure 2. Experimental set-up for the concrete testing.

The plain concrete specimen has the same cross-section as the aluminium bars, to which it is glued with an epoxy resin (tensile strength > 30 MPa). Aluminium was chosen as the bar material because of its acoustical impedance, which is not far from that of plain concrete. This last fact minimizes the constraint to transverse deformation of the concrete specimen that depends on the ratio between Poisson and Young modulus. The strain gauge stations on the input bar measure the incident pulses ε_I and the reflected pulses ε_R. The strain gauge stations on the output bar measure the pulses ε_T transmitted through the specimen.

By the measure of the reflected and transmitted pulse the stress, the strain, and the strain-rate history were obtained using the formulation of the Hopkinson theory (Lindholm 1971):

$$\sigma(t) = E_0 \frac{A_0}{A} \varepsilon_T(t) \tag{1}$$

$$\varepsilon(t) = -\frac{2 \cdot C_0}{L} \int_0^t \varepsilon_R(t)\, dt \tag{2}$$

$$\dot{\varepsilon} = -\frac{2C_0}{L} \varepsilon_R(t) \tag{3}$$

where: E_0 is the elastic modulus of the bars; A_0 their cross-sectional area; A is the specimen cross section area; L is the specimen length; C_0 is the sound velocity of the bar material.

The concrete specimens have been instrumented with a strain-gauge and glued to the bars as shown in Figure 3a.

a.

b.

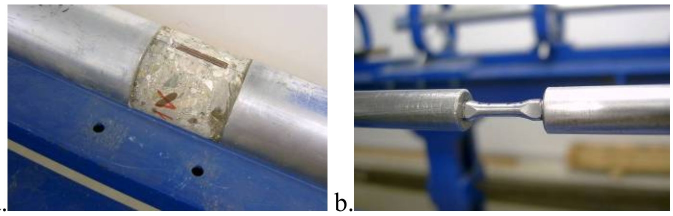

Figure 3. Specimen used: a. concrete, b. steel

4.2 *Set up for high strain-rate tests of steel*

The system consists of two circular high strength steel bars, having a diameter of 10 mm, with a length of 9 and 6 m for input and output bar, respectively.

The steel specimen is screwed to the input and output bars. The steel specimens have a diameter of 3 mm (see Fig. 3b). The input and output bars are instrumented with semiconductor strain gauges which measure the incident, reflected and transmitted pulses acting on the cross section of the specimen (see Fig. 4). The testing set-up is shown in Figure 5.

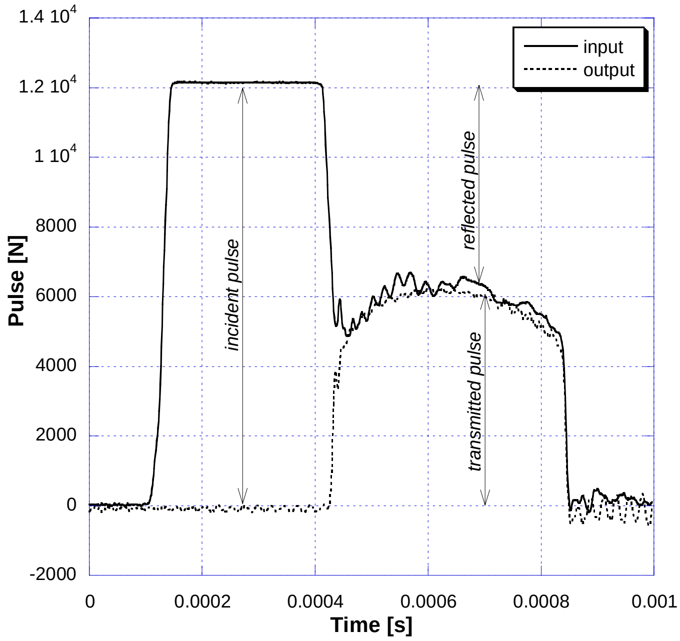

Figure 4. No-filtered signal measurement by input and output bar strain-gauge stations.

Figure 5. Set up for high strain-rate tests on steel specimens

The acquisition of two images before and after the failure of the reinforcing steel specimens, has allowed to obtain the characteristics variable as the diameter after failure in the necking zone as well as the curvature of the same zone, and the fracture strain (Fig. 6).

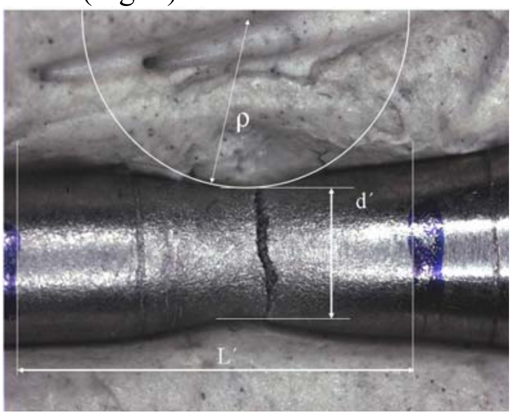

Figure 6. Measurement after failure

4.3 *Set up for medium strain-rate tests of steel*

In order to analyse the behaviour of the steel at a medium strain rate of 10 s^{-1} a Hydro-Pnuematic Machine (HPM) has been used (Albertini, 2005). The scheme of the HPM is shown in Figure 7 where it can be seen that the machine (Fig. 8) consists of :

- a cylindrical tank divided in two chambers by a sealed piston; one chamber to be filled with gas at high pressure (e.g. 150 bars), the other chamber to be filled with water. The water chamber can discharge the water through a calibrated orifice when the fast electro-valve is opened in order to start the test.
- the piston shaft which extends out of the gas chamber through a sealed opening, and its end is connected to the material specimen; on the shaft is attached a target whose movement is measured by a displacement transducer.
- the elastic bar, one end of which is connected to the material specimen and the other end is rigidly fixed to the machine supporting structure; the elastic bar is instrumented with a strain-gauge and its function is the measurement of the load resisted by the specimen during the test.

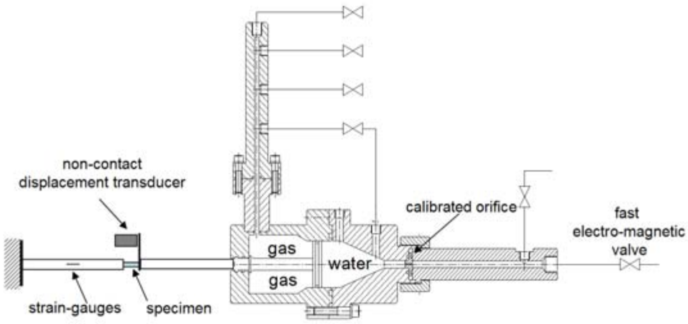

Figure 7. Hydro-pneumatic device scheme

The functioning of the machine is the following:

- water is filled in the upper tank-chamber and gas is filled in the lower tank-chamber; equal pressure is established in the water and gas chambers so that the forces acting on the two piston faces are in equilibrium.
- the specimen is fixed to the piston shaft and to the elastic bar.
- by activating a fast electro-valve closing the water chamber the force exerted by the gas pressure on one face of the piston prevails, accelerating the piston, which simultaneously loads the specimen and pushes water to flow out through the calibrated orifice at a constant speed, with the result of imposing a strain to the specimen with a constant strain-rate.

The movement of the piston at constant speed and therefore the constancy of the of the strain-rate during the test, depends mainly from the constancy of the force exerted by the gas pressure on the piston face; a good result in that sense has been obtained by maintaining small the gas volume change during the test so that small is also the gas pressure decrease and the piston force decrease.

This result has been obtained by maintaining small the stroke of the piston movement in order to limit to about 10 % the gas chamber volume decrease during the test.

The load P resisted by the specimen is measured by the dynamometric elastic bar; the specimen elongation ΔL is measured by the displacement transducer sensing the displacement of the plate target fixed to the piston shaft.

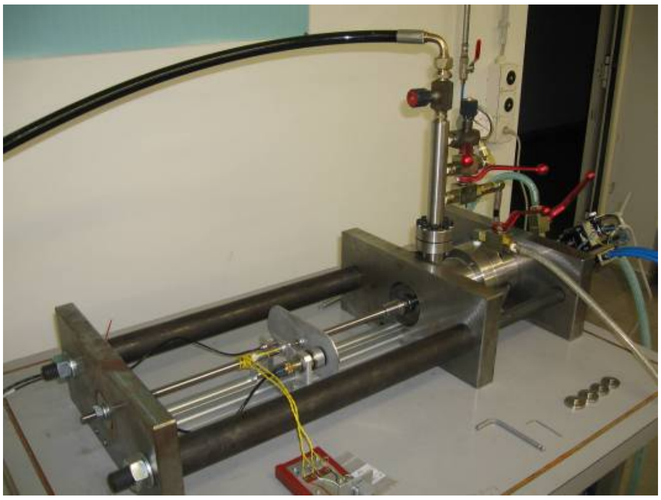

Figure 8. Hydro-pneumatic device for medium strain-rate test.

5 STRAIN–RATE TESTS RESULTS

5.1 *Concrete*

The behaviour of the concrete at high strain-rates is shown in Figure 9 as stress vs. strain curves in tension. The tensile strength, the ultimate strain and the fracture energy (considered as the area under the curves) increase as strain-rate become larger. Even though the concrete was taken from an existing structure and was about forty years old, the trend observed was coherent with the results described in the literature (CEB 1988, Malvar & Ross 1998, Albertini et al. 1998, Cadoni et al. 2000, 2001, 2005a, 2006).

The different concrete coming from arch and piers were tested at the different strain–rate regime. In Figure 10 are shown some stress versus time curves of the Ø= 60mm core concrete. In Figure 11 a specimen after failure is shown.

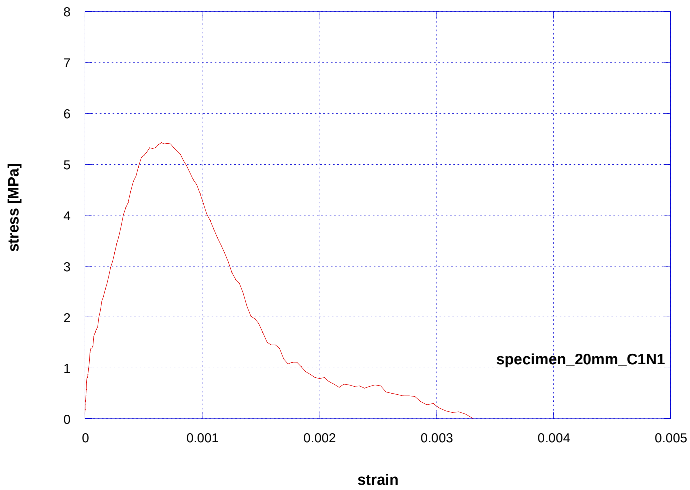

Figure 9. Stress vs. strain of concrete specimen tested at a strain-rate of 52 s^{-1}: fracture strain: ε_u = 654 $\mu\varepsilon$; ultimate strain: ε_f = 3309 $\mu\varepsilon$; fracture stress: f_{ct} = 5.43 MPa

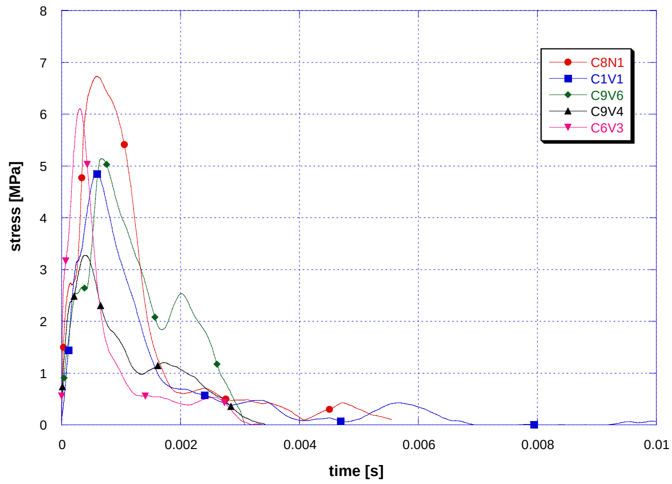

Figure 10. Comparison between the stress vs. time curves of the large concrete specimens (10 s^{-1}).

Figure 11. Large concrete specimen after failure

In table 1 are shown some results concerning the concrete of the piers both old and strengthening material.

Table 1. Results of concrete tests

Material	Strain–rate [s^{-1}]	f_{ct} [MPa]	ε_u [$\mu\varepsilon$]
	10^{-4}	1.87	-.-
Piers	2.14	2.38	644
original	6.1	3.82	377
concrete	6.95	4.95	638

	9.14	4.85	600
	10^{-4}	2.91	-.-
Piers strengthening concrete	4.31	5.87	1090
	57	7.35	876
	70	11.67	862

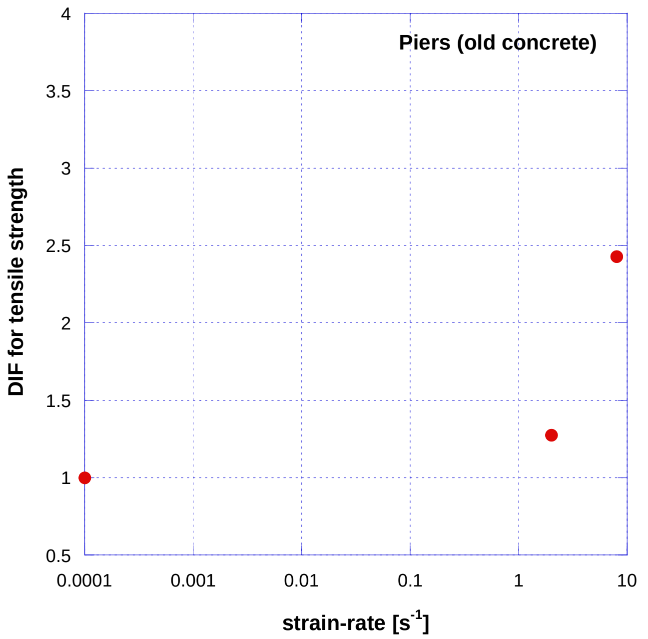

Figure 12. DIF for the fracture stress of the Piers (original concrete)

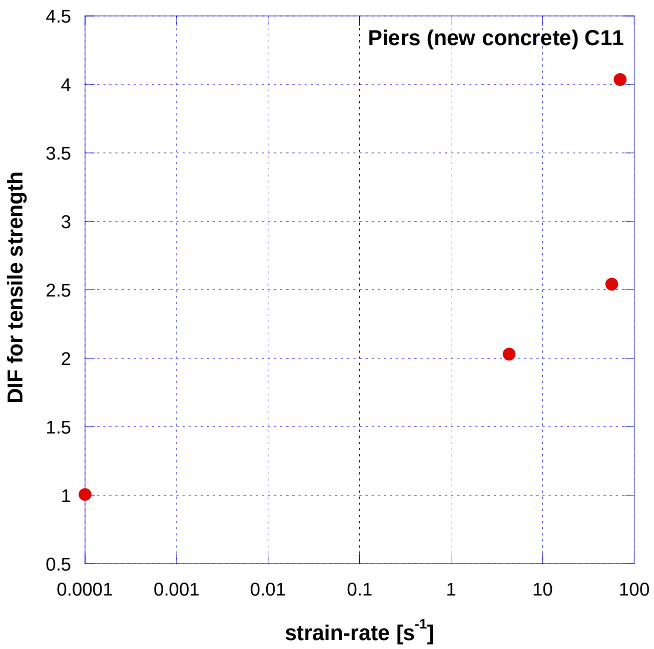

Figure 13. DIF for the fracture stress of the Piers (strengthening concrete)

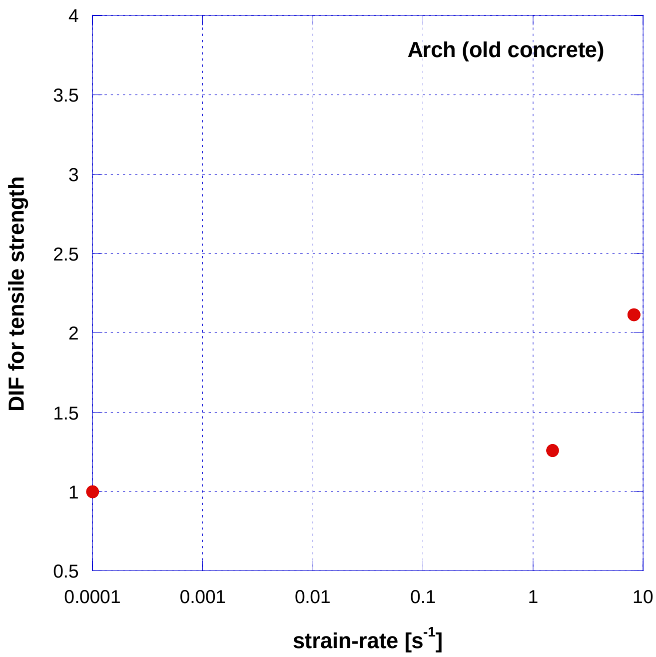

Figure 14. DIF for the fracture stress of the Arch (original concrete)

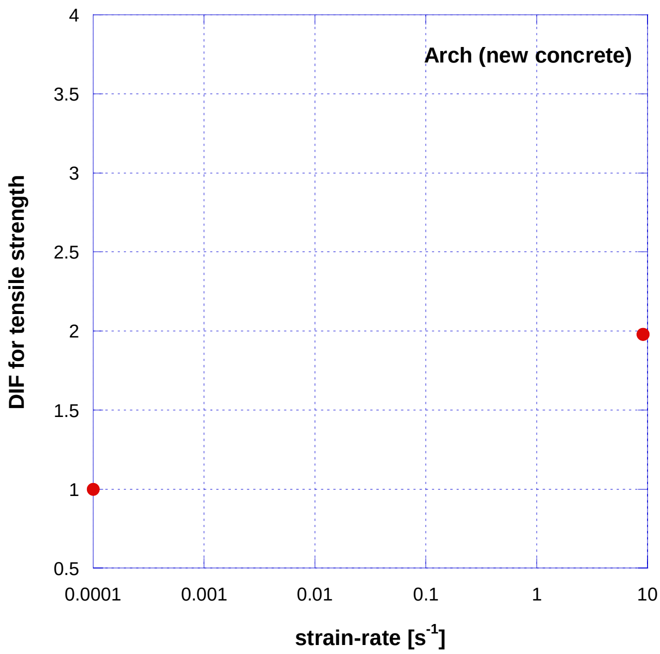

Figure 15. DIF for the fracture stress of the Arch (strengthening concrete)

In Figures 12-15 are shown the behaviour of the tensile strength of the different concretes in terms of Dynamic Increase Factor (that is the ratio between the dynamic strength and the static value) in function of the strain-rate.

5.2 *Reinforcing steel*

The analysis of the specimen has been carried out studying both the experimental results in terms of stress versus strain curves and fracture, in statics and dynamic (Figs. 16-18).

The steel specimens were obtained from the old reinforcing rebars. The specimens have been tested at high strain-rate regime 200 and 600 s^{-1}, 5 samples for each velocity. As reference the same specimens have been tested in quasi–static regime. In order to analyse the behaviour of the reinforcing steel in a large range of strain-rate additional specimens have been tested at medium strain-rate of 10 s^{-1} and at higher strain-rate of 1000 s^{-1}. The results are shown in Table 2.

Table 2. Results of reinforcing steel tests

Strain-rate [s^{-1}]	$f_{y, ave}$ [MPa]	$f_{u, ave}$ [MPa]	$\varepsilon_{u, ave}$ [%]
10^{-4}	388.3±3.8	708.0±18.0	10±0.7
10	-	772.3±5.0	-
173.8±17.6	547.2±42.8	789.4±42.6	10.7±1.1
561.6±55.2	626.4±9.5	829.6±8.1	15.3±0.7
1000	636	880	-

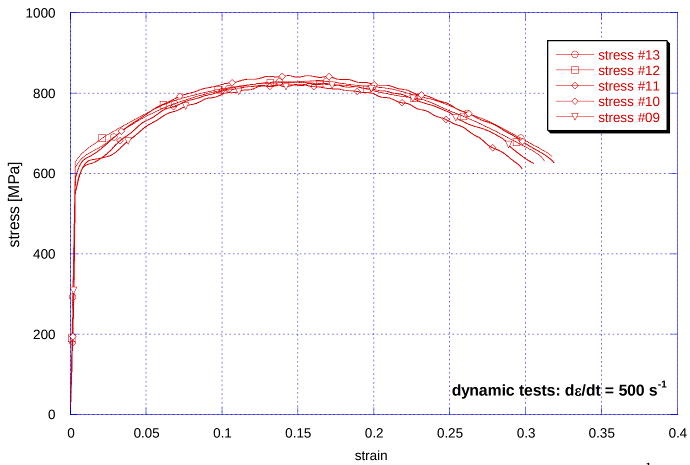

Figure 16. Stress vs. strain of reinforcing steel (about 600 s^{-1})

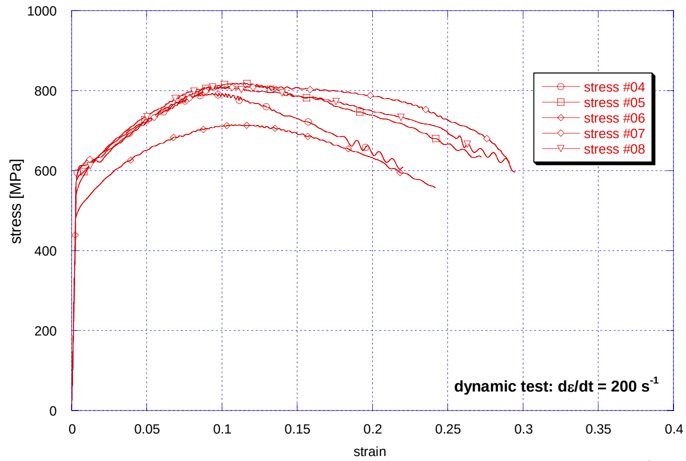

Figure 17. Stress vs. strain of reinforcing steel (about 200 s^{-1})

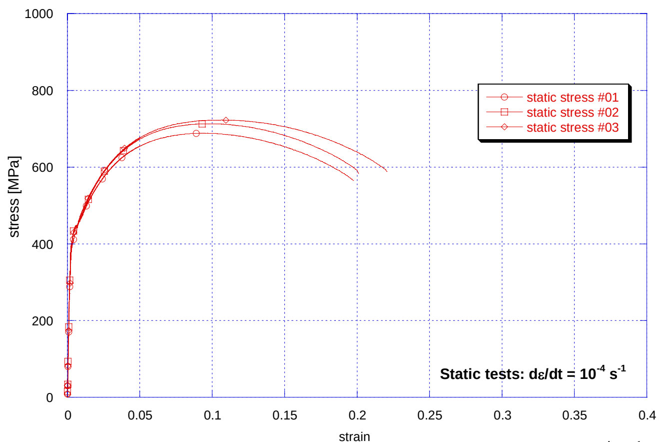

Figure 18. Stress vs. strain of reinforcing steel (about 10^{-4} s^{-1})

In Figure 19 are shown the stress versus time curves for the specimens tests at high strain-rate.

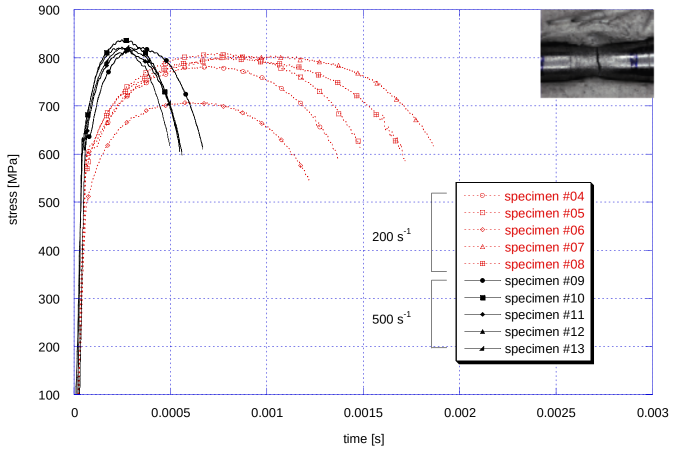

Figure 19. Stress vs. time of reinforcing steel

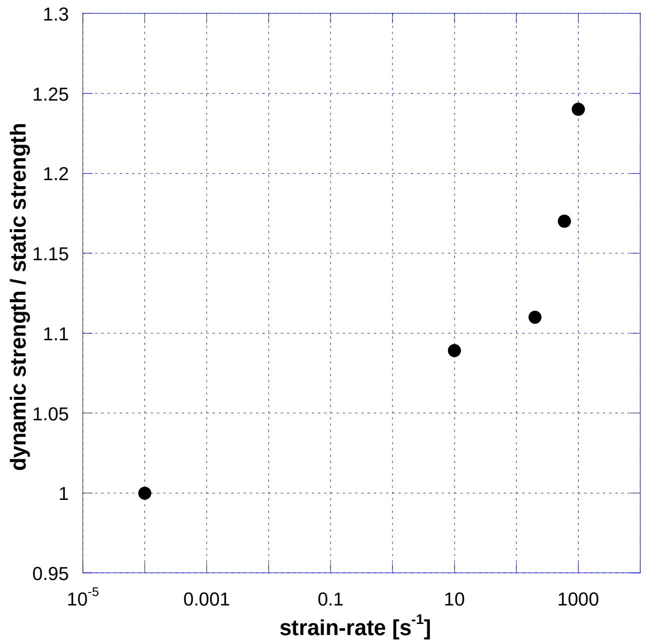

Figure 20. DIF vs. strain-rate of reinforcing steel

The behaviour of the tensile strength of the steel at four velocities is described in Figure 20 where the DIF in function of the strain-rate is shown.

6 CONCLUSION REMARKS

After this first phase of the project the results can be summarized as follows:

– A strain-rate sensitiveness is present for the two different types of concrete (original and strengthening concrete) analyzed in the project. The two concrete had surely a different micro-structure due to composition and grading (alluvial aggregate for the original concrete and crushed for the strengthening one), but they have shown similar characteristics in terms of compression resistance and elastic modulus. An increment of strength at high strain rate from

100% to 400% has been found, which implies an increase of ductility and energy absorption capability. Actually at high strain-rate the strengthening concrete seems to have a better behavior than the original one.

– A strain-rate sensitiveness in term of stress and strain has been found also for reinforcing steel. The yielding stress growths more than 60% from the quasi-static regime to the impact one (>500 s^{-1}), the fracture stress increases its value less than 20%, the ultimate and fracture strain have an increment of 50% of their values.

– The results will be used to implement realistic constitutive laws for computational codes; then numerical simulations will be carried out to predict the behavior of the structure subjected to extreme loads.

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