RESPONSE TO THE REVIEW #3 OF MANUSCRIPT NUMBER: NHESS-2022-93

We thank the reviewer for these numerous and detailed comments. Responses to these comments are provided below in red.

The paper investigates the collapse of a steel building, built in 1982 in Montpellier in the south of France, under snow and rain loads occurred in 2018, providing detailed information of the meteorological event, its features, influence on snow accumulation on the building as well as on the subsequent rain event, heavily affecting the snow density and the resulting load acting on the roof.

The paper continues with the FE modelling of the structure, to simulate the collapse condition trying to estimate, by means of back analysis, the actual load intensity which led to the collapse.

On the following parts clarifications are needed.

RC3#1. In Annex C it is stated that the structure is not known in detail and some simplifications and assumptions on the real geometry are introduced in the FE model, the influence of which in the results is also checked by means of a “virtual”. This kind of assumptions may significantly affect the validity of the FE results and more explanations are needed. In particular, a detailed list of missing information should be added, commenting on the potential impact of the induced uncertainty in the FE model. Some drawings showing the structure and its elements (cross sections, dimensions, etc.), possibly from the time of the construction, could help in better understanding the structural behaviour.

The elevation plans of the building that could be recovered from archives do not explicitly show the structure of the eastern façade, covered by cladding. Only the sections of the columns surrounding the doors are provided; these are the sections that have been taken into account for all the columns of the eastern façade in the FE model. Following your remark and the comments of the other reviewers, we looked for new information about the geometry of the structure and ended up finding a top view of the building on which the characteristics of the supporting columns of the eastern facade are provided. They are in fact HEA 160 profiles. New simulations that take into account this information have therefore been carried out. So, we propose (i) to integrate these new data and the results in a revised version of the manuscript, including more details on the dimensions and sections of the eastern facade elements, and (ii) to remove the Annex C about the virtual model of the structure (which thus becomes much less relevant).

RC3#2. Steel properties are reported in Table 1, clearly referring to nominal values for S235 steel. In a static non-linear analysis, the actual mechanical properties of steel play a fundamental role in the determination of ultimate loads leading to the structural collapse. A clarification on this aspect should be introduced, possibly referring to test results on specimens extracted from the steel members after the collapse or, at least, by making reference to mean values of resistances instead of characteristic values, as it is the case in Table 1.

Unfortunately, no tests have been carried out on steel elements after collapse. This is why the steel characteristic values were used in the FE model. We agree that these values, chosen to (somewhat) balance the fact that the initial state of the structure is considered perfect in the FE model, are pessimistic. As you and another reviewer suggest, we performed new simulations that take into account the mean values of the steel strengths according to the new Eurocode for design of steel structures: fy = 1.25 x 235 = 294 MPa and fu = 1.2 x 360 = 432 MPa and an ultimate strain εu = 20%. The results of the new simulations will be presented and discussed in the revised version of the manuscript. Preliminary results, that take into account those new values of the steel behaviour and the new characteristics of the eastern facade, are presented further (see RC3#6).

RC3#3. The mesh sensitivity study, mentioned in line 170-175 and illustrated in Annex A, does not seem appropriate for a truss system, with hinged beams.

As mentioned in lines 188-189 of the original manuscript, the roof frame elements are either welded or bolted together and not hinged. This point will be further emphasized in the revised version of the manuscript.

RC3#4. Collapse criteria illustrated in 3.3 (lines 195-203) are not clear, as it could be interpreted that the collapse is reached as soon as one steel beam yields or reaches the ultimate strength (which is then not expected to occur as this happens only after yielding). In pushover analyses the final collapse mechanism is identified under non-linear static analysis under increasing loads, which is not evident in methodology illustrated in the paper. A clarification is needed.

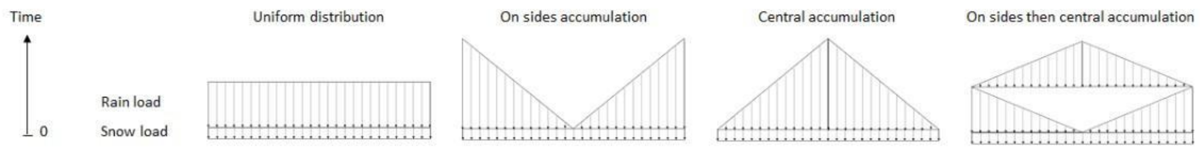
Collapse criteria as referred in the manuscript are indeed satisfied when the steel yield or ultimate strengths are reached in at least one cross-section. As you are pointing out, it therefore does not correspond to the actual collapse of the structure but rather indicates an initiation of deterioration that can then have a significant impact on the structure. This point will be clarified in the revised version of the manuscript and the term 'collapse criteria' will be systematically replaced by the more appropriate term 'failure criteria'.

RC3#5. Among the collapse criteria no mention is made on buckling of compressed members, which as expected and as confirmed by the photos of the collapsed structure, has occurred. Buckling anticipates the failure of members with respect to the uniform compression till yielding and this aspect could lead to a significant reduction of the ultimate load in the FE analysis. A clarification is needed.

We thank the reviewer for this comment. In so far as the photos of the collapsed structure show compressed elements that have undergone buckling and even if this buckling is not necessarily the cause of the rupture and may have occurred during the collapse, it is important to present an analysis of this failure mode. We therefore propose to introduce the criterion of normal displacement equal to H/150 for each of metal columns. This analysis will be presented in the revised manuscript.

RC3#6. Considering the flexibility of the structural system of the roof, the assumption of the uniform distribution of the snow load, and moreover of the rain load, all over the roof surface is a strong assumption, which could lead to wrong unconservative results. This aspect is only mentioned as a limitation of the analysis, but should be better illustrated as ponding effects could have caused a significant redistribution of the load, with concentration in the centre of the roof area, i.e. where its effects are more onerous for the system.

We remind here that the roof is not really flexible, because its frame elements are either welded or bolted together. However, we agree that the assumed uniform load distribution for the total load is maybe too simplistic. As indicated in the response to reviewer 1 (who also pointed out this point), we think that the distribution of the initial snow load was nearly uniform, because of the low slope of the roof (1% slope on each side of the ridge line of the roof facing north-south) and of the very little wind during the snowfall. Then, rain probably first accumulated on the west and east edges of the roof until the direction of the roof slope reversed due to the increase of the deflection. After that, the rain accumulated in the center of the roof. Thus, rather than considering only one scenario of uniform distribution for the total load, we propose to study four different cases of pressure distribution, as shown in the figure below, for snowfall of 30 and 35 cm.



The analysis of results of these different scenarios will be presented in the revised manuscript. Preliminary results are presented here, corresponding to loads leading to the different damage criteria for a 30 cm snowfall.

Failure criterion	Loads (N.m-2)		
	Uniform	Central	On sides
ymax=0.225	1360	1210	1660
ymax=0.27	1660	1415	2090
Elastic limit	1320	1330	1255
Ultimate limit	Not met	Not met	Not met

RC3#7. Based on the above comments the discussion of the FE results in 4.4 may need to be reconsidered.

As it is proposed to modify the eastern facade of the FE model (see response to your comment 1) and the steel behaviour law (see response to your comment 2), as well as to integrate a collapse criterion on buckling (see response to your comment 5), the discussion of the FE results will be carefully reconsidered in the revised manuscript.

RC3#8. Paragraph 4.2. It is claimed that the structure respected the structural design codes at the time of the construction as well as at the time of the collapse (2018). Later on in the Appendix it is stated that the SLS limit states were not verified. The particular structural scheme, a steel 3D truss plate with no intermediate supports, is particularly prone to deformation effects, which generally end up in governing the design. Some more details on these aspects are needed, to better understand the validity of the drawn conclusions about the compliance with the design standards.

We agree that the conclusions about structure's compliance with design standards are unclear. In fact, the structure was deemed to respect the structural design codes at the time of the construction but not totally at the time of the collapse. Indeed, according to Eurocode, the ultimate limit states were respected but not the service limit states. The simulations presented in the manuscript show that yield occurs for a load of 1 000 N.m-2, which is less than the exceptional load recommended by Eurocode (equal here to 1 280 N.m-2). Moreover, the service limit state corresponding to an excessive curvature is reached for a snow load of 1 275 N.m-2, which is largely above the value for a permanent project situation (that corresponds to a snow load equal to 640 N.m-2) but of the same order of magnitude as the one for an accidental project situation (that corresponds to a snow load equal to 1 280 N.m-2). We will revise the manuscript accordingly, to clarify these aspects. The conclusion about structure compliance with design standards will be qualified and more explicit and will take into account the new simulations results.

RC3#9. Line 315: recent climate models provide also snow variables, such as snow depth or SWE.

We are aware of convection-permitting climate models which are able to simulate high-resolution meteorological variables, including snow variables such as snow depth or SWE (https://doi.org/10.1002/joc.7637). However, to our knowledge, these simulations are not available in the south of France where snow events are rare. These simulations have been obtained in priority in high-latitude or mountainous regions (e.g. the pan-alpine region).

RC3#10. In the conclusions it should be better highlighted which are the main outcomes of the study, i.e. which sort of recommendations are proposed by the Authors also in view of the revision of structural design standard or for the analysis of existing buildings.

We thank the reviewer for this proposition and we will follow it by wrapping up the main outcomes and recommendations in the concluding section of the revised manuscript.