BACK-ANALYSIS OF AN AVALANCHE DESTRUCTIVE IMPACT ON BUILDINGS

Valerio De Biagi*, Bernardino Chiaia, Barbara Frigo
DISEG, Politecnico di Torino, Italy

ABSTRACT: This paper aims to perform a back-analysis of a snow avalanche event occurred in the Italian Alps on December 2008, in order to estimate impact pressure of a dense avalanche on buildings.
In the second decade of December, 2008 the Italian Alps underwent extreme meteorological conditions, i.e. large snowfalls coupled with significant wind accumulation effects. Many extreme snow avalanches, seldom or never observed in the past, naturally triggered in that occasion. On December 15th, 2008 a thick and soft snow slab of about 50000 cube meters released from the upper part of a big wall and impacted a series of chalet houses of Les Thoules in the municipality of Valsavarenche, Aosta Valley. The avalanche totally destroyed seven houses and cut-off the infrastructures (roads, power and telephone lines) for many days.
The damage effects caused by the avalanche are here investigated and analyzed under a structural framework. Following inverse forensic approaches based on collapse mechanisms and debris arrangement, an estimation of the dynamic pressure and impact direction is performed. A comparison the results of the back-analysis is done and suggestions are proposed.

1. INTRODUCTION

In December 2008, northwestern Italian Alps underwent extreme meteorological conditions, i.e. large snowfalls coupled with significant accumulation winds, [Maggioni *et al.* (2009)]. Many extreme snow avalanche triggered and, occasionally, interested built-up areas of mountains. This is the case of “Les Thoules” snow avalanche in Valsavaranche (Aosta Valley, IT). The effects of the impact of snow avalanches are important both for understanding avalanche dynamics, and for managing avalanche hazard, in particular in case of possible damages of public services (i.e. transportation, infrastructures) [Bovet *et al.* (2011)].

Back-analysis represents one of the most effective ways to understand natural phenomena. The analyses of damages generated by snow avalanches can be very valuable to improve the knowledge of the avalanche pressure against a structure [Margreth and Ammann (2003)]. This process may, moreover, lead to a better evaluation of the risk, De Biagi *et al.* (2012a).

In this paper, the damages of “Les Thoules” snow avalanche on four constructions (both partially damaged, and totally collapsed) are analyzed under a structural framework following forensic approaches and surveys onsite. At the end, some considerations are drawn and a comparison between numerical simulations and the results of the back-analysis is done.

**Corresponding Author Address*: Valerio De Biagi, Department of Structural, Building and Geotechnical Engineering, Politecnico di Torino, C.so Duca degli Abruzzi, 24, I – 10129 Torino. Tel: +39 011 0904889, email: valerio.debiagi@polito.it

2. “LES THOLUES” SNOW AVALANCHE

Very steep mountain slopes characterize Valsavarenche, a valley in the region of Aosta, northwestern Italian Alps. On December 15th, after heavy snowfalls (150 cm of fresh snow at 2000 m a.s.l.), a thick and soft snow slab of about 50000 cube meters released from the slope “La Tour” and stopped with a big jump over Les Thoules village at 1600 m a.s.l.. The flow split into two branches and impacted the built-up area. The left-hand side branch destroyed four houses whereas the right-hand side one impacted two chalets and one house. Hopefully no injured were recorded. The avalanche cut off telephone and power lines, and the traffic on the main road was stopped for many days.

Figure 1 shows a map of the study area. The four houses are highlighted and numbered in red. Construction no. 1 was partially damaged; construction no. 2 was seriously damaged. On the contrary constructions no.3 and 4 were totally destroyed by avalanche flow. All the considerations herein made were preceded by a detailed survey on the first days following the event and by the retrieval of the available drawings of the buildings.

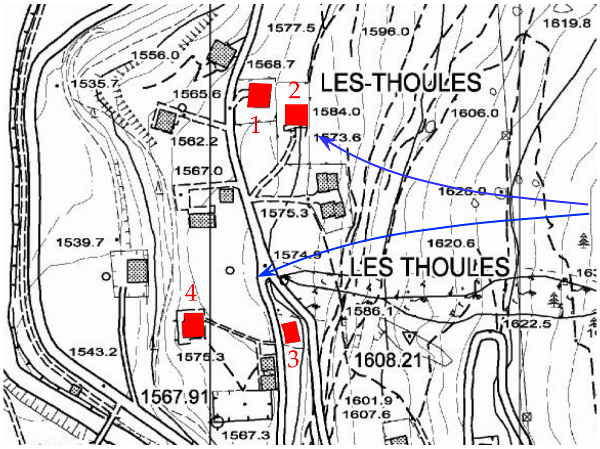

Figure 1: Map of "Les Thoules" village. Blue arrows indicate avalanche path with its two branches. Red marks and numbers indicate the back-analyzed constructions, source RAVA.

3. CONSTRUCTIONS NO.1 AND 2

Constructions no.1 and 2 had similar architecture. They can be classified as traditional wooden houses ("chalet") resting on a concrete basement with large terraces. Two separate parts composed the buildings: a lower concrete basement and an upper part (with two floors) made of softwood beams blocked each other by carved hinges (*blockbau* technique). At the corners of the buildings, solid wooden column were fixed. Construction no.1 had an internal concrete pillar that supported the roof. In both houses, the timber roof was covered by "lauzes", a foliated rock traditionally employed in mountain areas.

As shown by post-event pictures, avalanche flow impacted first construction no.2 in its north-east corner and caused a combined rotation and translation of the wooden upper part. Then, the flow was deviated and stroke construction no.1 in its north-east corner. Since construction no.2 suffered large damages, construction no.1 was interested by the local collapse of part of the roof.

For the back-analysis of construction no.1, the roof dynamic is modeled. A plastic hinge on top of an 8 meters timber beam caused the rotation, θ, of part of the roof. The dynamic equilibrium can be simply written as

$$I \times d^2\theta/dt^2 = 4 \times mg \qquad (1)$$

where I is the inertia around the hinge, m is the mass of the roof. For a given mass-inertia configuration, the collapse time can be computed. The evaluation of the amount of snow packed into the house in the collapse time gets an estimate of the flow rate and velocity.

Referring to construction no.2, a rigid body motion is supposed. Translational and rotational dynamic equilibrium equations are written. The pressure due to avalanche interaction represents the active force, while the resisting components are the ones due to friction on the interface wood-concrete. The most important aspect in the back-analysis of construction no.2 is the fact that the initial and final positions are known (the latter from a photographic survey). In that sense, it is possible to associate to each value of impact duration a specific value of impact force, which is able to produce a given displacement (translation + rotation) in a prescribed time, via the following system of equilibrium equations

$$m \times d^2x/dt^2 = F - \mu mg$$
$$I \times d^2\theta/dt^2 = Fb - \Sigma M_{w,I} \qquad (2)$$

where F is the impact force at a distance b from the center of mass of the system, m the mass of the upper part of the construction, I its rotational inertia, $M_{w,I}$ is the resisting torque due to friction of walls on the concrete basement.

The results of the back-analyzes on the two constructions estimate an average impact pressure of 54.5 kPa and a flow velocity of about 13.2 m/s. Further details on damages, modeling and calculations can be found in Bovet *et al.* (2011).

4. CONSTRUCTION NO.3

Two different parts composed construction no.3: an upper one-story masonry structure with no pillars, a lower concrete basement. The plans showed a 30 cm thick structural masonry with large openings. Plan dimensions of the house were 8.10 x 5.10 meters, with the larger side facing towards avalanche path. Architecturally, the rooms were separated by partition walls which acted as stiffeners. During the avalanche event, the whole upper masonry part collapsed and debris were scattered by avalanche flow downwards across a large area. As a result of a detailed survey on the remaining part of the construction, no bricks or mortar were found in correspondence to shear walls. Only fractured clay elements, with inclined rupture surface, were found on the impact wall. The concrete basement was partially damaged, i.e. part of the floor collapsed.

The structural back-analysis conduced on the event considers two different modes of rupture. The former supposes that longitudinal walls (with respect to the avalanche path) acted as shear resisting elements. The latter supposes that the internal partition walls stiffened the whole upper part, which behaved like a rigid body on the concrete basement. Horizontal and rotational equilibrium, respectively, give an estimate of impact pressure on the construction. In particular, the unknown parameters are the impact pressure, *p*, and avalanche flow depth, *h*. Due to house position and the surrounding terrain, avalanche flow is supposed to impact the house at a minimum height of 2 m with a vertical angle of 10°. Masonry mechanical properties were evaluated from the values available in literature (EN1996-1), in particular we suppose

$$\tau_{vk0} = 0.18 \text{ MPa}. \quad (3)$$

A sketch of the structure is proposed in Figure 2. Total weight at upper-lower part interface is estimated from original drawings in 820 kN, with an average vertical stress of about

$$\sigma_v = 0.11 \text{ MPa}. \quad (4)$$

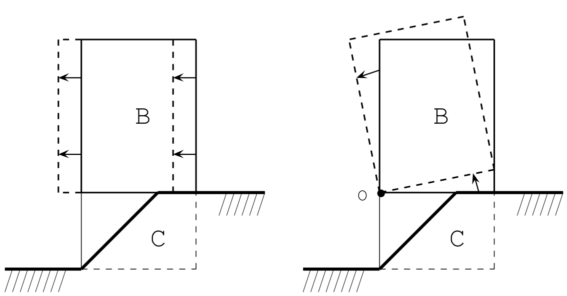

Figure 2: Sketch of the two collapse modes. The upper part made of bricks is (B), the lower concrete basement is (C), thick line represents ground profile. On left-hand side the translational collapse mode (Coll.mode #1), on right-hand side the rotational collapse mode (Coll.mode #2), with pole O.

Referring to the first collapse mode, a basal sliding has been supposed to better describe the collapse of the construction. Thus, Mohr-Coulomb criterion is applied to the interface between the longitudinal shear walls and the basement. Turnsek – Cacovic model [Turnsek and Cacovic (1971)], has not been considered because of the absence of residual parts of the structure, which have suggested a bulky-like behavior. The limit value, τ_{vk}, considering a friction coefficient equal to 0.40 [EN1996-1], is

$$\tau_{vk} = \tau_{vk0} + 0.40\ \sigma_v = 0.22 \text{ MPa}. \quad (5)$$

Shear walls are, in plan, 3.06 square meters large. Therefore, a total horizontal force of 856.8 kN is required to activate this first collapse mechanism. The equilibrium condition implies that

$$p \times h \times 8.10 = 856.8 \text{ kN}, \quad (6)$$

since 8.10 m is the width of the impacted wall. On the other hand, the presence of inner partition walls that acted as stiffeners, induced a rigid-body behavior of the whole upper part of the construction. In that sense, a rotational equilibrium around pivotal axis “O” of Figure 2 is imposed in order to get the impact pressure. Referring to Figure 2, the stabilizing torque,

$$M_s = 820 \times 2.55 = 2091 \text{ kNm}, \quad (7)$$

is represented by the weight of the building acting in its center of mass; the destabilizing torque,

$$M_d = [p \times h \times 8.10][h/2 + 2], \quad (8)$$

is represented by the resultant force due to the impact applied in its center of distribution. For equilibrium, the sum of all torques has to be equal to zero, i.e.

$$M_s - M_d = 0. \quad (9)$$

As reported in Table 1, it is possible to compute the impact pressure which activate the two collapse mechanisms for a given flow depth, *h*.

As a result of the survey performed on the site, some tree branches and light spots taller than 3 m were not broken by avalanche flow. Therefore, we set 3.00 m as an upper limit value of flow depth. In that sense, 25 kPa can be considered as a lower estimate of impact pressure, as shown in Figure 3.

h [m]	Coll. mech #1 [kPa]	Coll. mech #2 [kPa]
0.50	168.74	233.01
1.00	84.37	104.85
1.50	56.25	63.55
2.00	42.18	43.69
2.50	33.75	32.26
3.00	28.12	24.97
3.50	24.11	19.97

Table 1: Impact pressures related to the two different collapse modes considered.

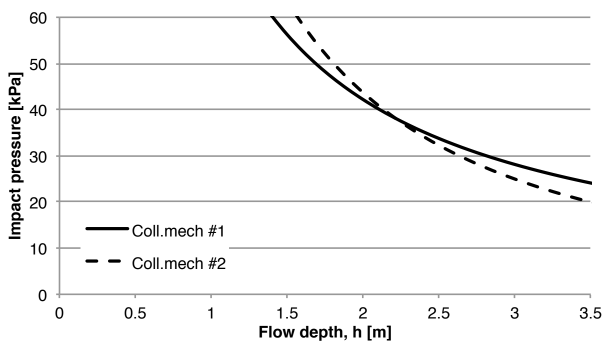

Figure 3: Impact pressures *vs* avalanche flow depth for the two considered collapse mechanisms.

5. CONSTRUCTION NO.4

Construction no.4 was a large structure constructed on a concrete plate on the edge of a scarp. It was composed by two floors and by a basement partially opened with a terrace. The plans and the surveys showed a 50 cm thick structural masonry made by stone blocks and clay bricks. Large windows were opened in the bearing walls. Floors were in reinforced concrete and there were two 50 x 35 inner pillars, which support floor beams and the timber roof covered with “lauzes”. Maximum plan dimensions of the house were 12.5 x 10.2 meters, with the large side facing towards avalanche path. Since the plan was irregular, there were four shear walls. Further to avalanche impact, the house was totally destroyed and debris were carried down the scarp. Only few parts of the retaining wall were found intact. A survey on the remaining parts showed the poor quality of the construction and the presence of hollow bricks, unsuitable for structural walls.

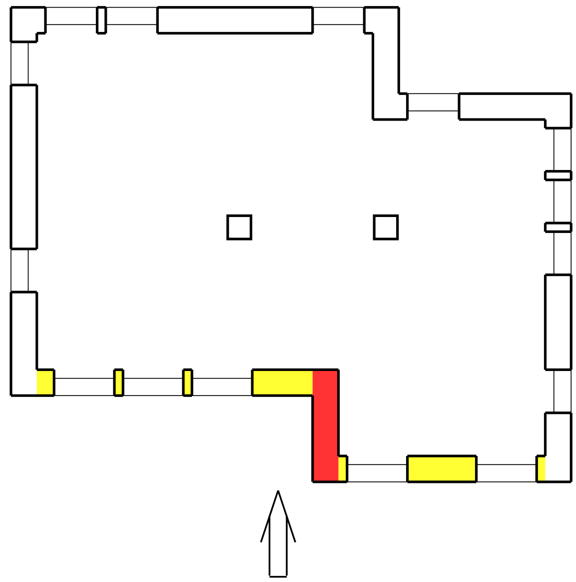

Figure 4: A sketch of the first floor of the construction. Thick lines represent walls cross-sections. The frontal walls of rupture mode #1 are in yellow; the shear wall of rupture mode #2 is in red. Note the large openings. The arrow indicates avalanche flow direction.

The structural back-analysis considers two different modes of rupture. Since the structure is large, we can easily imagine that avalanche caused a local damage, which evolves into a global collapse. The former local collapse is a large out-of-plane displacement of impacted walls, the yellow ones in Figure 4. The latter supposes that one of the shear walls, in particular the one in red in Figure 4, was not able to sustain the force exerted by the avalanche on the walls. Since we refer to local damages, in order not to create misunderstandings with the previous results, we prefer to call these local collapses as rupture modes. The unknown parameters are the impact pressure, p, and avalanche flow depth, h. The presence of a snow layer lying on the ground presupposes that avalanche flow did not impact directly the first meter of the construction.

Masonry mechanical properties were evaluated from the values available in literature [EN1996-1], in particular we suppose

$$\sigma_k = 1.92 \text{ MPa}, \quad \tau_{vk0} = 0.20 \text{ MPa}. \tag{10a-b}$$

Referring to rupture mode no.1, a mechanism within the external walls is supposed. In particular, as illustrated in Figure 5, three hinges form: two at the ends of the panel, the other in the middle point. The described above hinges configuration represents a kinematically admissible collapse mechanism. Therefore the hypothesis of the upper-bound limit theorem is satisfied and an estimate of the collapse load can be found. This mechanism can form around windows openings, where the aspect ratio of the wall is high. Vertical gravity forces distribution give an axial force equal to 75.75 kN/m at top hinge, 89.27 kN/m at middle hinge, and 104.80 kN/m at bottom hinge. Considering that masonry cracks in tension, the resisting bending moments are 16.94 kN, 19.55 kN, and 22.39 kN, respectively [Tassios (1986)]. Referring to Figure 5, the work performed by external forces, i.e. avalanche impact force, is equal to

$$W_{ext} = [1/2 \times p \times h^2]\ \varphi, \tag{11}$$

where φ is the rotation angle with respect to the vertical. Similarly, the work performed by the internal forces is the product of the rotations and the resisting bending moments, i.e.

$$W_{int} = [16.94 + 2 \times 19.55 + 22.39]\ \varphi = 78.43\ \varphi. \tag{12}$$

Since internal and external works are equal, as a result of the theorem, for any value of flow height, *h*, the impact pressure is found.

Referring to rupture mode no.2, which account for the resistance of the central shear wall, Mohr-Coulomb failure criterion is applied to the masonry. In particular, the average vertical stress, σ_v, is equal to 0.21 MPa, thus the shear resistance is assumed equal to

$$\tau_{vk} = \tau_{vk0} + 0.40\ \sigma_v = 0.28 \text{ MPa}. \tag{13}$$

The total resisting shear, V_r, at the base of the shear-wall is equal to

$$V_r = 0.28 \times 10^3 \times 2.00 \times 0.50 = 280 \text{ kN}. \tag{14}$$

Considering a 6 m wide influence area, the translational equilibrium gets the impact pressure.

As reported in Table 2, it is possible to compute the impact pressure, *p*, which activates the two rupture mechanisms for a given flow depth, *h*.

It follows that rupture mode no.1, related to out-of-plane displacements, is activated for lower impact pressures. This aspect can be shown in the plot of Figure 6. Since the supposed flow depth is smaller than 1.50 m, the impact pressure should have been larger than 12 kPa.

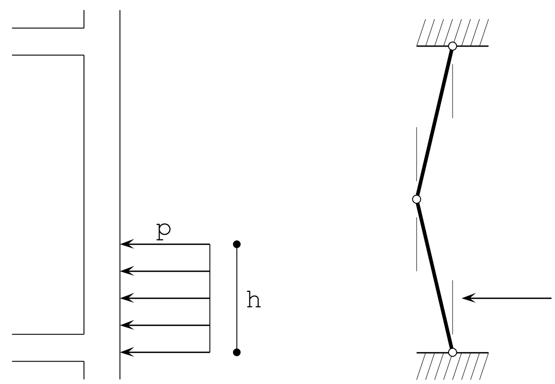

Figure 5: Rupture mode no.1. The mechanism is detailed in right-hand sketch, the rotation angle of each hinge is equal to φ.

h [m]	Rupt. mode #1 [kPa]	Rupt. mode #2 [kPa]
0.50	23.78	93.33
1.00	15.62	46.67
1.50	11.21	31.11
2.00	8.50	23.33

Table 1: Impact pressures related to the two different rupture modes considered.

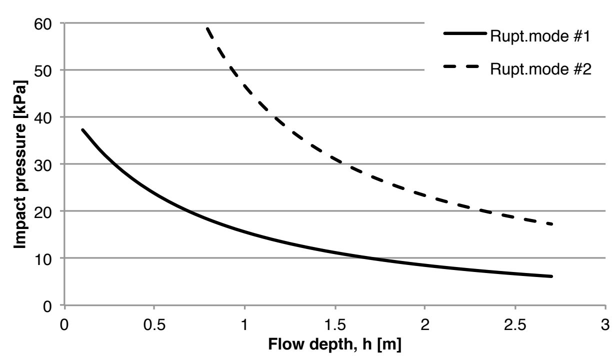

Figure 6: Impact pressures *vs* avalanche flow depth for the two considered rupture modes.

6. CONCLUSIONS

As a result of the back analysis, the following considerations can be drawn. First, two constructions were totally damaged. The limit impact pressures found for the activation of collapse/rupture mechanisms, even if computed by means of upper-bound theorem, still have to be considered as lower-bounds of the impact pressure. On the contrary, the results of the back-analysis conduced on the partially damaged constructions no.1 and 2 can give an upper-bound estimate of the impact pressure.

Following the results of the present investigation, the impact pressures due to avalanche event of December 15th, 2008 on Les Thoules village range from 12 kPa (for construction no.4) to 54 kPa (for construction no.2). Although the lower value is smaller if compared with the magnitude of damages recorded onsite, the quality of construction was definitely poor, i.e. low impact pressures cause the rupture. For these reasons, it is better to consider a range of impact pressures oscillating between 25 – 54 kPa, which has more sense for the size of this extreme avalanche event.

As an outcome of the proposed work on a real event, we can draw some general considerations. First, there are many possibilities for performing avalanche studies through structural back-analyses. The results can be the base for risk assessment and a support to local Authorities in their decisions. Then, the studies on avalanche impacts on constructions can help technicians in the design of rehabilitation interventions on damaged buildings [De Biagi *et al.* (2012b)]. At the end, in order to conduce an effective back-analysis on an avalanche event, it is important to consider both damaged and undamaged elements. The estimated impact pressure on the former represents a lower bound of the real value. On the contrary, the latter gives and upper estimation of the real magnitude of the event.

Future developments on back-analyses related to avalanche events might consider breaks in natural elements, like trees branches, for the assessment of flow pressures and velocities.

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