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RINTC PROJECT: NONLINEAR ANALYSES OF ITALIAN CODE CONFORMING PRECAST R/C INDUSTRIAL BUILDINGS FOR RISK OF COLLAPSE ASSESSMENT

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Abstract. *Most of modern seismic design codes adopt the performance-based approach by defining a pre-defined level of safety for different limit states. These safety margins are implicitly defined into the definition of detailing provisions as well as in partial safety factors for both actions and materials strength. Moreover, such implicit provisions are related to the specific structural typology; hence, the structural safety can change for different typologies designed for the same site. This study investigates the seismic performance of typical Italian RC precast buildings by means of an extensive parametric study. Both the structural typology and the geometrical configurations are defined in order to represent the most common practice in the country. The investigated buildings are located in five sites with different levels of seismic hazard and two soil types are also considered. The collapse assessment is investigated by means of multi-stripe analyses, performed by non-linear dynamic analyses at 10 intensity levels.*

1 INTRODUCTION

According to the modern building codes [1, 2], the seismic design should ensure that, under seismic actions of a given intensity level, the structures reach a corresponding performance level, i.e. limit state. The adequate safety level for a limit state is implicitly obtained by considering: a) a probabilistic definition of design values for both actions and material properties; b) specific provisions (detailing) related to the structural typology. Therefore, the influence of such provisions cannot be defined a priori for different sites and/or for different structural typologies. Some research studies have been performed for existing [3, 4] and new RC frame structures [5] in order to investigate the reliability of modern design approaches. However, few studies have been performed for RC precast buildings in terms of seismic assessment [6] and risk study of existing precast buildings [7]. Several experimental tests were performed on new buildings and connection systems [8-10]; on the contrary, the code improvements have not been fully investigated as well as their influence on the global seismic vulnerability. Moreover, an important scientific debate has been opened on the safety of this structural typology in the last years because of two main reasons: 1) the widespread damage in precast buildings due to recent seismic events in Europe [11, 12] and the significant changes in the current codes for the design of this structural typology.

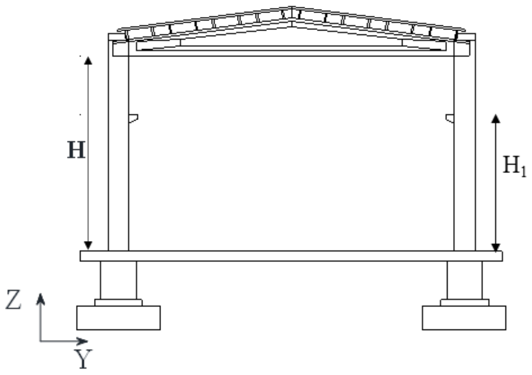
This paper aims to define the seismic vulnerability at the collapse limit state of single-story RC precast buildings. The case-studies are designed according to the Italian building code [1] in order to simulate realistic industrial RC precast structures. The results of the design demonstrate the influence of specific code provisions recently introduced in the building codes. An extensive parametric study is performed on four geometrical configurations of single-story precast buildings. These structures are designed for five seismic-prone areas (seismic intensity levels) by adopting two typology of soil. The vulnerability assessment is investigated by means of multi-stripe analyses at 10 intensity levels. The Demand/Capacity ratios are evaluated for all the case-studies.

2 PARAMETRIC STUDY

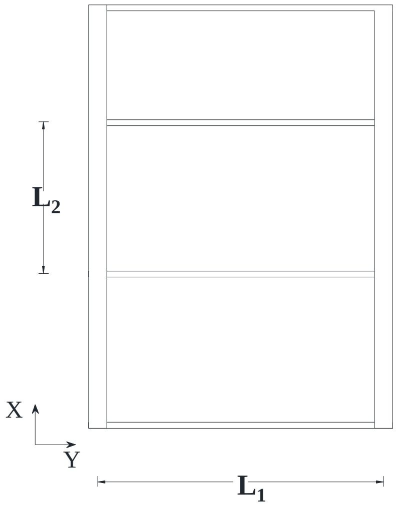
The seismic performance of single-story RC precast buildings is investigated by means of nonlinear multiple stripe analyses. The investigated structures (Figure 1) consist of isostatic columns, prestressed principal and secondary beams pinned to the columns, roof elements pinned to the beams and vertical cladding panels.

A parametric study is performed by considering a set of European industrial precast buildings. Figure 1 shows the frontal view of the case-studies in Y direction (a) and the plan view (b). All the case-studies consist of 4 bays in X direction and 1 bay in Y direction. Four geometrical configurations (Table 1) are assumed in this study by varying the height of the columns (H in Figure 1(a)) and the width of the bays (L_1 and L_2 in Figure 1(b)). Since the single-story precast structures usually host industrial activities, the presence of the crane is assumed in the case studies. This element is connected to a bracket, placed at 4.5m column height for the geometries 1 and 2 and at 7.5m column height for the geometries 3 and 4 (H_1 in Table 1).

The above-described geometrical features are chosen as the most spread in Italy, according to a wide-ranging database, which was produced by collecting the data of Italian reports concerning typology classifications and post-earthquakes assessment surveys [11].



(a)



(b)

Figure 1 Layout of the case-studies: (a) frontal view (Y direction) and (b) plan view

Geometrical configuration (Geo)	L_1	L_2	H	H_1
	[m]	[m]	[m]	[m]
1	15.00	6.00	6.00	4.50
2	20.00	8.00	6.00	4.50
3	15.00	6.00	9.00	7.50
4	20.00	8.00	9.00	7.50

Table 1 Geometrical configurations of the case-studies

2.1 Design approach

The case-studies are designed according to Italian building code [1] in five sites in Italy (Table 2); for each site two typologies of soil are considered: type A (the average velocity of S waves in the upper 30 m, $V_{s,30}$, is larger than 800m/s) and type C ($V_{s,30}$ is in the range: 180m/s - 360m/s). The concrete has a cubic cylinder compressive strength of 45N/mm^2 and the reinforcement steel has a characteristic yielding strength of 450N/mm^2.

Site (S)	Latitude	Longitude	a_g [g]-	a_g [g]-
L'Aquila (AQ)	13.399	42.349	0.104	0.261
Napoli (NA)	14.268	40.854	0.060	0.168
Roma (RM)	12.479	41.872	0.055	0.123
Caltanissetta (CA)	14.060	37.480	0.034	0.073
Milano (MI)	9.186	45.465	0.024	0.050

Table 2 Sites of the case-studies

Two limit states are considered under seismic loads: the Damage Limitation (DL) Limit State (LS) and the Ultimate (U) Limit State (LS). The live load is equal to 0.5kN/m^2 (roof of an industrial building) and the snow is evaluated as a distributed vertical load, according to the site. Secondary beams and cladding panels design is not performed in this study, the secondary beams have rectangular-shaped section (30cm x 60cm) and the vertical panels have a self-weight of 4kN/m^2. The connection systems are not designed in this study and their seismic behavior is not modelled in the numerical analyses; they are assumed strong and stiff enough to avoid their brittle/premature failures. The foundation system is not designed; in the numerical analyses a fixed constraint is assumed at the base of the columns. The column fork is constant in all the buildings: the height is equal to 60cm and the thickness is 15cm.

The roof elements (Figure 2) are designed only for vertical loads because of the pinned connections with the beams. Table 3 shows the dimensions of the TT roof elements in all the case-studies: B is the width of the roof element and H is the height. The details about the reinforcement bars are not reported in this paper since they are not required to perform the non-linear analyses.

The principal beams are prestressed RC elements with variable cross-section in height and shape. These elements are designed under two load combinations: 1) the combination of vertical loads (self-weight, live loads and snow) and 2) the combination of the seismic vertical component. For these elements the vertical component is considered for the sites AQ and NA (a_g larger than 0.15g for ULS) and for all the geometrical configurations (the span of the beams is always larger than 8m). Table 4 shows the mean values of breadth and depth of the principal beams.

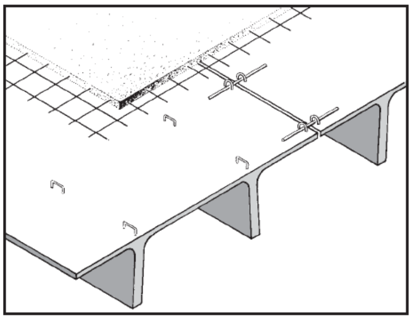


Figure 2 Layout of the roof: TT elements and cast in-situ slab

B	H	Site	Soil	Configuration
[cm]	[cm]	[-]	[-]	[-]
160	40	All	A, C	1, 3
240	40	All	A, C	2, 4

Table 3 Dimensions of the TT roof elements

B_{mean}	H_{mean}	Site	Soil	Geometry
[cm]	[cm]	[-]	[-]	[-]
19	114	All	A, C	1
19	150			2
19	114			3
33	140			4

Table 4 Mean dimensions of the principal beams for all the case-studies

The columns are precast square-shaped elements; both the cross-sections and the reinforcement ratios are constant in each structure. In order to evaluate the seismic demand on the column, dynamic linear analyses are perfomed by means of the above described elastic model in OpenSees [13]. The analyses are performed in both the horizontal directions and the seismic effects are then combined. In this study the behavior factor is assumed equal to 3.5 for all the case studies, according to the Italian code provisions for isostatic columns in ductility class “B” (medium ductility in EC8). P-Δ effects are taken into account according to the Italian code (and Eurocode 8) provisions: 1) amplification of the effets according to the value of the stability factor, θ; 2) mimimum dimensions of the column section, i.e. larger than 1/10 of the shear length, if P-Δ effects are not negligible (θ larger than 0.1). The column dimensions are reported in Figure 3 for all the case studies and Figure 4 shows the ratio of the logitudinal reinforcement. It is worth to hihglight that most of the columns provide a minimum reiforcement ratio required by the code ratio (i.e. about 1%). This minimum requirement can lead to a significant overstrength of the structures. The reinforcement ratio is larger than 2% if the structure is located in a high seismic zone (e.g., AQ) on flexible soil ("C") and the column height is 6m (Geo1 and Geo2). Only in few structures the reinforcement design is influenced by seismic actions rather than by other loads.

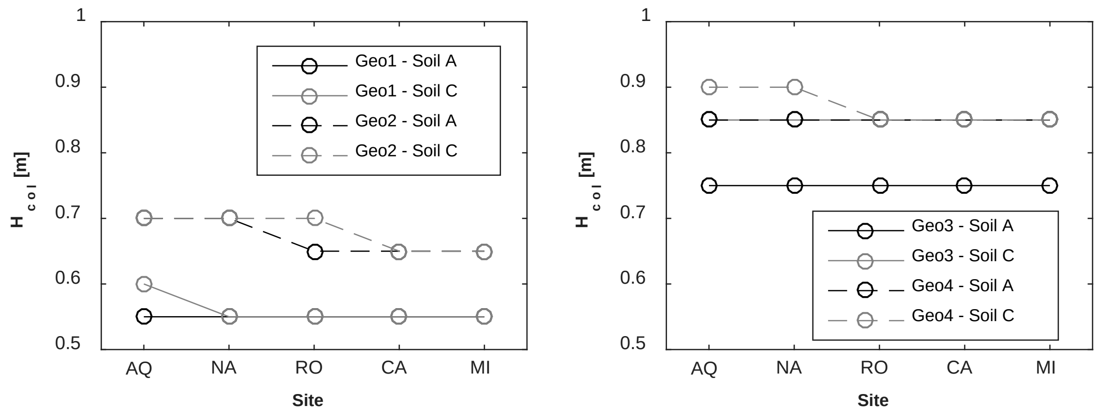


Figure 3 Column dimensions

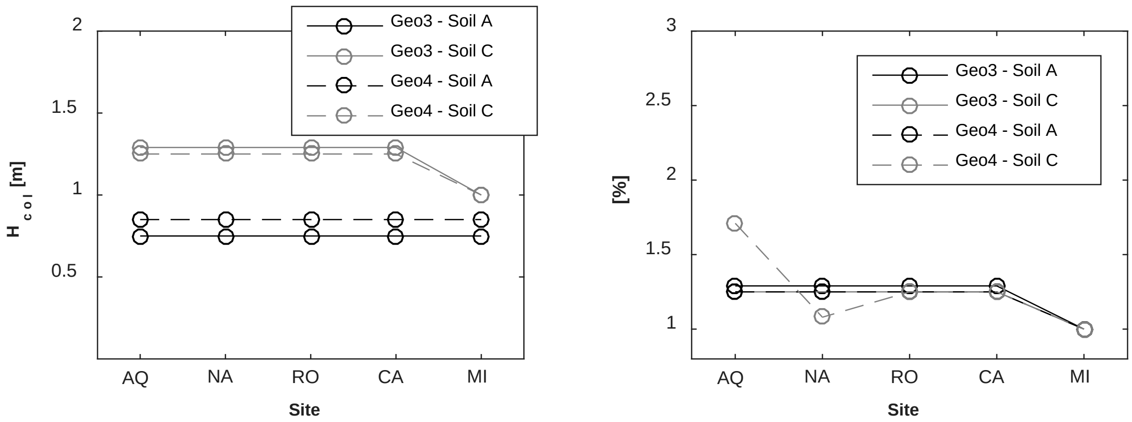


Figure 4 Longitudinal reinforcement ration in columns

3 NONLINEAR ANALYSIS

Nonlinear dynamic analyses are performed for all the case-studies in order to evalaute their seismic performance. The nonlinear model is implemented in OpenSees; it consists of columns, secondary beams and principal beams.

3.1 Nonlinear model

The nonlinear model consists of the horizontal elements (beams) and the columns. The beams are modelled as elastic elements and the nonlinear behaviour is concentrated at the columns base by means of a lumped plasticity approach. The plastic hinge is defined by means of a tri-linear moment-rotation curve, consisting of: the yielding point, the capping point and post-capping rotation, evaluated according to Haselton [14]. The yielding moment is defined by performing a fiber analysis on the RC section and the yielding rotation is evaluatated according to Fardis [15]. The hysteretic behaviour is modelled according to Ibarra [16].
The seismic response of precast buildings is investigated by means multi-stripe analyses [17] at 10 intensity levels (i.e. 10 return periods in Table 5). A set of 20 records is selected [18] at each intensity level for the five sites and both the soil types. The records were selected by means of the Conditional Spectrum (CS) approach [19-21] at a reference period of 2.0 seconds, given the structural typology.

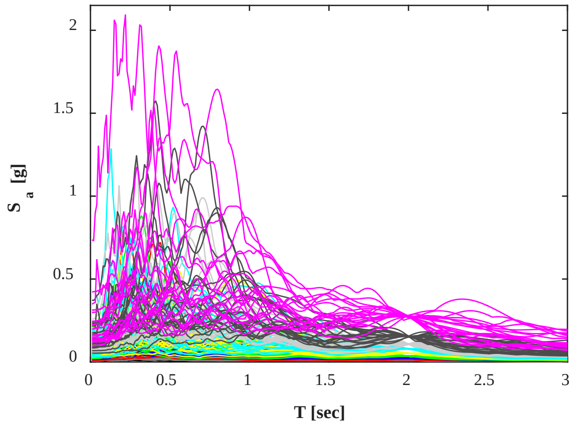


Figure 5 Spectra of the records for Caltanissetta on soil A

IM [-]	**1**	**2**	**3**	**4**	**5**
T_R [years]	10	50	100	250	500
IM [-]	**6**	**7**	**8**	**9**	**10**
T_R [years]	1000	2500	5000	10000	100000

Table 5 Return periods for 10 intensity levels

3.2 Results of stripe analysis

The multi-stripe analyses are performed on all the case studies and the results are reported in terms of demand/capacity (D/C) ratios along with the reference (T=2.0sec) spectral acceleration at each intensity level. Figure 6 shows the maximum ratio in the two horizontal directions between the demand and the capacity in terms of top displacement. The capacity of the structures (collapse force and displacement) is evaluated on the pushover curve at 50% reduction of the peak strength. Figure 6 shows the D/C ratios for all the case studies, intensity levels and sites:

- the empty circle markers are the cases of safe performance (D<C);
- the filled circle markers are the failure cases (D>C) due to the columns rotations;
- the square markers are the cases of dynamic instability ((D>>C)); for these points, a fictitious displacement is plotted since they correspond to very large demand in terms of drift in the structures.

According to the results of the multi-stripes analyses, very few collapses (D/C>1) are recorded for the investigated case-studies. By changing the seismic hazard of the site, the safety of the structure significantly changes. This can be justified by the adopted design approach. The column cross-sections and the reinforcement ratios do not change in most of the structures (Figure 6) because of the code minimum requirements in seismic prone areas. The geometry does not significantly influence the seismic behavior of the structure. The higher structures (H=9m) are safer in L'Aquila (high seismic hazard): the design code provides severe requirements for high flexible structures (minimum section of columns because of P-Δ effects). The soil type influences the structural response in terms of number of collapse cases (e.g. Geo2-AQ). This means that the increment of design reinforcement and section dimensions due to the soil C with respect to the soil A is not able to guarantee the same safety against the collapse.

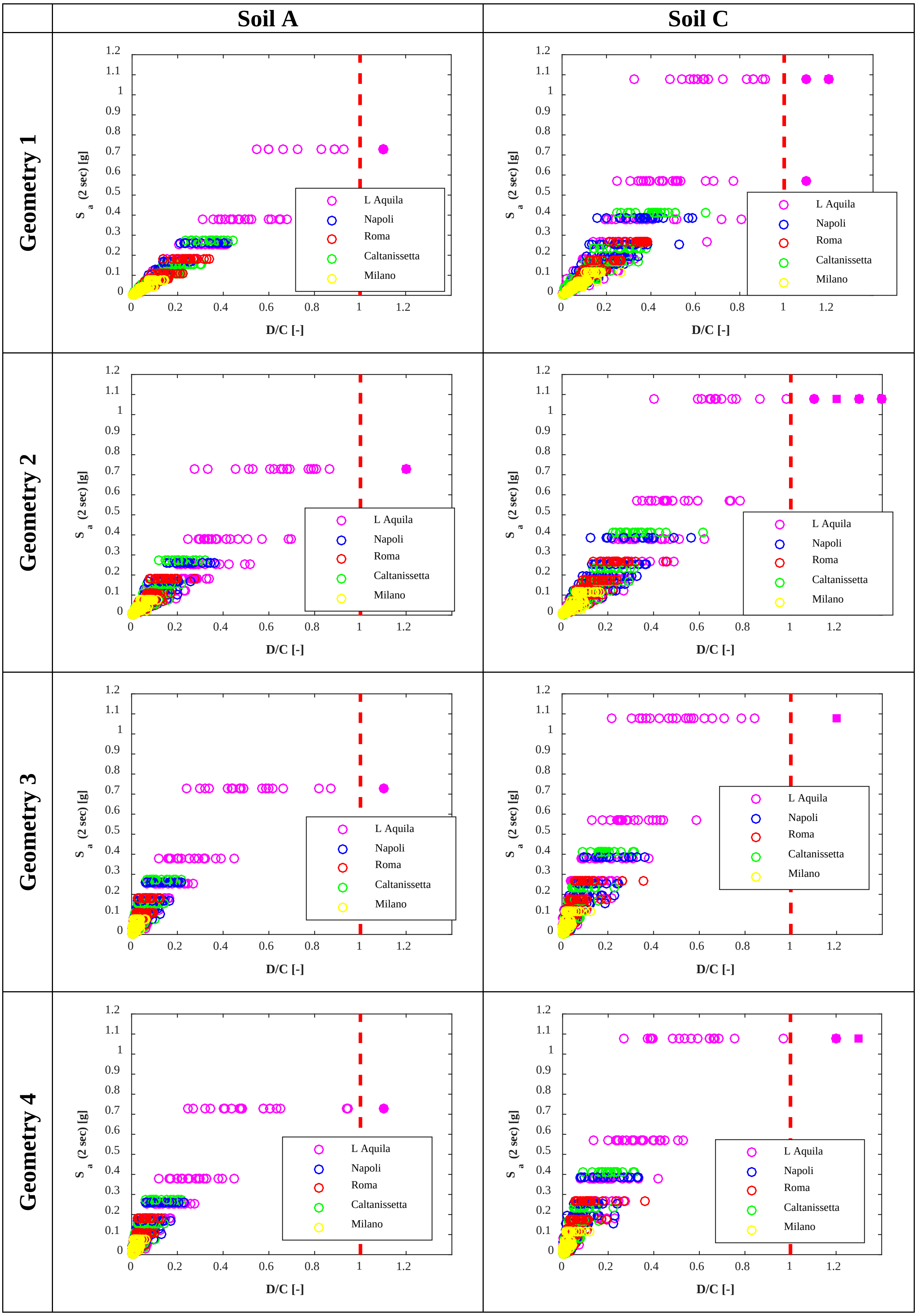


Figure 6 Results of the multi-stripe analysis: D/C ratios along with the spectral accelerations (T=2sec)

CONCLUSIONS

A parametric study investigates the seismic performance of industrial single-story RC precast buildings, designed according to modern building codes. Multiple stripe analyses are performed for ten intensity levels in five seismic-prone areas in Italy. The results of the nonlinear dynamic analyses demonstrate that the capacity of the structures is quite constant with the geometry and it is lightly influenced by the seismicity of the site because of the seismic details and the design overstrength. Moreover, a large overstrength of the structures caused very low value of the Demand /Capacity ratios for many case-studies. The overstrength decreases with the site seismicity increases; in the case of low seismicity, the seismic demand on structures can be very low even for an intensity measure with a return period of 100000 years. Only for the case-studies in L'Aquila on soil C at the maximum intensity level some collapses occurred.

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