A reliability-based method for taking into account snowfall return period in the design of buildings in avalanche-prone areas

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Valerio De Biagi, Monica Barbero, Mauro Borri-Brunetto
DISEG Politecnico di Torino, Torino, Italy

For many natural hazards, design codes include simple approaches, based on semi-probabilistic methods, which allow the engineer to check the structural safety of the construction. This framework implements the socio-economic optimization of the resources to be used to build construction works. At present no simple approaches have been formulated for snow avalanche hazard. Recent research focuses on fully probabilistic assessment of structural safety, aimed at the estimation of the failure probability of the element at risk. This strategy requires a large amount of data and complex simulation techniques. In order to propose the implementation of snow avalanche hazard into the framework of modern design codes, we suggest an approach based on different avalanche scenarios with different snowfall return periods depending on the consequences of failure of the designed construction.

1 Introduction

Any structure, or structural element, should be designed in such a way that it is suited for the use during its design working life. In other words, it has to adequately resist the expected actions, to withstand extreme actions, and not to be damaged by extreme natural events in a way disproportionate to the original cause [1]. These are the basic principles of a reliability-based structural design [2, 3, 4, 5]. Standards, like Eurocode, ASCE rules, as well as National Prescriptions, have implemented these principles in such a way that, nowadays, the design of a structure follows a probabilistic framework, rather than a deterministic one. For example, wind speed has been measured during a given period, and a characteristic value corresponding to a given exceedance probability has been assigned to each site [6].

Actions are divided into three classes: permanent (acting continuously, i.e., dead loads), variable (for which the variation of magnitude in time is small if compared to the mean value, i.e., wind loads), and accidental (actions unlikely to occur with a significant magnitude during the life of the construction). A natural phenomenon is classified either as a variable or accidental load, depending on site conditions. For example, in the majority of European countries, high speed winds are considered as extreme meteorological events, thus accidental actions. On the contrary, in US tornado-prone areas, cyclones are regarded as variable actions [1]. Similarly, earthquakes have been extensively studied and are considered as variable actions. For each site, a ground motion intensity is related to the probability of occurrence of the natural event and, thus, a probabilistic approach to the design of the construction is possible.

In general, risk analysis considers the probability of a structure to be damaged one or more times in a given number of years, ℓ, by a natural hazard having a known return period. If the hazardous events are independent between the years the probability of occurrence of an event, P_e, with a magnitude m larger than a reference value M, i.e., $P_e = p\,(m \geq M)$, is related to the return period (measured in years) of the threshold event, T, by the following expression [7, 8]

$$P_e = p\,(m \geq M) = 1 - \left(1 - \frac{1}{T}\right)^{\ell}. \tag{1}$$

The previous expression is the probability of at least one occurrence, i.e., one minus the probability of no occurrences. At present, the loads generated by snow avalanches are considered as accidental, independent of the geographical region in which the structure is built. Different return periods events should be considered depending on the use of the construction in order to make the *reliability differentiation* possible. In its most general definition, the reliability is the ability of an item to perform a required function under stated conditions for a specified period of time. The reliability differentiation is the set of measures intended for the socio-economic optimization of the resources to be used to build construction works, taking into account all the expected consequences of failures and the cost of the construction works [9].

In the framework of the current methods for assessing structural safety, reliability evaluation can be performed at different levels of accuracy and complexity. Fully probabilistic approaches, leading to the calculation of the probability of failure for a given structure under certain actions, amount to multi-dimensional integration in the space of certain random variables describing the statistics of actions and strengths (e.g. [9], Annex C). These methods require a deep knowledge of the statistics of both actions and material properties, which are usually unavailable to the structural engineer involved in the design of an ordinary construction. To overcome this lack of knowledge, regulations and building codes are based on the so called *semi-probabilistic method*, which introduces characteristic values of both actions and material properties and partial safety coefficients. In this framework, the structural safety is checked through simple inequalities; i.e., for each structural member, the resistance must be larger than the force resulting from the applied external loads.

The aim of this paper is to propose a first step towards the implementation of a semi-probabilistic procedure for checking the safety of constructions impacted by snow avalanches, following approaches common to other natural hazards.

In what follows, the current practice for evaluating design loads in presence of natural hazards is summarized (Section 2), with regard to areas affected by snow avalanches. The proposed strategy for reliability differentiation in snow avalanche design is then described (Section 3) and the introduced improvements discussed (Section 4).

2 Reliability-based analysis and importance factors

From the point of view of structural safety, buildings and other constructions should be classified on the basis of the risk to human life, health, and welfare associated with their damage or failure, and by the nature of their occupancy or use [10]. Thus, major care has to be paid to those constructions in which the number of occupants and their exposure are higher (greater is the exposure, higher is the risk). As a design principle, design rules foster reliability differentiation: taking as example seismic excitation, a temporarily occupied construction should be designed to resist events with lower return period, i.e., lower intensity, than the one related to permanently occupied buildings. As a result of the previous consideration, design codes identity different consequences of failure or malfunction of a structure related both to the loss of life and to economic, social or environmental consequences. Table 1 illustrates the consequence classes which relate to the occupancy of the building [9]. In seismic design, the previously mentioned classification turns into the importance of the activity performed in the building [11]. That is, constructions devoted to public safety and hospitals, in case of earthquake, and despite their size, have higher importance than ordinary structures (houses, hotels) even if highly occupied (see Table 2). Infrastructures, such as roads and railways, are classified similarly.

For any structure (or infrastructure) subjected to a natural hazard, the reliability, R, can be calculated as

$$R = S - A \tag{2}$$

where S and A are the variables expressing the resistance and the action, respectively. Considering that both the variables are expressed through probability functions, R would be probability-based [13, 14]. In this sense, R can be negative. Structural safety requires that the probability of having $R < 0$ must be kept lower than an annual threshold value. Depending on the type of structure and the characteristics of the considered failure mode the threshold value varies between 10^{-3} and 10^{-6} per year. In the most common situations the following inequality should hold [15, 16, 17]:

$$\Pr(R < 0) < 10^{-5} \text{ per year.} \tag{3}$$

Clearly, since reliability requirements can be different for different limit states to be considered, e.g., serviceability is less stringent than structural failure, the threshold value differs from case to case.

The principle of reliability differentiation is achieved by varying load intensity. For example, the increment (or decrement) of load magnitude can be achieved by multiplying the design load by a factor, γ_I, usually greater than one (e.g., seismic analysis in ASCE 7 [10, Ch.1] and in EN 1998 [11]). The reference peak ground acceleration, a_{gR} corresponds to the reference return period of the seismic action for the no-collapse requirement (2450 years). The design ground acceleration on outcropping bedrock, a_g, is equal to

$$a_g = \gamma_I a_{gR}. \tag{4}$$

where γ_I is the importance factor related to the consequences of a structural failure. For CC2a constructions, $\gamma_I = 1.0$. This strategy is not exclusive to seismic design. For example, static snow load is the result of the product of ground snow load and an importance factor depending on the occupancy of the building [10, Ch. 7].

A different approach is proposed by the Italian seismic rules [18], where new terms to describe seismic hazards and seismic actions on structures were introduced to associate to each consequence/importance class a design reference event with a different return period [19]. The importance of the construction affects the final value of the ground acceleration through its reference life [18]. The probability of occurrence of the earthquake, which can be described by means of an extreme-values distribution, obviously depends on the length of the period during which the construction is in use, i.e., the design working life of the structure, L_D (see Table 3). The more important the construction, the longer its reference life, L_R, is. In other words, the design working life is corrected (either increased or decreased) by a multiplying factor, C_U, the coefficient of use, in order to consider the importance of the building, i.e.,

$$L_R = L_D C_U. \tag{5}$$

Defining the reference life as the period during which a structure can be experience an event with magnitude m greater than a reference one, M, Eqn.(1) turns into

$$P_e = p(m \geq M) = 1 - \left(1 - \frac{1}{T}\right)^{L_R}. \tag{6}$$

For example, in a reference period of 50 years ($L_R = 50$), the probability of occurrence of events having magnitude larger than an event with return period $T = 475$ years, is $P_e = 10\%$. This is the requirement for the Ultimate Limit State (ULS) invoked in European prescriptions [18, 11].

Usually, the target probabilities, which depends on cost and benefits analysis, are given by design rules. European and American guidelines give the performance objectives and identify the design events with the respective occurrence probabilities. In EC 8, P_e equal to 10% and 2% in 50 years refer to Damage Limitation State and Collapse Limit State, respectively [20]. In FEMA 356 [21], a 4 x 4 performance matrix is illustrated. Its rows relate to seismic events with a given probability of occurrence, while the columns refer to target performance levels. The Basic Safety Objective includes the same seismic excitations as the European prescriptions. In addition, enhanced (or limited) rehabilitation objectives provide higher (or lower) performance levels for given seismic hazard levels.

3 Snow avalanche impact design

In the same way as tornados in specific coastal areas of the US, one might assume the impact of snow as variable action. Since this natural hazard is common in alpine environment, it would be adequate to consider various performance levels (similar to seismic ones) for the interaction between structures and snow flow. The reference events should take into account both the occupancy and the importance of the construction, as described in the previous section. The approach herein described implements the reliability concepts suggested by EC 0 [9].

The occupancy of buildings in mountain areas is highly variable. Mountain resorts are full of people in winter season. On the other hand, mountain agricultural constructions are exclusively occupied in summer, when avalanche hazard is null. In this sense, serviceability standards have to be fulfilled instead of requirements for people safety. The previous consideration would necessarily implies a design oriented towards the reduction of construction costs, as fostered in the rules [9].

3.1 Current practice

Norwegian regulations state that new houses should not be built where avalanches fall more frequently than once every 1000 years [22]. Icelandic studies showed that different return periods compete to different risk for the constructions [23]. Swiss regulations related to the design and construction of buildings in snow avalanche hazardous areas act in two complementary ways. First, land planning is considered [24].

The Swiss procedure provides mapping criteria for dense-snow avalanches depending on impact pressures over return periods of 30 and 300 years. If the impact pressure is larger than 30 kPa, no construction work is allowed (neither the construction of building occupied only in summer). If the impact pressure is smaller than 30 kPa, the structure has to resist the forces caused by the interaction between flow and construction. The impact pressures estimated for both the previous cases are calculated for a snow depth in the avalanche release zone assumed to coincide with the snow depth precipitation in 3 days before the event, or 3-day snowfall depth, H_{72} [25, 24]. The choice of three successive days snowfall is due to the fact the damages are often caused by heavy snowfall over several days rather than a huge one-day snowfall [26, 27]. In [28], a comparison between different forecast variables, say new snow in a single day, in 3, 5 or 10 days, is presented. The new snow in three days seems the most appropriate for forecasting large and infrequent avalanches. Meanwhile, from the analysis of a database of avalanches along a single path in French Alps it results a weak correlation between the release probability and 3-day snowfall depth because major avalanches can be released after snowfalls that were not necessarily intense but sustained over several days [29]. The numerical simulations illustrated in [30] show that, given a snow depth, a change in friction coefficients implies a wide range of impact pressures.

Despite the previous considerations, a procedure that considers 3-day snowfall depth is currently applied in Italy [31] and in France. Avalanche hazard mapping based on these criteria requires as input the evaluation of H_{72} for T = 30 and T = 300, at least [32, 33]. Although the return periods are different, Jónasson and others [23] stated that Norwegian and Swiss reference events are roughly comparably to a return period of a thousand years Besides, there are area outside western Europe where neither weather records nor weather models are adequate to estimate H_{72} for planning purposes.

Besides land planning, the structure must be designed to resist an impact force due to a H_{72} release snow depth with return period equal to 300 years. At present, since 300 years return period is used independently from the use and the occupancy of the construction, regulations do not implement entirely the previous considerations on reliability differentiation.

The research conducted in the last decade focused on the development of an individual risk framework for snow avalanche hazard. The estimation of the individual risk due to snow avalanche hazard in densely populated areas is currently practiced in Iceland [34]. It was found that the average annual risk due to snow avalanches for people living in hazardous areas was five times higher than the risk due to traffic accident. The risk within the most endangered areas was much higher and thus unacceptable by any measure. Hence, a framework for the evaluation of the individual risk was developed [35] supporting the thesis that the reduction of the individual risk would reduce the aggregated risk to the society.

In 2010, Bertrand et al. [36] deterministically quantified the vulnerability of a simple reinforced concrete building (made by two orthogonal walls) considering the geometry and the percentage of steel reinforcement. Eckert et al. [37] used the results of [36] and presented a relationship between reliability-based failure probabilities and individual risks. In addition, they shown that the difference between vulnerability and fragility curves is not that important when they are used within a risk framework. Following that, Favier et al. [38] obtained a set of fragility curves for reinforced concrete walls exposed to a uniform and quasi-static snow avalanche pressure load. They accounted for different limit states: depending on the safety of people and the real collapse, they found different risk thresholds for the assessment of the structure. In their considerations, the winter usage of each building was accounted for. Focusing on individual risk, they showed that the usual tricentennial return period for the reference event may be seen as optimistic; in their calculations based on a set of various buildings, only events with return period larger than a thousand year are below standard risk acceptance levels [39].

3.2 Proposed practice

It must be mentioned that meteorological databases are the only continuous source of data related to avalanche [26]. In some cases, a record of the run-out distances of past avalanches is available; exceptionally, it is possible to have an estimation of the volume of snow involved, velocities, impact pressures from back-analysis on constructions or forests [40, 41]. Anyway, it is possible to numerically estimate the parameters of dynamic models of snow avalanche motion that fit the observations in the run-out zone, given the release volume through the statistics of meteorological records [42].

Following the approach illustrated in Section 2, a design strategy able to account for reliability differentiation is proposed. The use of different scenarios accounting for different snow depth is fundamental to the proposed design approach. Once this parameter is defined, the expert on snow engineering would perform numerical simulations, which consider both the topography and the roughness of the site, the presence of vegetation and the dynamical parameters, and would give an estimation of the impact pressure on the building. The effects of the impact are then evaluated through common procedures, according to the personal choices of the structural designer about methods of analysis and construction materials [40]. This approach follows the typical design flowchart for structures subjected to natural hazards and anthropic loads.

According to the current practice, an additional coefficient is introduced into Eqn.(5), which is rewritten as

$$L_R = L_D C_I C_A, \tag{7}$$

where C_A is a reducing coefficient. The purpose of this coefficient is illustrated in the following.

The performance objectives invoked by the American and European rules for seismic design [21, 11] and converted for avalanche hazard design are four:

- *Operativity limit state* (OLS): the event does not cause damage to the structure nor interruptions of the activities. The probability of exceedance of avalanche action during the reference life of the construction, P_e, is set equal to 50% [11];

- *Limit state of prompt use or Damage limitation* (DLS): the event causes slight damages that does not compromise the stiffness and the whole capacity of the structure. The activity has to be temporarily suspended since the apparatuses might be subjected to malfunctioning. The probability of exceedance of avalanche action during the reference life of the construction, P_e, is set equal to 20% [11];
- *Limit state for the safeguard of human life or Ultimate state* (ULS): after avalanche impact, the construction is affected by failures and collapses of nonstructural components. The construction retains significant stiffness and resistance against vertical actions and moderate resistance to horizontal actions. The activity has to be interrupted in order to make large rehabilitation works of the apparatuses. The probability of exceedance of avalanche action during the reference life of the construction, P_e, is set equal to 10% [11];
- *Limit state for collapse prevention* (CLS): after the impact event, the construction has suffered serious failures and collapses of nonstructural components. The construction retains a significant stiffness and resistance against vertical actions but has a small safety margin against collapse from horizontal actions. Large structural works are needed in order to fulfill safety requirements. The activity has to be suspended. The probability of exceedance of avalanche action during the reference life of the construction, P_e, is set equal to 2% [11].

In the present framework, the current design procedure could be interpreted as follows. We assume that the construction referred to in the Swiss standards [33] has a design working life, L_W, equal to 50 years and an importance factor, C_I, equal to 1.0, and it has to be designed to resist an event with return period T equal to 300 years. The previous statement implies that at the Ultimate Limit State, for which the probability of exceedance, P_e, in the reference life represented by Eqn.(7), is equal to 10%. Therefore, Eqn.(6) can be written as

$$0.10 = 1 - \left(1 - \frac{1}{300}\right)^{50\times 1.00\times C_A}. \tag{8}$$

The previous equality is satisfied for $C_A = 0.63$.

Reliability differentiation is obtained by assigning to the coefficient of importance the values reported in Table 4. The American rule [10] suggests multiplicative coefficients for snow loads that have to be applied to the load, not to its return period. At present, no straightforward links between return period and H_{72} snow depth exist. That is why we propose to use the importance factors of Italian seismic rule [18], which operates directly on return periods.

Table 5 reports the return periods of the threshold events for the design of structures in avalanche hazardous area. The return periods are computed by inverting Eqn.(6) and assigning the probabilities of exceedance previously reported to each limit state. For example, in the design of a construction, such a private house, in a snow avalanche hazard area, the engineer must design the structure considering the forces due to an impact of an avalanche that has a 300 years return period release depth, H_{72} (obtained in Table 5 at $L_D = 50$ yr, CC2a, ULS). On the contrary, if an hotel has to be built (at the same site) the reference event has return period equal to 450 years (50 yr, CC2b, ULS), i.e., larger impact forces are expected. Similarly, in case of an agricultural building, say a barn, which has design life equal to $L_D = 30$ years, the reference event for estimation of the impact forces for the design of structures has return period equal to 126 years (30 yr, CC1, ULS).

For the design of the construction, the variable of interest might be represented by the impact pressure, i.e., the interaction between snow avalanche and construction is represented by dynamic forces. To get the value of the impact pressure, various procedures are possible, depending on the amount of data available for the investigated snow avalanche phenomenon.

If little information is available, first the return period of the snow avalanche is determined, e.g., using Table 5. Then, the snow depth in the release zone (and, thus, the release volume) is estimated through statistical models based on meteorological data. Following that, a numerical simulation of the snow avalanche phenomenon is required. The values of the friction parameters to be used in the numerical model depend on the return period, as already done in the current practice [43]. In the proposed strategy, the values corresponding to the chosen return period should be used, and the impact pressures at the construction site are estimated from the numerical model.

If the avalanche path was previously studied and a return period was assigned to a run-out distance, the friction parameters along the path could be determined in such a way that the numerical simulation fits the observations. In this case, the release volume is still determined with the statistical models based on meteorological data for a given return period. As in the previous case, the impact pressures at the construction site are estimated numerically.

After the evaluation of the impact pressure, the structural engineer will use the most appropriate design choice of materials (concrete, steel, masonry, timber, or a combination) and structural shapes (e.g., frame, wall, slab on columns, trusses), according to the magnitude of the impact pressure and the architectural desiderata. Different impact pressures, referring to different return periods, would presuppose a different structural design. That is, the design is based on impact pressure, and not on snow depth in the release zone.

4 Conclusion

The present paper deals with reliability differentiation in the design of structures able to resist snow avalanche impacts. At present, no reliability differentiation is performed, resulting in oversized structures, with increased costs for the community.

Following the strategy already used in seismic design rules, the design principle is implemented through a coefficient of importance that multiplies the design working life of the construction depending on the degree of occupancy and the relevance of the activities performed herein. In order to ensure an adequate degree of reliability of the construction against the natural hazard, current practice indicates as design event that whose magnitude has 10% probability of being exceeded in the reference life. For ordinary structures with an expected design life of 50 years, this threshold event has a return period of 475 years, i.e., an annual probability of exceedance equal to 2.1×10^{-3}.

Nevertheless, the actual design practice in the Alps indicates that the construction has to be designed for events due to meteorological conditions with return period of 300 years. Inserting the corresponding annual probability of exceedance in Eqn.(6), without considering any reducing coefficient, would imply a probability of exceedance of 15.4% in 50 years, that is not acceptable since grater than the threshold of 10% in 50 years imposted by the Codes. Considering 300 years as the correct return period for the design of ordinary constructions, a reducing factor is easily applied to the design working life in order to uniform the probability of exceedance to 10% in 50 years. With the procedure previously described, thanks to the computation of H_{72} at various return periods, the reliability differentiation is possible.

Questions are still open. Is the probability of occurrence of a given snow pack thickness linked to the probability of occurrence of the impact pressure? In other words, there is the possibility that an avalanche due to a 300 years H_{72}-snow depth has an annual probability of occurrence different from $1/300$. In case the annual probability of exceedance was smaller, the design would be conservative. On the opposite, higher annual probability of exceedance would imply lower structural safety. As highlighted by Schweizer et al. [28], the average rate of occurrence of a given critical new snow depth (for which large avalanche are expected) is different from the average rate of occurrence of a large avalanche event. In addition, with increasing return periods, the ratio of the return periods of avalanche event and critical snow depth is expected to increase, i.e., even for the very rare and extreme events the critical new snow depth will often not be extraordinary. Besides that, changing mountain slope conditions during the design life of the construction has to be considered. In the past centuries, mountain slopes were used for agriculture and wood exploitation more intensively than nowadays. That induced large snow avalanches, just because the conditions in the release and running zone were different, as suggested in [44]. The development of procedures able to consider human involvement in snow avalanche hazard are the natural continuation of the studies herein presented.

The proposed approach is general, i.e., independent of the particular avalanche case, so that it could be applied to any situation. To this end, a robust correspondence between snow cover conditions at the release zone and impact pressure on the elements at risk would allow the practical implementation of the method in design codes. In the future, the knowledge of such a relationship would lead to a simple strategy for the assessment of structural safety of constructions and infrastructures in mountain hazardous areas.

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Table 1: Categorization of Consequence Classes [12, Table A.1].

Consequence class	Example of categorization of building type and occupancy
1 (CC1)	Single occupancy houses not exceeding 4 storeys. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of $1\frac{1}{2}$ times the building height.
2a Lower Risk Group (CC2a)	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys. Retailing premises not exceeding 3 storeys of less than 1 000 m^2 floor area in each storey. Single storey educational buildings All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2 000 m^2 at each storey.
2b Upper Risk Group (CC2b)	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Educational buildings greater than single storey but not exceeding 15 storeys. Retailing premises greater than 3 storeys but not exceeding 15 storeys. Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which the public are admitted and which contain floor areas exceeding 2 000 m^2 but not exceeding 5 000 m^2 at each storey. Car parking not exceeding 6 storeys.
3 (CC3)	All buildings defined above as Class 2 Lower and Upper Consequences Class that exceed the limits on area and number of storeys. All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5 000 spectators. Buildings containing hazardous substances and /or processes

Table 2: Comparison between the Importance classes for buildings reported in Eurocode 8 for seismic design [11] and the Consequence classes reported in Eurocode 0 as base of design [9].

Importance class	Consequence class	Building
I	CC1	Buildings of minor importance for public safety, e.g., agricultural buildings, etc.
II	CC2a	Ordinary buildings, not belonging in the other categories.
III	CC2b	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g., schools, assembly halls, cultural institutions etc.
IV	CC3	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g., hospitals, fire stations, power plants, etc.

Table 3: Indicative design working life, from [9].

Indicative design working life (years)	Examples of the structures
10	Temporary structures
10 to 25	Replaceable structural parts, e.g., gantry girders, bearings
15 to 30	Agricultural and similar structures
50	Building structures and other common structures
100	Monumental building structures, bridges, and other civil engineering structures

Table 4: Coefficient of importance in snow avalanche design, derived from Italian seismic design code [18, Sec. 2.4.3].

Consequence class	C_I
CC1	0.7
CC2a	1.0
CC2b	1.5
CC3	2.0

Table 5: Return periods, in years, for 3 consecutive days snow depth value, H_{72}, suggested for snow avalanche design for various limit states for design working life equal to 30 and 50 years. The probability of exceedance related to Operativity Limit State (OLS) is 50% in the reference life. The same value is 20% in Damage Limit State (DLS), 10% in Ultimate Limit State (ULS) and 2% in Collapse Limit State (CLS).

$L_D = 30$ yr

Consequence Class	OLS	DLS	ULS	CLS
CC1	20	60	126	656
CC2a	28	45	180	938
CC2b	41	128	270	106
CC3	55	170	360	1875

$L_D = 50$ yr

Consequence Class	OLS	DLS	ULS	CLS
CC1	32	99	210	1094
CC2a	46	142	300	1562
CC2b	69	213	450	2343
CC3	92	283	599	3124