Controlled Demolition
of the Pietrastretta Bridge in Italy

by Roberto Folchi and Gianluca Auletta

Introduction

This article details the controlled demolition of one of the two carriageways of the Pietrastretta bridge in the "Sicignano – Potenza" highway in Italy. This bridge was built in the 1970s with reinforced concrete. Single slim pylons support on transverse cantilevers, two carriageways on three beam spans. A total of 21 spans cover a length of 1 km (0.62 mile).

The demolition became necessary to replace the old carriageways with a new one in COR-TEN steel (about half the weight of reinforced concrete, higher seismic resistance, reduced need of maintenance, and longer shelf life). All the spans were damaged, with cracks, loss of concrete volume, exposed re-bars, corrosion, and rupture. Less relevant damages were affecting the pylons.

The Southern carriageway was put out of service and traffic in both directions was moved to the Northern carriageway. Vehicles exceeding 7.5 tons (16,500 lb) were not permitted. Reconstruction consisted of replacement of both carriageways, starting with the southern one, and also in the treatment of the pylons concrete surfaces and rebars.

A first attempt to remove the spans by means of a launching girder was stopped due to the occurrence of an accident. One of the three beams of the span, cut from the other two and broke while lifting and in its fall almost dragged the launching girder and its personnel down with it.

The failure was connected to a crack that created a hinge triggering the collapse with a strong tear on the fixing spots for lifting. This beam, 185 metric tons (200 imperial tons) and 45 m (150 ft) long, was originally tied to the other two by means of six transverse post-tensioned reinforced concrete beams and by the reinforced concrete slab. When the slab and transverse beams were cut to permit beam lifting, all the dead weight had to be self-sustained, having lost the cooperation of the other two beams.

To prevent risks of another collapse, it was decided to demolish all remaining spans with explosives. All demolition activities were foreseen by means of radio-controlled equipment with personnel secured to a safety line fixed to the pylons.

The blast was designed to prevent damage to the northern carriageway and pylons, and to close standing housing.

Geometrical, mechanical, and dynamic characterization of the bridge were checked to permit development of a 3D model of the two acceptors "bridge-to-be-weakened-for-blasting" and "bridge-to-be-left-in-place." The main acceptors were modeled with the Applied Elements Method (AEM). They were proven to be stable after weakening of spans and also transverse overturning of the bridge at sudden release of the weight after blasting the spans.

Charge and firing sequence were dimensioned in order to keep blast induced impact below thresholds according to DIN 4150-3 and state of the art.

Due to the low safety margins for spans resulting from the AEM model computation and also risks for local undetectable extreme reduction of the resistance parameters, maximum care was given to accuracy in execution of cuts and holes to prepare the structure for blasting. It was important to maintain execution as much as possible matching the model and minimizing the risk of a sudden collapse.

The personnel was secured against the hazard of an accidental collapse of the span by means of a safety line fixed at the pylons and procedure based activities with remote controlled equipment.

The demolition job was completed in two months.

Continuous displacement and seismic monitoring during blasting helped to prove the model and to keep track of the impact induced by the demolition.

The Pietrastretta Bridge Geometry and Structure

The Pietrastretta viaduct is part of the motorway junction Sicignano-Potenza, in the municipality of Vietri di Potenza. It is a 1 km (0.62 mile) long bridge, with a bend layout, halfway on a steep slope, with 21-spans **(see figures 1 - 4)**.

It was designed in the late 1960s and built in the early 1970s. Design of this bridge, as many other bridges of that time, was focused on concrete and reduction in iron quantities, to save money, taking advantage of the low cost of highly qualified manpower. This was observed by the number of beams (three instead of the standard four) as well as from the beams geometry, with variable height of the cross section (in-

Figure 1. Profile of Pietrastretta bridge, spans from 1st to 16th (on a total of 21).

Figure 2. Detail of the bridge.

Figure 3. Detail of the cantilevers supporting the two carriageways and of the post-tensioned transverse beams tying together the three main beams.

Figure 4. Northern (left, to be left in place in service) and Southern carriageways of the Pietrastretta bridge.
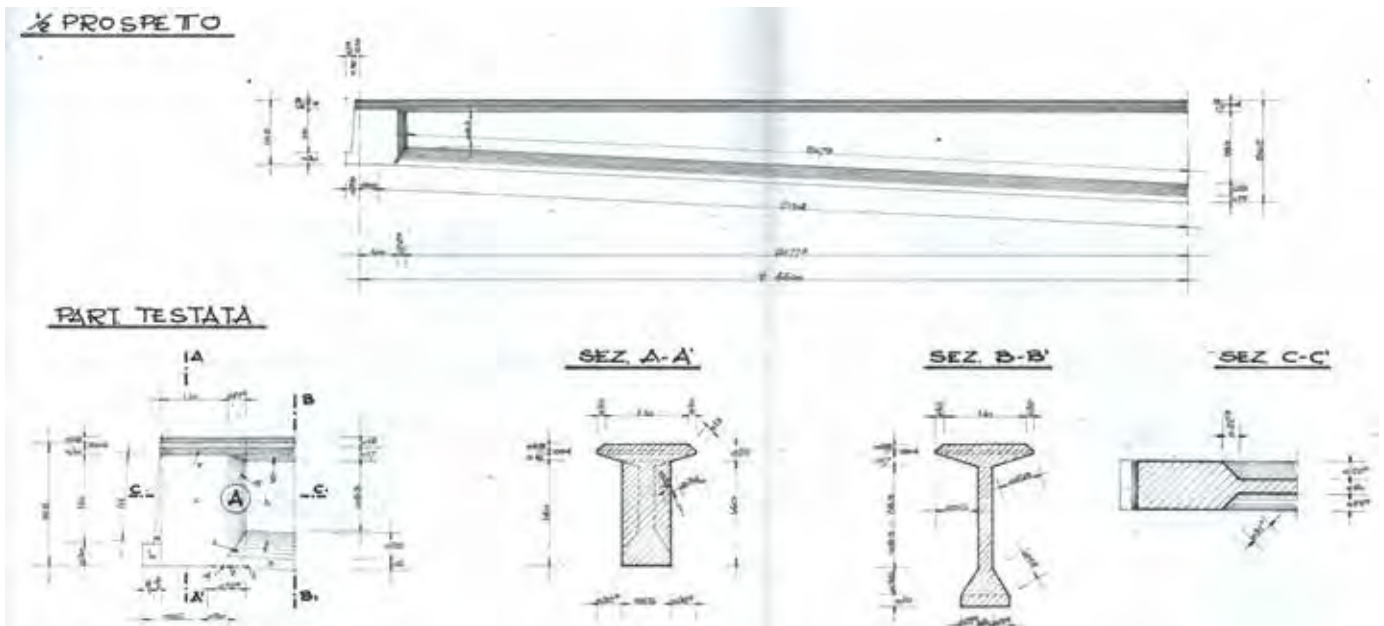
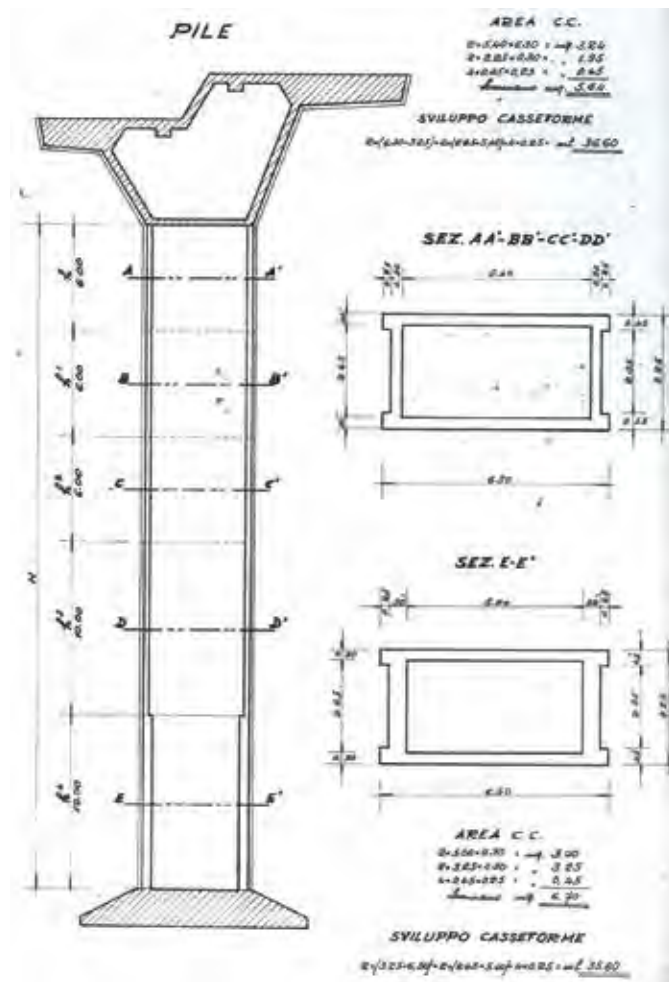*Figure 5. Drawings of the reinforced concrete beam.*

Figure 6. Drawings of the reinforced concrete pylon.

creasing from the supports to the centerline, instead of being equal for the whole length) and from the slim and elegant geometry of the pylons with the many angled surfaces requiring extra working hours for scaffolding and form-working **(see figures 5 and 6).**

The bridge has two independent carriageways, 3 meters staggered to match the slope profile. Both carriageways are supported on a transverse box-cantilevers from a single slim box-pylon. Height of pylons ranges from 12 to 50 meters (40 to 160 ft). Spans are built with 3 "I" beams of pre-stressed concrete with a height of 1.75 m (70 in) at the support and 2.7 m (106 in) in the middle. Six post-tensioned transverse beams and a slab 9.5 meters (31 ft) wide and 20 cm high (8 in) bind together the three main beams.

The beams upper flange are 120 cm (47 in) and lower 65 cm (26 in). Beams are standing on a fixed support at one end and a longitudinal support at the other. A seismic dumper was not installed because it was not the law at the time of construction, nor was it implemented later as in many other bridges of that time.

Center of mass of each carriageways lays outside the trunk of the pylon. This is because of the pylons shallow foundations, at first glance gave concerns for transverse overturning after removal of one of the two carriageway before deployment of the new carriageway to counterbalance. This scenario was analyzed in the 3D model and excluded.

Survey

In a preliminary survey it was confirmed that all beams were inflicted by widespread degradation with loss of concrete volumes, exposition of reinforcement bars, and tensioning wires with consequent corrosion **(see figures 7 and 8).** Local reinforcement appeared to be deeply rusted and also cracked. This degradation showed a lower grade in the middle beam in comparison with the external ones. A higher degradation was noticed in the lower flange in comparison with the upper one. Similar deep degradation also involved the slab but

Figure 7. Strong degradation of the beams at 12th span southern carriageway, lower flange, with lost of concrete layers, extended corrosion and broken re-bars and tensioning wires.

Figure 8. Detail of degradation.

not the cross beams. Three of the 21 spans appeared close to ultimate resistance.

Differences in the intensity of the degradation among one span and the other depended on different care in execution and consequent different mechanical resistance of the concrete, but also from different exposition to freeze/thaw weathering from road salt and bad drainage.

Geometry of the structural components was found matching to the construction design. At the time of the construction, it was typical to adjust the design with work in progress. This was to adjust for construction mistakes and/or to

SPAN	Unit weight [kg/m³]	Uniaxial compression strength [MPa]
1	2.378	32,1
2	2.411	33,0
3	2.412	40,7
4	2.402	45,9
5	2.409	31,9
6	2.379	36,6
7	2.404	38,4
8	2.397	51,8
9	2.423	37,0
10	2.404	38,3
11	2.414	30,9
15	2.331	44,8
16	2.381	35,9
17	2.412	33,0
18	2.387	36,0
19	2.394	41,8
20	2.394	35,0

MINIMUM VALUE AMONG THE 17 SPANS

min AVERAGE =	2.322	38,9
max STD DEV =	63	14,1
min AVERAGE - STD DEV =	2.266	31,8
min MIN =	2.232	30,9
min AVG - max STD DEV =		24,8
max AVG - min STD DEV =	2.387	

MAXIMUM VALUE AMONG THE SPANS

max AVERAGE =	2.401	58,5
min STD DEV =	15	3,4
max AVERAGE - STD DEV =	2.377	53,6
max MIN =	2.374	51,8
max AVG - min STD DEV =		55,0
min AVG - max STD DEV =	2.259	

AVERAGE VALUE AMONG THE SPANS

AVERAGE =	2.369	47,9
STD DEV =	30	9,0
AVERAGE - STD DEV =	2.342	39,1
MIN =	2.334	37,9
average AVERAGE - average STD DEV =	2.339	39,0

Figure 9. Summary of values found in testing for mechanical characterization of the concrete.

compensate for lack of specific materials in the market. Those changes were not always recorded in the official documents so that for old bridge demolition, a control of the geometry and structure consistency is always needed. However, no control was however possible for the tensioning wires due to concrete covering the tensioning bulbs on the beams heads. Also, no data was found on wires post-tensioning sequence if executed before or after the connection of the slab to the beams. Therefore ,assumptions had to be made for the 3D model.

Mechanical Characterization

One hundred ten samples of concrete were cored, not less than two in each beam (one in the middle, one by the support). This was to permit execution of approximately 340 tests to define mechanical resistance parameters of each single beam and, consequently, of the span **(figure 9)**. Cores were sampled in the above flange, one close to the support and the other in the middle of the beam. The purpose of this was to check the uniformity of resistance from different amplitudes of the traffic induced stresses during life service. Tests included: unit weight, uniaxial compression, triaxial compression with confining pressure 4 MPa (580 psi) - approximately 10% of the expected ultimate resistance, static elastic module according to the norm EN 12390-13, and ultrasound for the computation of the tangent dynamic elastic modulus according to the norm EN 12504-4.

A minimum ultimate uniaxial compressive strength (UCS) of 30 MPa (4,350 psi) was found. This is low if compared to the minimum of 45 MPa (6,525 psi) foreseen by law. Higher UCS found was of 46 MPa (6,670 psi). For all the spans, the minimum UCS's were equal to the mean value less the stan-

Figure 10. Drawing of the rebars of the beam.

Figure 11. Drawing of the pre-stressed cable layout.

dard deviation. The standard deviation reached 30% of the average value, never falling below 11%. This showed a high variation of the resistance in the beams of the same span. Average tensile strength from "Brazilian" test were generally below 10% of UCS. The triaxial compression strength were approximately 40% higher than UCS (57 to 77 MPa – 8,265 to 11,165 psi).

The reinforcement bars **(figure 10)** found in beams and slabs were the improved adhesion type, both having circular and square cross section. Tensile tests showed an average yield strength of 498 MPa (72,210 psi) and rupture at 672 MPa (97,440 psi), with a standard deviation respectively of 54.5 MPa (7,900 psi) and 72 MPa (10,440 psi). The six pre-stressing cables **(figure 11)** were made with steel strands with strength resulting to 0.1% of residual deformation found in test equal to 1607 MPa (233,000 psi).

3D Model of the Bridge

A 3D model of the bridge structure was made, based on

Figure 12. 3D model of the bridge.

the data from survey and testing **(figure 12)**. Above the span's dead weight, live concentrated loads were added, those consisting in the 2 pieces of equipment simultaneously used on the same span: wagon drill or mini excavator or floor diamond saw not exceeding 3,000 kg (6,600 lb) increased conservatively to 4,500 kg (9,920 lb) to keep into account dynamic forces, and telescopic lifter 15,000 kg (33,000 lb).

Two demolition scenarios were analyzed, both including drilling of holes for explosive and cutting of the slab and beam upper flange for weakening, one of the two including a span divided in three longitudinal sections to be removed by blasting one by one (triple-step blasting) instead of the whole span at a time (single-step blasting).

A static analysis was performed on the 3D model to define safety factors for ultimate resistance for the weakened structure. A dynamic analysis was also performed to assess risks of transverse overturning of the pylons due to the sudden removal of the spans being blasted.

Results showed how the span would have sustained the foreseen weakening without failure in both single-step and triple-step blasting demolition scenarios. The minimum factors of safety for ultimate resistance of the concrete, steel reinforcement and pre-stressed cables were respectively 3.60, 3.48, and 2.37 for the triple-step scenario (25% lower than those for the single-step scenario). The triple-step scenario was found with a maximum downward deflection of the beam 28% higher than that for the single-step scenario. Vibrations induced by the single-step blast scenario was found to be 1/3 higher than the one for the triple-step scenario. However, for both scenarios vibration was found to produce no uplift of the pylon foundation and, therefore, no risk of pylons transverse overturning **(figures 13 and 14)**. Being that the static analysis was more sensitive than the dynamic analysis to local variations materials strength, although both scenarios were found with an acceptable safety factor, the single-step scenario was chosen because of its higher static safety factor.

A video simulation was presented to all concerned to better illustrate the demolition process and to demonstrate the reduced risks of spans sliding after impacting the ground, and also the kinematic of the fall after the blast **(figure 15)**.

Demolition Execution

Care was given to accuracy in execution of cuts and holes to prepare the structure for the blast. This was important to maintain actual execution as much as possible matching the model and therefore minimizing risks of occurrence of unwanted extra weakening, triggering a collapse. To set risk to zero, personnel on the spans were secured against the hazard of an accidental collapse by means of a safety line fixed at the pylons and equipment was all remote-controlled, operated at a safe distance **(figure 16 and 17).**

The spans, after having been drawn down, had to remain stable on the steep slope. This prevented sliding back to the pylon or away down on the slope with consequent difficult access for the excavators to complete the demolition with breaker and jaws crunch, and also for mucking. For this purpose ,drilling and preliminary slab demolitions were extended to the whole span, for a widespread fragmentation with the blast.

The spans from the higher pylons impacted the ground with a speed exceeding 20 km/h and disintegrated. Spans fall-
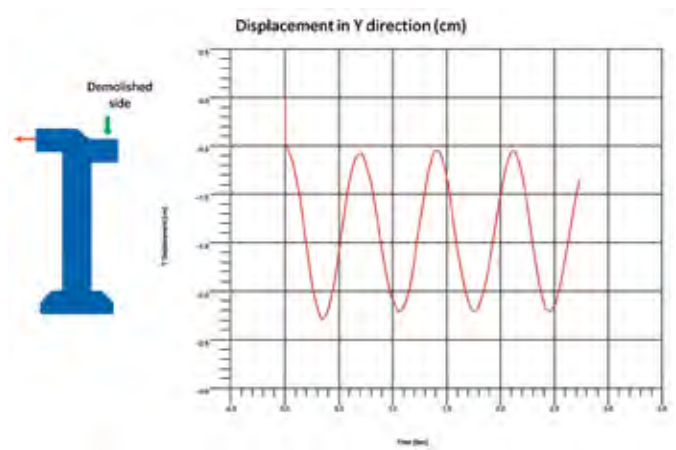

Figure 13. Displacement induced in the pylon for the horizontal transverse direction by the sudden release of the weight of the span on the cantilever, in the single-step blasting scenario.
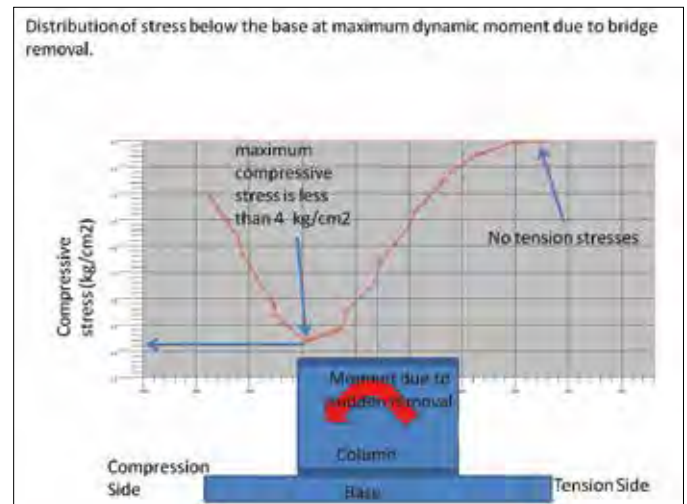

Figure 14. Results of stresses at pylon basement to assess lack of risk for transverse overturning.
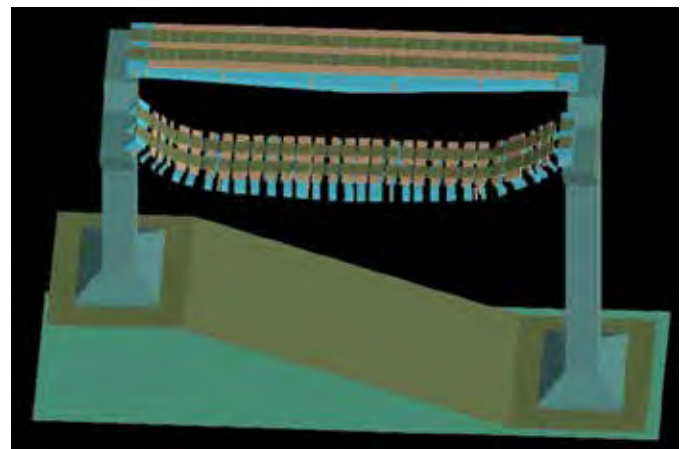

Figure 15. Picture from the video of the model simulation of the single-step blasting scenario for the span being drilled and cut along its whole length.

ing from the lower pylons, at lower impact speed, remained in pieces tied together by reinforcement steel, juxtaposed to the slope contour.

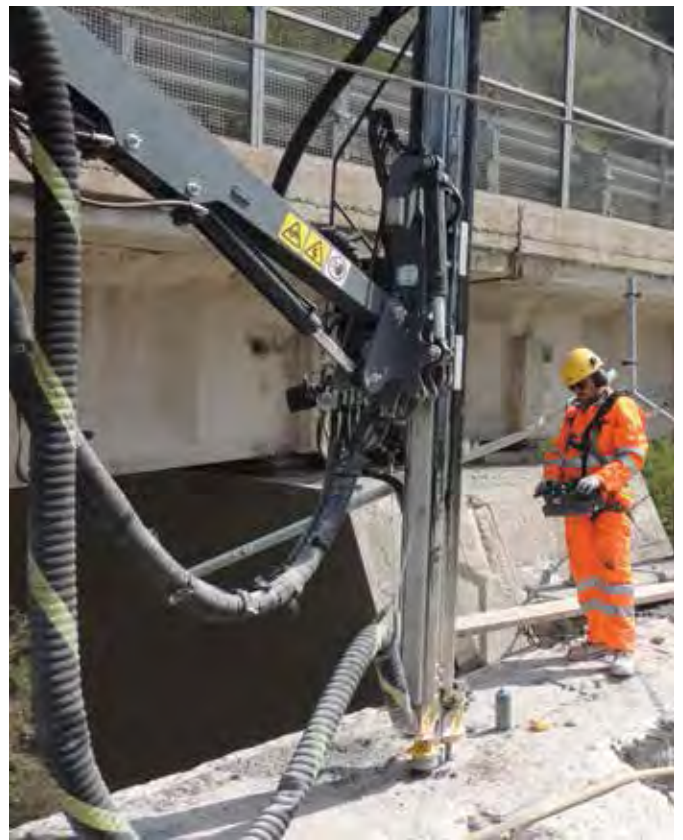Figure 16. Cut of transverse beams by means of a hydraulic crusher on a remote-controlled electric excavator.

Figure 17. Drilling with a remote-controlled wagon drill.

Figure 18. Span prepared and loaded for triple-step blasting.

Figure 19. Span blasted in triple-step at the ground.

Conclusion

By controlled blasting with remote-controlled equipment, it was possible to perform quickly and safely the demolition of the bridge Pietrastretta as an alternative to deconstruction by means of a launching girder. This last technique, due to a residual resistance of the structure close to the ultimate one due to extended and concentrated degradation, proved to be an unacceptable level of risk, especially after having shown occurrence of an accident.

Controlled blasting allowed not just the best safety conditions for personnel due to hazards and risks minimization but also lowered the cost and time needed to complete the project.

This successful result was achieved with a stringent approach with accurate and extensive surveys, proper modeling, competence and experience in designing as well as precise execution.
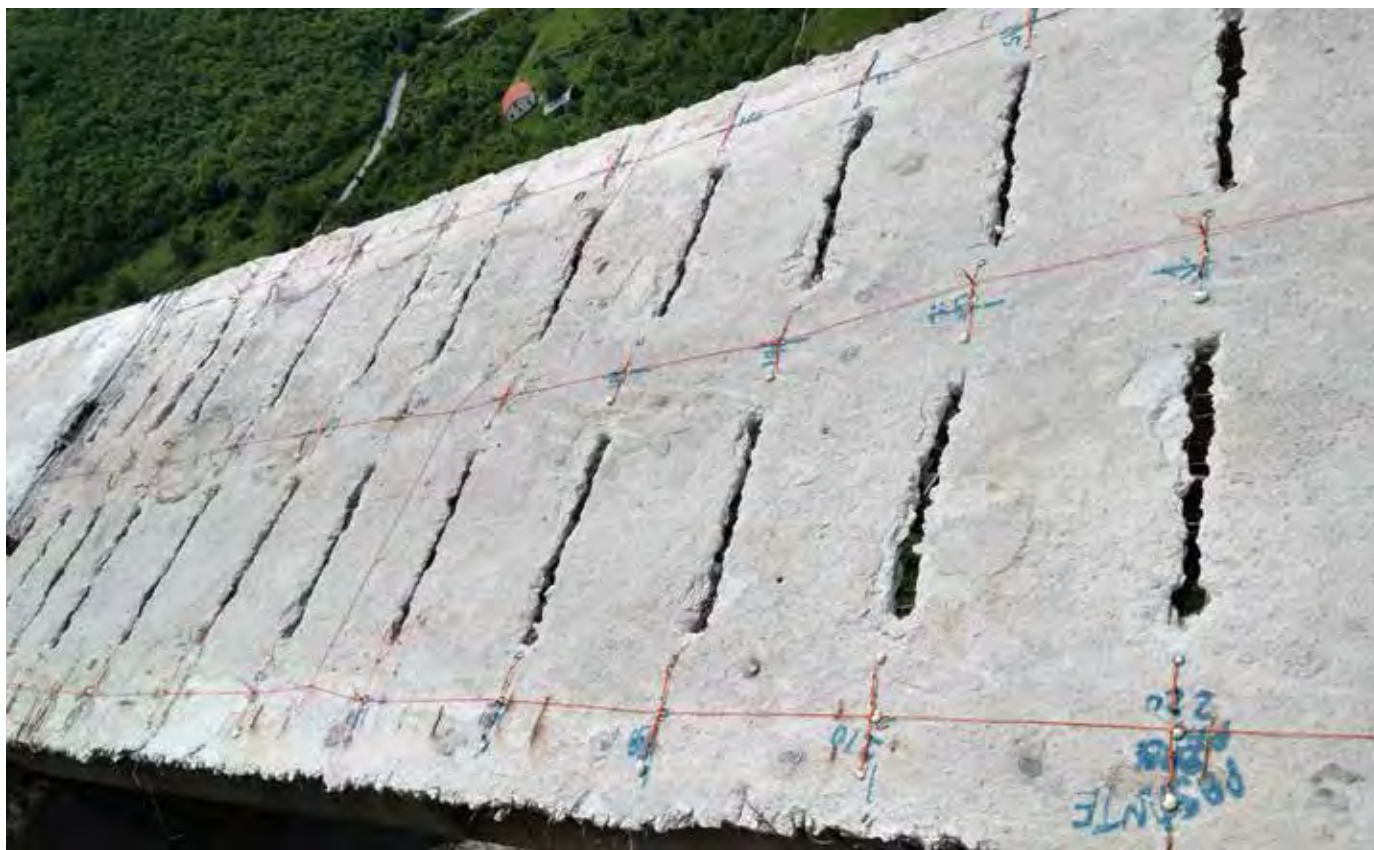
Figure 20. Span prepared and loaded for triple-step blasting.

Figure 21. Span blasted in one single-step at the ground.

Figure 22. Several spans blasted in one single-step at the ground.

Figure 23. Detail of the tensioning heads being exposed after the blast.

Impact induced in the below standing pylons and the by side standing spans, as well as the close-by housing, were kept below thresholds such to guarantee 100% no occurrence of damage according to DIN 4150-3. Seismic and strain monitoring, also executed extensively, confirmed the models at the design stage.

Acknowledgment

About the Authors

Roberto Folchi is founder and main shareholder of NITREX, a contracting company operating in the field of explosives demolition, underwater blasting, tunnelling and controlled blasting for open pit projects. He graduated in Mining Engineering in Rome in 1984 and has over 35 years experience in the explosives industry.

Gianluca Auletta is a structural civil engineer and PhD in seismic risk - earthquake engineering, with experience in structural modeling and monitoring. He works as a freelance in the field of structural design of buildings and bridges and as a researcher for structural design, analysis of seismic vulnerability of relevant buildings and bridges. He is co-author of about thirty scientific publications.

Figure 24. Sequence of blasts.
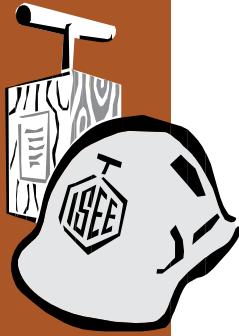
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