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Seismic damage and loss evaluation in precast industrial buildings through low-cost accelerometers

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Abstract

Past seismic events, in particular the Emilia earthquake (2012), showed how the damage caused in highly industrialized areas could cause serious problems with large-scale economic effects. It is interesting to highlight how these effects are both immediate, due to the seismic vulnerability of old precast industrial buildings, and in the long term since the restoration operations are typically long due to the difficulty of assessing the real structural health state, particularly for the difficulty in reaching and inspecting components sensitive to damage, such as the connections. This work has the objective of finding a procedure, through relatively low-cost MEMS accelerometers, which allows to record and process the seismic accelerations experienced by the various structural elements. The paper considers the application of a procedure that allows identifying the state of health of the system through the application of damage indexes found in the literature and a first estimate of costs and recovery times to bring the building back to its pre-earthquake conditions. The procedure has been applied to a selected case study.

Keywords: Precast Building, Seismic damage, Economic losses, Downtime, Mems accelerometers

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1. Introduction

Recent advances in low-cost sensors manufacturing, computational capability and structural monitoring (Abdo and Hori 2002, Carden and Fanning 2004, Farrar and Worden 2007) have extended the interest in continuous damage detection in the civil engineering sector. For instance, the detection of damage after seismic events is of considerable importance to allow for more accurate inspections, to determine the safety or unsafety of the damaged building or to set an adequate retrofit plan, considering that, sometimes, some of the structural elements or connections cannot be inspected directly. It is also highlighted that, as in the case of precast industrial buildings, the interruption of service may cause huge indirect economical losses, therefore a more rapid and accurate knowledge of the effective state of damage would help in reducing such losses. In this regard, the introduction of low-cost sensors in a building can provide information for defining the main parameters of the earthquake (Trifunac and Novikova 1998), as well as to estimate the state of damage (Cosenza and Manfredi 2000, Azhdary and Shabakhty 2013, Datta and Ghosh 2008, Sinha and Shuradhonkar 2012). Through a real-time estimate of the state of damage it would be possible to reduce the time for the damage assessment, to reduce downtime and to assess the building state-of-health from the recorder data in a more accurate way. To make this system possible, it is necessary to identify a compromise between the accuracy of the recorded data and the cost of the system. The present paper aims at assessing the use of a net of MEMS sensors in industrial buildings and to define suitable strategies for structural health monitoring.

2. Damage detection and loss estimation

2.1. PEER-PBEE methodology

PEER-PBEE methodology (Günay and Mosalam, 2012) represents an accurate probabilistic procedure for the seismic loss estimation of any structural system. The first step of the method is the definition of the hazard curve. In the next step the seismic analysis of the considered system is carried out to obtain the values of engineering demand parameters (EDP) used to relate the damage states of structural and non-structural components to repair costs. The last two steps are the damage and loss analyses: the first aims to define the probability of damage of each damageable element; the second relates the damage probability to losses (repair cost, downtime, casualties among others).

This methodology needs to be adapted for the considered building typology (i.e. precast industrial buildings) due to the occurrence of local collapses. In fact, such buildings are typically statically determinate structures with dry connections and the collapse of one element leads to the collapse of all the supported elements. For this reason, the concept of collapse hierarchy was introduced (Bosio et al. 2020), which is a logical set of relationships to account for the collapse of the various elements and of the supported ones. The PEER-PBEE method was therefore adapted by calculating the probability of damage of a given element as the product of its probability of damage times the probability of not-damage of the element that is at the lower level of the collapse hierarchy:

$$\begin{gathered} P^G_{DS-i,j} = P^L_{DS-i,j} \cdot P^G_{NC-sup} = P^L_{DS-i,j} \cdot \left(1 - P^G_{C-sup}\right) \\ P^G_{NC-sup} = \prod P^L_{NC-sup} = \prod \left(1 - P^L_{C-sup}\right) \end{gathered} \tag{1}$$

Where $P^G_{DS-i,j}$ represents the overall probability of exceeding the i-th limit state of the j-th element; $P^L_{DS-i,j}$ represents the local probability (calculated from the EDP) of exceeding the i-th limit state of the j-th element; P^G_{NC-sup} represents the probability of not collapse of the element that is at the lower level of the collapse hierarchy, obtained as the complement to 1 of the probability of collapse of the same element. This probability can be expressed as the product of the non-collapse probability $\left(1 - P^L_{C-sup}\right)$ of all the elements that are at a lower level of the collapse hierarchy.

2.2. Damage indexes

Another way for damage assessment is through the adoption of the so called "damage indexes" (DI) (Sinha and Shuradhonkar 2010) which, using recorded and derived data (such as floor absolute acceleration and floors relative displacement), allow for a rapid estimate of the structural health. Herein, two of the damage indexes which only require the displacement value for the damage assessment have been considered:

- Wang and Shah

$$D = \frac{e^{\eta\beta}-1}{e^{\eta}-1} \quad \text{with} \quad \beta = C\sum_{i=1}^{n}\frac{d_i}{d_u} \tag{2}$$

Where d_i and d_u are the i-th displacement and ultimate displacement, respectively; η can take the value of -1 or -3; C is a constant value equal to 0.15.

- Powell and Allahabadi

$$D = \frac{\mu_m-1}{\mu_u-1} \tag{3}$$

Where μ_m and μ_u are the ductility at maximum value and ultimate ductility, respectively.

Such indexes have been subsequently normalized (Fig. 1a) to get a value equal to 1 for ultimate conditions and 0 in the case of no damage. It should be noted that for the Wang and Shah index, the normalization was carried out on β. In addition, this index does not depend on the maximum displacement obtained from the seismic event but takes into account all the inelastic cycles experienced by the building. The normalized value (β_n) of β is:

$$\beta_n = \frac{\beta_i}{\beta_u-\beta_e} - \frac{\beta_e}{\beta_u-\beta_e} \tag{4}$$

Where $\beta_i, \beta_e, \beta_u$ represent the coefficient β in correspondence to the maximum, elastic and ultimate displacement of the structure, respectively. For the Powell and Allahabadi's index, it is observed how this index has been already defined between 0 (yielding displacement) and 1 (collapse), for this reason a value equal to 0 is placed in the case of elastic displacements.

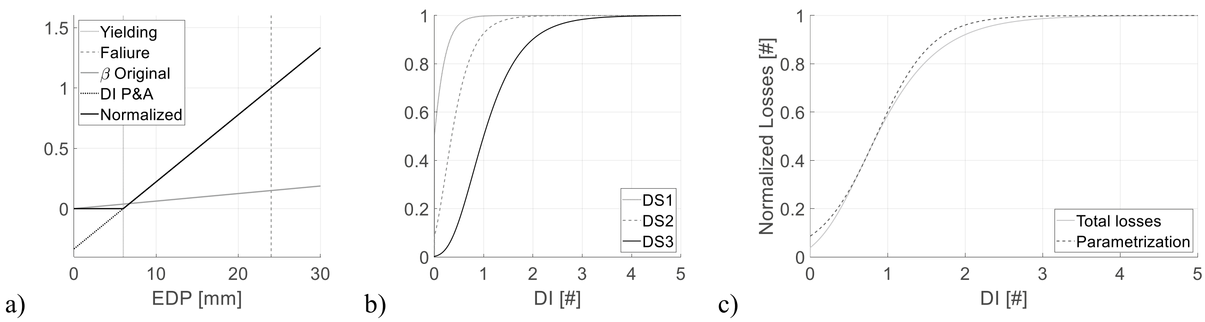

Fig. 1. a) Normalization of the β coefficient and of the damage index of Powell and Allahabadi (DI P&A); b) example of curve of losses for each damage state (DS); c) curve of total losses and parametrization.

2.3. Loss estimation through damage indexes and recorded data

The proposed simplified procedure for both damage and loss assessment consists in combining the PEER-PBEE method and the damage index approach through the data received from a series of sensors installed on the structure.

Recalling that a damage index equal to one corresponds to failure conditions evaluated in average terms, it can be assumed that this situation corresponds to a probability of collapse of a given element equal to 50%. In addition, it is possible to directly relate the damage index to the losses by considering that a given engineering demand parameter (EDP) is associated with both a damage index and a loss value. Herein, the loss value is obtained as the sum of the contribution of each damage state (DS) that an element can experience (Fig. 1b-1c). The shape of the total losses resembles a sigmoid function, therefore a logistic function has been selected to fit the total loss curve and allow for a faster loss estimation. The loss curve fitting could be carried out for each structural and non-structural element by directly applying the PEER-PBEE approach.
Once the damage-loss correlation has been determined and applied based on the data recorded on the building, the collapse hierarchy criterion must be applied. The associated loss value at the i-th element is:

$$L_i = L_i^L + L_i^G = l_i \cdot L_{max,i} \cdot (1 - PC_i^G) + L_{max,i} \cdot PC_i^G \quad (5)$$

where L_i^L represents the losses of the i-th element evaluated through the EDP; L_i^G represents the losses of the i-th element caused by damage of the supporting elements; l_i represents the share of the total loss calculated using the proposed methodology; $L_{max,i}$ represents the maximum losses of the i-th element; PC_i^G represents the global probability of collapse of the i-th element calculated through the collapse hierarchy (Eq. 1).

3. Reference case study

To validate the proposed procedure, a finite element (FE) model was created considering a typical configuration of a precast industrial building. The building has a rectangular plan (20m x 42m) and the load bearing structure consists of eight one-direction frames in the longitudinal direction equally spaced with a total height equal to 8.6m. Fig. 2 shows the details of the main structural elements. The cross-section AA (50cm x 40cm) has the largest inertia in the transversal direction with 12 ϕ14 longitudinal rebars and ϕ8 stirrups at 200mm centre to centre. The roof system is made of double-tee prestressed RC elements supported by double tapered beams. The roof element-beam connection is a steel bracket with 4 ϕ10 dowels. The cladding system is made of RC panels. Along the longitudinal direction, 4 levels of horizontally spanning panels are present with ribbon windows between the 3rd and the 4th row. Such panels are connected to the columns by 2 bearing connections at the bottom of the panel (a bearing bolt, ϕ24, connected to a stiff steel bracket) and 2 retaining hammer-head anchor bolts at the top (ϕ16 bolt and 40mm x 2.5mm anchor channel) as reported in Belleri et al. (2016). In the transversal direction, vertically spanning cladding panels are present and anchored to the grade beam by a L-shape steel plate and to the top beam by a hammer-head stripe connection (Zoubek et al. 2016, Belleri et al. 2017).

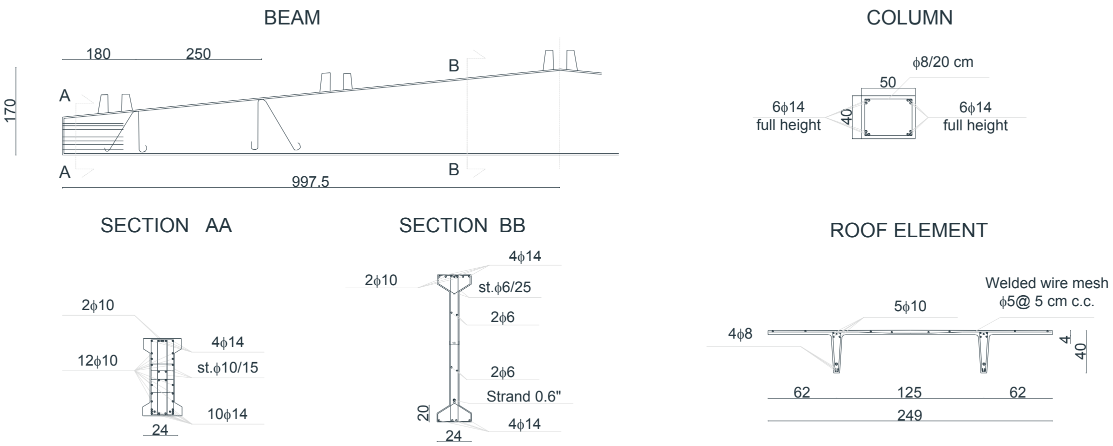

Fig. 2. Construction details for the main structural elements.

The FE model of the considered buildings was developed with the software OpenSees (McKenna and Fenves, 2001). In the FE model definition, the focus was made on a single span, in addition to the two columns and the main girder; the model also includes the tributary portion of roof elements. The cladding system was modelled as lumped masses in correspondence of the connections to the supporting elements. The main seismic vulnerabilities have been considered in the FE model in terms of plastic hinge at the base of the column, hysteresis of the beam-column dowel connection, hysteresis of the beam-roof element connections, friction connections for the beam-column and beam-roof element connections. The Modified Ibarra-Medina-Krawinkler Deterioration Model with Peak-Oriented Hysteretic Response ("ModIMKPeakOriented" Material) (Ibarra et al., 2005) was considered for the plastic hinge in terms of moment-rotation response and in terms of force displacement response for the dowel connection (Bressanelli et al., 2019). An elasto-perfectly plastic model was considered for the beam-roof element mechanical connections in parallel with a friction behaviour: flatSliderBearing model with a friction coefficient equal to 0.13 (i.e., neoprene-concrete surface) and initial stiffness equal to 490 kN/m (i.e., transversal neoprene pad stiffness). The other main hysteretic parameters are reported in Table 1. The FE models were subjected to time history analyses using a set of 52 accelerograms (Appendix A) with increasing intensity and various epicentral distances obtained from the database Itaca (INGV).

Table 1. Modelling parameter for nonlinear behaviors.

Element	**K_0**	**K_1/K_0**	**M_y**	**θ_p**	**θ_{pc}**	**θ_u**	**M_{res}**
Column	43100	0.07	262.7	0.0174	0.0432	0.2	0.2
Element	**K_0**	**K_1/K_0**	**F_y**	**d_p**	**d_{pc}**	**d_u**	**F_{res}**
Beam-Column connection	59600	0	54.23	0.0111	0.014	0.028	0
Beam-Roof connection	520	-	10.4	0.02		0.05	0

To evaluate losses, the following elements and the associated damage states (DS) were considered: a) columns (DS: cracking, concrete spalling and collapse); b) beam-column connections (DS: cracking, concrete spalling and failure of mechanical connection); c) beam-roof element connection (DS: relative displacement, connection failure and element falling); d) cladding system (DS: yielding of connection, connection failure, element falling). Table 2 shows the values obtained following the parameterization (logistic function) carried out according to the proposed approach.

Table 2. Parametrization of the loss curve for the considered seismic vulnerabilities.

Element	**Normalized Losses**		
	Mean	**Dispersion**	**Maximum Error**
Column	0.8555	0.362	±5%
Beam-column connection	0.0302	0.200	±9%
Roof element	0.8446	0.371	±1%
Horizontal cladding	1.2504	0.530	±2%
Vertical cladding	1.3846	0.524	±3%

4. FE results

The investigated procedure requires the structural response time histories. For this reason, the building was supposed instrumented with MEMS accelerometers whose recordings can be integrated twice to get absolute and relative displacements and consequently the damage suffered by the building itself. The following sensors have been supposed:
• 1 sensor placed at the bottom and 1 at the top of the column;
• 1 sensor placed at each side of the beam;
• a set of sensors placed along the beam in proximity of the supports of the roof elements;
• a set of sensors placed along the column and along the horizontal cladding.

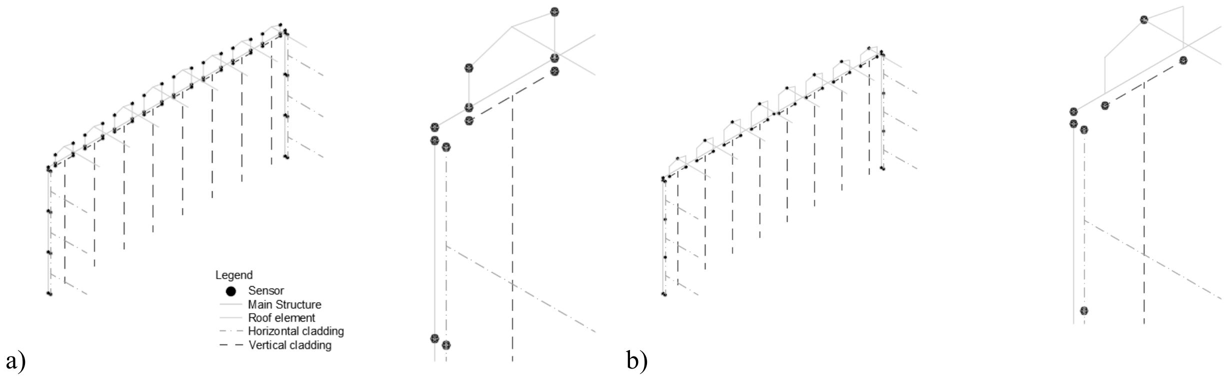

Fig. 3. Position of the sensors in the FE model: a) full sensors configuration; b) reduced number of sensors.

To compare the losses estimated from the proposed approach and the ones obtained from the PEER-PBEE method, a set of 10’000 simulations was carried out considering various realizations of the capacity of the structural elements and connections (assumed normally distributed with coefficient of variation of 20%). Fig. 4 shows the results of such a comparison highlighting how the Wang and Shah’s DI is less reliable since it leads to an underestimation of the damage; on the contrary, the Powell and Allahabadi’s DI is closer to the PEER-PBEE results.

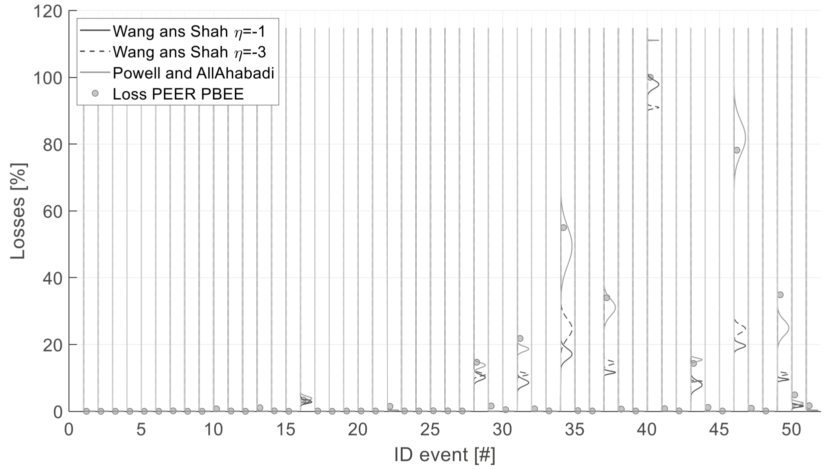

Fig. 4. Comparison between losses estimated with PEER-PBEE method and the proposed approach: from FE data.

To consider the influence of placing a real set of sensors with a given sensitivity and spectral noise, the numerical FE results were modified by adding a Gaussian white noise with a root mean square equal to 0.1mg, 0.5mg, 1mg, 2mg, 5gm, and 10mg. It is worth noting, that the use of real low cost sensors may introduce further uncertainties as the loss of some acceleration points that can reduce the accuracy of the double integration, the influence of the temperature and the presence of drifts in the signals. Herein, the obtained data were digitally filtered with a low pass filter with a cut-off of 15 Hz, then the mean value of the signal was removed. The losses were estimated with the Powell and Allahabadi’s DI due to its previous better performance compared to Wang and Shah’s DI. Figure 5 shows how in the case of high-quality sensors the error is negligible, similar results are obtained from the reduced number of sensors. In the case of noisy sensors, a loss overestimation is recorded for the high intensity earthquakes. The reason of such overestimation is associated with the integration of the accelerations to obtain displacements (through Simpson’s quadrature formula). It is worth noting that in the absence of damage, the procedure does not provide false positive results in the full sensor configuration, while for the reduced number of sensors some false positives results are obtained (up to a maximum loss value equal to 20% of the total repair cost) for the earthquakes of medium intensity. In such conditions, we can therefore state that the simplified approach is more reliable in the case of significant damage, introducing a loss uncertainty in the 0-25% range for the reduced number of sensors.

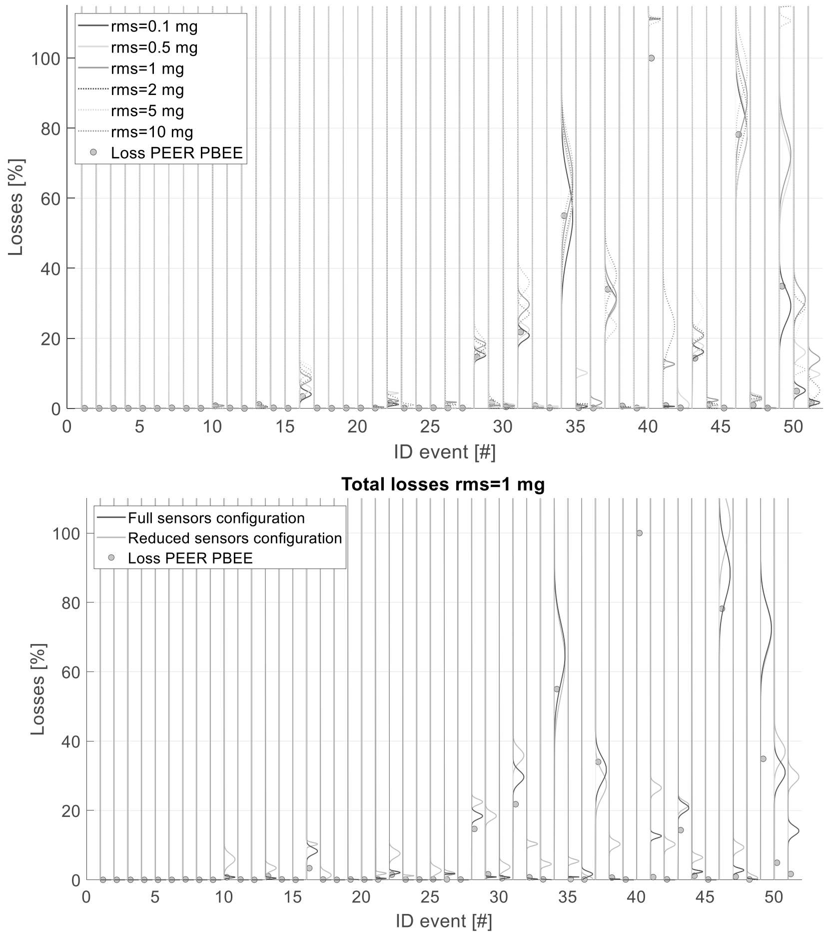

Fig. 5. Comparison between losses estimated with PEER-PBEE method and the proposed approach, in the case of simulated sensor data: a) full sensors configuration; b) reduced number of sensors.

5. Conclusion

This paper investigated the seismic loss estimation through sensor data (accelerometers) in precast industrial buildings. In this regard, a procedure has been defined starting from damage indexes formulations available in the literature. Each damage index was normalized to get 0 for no-damage and 1 for failure and a logistic function was defined to directly relate the damage to losses. The proposed approach was applied to a reference case study made by a precast frame resembling the structural system of an Italian industrial building. A set of finite element time history analyses was conducted considering ground motions with various intensity and epicentral distances, the uncertainties related to the capacity of the structural elements and of the connections. The results were compared to the losses estimated applying the PEER-PBEE method and allowed to highlight the best performance of the Powell and Allahabadi's damage index. A further set of simulations was carried out to evaluate the influence of real sensor data. In this regard a Gaussian white noise was added to the finite element acceleration results and the displacements were derived from double integration. Spectral noise with increasing intensity was considered. The results show how the noise addition leads to a loss overestimation for high intensity earthquakes while no false positive losses were recorded for no-damaging earthquakes in the case of a full sensors configuration, while some false positive results (for medium intensity earthquakes) were recorded in the case of a reduced number of sensors.

Appendix A. Considered earthquakes

ID Event	Place	Soil	Mw	Date	Epicentral Distance (km)		
					1st ID	2nd ID	3rd ID
1-3	Gubbio	B	3.1	08/01/2014	7.2	34.4	82.3
4-6	Fiordimonte (MC)	D	3.9	27/11/2016	5.8	29.9	61.1
7-9	Norcia	A	4.0	31/10/2016	5.2	31.6	81.4
10-12	Ponte San Pellegrino (MO)	C	5.0	20/05/2009	7.4	31.5	80.4
13-15	L'Aquila	B	5.3	18/01/2017	9.7	20.2	74.3
16-18	Norcia	E	5.4	18/01/2017	17.7	34.1	76.4
19-21	Castelsantangelo sul Nera (MC)	C	5.4	26/10/2016	3.2	22.4	59.5
22-24	Mirandola (MO)	C	5.5	29/05/2012	6.7	24.6	57.2
25-27	Foligno (PG)	A	5.6	14/10/1997	8.7	23.5	70
28-30	Colfiorito (PG)	D	5.7	26/09/1997	2.8	25.0	66.0
31-33	L'Aquila	B	5.9	06/04/2009	4.9	30.8	80.1
34-36	Finale Emilia (MO)	C	5.9	25/05/2012	16.1	24.4	62.7
37-39	Accumuli (RI)	B	6.0	24/08/2016	8.5	30.2	62.0
40-42	San Felice sul Panaro (MO)	C	6.0	29/05/2012	0.5	25.7	65.7
43-46	Colfiorito (PG)	D	6.0	26/09/1997	4.8	27.0	59.1
47-59	Capo del colle (PG)	A	6.1	30/10/2016	11	27.0	59.1
50-52	LAviano (SA)	B	6.5	23/11/1980	18.9	42.6	58.7

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