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PREFABRICATED REINFORCED CONCRETE CLADDING PANELS SUPPORTS: DESIGN AND NUMERICAL MODELLING

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Abstract

Prefabricated Reinforced Concrete (RC) cladding panels are often adopted in industrial facilities. While they are usually characterized by high strength and easiness of construction, their supports can represent a weak point. Indeed, the connections between the panel and the structure play a key role for the cladding safety. Without an appropriate design the connection can presents a brittle behaviour. High stress concentration, aging and exposure to aggressive environmental conditions that can accelerate the materials degradation represent some of the critical aspects in the connection design. This work, starting from a real case study describing the collapse of a RC cladding panel of a cooling tower, presents new simple design approaches for the supports. An accurate numerical analysis of the mechanical behaviour is also performed with a nonlinear Finite Element model developed in ANSYS environment.

Keywords: reinforced concrete, supports, durability, connections, fragility

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1 INTRODUCTION

Prefabricated Reinforced Concrete (PRC) elements exhibit in some cases a brittle failure. The technology of precast concrete elements has become wide-spread particular from the 70s. Its main advantages are the reduced costs, the high quality of construction products and the speed of the entire construction process. Prefabricated buildings have been widely used in industrial/production plants. Their modular solutions, with shorter execution times and the possibility to use a reduced number of components have been adopted in several cases. The technology of extrusion consent to furnish low-cost panels with the same cross sections and different length. The designer of PRC elements preferred in time the application of quasi-isostatic structure with punctual connection. The reasons must be found in the possibility to absorb the viscous deformation. So, the joints between the various components are often represented by punctual elements (bars, plugs etc.) and so the support areas are reduced to the possible minimum value to better exploit the materials and to reduce the overall dimensions and costs of the connections and of the structure.

Recent earthquakes and, in particular, those that recently hit Italy (Abruzzo in 2009 and Emilia 2012 [1–3]), have highlighted the high vulnerability of the connections: numerous collapses or disasters occurred because of their fragility and inadequate capacity. The consequence, in Italy, was the development of specific guidelines and research activities for structural retrofitting of the joints [4–7]. Another relevant point are the ongoing changes in climate that produces significant effects, sometimes unexpected, on environmental actions [8–10]

However, the seismic event is not the only trigger for collapses or instability in such structures: in many cases the degradation of the material accompanied by a reduced maintenance activity non exhaustive code indication and improper structural design has produced situations of instability even in the presence of service loads only [11]. This is due to the reduced redundancy with which the support was conceived. This elements, joined to degradation due to aging [12–14], uncertain mechanical property of materials [15, 16] and poor cure of structural details [17, 18] produce dangerous static and seismic vulnerability. Indeed, without a conservative approach, there is an high sensitive to construction, material, installation or maintenance imperfections that can lead to failure [19–21]. From this the strategic relevance of specific survey and on site investigation to detect signals [22, 23].

This work reports on fall of RC cladding of a cooling tower located in an Italian industrial plant built in the early 2000s. This case is representative of the brittle behavior worsen by micro-climate conditions (marine salt vapors) reducing the durability of the elements, due to inadequate structural design. Indeed, in the original design the possibility of imperfections in the supports has not been considered, nor any strength or residual bearing capacity have been attributed to the connection [24, 25].

The examined collapse happened on the north-western Italian coast which, for reasons of privacy, are not explained here. The examination of the deficit conditions put in lights some elements of weakens in the conception of the structure, which is the result of the culture of "allowable stress", the precedent design criteria that provide a limitation in the stress. Indeed, this approach do not paying attention to aspects of redundancy and sensitivity to imperfection.

Consolidation strategies are presented together with a proposal of a simple design criteria. In addition, a numerical model, in view to extend the results of the example in other similar situations, is developed with Ansys rel 18.1.

2 COOLING TOWER CLADDING PANEL COLLAPSE

A collapse of some cladding panels (Figure 1) is happened in a cooling tower of an energy production facility (47 x 32 m plant dimensions and 15.7 m height) built in the early 2000s. This event was triggered by the failure of the edge of one of the support cantilevers of the panels, on the south-west corner of the building. The damaged condition of the corner appears in a visible defect in the panels support (Figure 2). The small dimension of collapsed support is the main cause of the collapse. The microclimatic condition of nebulized sea water, percolating into the cooling tower, had a not negligible influence. This sea water, insinuated in the porosities of the concrete, has accelerated the structural degradation in a detail in which the sensitivity to imperfection plays a significant role.

A survey was carried to obtain proper information about the:

- mechanical characterization of the concrete of the various components (panels, columns, support);
- reinforcements emerging from the collapsed panels and cantilever;
- cracks on the lower edge panels still in position.

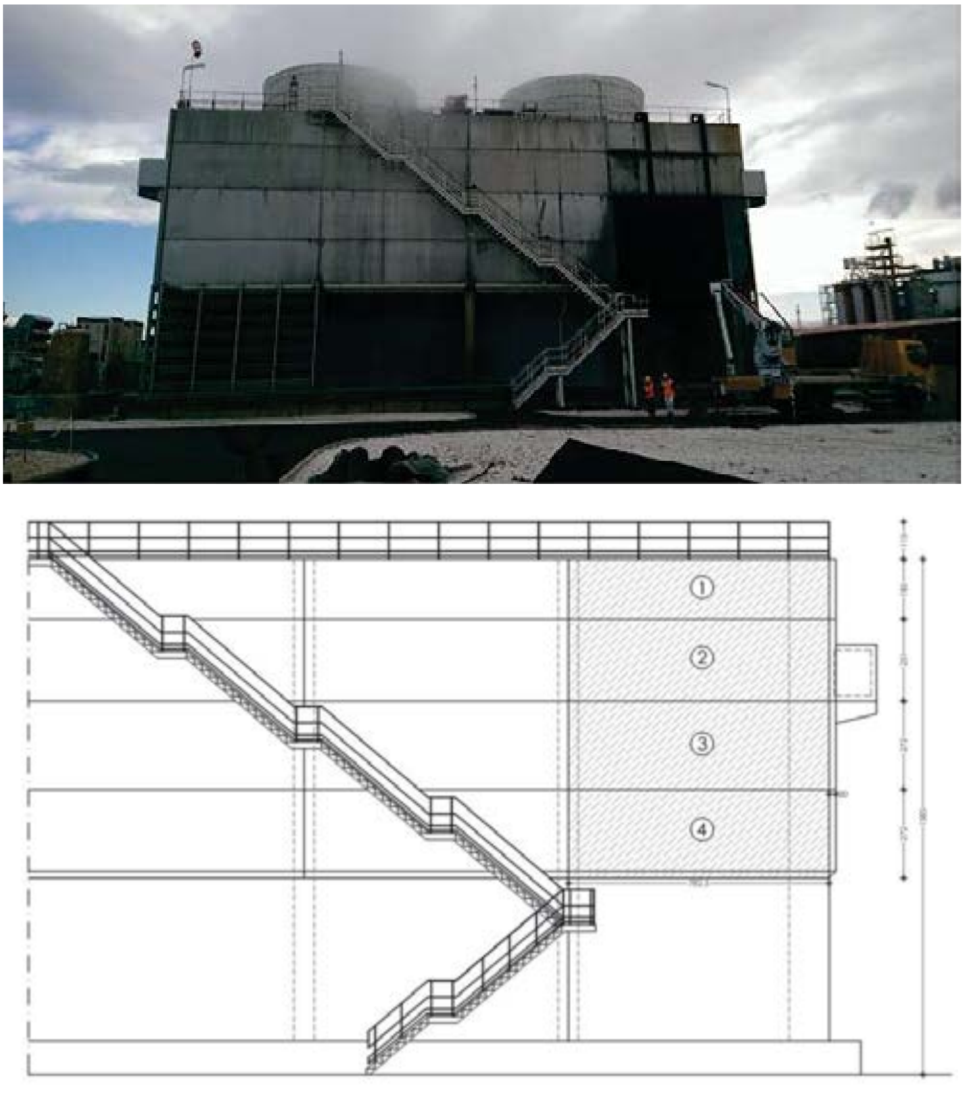

Figure 1: Cooling tower view with collapsed panels (top), drawing with highlighted collapsed panels (bottom).

The concrete of panels, cantilever and columns is characterized by cubic compressive strength equal to 50 MPa. This has been assessed using rebound hammer, pull out tests and destructive compressive strength tests on cores. The analysis of the reinforcements emerging from the concrete cantilever confirms that the collapse was triggered by the failure of concrete cover and of a 16 mm diameter steel threaded bar designed just to enable an accurate assembly of the panels but without structural role. The analysis of the cracks on the other corner panels (Figure 3) showed how all the corners not yet affected by collapses present visible cracks (up to almost 1.0 mm wide). This highlights that the conditions of the other panels support are not safe. In addition, the support zone of all the panels is very small.

A simple explanation of the collapse can be obtained with elementary equilibrium evaluation. On the concrete cover of the RC cantilever there is a permanent load of about P=106.35 kN. Indeed, the density of concrete is 25 kN /m^3 while the volume of the collapsed panel is 8.51 m^3.

The reinforcements of the cantilever could not offer any resistance to shear. Indeed, Figure 4 show that the collapse crack developed quite far from the reinforcement. Thus, the 16 mm diameter threaded bar, designed just to place the cladding panel, was hugely deformed and cannot bear the panel load. Indeed, its cross section A_{res} is too small to bear the load. The force P, representing the panel load, induces an average shear stress τ equal to:

$$\tau = P/A_{res} = 106350/154 = 705 \text{ MPa} \qquad (1)$$

This stress is beyond the material limit (360 MPa) even without considering any additional bending. To further adjust and make more uniform the stress between panel and cantilever a polymeric foam, based on polyurethane, was applied. However, this produced an imperfect contact between the panel and the cantilever.

Figure 2: South-West corner before (left) and after (right) the collapse.

Figure 3: Cracks on the remaining panels.

2.1 Retrofitting

To improve the safety of the other RC panels several additional connectors, realized with AISI 316 L steel (ϕ27mm, L=500mm), where applied throughout an injection with epoxy resins. These elements can guarantee structural integrity over time even in aggressive microclimatic conditions (Figure 4). Their external position (Figure 5) allows to carry out periodic maintenance inspections and to keep the support condition under control. The philosophy of intervention is to introduce redundance in the system. Instead, the original detail did not have structural robustness, and, in case of material degradation, the failure is incipient.

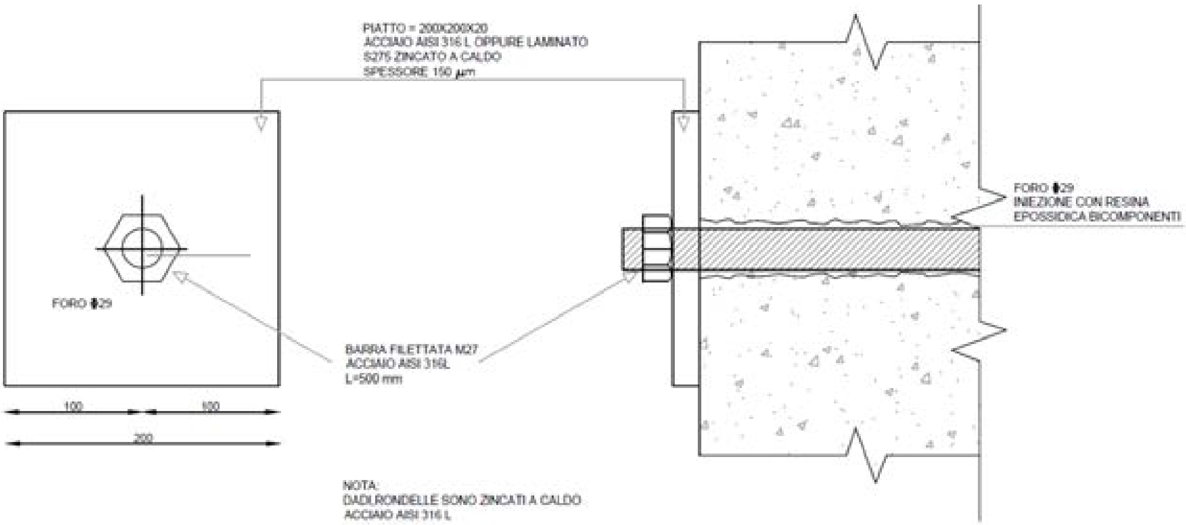

Figure 4: Details of additional steel connectors

Figure 5: Details of on-site installation of the connectors.

3 DESIGN CRITERIA PROPOSAL

To reduce this kind of fragile collapse new design approach should be introduced in a stochastic framework [26]. In the following two of them are described.

3.1 Effective trace

The effective trace of the support surface a_{ef} should be evaluated as:

$$a_{ef}=a-(s+\sum d_i) \quad (2)$$

Where s is the horizontal gap between the panel and the column, while d_i are the distances presented in Figure 6.

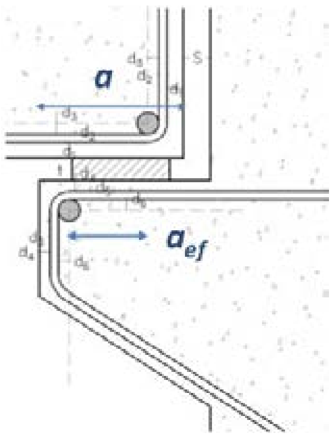

Figure 6: Details of effective trace approach.

Considering that a can be measured in a deterministic way, the uncertainties are in $(s+\sum d_i)$. The latter expression can be described with a stochastic approach. Indeed, it is possible to assume that this term it is defined by a log-normal probability distribution. For example, considering the case described in Section 2 $s_k = 1.3$ cm, $d_{i,k} = 1.3$ cm the following characteristic value of the effective trace is obtained:

$$a_{ef,k} = a - 1.3(s + \sum d_i) \tag{3}$$

Now it is interesting to point out that for the previous case a=15 cm, while a_{ef} =2 cm and $a_{ef,k}$<0.

Equation (3) allows to develop a probabilistic reliability analysis of this structural detail.

3.2 Sliding surface

Considering the common "sliding surface" approach a simple geometric safety criterion. The fracture line is here characterized by an inclination of 45° with respect to the horizontal plane. Then it is overlapped on the reinforcement details of the support cantilever. If this line crosses the horizontal distance between the reinforcement, (Figure 7), it is possible sustain the load. Vice versa if it cannot cross this horizontal line the collapse is expected.

Clearly this graphical criterion can be expressed through a mathematical formulation, see equation (4) and Figure 7 for the symbols meaning.

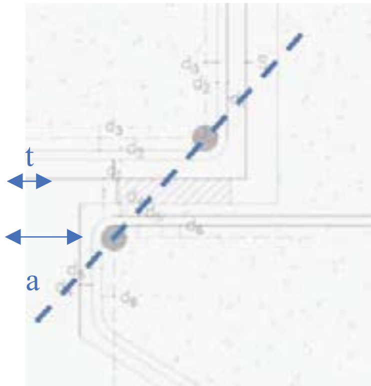

Figure 7: Details of sliding surface approach.

$$a > t + \sum d_i \tag{4}$$

Considering a log-normal probability distribution is assumed for each geometrical variable it is possible to obtain also in this case a stochastic value of the safety coefficient.

4 NUMERICAL MODEL

A fem model of the structural detail is carried out with Ansys (rel 18.1). Three different conditions with different values of the horizontal distance between the bar in the cantilever and the bar in the panel (a_{ef} see equation (2) have been analyzed. The geometrical description with loads and boundary conditions is reported in Figure 8. The cantilever is fixed in the back and in the bottom surface while the load is applied on the top of a panel section. The whole model is constrained in the z direction (transversal to the cantilever plane) to consider a plain problem. In any case a 3D modelling strategy has been developed to model the transversal rebars inside the concrete mesh.

The numerical model has been developed in ANSYS environment using SOLID65 elements for concrete and LINK180 for rebars. Concrete class is C20/25 while steel for rebars is B450C, see NTC18 [27]. The nonlinear behaviour of concrete and its progressive failure has been modelled with the Willam e Warnke criterion [28] while the steel rebars behavior is always linear elastic. Two transversal rebars (ϕ10 mm) have been modelled following the scheme presented in Figure 6: one in the bottom right corner of the panel and one in the top left corner of the cantilever.

A nonlinear quasi-static analysis has been performed with a progressive load applied as a uniform pressure on the panel top surface. This load has been varied to test what is the ultimate capacity for the three different geometrical configurations (denoted Left, Central and Right) characterized by different a_{ef} parameter values, see Figure 6 and Table 1.

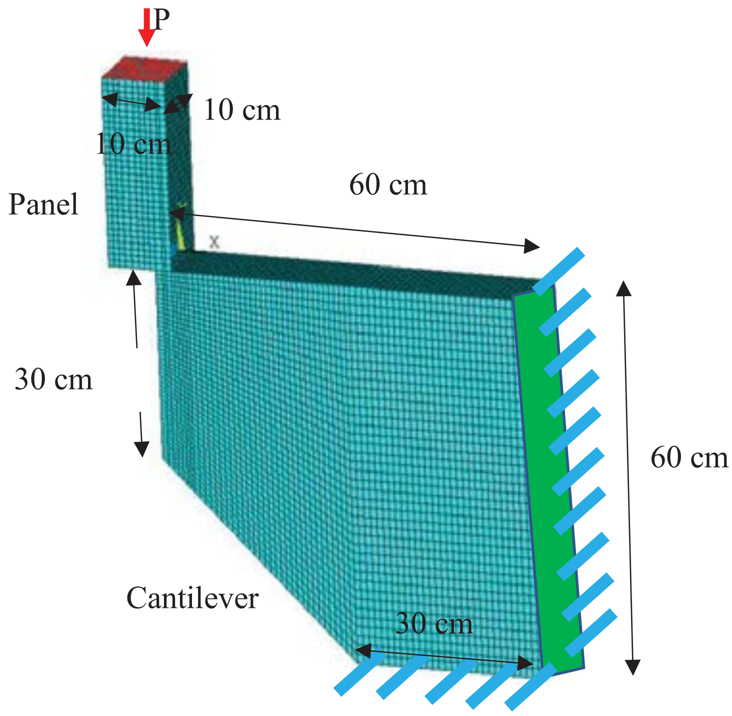

Figure 8: Model mesh, loading and boundary conditions.

	Left	Central	Right
a_{ef} (cm)	0	2.4	4.4
P_u/P_{MAX}	0.05	0.60	1.00

Table 1: Geometrical parameter *a* and relative ultimate load for different configurations.

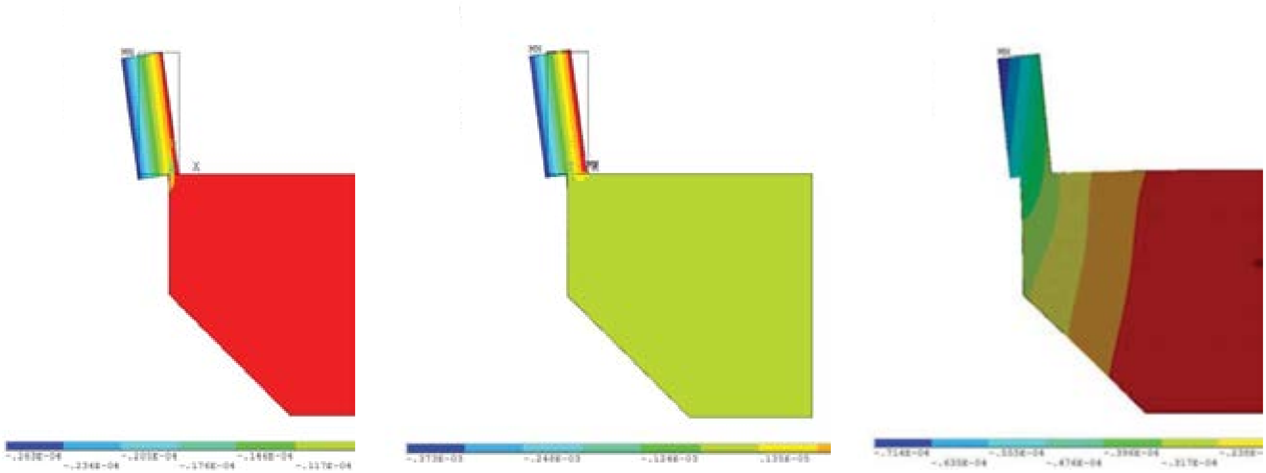

Figure 9: Displacement Contour vertical direction, measures are in m.

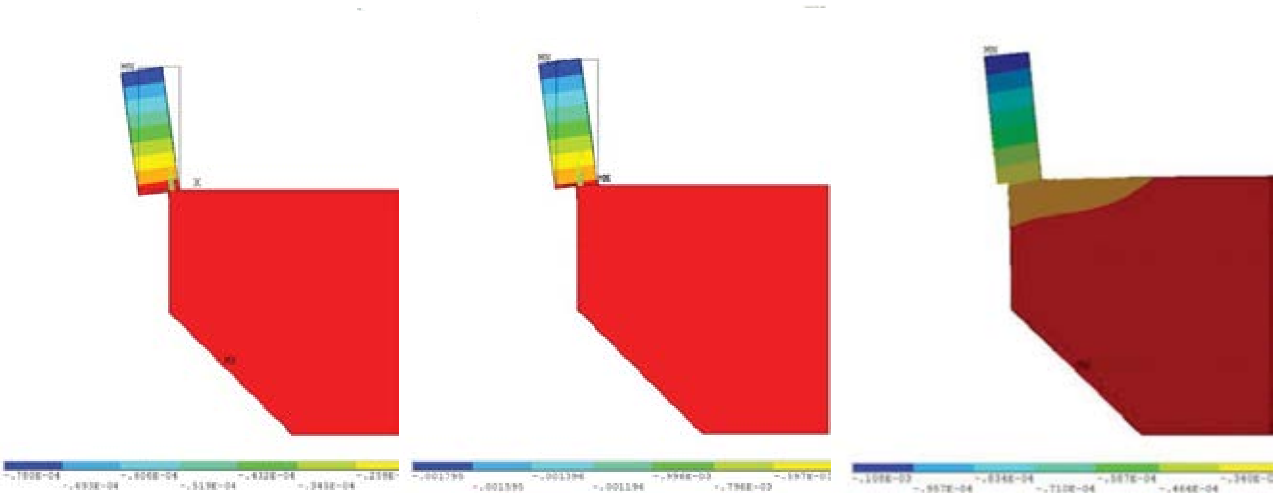

Figure 10: Displacement Contour horizontal direction, measures are in m.

Both displacement and convergence criteria has been enforced with a Newton-Raphson approach to solve the nonlinear problem. When is not possible to find an equilibrium configuration the simulation is stopped and the maximum load is measured P_u. Its relative values are reported in Table 1. As expected, the configuration with the maximum a_{ef} (denoted as Right) presents the maximum load P_{MAX}, the left and central configuration instead produced a reduced ultimate load respectively equal to the 5% and 60% of the P_{MAX}.

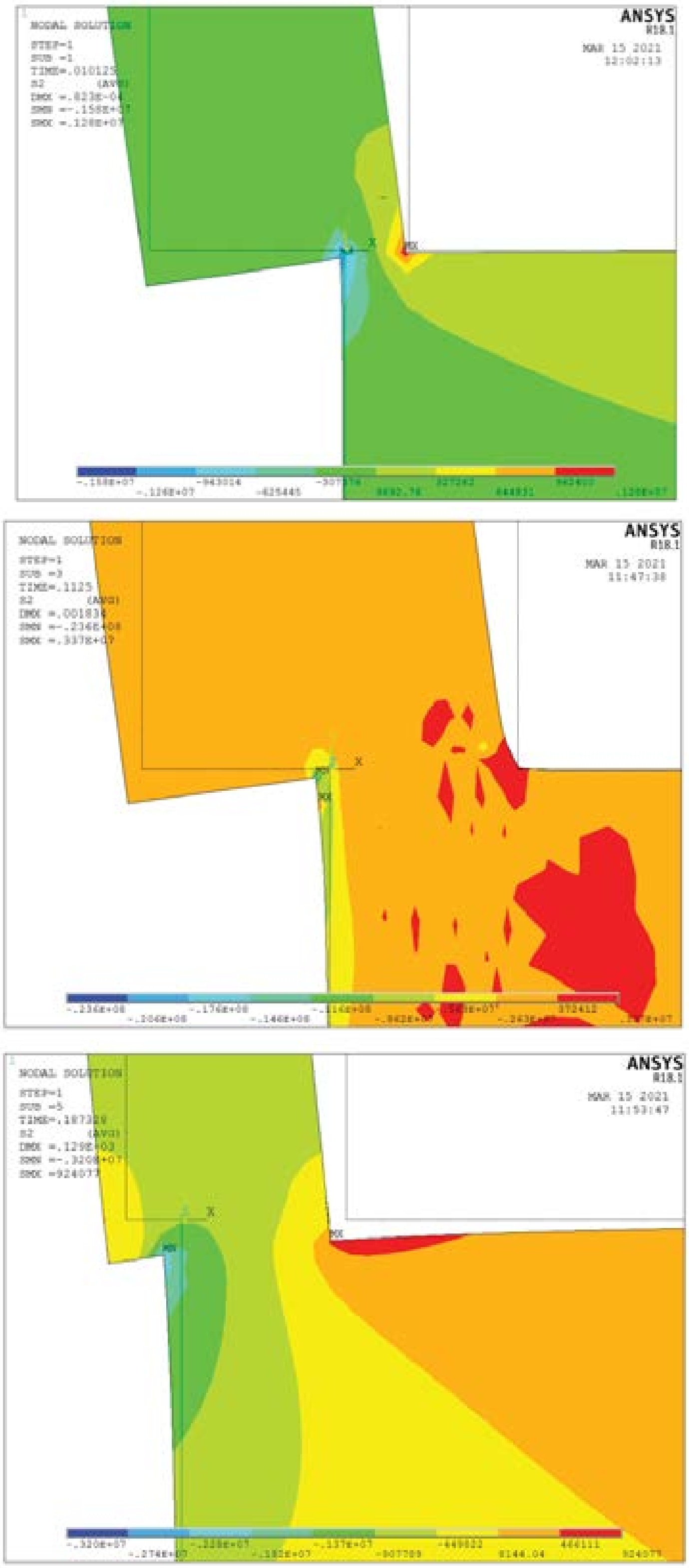

Figure 11: 2nd principal stress contour.

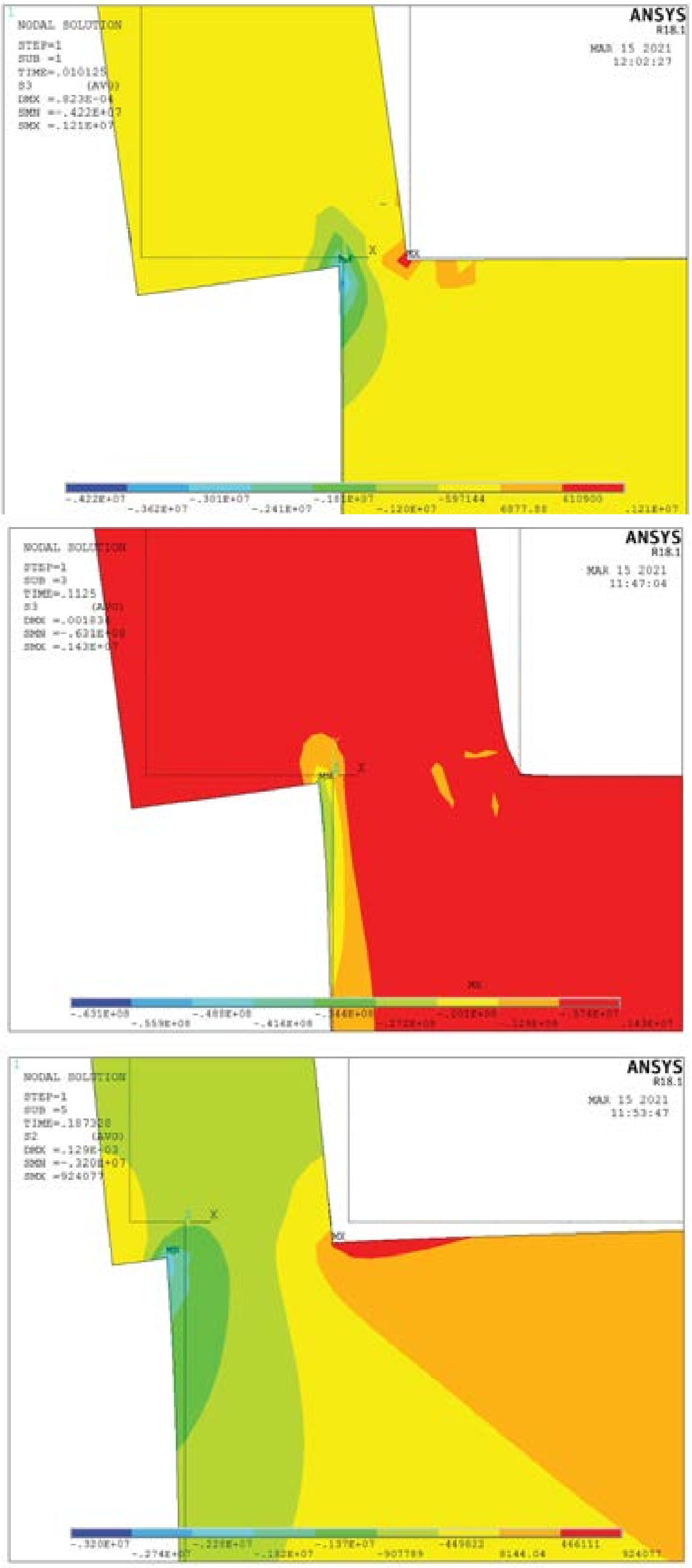

Figure 12: 3rd principal stress contour.

The vertical and horizontal displacement contours in the ultimate load step are reported in Figure 8 and 9 for the three configurations. It is clear that the left configuration (a_{ef}=0 cm) arrive at collapse with most of the deformation concentrated in the panel while the cantilever is still almost undeformed. Instead, the right configuration (a_{ef}=4.4 cm) requires that also the cantilever is deformed to reach the collapse condition.

Figures 10 and 11 presents the contours of 2nd and 3rd principal stress highlighting that the edge of the contact surfaces between the panel and the cantilever are the most stressed part of the element. Maximum compression is generally reached on the left part while maximum tension can be found on the right part.

The failure mechanism is almost the same for the three cases: the tensile crack appears in the top part of the cantilever on the right side of the contact edge between panel and cantilever. A representative picture of the crack propagation can be found in Figure 13 that points with red marks the tensile cracks. Clearly the crack pattern starts on the right and it develops toward left till the structure is not capable of finding an equilibrium condition.

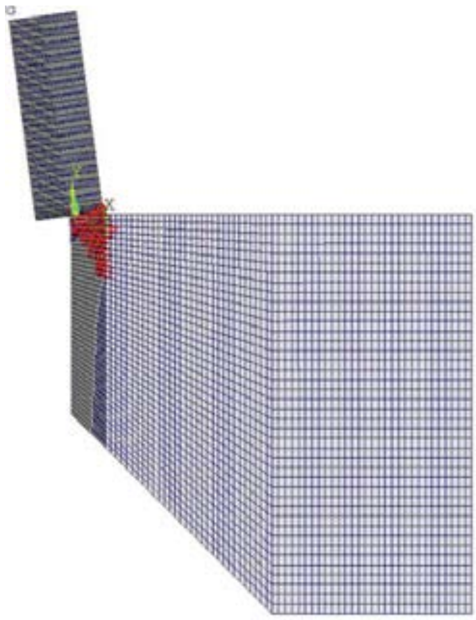

Figure 13: Typical Crack pattern.

5 CONCLUSIONS

This paper reports on recurrent situations typical of a poor concept of the prefabricated elements supports. In the modern performance-based design ductility, robustness hierarchy of structural element importance are key aspects. The possibility that a local collapse can trigger macroscopic collapses must be considered in existing PRC Stricture. A structural retrofitting as well as proper sealing strategy of the existing joints must be taken into account [29].

The numerical model showed how a bigger contact surface and a bigger distance between the transversal rebars a_{ef} yield to high load capacity. In addition, the failure mechanism highlights how it is critical the left end of the contact between the panel and the cantilever suggesting improving the resistance of this mechanism using appropriate reinforcements.

A modern trend is nowadays to evaluate also minor details in a resilience perspective. Thus, also the design of constructive details [30] should take into account the needs of inspection, the possible faults or malfunctions warning signs, and should avoid that any local collapse does spread in an uncontrolled global collapse.

Further developments of this research are expected for a stochastic evolution of the proposed design criteria or considering the effect of material irregularities.

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