Received: 15 February 2021 | Revised: 19 July 2021 | Accepted: 25 July 2021

DOI: 10.1002/eqe.3530

RESEARCH ARTICLE

WILEY

Towards a practical loss-based design approach and procedure

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Funding information
Ministero dell'Università e della Ricerca

Abstract

Performance-based design was first envisaged in the early 1990s and has become a mantra in conceptual seismic design. However, practical approaches to be possibly introduced in building codes are not readily available, to say the least, though some can be found in the literature. What is essentially missing is some correlation between structural response parameters and expected monetary losses, at a level of simplicity comparable with the force- or displacement-based approaches applied to design in everyday practice. The purpose of this conceptual paper is to provide the basics of a formulation that may evolve into such a practical loss–based approach.

KEYWORDS
damage, loss, performance, seismic design

1 | INTRODUCTION

Following the devastating economic impact of the 1994 Northridge earthquake in the US due to damage and disruption, it was clear that further work needed to be done to mitigate the devastating effect of earthquakes onsociety. One of the main issues to be addressed was how seismic codes' focus on life safety and collapse prevention was not sufficient to avoid the problems observed in events like Northridge and a new conceptual way of thinking was required.[1]

This led the way for what became known as performance-based earthquake engineering (PBEE) in the 1990s and its rise in popularity can be observed through its discussion in the literature and integration in seismic guidelines around that time. Guidelines like the Vision 2000 framework[2] outlined this PBEE approach by relating required building performance to several seismic hazard levels and FEMA-356[3] later integrated this approach in its acceptance criteria, for example. Other works like Priestley and Calvi[4] and Priestley and Kowalsky,[5] for example, discussed how specific design methods like displacement-based design could help bridge the gap in seismic design philosophy put forward via PBEE and practical implementation for designers.

Later research efforts [e.g., 6–8] began to integrate aspects like monetary loss and downtime and also bring the probabilistic nature of the problem to the forefront [e.g., 9, 10]. This saw some considerations on how these performance metrics may be integrated into the performance-based design of structures, with more recent research[11–14] also encompassing risk-targeted methods, and one approach in particular by Luco et al.[15] has been incorporated into the US guidelines[16] via risk-targeted design maps, for example.

While many approaches have been studied and developed, there is still a gap between what is possible in the current state of practice via available design tools and what is generally needed by society. Issues like how to correlate the expected repair costs of structural and non-structural elements to tangible structural design parameters for structural engineers are not well-established. In addition, the potential for notable indirect losses as a result of downtime and disruption is yet to

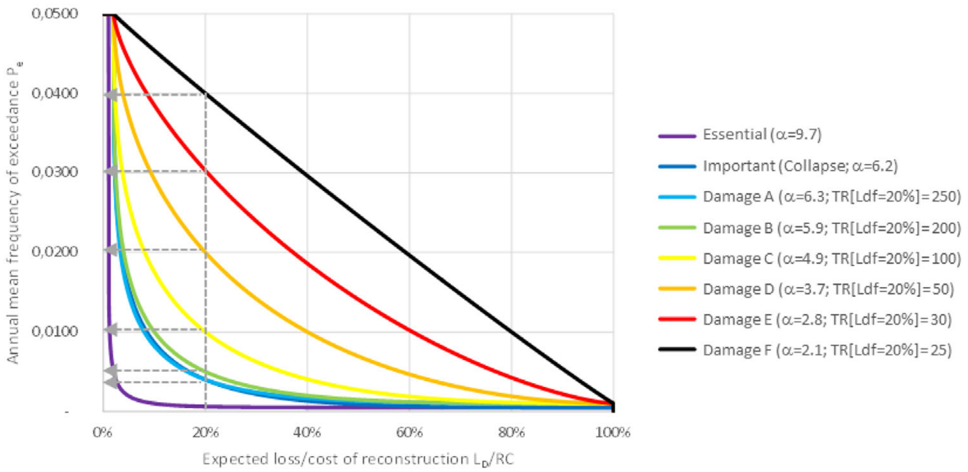

FIGURE 1 Correlation between P_e and L_D according to Equation 1 for different return periods (T_R) of the ground motion at the specific level of direct loss $L_{Df} = 20\%$ of the cost of reconstruction (C_{RC})

be tackled in a meaningful way, despite its relevance, as highlighted in recent earthquakes such as the 2016 Central Italy earthquakes.

This article aims to tackle these issues and provide some conceptual discussion on how approaches to deal with them may be formulated to make them more accessible in future performance-based approaches.

2 | DEVELOPMENT OF A PRACTICAL FORMULATION FOR LOSS CURVES

The first basic ingredient of the formulation is a simple correlation between mean annual frequency of exceedance (or average return period) of a ground motion intensity and expected level of damage and subsequent loss. Such a correlation is depicted in Figure 1 and may be based on three main points: one related to the ground motion intensity inducing a first onset of damage, one related to the intensity inducing structural collapse or total economic loss and an intermediate one controlling the shape of the resulting curve. The expected annual loss (EAL) for a structure is defined as the area beneath this curve. This type of curve, defined by a few key points, has been used extensively in past works related to loss assessment of many different kinds of buildings [e.g., 17–22] and has been seen to work reasonable well; hence, it is deemed a suitable start for the conceptual proposal described herein.

The return period (T_R) or the mean annual frequency of exceedance (MAFE $= 1/T_R \approx P_e$) of a ground motion that induces some first onset of damage, denoted T_{R0}, is purely conventional and has little relevance. It will be assumed $T_{R0} = 20$ years (i.e., $P_{e0} = 5\%$) and a corresponding damage level inducing direct losses (repair cost) conventionally set at $L_{D0} = 2\%$ of the reconstruction cost. Although these assumptions are conventional, they may influence the value of the EAL particularly when it tends to assume rather high values (i.e., when a relevant damage is expected for relatively frequent ground motion levels).

The same values at collapse are conventional too and depend on the relevance of collapse in terms of human life loss. A reasonable return period value for the case of Italy, with reference to the Italian building code,[23] may be assumed around $T_C = 1000$ years (i.e., $P_{eC} = 0.001$), which may be raised to 2000 years in case of relevant consequences and reduced to 500 years for low importance buildings. The decision on this threshold when directly linking to expected casualties could be associated with the acceptable fatality threshold, with recent work[24] having discussed the possibility of linking these two for a given occupancy type and building, for example. The corresponding level of direct loss is $L_{DC} = 100\%$, indicating that collapse implies a complete economic loss of the building, which is of more direct interest to the discussion presented here although when uncertainties are considered in estimating repair costs, this value may exceed 100% but the general point remains. Of course, these values may vary from region to region and also between building codes, but the fundamental idea of a high return period associated with the collapse, or complete economic loss, of the building that may be adjusted by the engineer depending on the level of building importance remains a fundamental point of reference.

In order to be able to define the entire shape of the curve using a single control point, it is convenient to define an equation that expresses P_e as a function L_D and of a single control parameter, to be set in order to impose a specific level

TABLE 1 Correspondence between the parameter α in Eq.(1) and the resulting expected annual loss for the curves shown in Figure 1

	α=9.7	α=6.2	α=6.3	α=5.9	α=4.9	α=3.7	α=2.8	α=2.1
EAL	0.12%	0.37%	0.39%	0.44%	0.67%	1.15%	1.74%	2.50%

of the direct loss (L_{Df}) at a desired probability of exceedance (P_{ef}). Such a function should have a form similar to loss curves typically observed in past studies and could be forced to pass through three points, at the assumed onset of damage (index 0), at collapse or total loss (index C) and at a loss control function point (index f). A convenient formulation can be expressed as per Equation 1, introducing a parameter α that varies as a function of point f.

$$P_e = P_{eC} + (P_{e0} - P_{eC}) \cdot sin^{\alpha}\left(cos^{-1}\left(\frac{L_D - L_{D0}}{L_{DC} - L_{D0}}\right)^{\frac{1}{\alpha}}\right) \tag{1}$$

As anticipated, the value of α can be calculated imposing the third point f on P_{e} and L_{D}, which is controlling a certain level of direct damage for a given probability of exceedance, or vice-versa. Typical shapes of the curves generated by the equation are reproduced in Figure 1, imposing different P_{ef} values for a damage level corresponding to $L_{\mathrm{Df}} = 20\%$ of the cost of reconstruction (C_{RC}). The return periods at $L_{\mathrm{Df}} = 20\%$ for each curve, from damage A to damage F, are 250, 200, 100, 50, 30 and 25 years. It is noted that Equation 1 is simply just one hypothesis of what this function may be and considering loss curves from past work[20] it seems to be reasonable and hence is elaborated on herein. Different situations of building typology and performance requirements evaluated through more rigorous loss estimation methods may highlight slightly different shapes but the overall trend of Equation 1 is still considered valid in a general sense.

For essential buildings, the collapse T_{RC} has been set at 2000 years ($P_{\mathrm{eC}} = 0.05\%$), which is similar to other collapse risk values examined in past studies [e.g., 25], and an $L_{\mathrm{f}} = 10\%$ is imposed at $T_{\mathrm{Rf}} = 1000$ years; the assumed loss level at onset of damage ($P_{\mathrm{e0}} = 5\%$) is set at $L_{\mathrm{e0}} = 1\%$. Non-essential buildings with important consequences in case of collapse have $P_{\mathrm{eC}} = 0.05\%$ ($T_{\mathrm{RC}} = 2000$ years), but $T_{\mathrm{Rf}} = 250$ years at $L_{\mathrm{f}} = 20\%$. For essential buildings, the collapse limit state is irrelevant, since a limit state of continuous functionality is imposed for a return period normally accepted for a collapse limit state. Therefore, the curve becomes extremely concave. It is evident from Figure 1 that a variation of the annual probability of collapse has a limited influence on the curve, and consequently on the predicted EAL (the curve "important construction" is essentially superimposed to that "damage A," being characterized by the same point f, but with collapse return periods set at 2000 and 1000 years, respectively. The area below each curve is expressing the total EAL, which can be calculated by integrating the corresponding equations. It is now clear why it was anticipated that the ground motion return period assumed at the onset of damage can strongly influence the resulting EAL for damage-prone constructions: the left part of the curve will raise or decrease with significant influence on the its integral. For the curves shown in Figure 1, the following correspondence listed in Table 1 between α and EAL can be obtained.

Considering the points reported in Table 1, a good correlationbetween EAL and α can be obtained using the expression reported in Equation 2.

$$\alpha = \frac{6\%}{EAL + 0.5\%} \text{ or } EAL = \frac{6\%}{\alpha} - 0.5\% \tag{2}$$

The correspondence between the curve expressed by Equation 2 and some points calculated by integration is shown in Figure 2. It is confirmed that a change of the return period conventionally assumed for the onset of damage has a significant influence on the relation between EAL and α at high loss values, while the modification of the conventional starting loss value has little influence. This is observed through the different points (e.g., 10/1% represents $T_{\mathrm{R0}} = 10$ years and $L_{\mathrm{D0}} = 1\%$) being represented well by the proposed Equation 2. At this stage, it is felt that assuming 20 years as the return period for the onset of damage is more reasonable than assuming 10 years, and this will be done in what follows. Further sensitivity studies may be undertaken, as has been done in Shahnazaryan et al.[26] to show the importance of this point on the resulting loss curve and EAL, for example.

The application of Equation 2 facilitates a practical application of EAL in design. Actually, it is possible to fix a target EAL level, calculate α and plot the corresponding curve, as shown in Figure 3. This type of approach has been implemented in some recent applications of conceptual seismic design to reinforced concrete structures,[1,14] but the purpose of this discussion is to describe the conceptual significance of it. High values[25] of EAL > 1% would typically not be desired in new

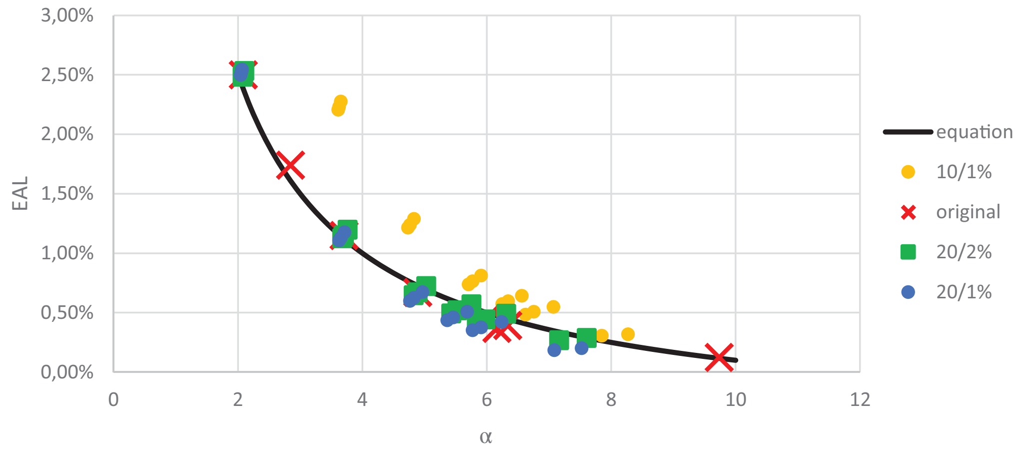

FIGURE 2 Correlation between EAL and α according to Equation 2 compared to points calculated by integration (yellow dots: 1% loss at $T_{R0} = 10$ years, green squares: 2% loss at $T_{R0} = 20$ years; blue dots: 1% loss at $T_{R0} = 20$ years)

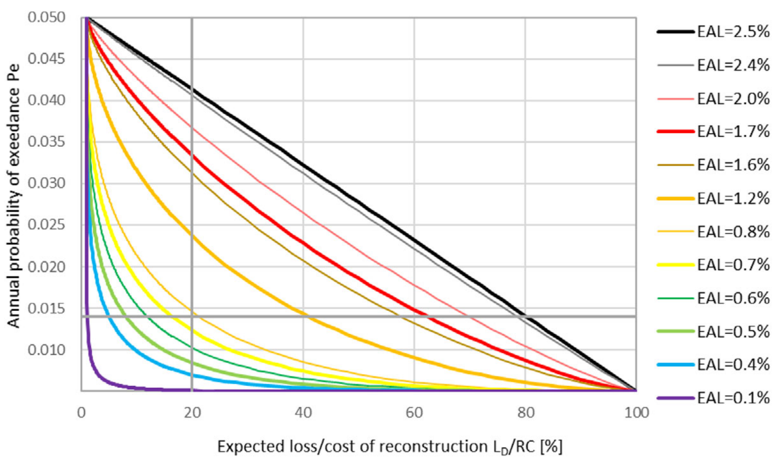

FIGURE 3 Correlation between P_e and L_D according to Equation 2 for different EAL values. The color of thicker lines is related to the curves in Figure 1

design but they are included in the graphs for reference as they cover the range of EAL values listed in the Italian seismic risk classification guidelines.[27–29]

The horizontal line shown in Figure 3 corresponds to $P_e = 1.4\%$ (i.e., $T_R = 72$ years) or a probability of exceedance equal to 50% in 50 years, a value sometimes used to prescribe a limit on the acceptable interstorey drift in order to protect non-structural elements. If one is able to associate a direct loss estimate to the interstorey drift (e.g., using storey loss functions[30]), they could decide for which interstorey drift they should design to limit the expected annual loss to the desired value, as has been explored in past research [e.g., 1, 6, 14]. Possibly more efficiently, one could use the vertical line, associated with a direct loss $L_D = 20\%$ of the C_{RC}, assuming that possibly this corresponds, say, to an interstorey drift $\delta_i = 0.5\%$, and thus evaluate for which ground motion return period this maximum acceptable δ_i should be imposed. The difference between these two approaches is essentially in how the problem is framed: the first asks what is the interstorey drift to be designed for to respect a certain loss limit at a given intensity, whereas the second asks which intensity should be designed for a given loss and interstorey drift. Under the latter assumption, the correlation between EAL and annual probability of exceedance of a ground motion for which $\delta_i = 0.5\%$ is the imposed design limit is approximately as listed in Table 2.

Figure 3 seems to indicate (under the arbitrary assumptions described above) that a code-conforming building may be characterized by an EAL in the range of 1.0% (approximately at the intersection of the two grey lines), which would be somewhat in agreement with the values put forward in the Italian seismic risk classification guidelines[27] initially outlined by Calvi et al.[28] and described in detail by Cosenza et al.[29] Clearly, the correlation between expected loss and drift is strongly influenced by the characteristics of the non-structural elements (NSE). Elements capable of accepting larger deformation without showing significant damage may allow larger drifts for the same loss (and thus the same EAL). The selection of a class of NSE will thus assume a meaning similar to the selection of a ductility class to define the force reduction factor

TABLE 2 Relationship between the annual probability of exceedance of shaking and EAL in order to respect a drift threshold of 0.5%

	EAL=0.1%	EAL=0.4%	EAL=0.6%	EAL=0.8%	EAL=1.2%	EAL=1.6%	EAL=2.0%	EAL=2.4%
$P_{ef}(\delta_i = 0.5\%)$	0.06%	0.33%	0.70%	1.18%	2.19%	3.01%	3.60%	4.02%
$T_{Rf}(\delta_i = 0.5\%)$ [years]	1700	305	143	84	46	33	28	25

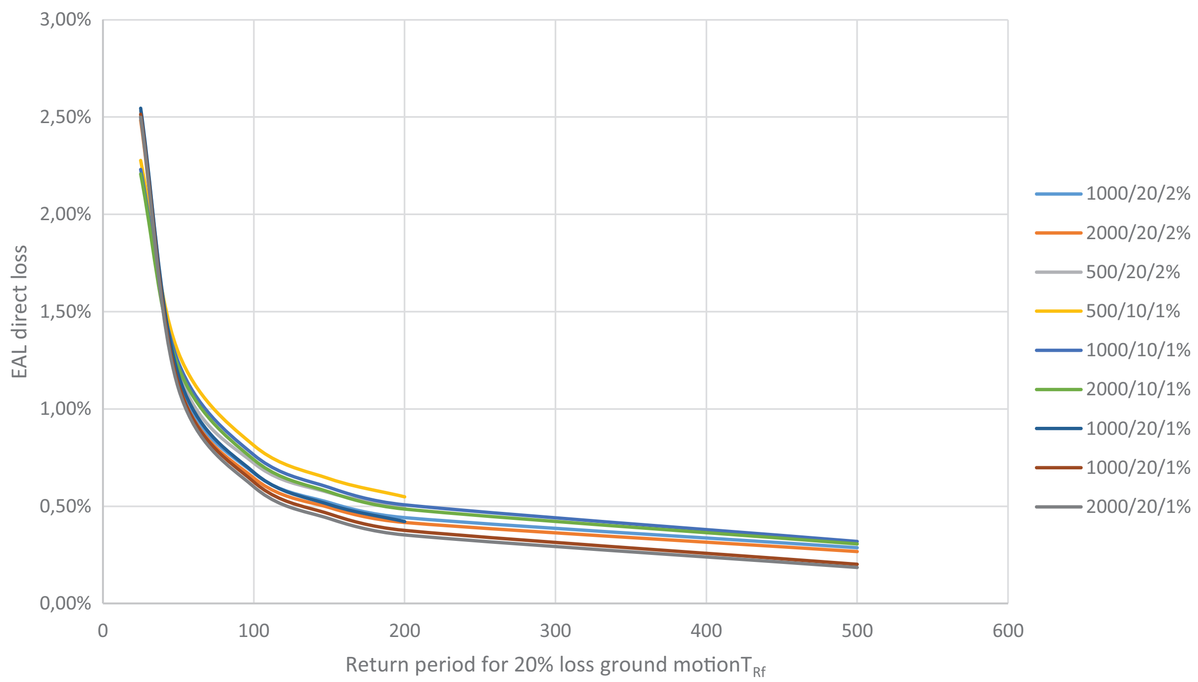

FIGURE 4 Correlation between EAL and damage control point f, as a function of the return periods of the ground motion inducing collapse and inducing the onset of damage

(q in the Eurocode system) of structural systems: lower class elements will require smaller drifts to assure the same loss level.

The resulting EAL is strongly influenced by the selection of the f point, while it is less sensitive to the assumption on onset of damage (point 0) and almost insensitive to the assumption on collapse (point C). This is evident in Figure 4, where the return period of a ground motion inducing losses corresponding to 20% of the reconstruction cost is represented in the horizontal axis, while each curve is characterized by different assumptions on points 0 and C. All curves are evidently packed together.

Designing for a ground motion return period between 100 and 200 years inducing a loss equal to 20% of the reconstruction cost seems reasonable, if only direct loss is considered; actually, all curves are descending quite steeply for lower return periods and remaining relatively flat for higher ones. It seems much easier to reduce the EAL from 2% to 0.5% than from 0.5% to 0.2%. Hence, these would seem to be reasonable range from which sensitivity studies could look to establish more refined data.

3 | CONSIDERATION OF INDIRECT LOSSES

The previous section discussed the limitation of direct economic losses in buildings and how they may be limited via EAL and consideration of drift limitations and ground motion intensities in performance-based design. While direct losses are undoubtedly of notable importance, the potential for indirect losses can have a significant impact and at times be orders of magnitude larger than the corresponding direct loss. For example, a production factory may suffer relatively low structural damage to the plant building and require some repairs of insignificant cost, but the loss of revenue due to the business interruption required to carry out these repairs may be quite substantial for the owner. Hence, the consideration of indirect losses in design is of critical importance and has thus far been lacking any serious consideration in the literature. A major problem has been the lack of data to accurately quantify it, but its relevance is beyond doubt. To tackle this, the approach used here is that generally known as the Fermi problem,[31] named after the Italian physicist, in which reasonable estimates of a quantity can be obtained with little to no data. It is based entirely on justified approximations whose purpose is to give order-of-magnitude estimates that help in understanding the problem. Given the conceptual nature of this paper and the issue at hand, it is deemed a suitable approach to shed further light for future consideration.

The simplest way to consider indirect losses is to assume some form of proportionality to direct loss, estimating the total value of expected indirect loss in case of complete collapse of the construction. Consider for example the case of residential buildings: the most obvious form of indirect loss is the cost of relocating the displaced building occupants, in the emergency phase and in the time required to repair and refurbish the damaged building. In this context one could reasonably estimate the potential maximum indirect loss assuming a daily cost of relocation per person, say $C_{\mathrm{rl}} = 35$ €/(day·person), a tributary area per person, say $S_{\mathrm{p}} = 25\ \mathrm{m}^2$/person, a cost of reconstruction $C_{\mathrm{R}} = 1000$ €/m^2, a required time for total reconstruction of, say $T_{\mathrm{rc}} = 730$ days. Using these arbitrary (but reasonable) values, the maximum indirect loss (L_{IM}) parametrized on the cost of reconstruction (C_{R}) would be given by Equation 3.

$$\frac{L_{IM(res)}}{C_R} = \frac{C_{rl} \cdot T_{rc}}{S_p \cdot C_R} = \frac{35 \cdot 730}{25 \cdot 1000} = 1.02 \tag{3}$$

This value indicates that the maximum indirect loss in case of total disruption of a residential building would possibly be in the same range of the reconstruction cost, that is, of the maximum direct loss, which may appear high and with more refined values may actually be a fraction of this for different building typologies and occupancies, but the general approach proposed would be anticipated to be the same. The same exercise indicates that the daily cost associated with the unavailability of a residential building is approximately 0.14% of the cost of reconstruction. Therefore, for example, a downtime of 3 months will imply an indirect loss approximately equal to 12% of the reconstruction cost. If such consideration were to be incorporated into the decision-making process during design, more informed decisions and mitigative actions could be taken to minimize the potential for such indirect losses.

In the case of a bridge, similar simple calculations could be based on the number of vehicles (N_{v}) crossing the bridge and on the required detour length (D_{d} [km]) in case of collapse, assuming a unitary cost per added travelled km (C_{km}), a cost of reconstruction (C_{R}) and a time required to reconstruct or repair the bridge (T_{rc}) is given in Equation 4.

$$\frac{L_{IM(bridge)}}{C_R} = \frac{N_v \cdot D_d \cdot C_{km} \cdot T_{rc}}{C_R} \tag{4}$$

In this case the only parameter with little variability is C_{km}, possibly in the range of 0.5 €/km. Some parametric analyses of possible combinations of the other parameters lead to reasonable ratios between indirect and direct loss for relevant bridges in the range of 2 to 5 and to a daily cost associated to downtime in the range of 0.3% to 0.8% of R_{C}. Considering the upper extreme, the implication is that the entire cost of reconstruction of an important bridge is repaid in each 4 months interval of downtime. Similar exercises are possible for all sorts of construction, such as commercial centers, school and universities, industrial buildings.

The correlation between indirect and direct losses is clearly non-linear. In fact, relatively minor damage would still imply some repair cost, possibly associated with some nuisance, but not necessarily requiring any interruption of the regular building activity. Quite the opposite, even if the direct loss is not complete, the time of interruption of the activity could well be extended and close to the expected maximum. Based on these considerations, the correlation between indirect (L_{I}) and direct (L_{D}) losses could be potentially described by a curve similar to that representing a cumulative normal distribution, expressed by Equation 5. It is noted that this is not expected to be the exact distribution of such loss, but more of an indication of how it may be anticipated to accumulate and expressed analytically for use in the practical design and assessment situations described here.

$$L_I = \frac{R_{I/D}}{\sigma\sqrt{2\pi}} \int_0^1 e^{-\frac{L_D^2}{2\sigma^2}} dL_D \tag{5}$$

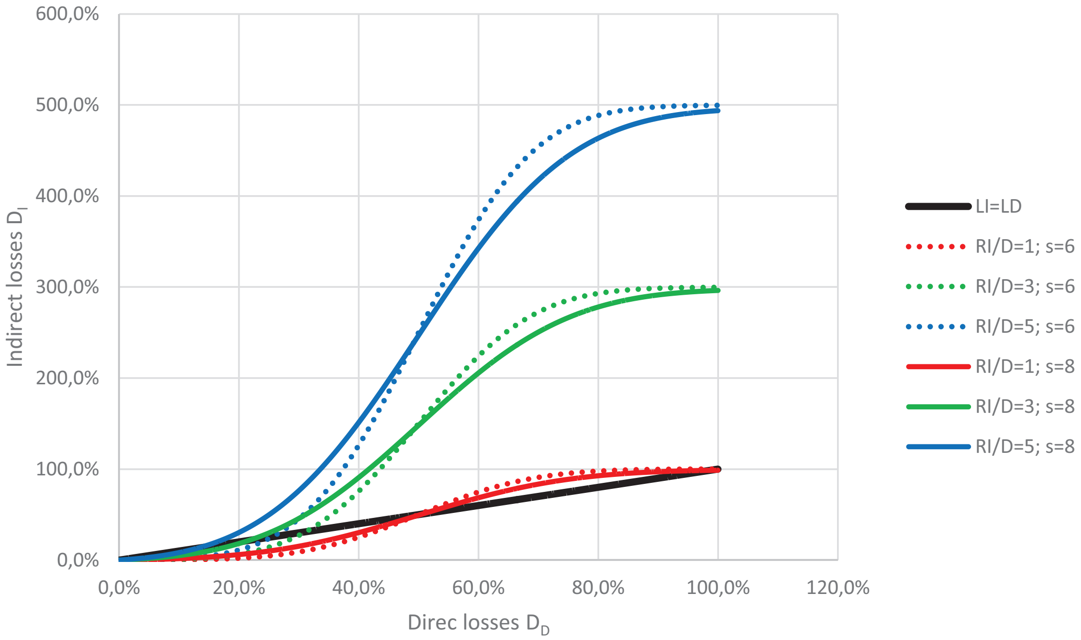

FIGURE 5 Correlation between indirect and direct losses according to Equation 5

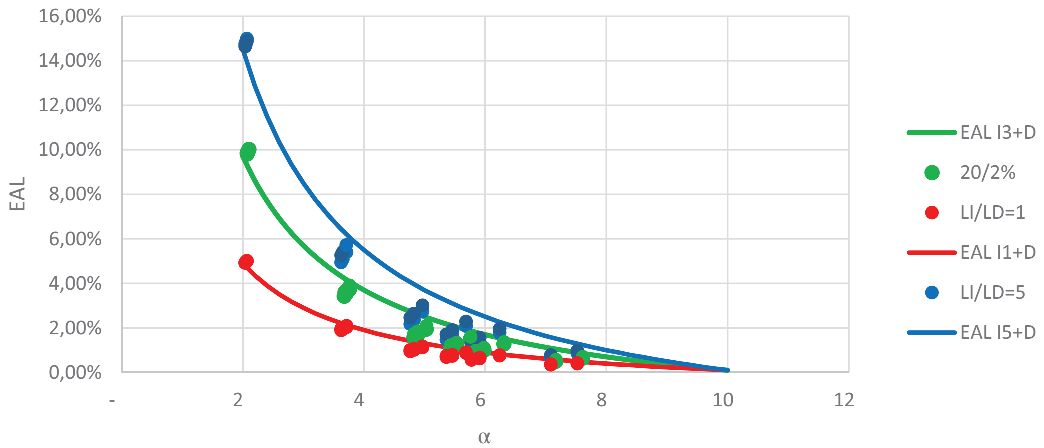

FIGURE 6 Correlation between EAL, including direct and indirect losses, and α according to Equation 6 compared to points calculated by integration. Points calculated with $\sigma = 6$ and $\sigma = 8$ are included, for indirect maximum losses equal to 1, 3, and 5 times the direct loss

Where $R_{I/D}$ is the ratio between the maximum values of indirect and direct losses, σ is a parameter that increases or decreases the proportionality between the growth of indirect and direct losses (possibly in the range 6 to 8, in a cumulative normal it would be the usual sigma). Sample curves derived from the application of Equation 5 are represented in Figure 5, considering ratios between maximum indirect to direct loss equal to 1, 3, and 5 and sigma values equal to 6 and 8.

Considering the correlation between indirect and direct losses described by Equation 5 and graphically represented in Figure 5, it is possible to recalculate all points associating the total (indirect + direct) predicted loss to corresponding α values, as a function of the ratio between maximum indirect and direct loss and of the adopted σ. A number of points obtained are depicted in Figure 6, without making a distinction between $\sigma = 6$ and $\sigma = 8$, since the differences seem negligible. In Figure 6, the points are compared with curves obtained revisiting Equation 2, including the parameter that

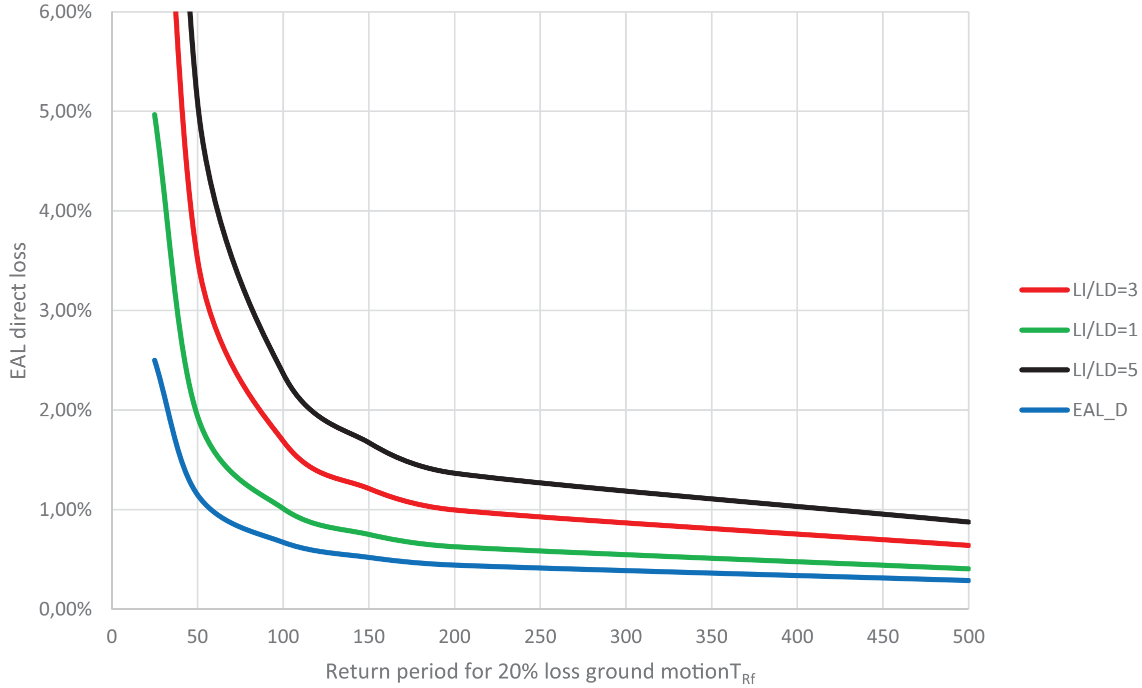

FIGURE 7 Correlation between EAL and return period of a ground motion inducing 20% of R_C direct loss. The curves include different levels of indirect losses. The ground motion inducing collapse is set at 2000 years R_p, the ground motion inducing onset of damage, at 2% of R_C, is set at 20 years R_p

expresses the ratio between maximum indirect and direct loss ($R_{I/D}$). The derived equation is given in Equation 6.

$$EAL_{I+D} = 6\% \left(1 + R_{\frac{I}{D}}\right)\left(\frac{10-\alpha}{10\alpha}\right) + 0.1\%$$

$$\alpha = \frac{6\% \left(1 + R_{I/D}\right)}{EAL_{I+D} + 0.6\% \left(1 + R_{I/D}\right) - 0.1\%} \tag{6}$$

The expression in Equation 6 needs to be tested and calibrated when suitable data become available. It can be improved and the adopted σ included as a correction factor, but it captures reasonably well the general trends and ideas being put forward.

The relatively modest influence of the assumptions about the return period of ground motions inducing the first onset of damage and inducing collapse on the EAL value, discussed with reference to Figure 4, is confirmed if indirect losses are included. The fundamental role of the assumptions about point f is confirmed, with a tendency of increasing the return period range at which the curve tends to become flat, as shown in Figure 7. This effect had to be expected: for higher predicted losses designing for a limited damage at higher return periods becomes more and more convenient. Similarly, including indirect losses has the effect of shortening the breakeven period for a given investment adopted to reduce the EAL. These points alone could have notable implications on the decision-making process surrounding performance-based design.

4 | CORRELATIONS BETWEEN LOSS, RESPONSE PARAMETERS AND INPUT GROUND MOTION

At the damage control point (f), it can be assumed that the structural response will still be essentially linear, that the structural damage is negligible and that the non-structural damage is responsible of direct and consequent indirect losses. Non-structural elements (NSE) can be acceleration- or displacement-sensitive.[32] Examples of acceleration-sensitive NSE are chimneys, parapets, hanging pipelines and ceilings; examples of displacement-sensitive elements are partitions, door

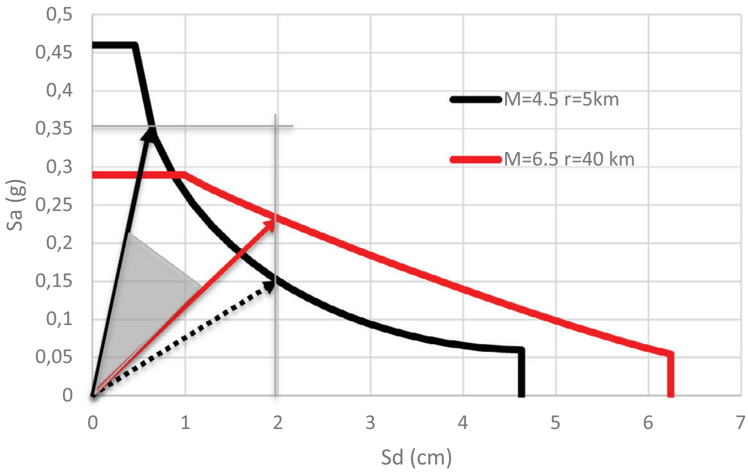

FIGURE 8 Acceleration and displacement design for loss control compared with elastic spectra at the correspondingly assigned return period. The building stiffness has to be designed between the black and the red arrows

and window frames, vertical pipelines. Clearly some NSEs can be sensitive to both floor accelerations and interstorey drifts.

Today, the sensitivity of NSE to actions is more a matter of guessing than a tested and certified property. It is desirable that this will change in the future, associating a function between floor acceleration or interstorey drift on one side and expected level of damage on the other side to each element. If this should be available, a single point of the function will define the level of acceleration and drift associated to, say, 20% damage or direct loss level. Other relevant points could be considered, for example the interstorey drift that will make difficult or impossible to open a door or a window, or the floor acceleration that will induce possible leaking in a pipeline. As a matter of fact, each NSE could be characterized by a couple of certified level of floor acceleration and interstorey drift that could be associated to a damage control point, conventionally set at, say, 20% direct loss level. Something like this implies the possibility of categorizing NSE classes, as a function of the acceptable action level, as recently proposed in O'Reilly and Calvi,[33] for instance. Considering the conceptual nature of this paper, let's consider an example class for which the damage control floor acceleration is set as $a_{F,f} = 0.35$ g and the damage control interstorey drift is set as $\delta_{i,f} = 0.5\%$. These values would not be too different than what would be found in design, with storey loss functions proposed in Ramirez and Miranda[34] for typical buildings in the US reaching notable losses around these demand levels, in addition to several fragility functions specified in the PACT library[35] showing median demands for NSEs around these thresholds, so they are not unreasonable.

To clarify the envisaged procedure it will be simply assumed that the displaced shape of the building is linear along the height; thus, deriving an equivalent displacement ($\Delta_{e,f}$) at the center of mass equal to 2/3 of the total height of the building multiplied by interstorey drift: $\Delta_{e,f} = 0.67{\cdot}H \cdot \delta_{i,f}$;. The limit design displacement will thus depend on the number of storeys ($\Delta_{e,f} \approx 20$ mm for a two story building; $\Delta_{e,f} \approx 60$ mm for a six story building). Discussing the correlation between input ground motion and response floor spectra is a complex matter, outside the scope of this work. Floor accelerations are influenced by higher mode contributions much more significantly and cannot be accurately linked to first mode response alone, as in the case of floor displacements. However, some proportionality does exist between the two, as noted in the prediction model provided in FEMA P-58,[35] for example, hence the logic of the discussion here remains valid. For the sake of argument, it is assumed that the floor acceleration coincides with the elastic spectral acceleration herein, although some additional factoring coefficients could be studied to refine this. These limits are obviously discussed here to generally illustrate the point but the trends are rational and more refined values could be adopted. For example, should the deformed shape at maximum response be known then the equivalent displacement is easily calculable with well-established methods.[36] Likewise, if some basic information about the first few modal shapes and period information can be established, then a good estimation of the expected floor acceleration or even floor spectrum can still be inferred using some simplified tool [e.g., 37, 38].

An example for a two-storey building case is shown in Figure 8, comparing the acceleration and displacement limits with two possible elastic spectra corresponding to the chosen probability of exceedance in a given location. In this example, it appears that the building secant-to-yielding stiffness should remain between the black and red arrows, but it is immediate to realize that for taller buildings the displacement limit could become irrelevant, being greater than the maximum possible demand, or that both the acceleration and displacement limits could become irrelevant in low seismicity zones, where the spectra may shrink inside the design limits. It is just the case of noting that analogous limits could be

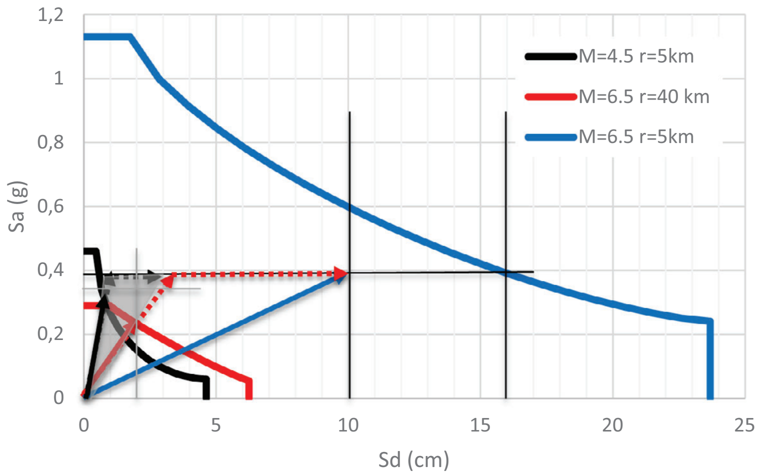

FIGURE 9 Design for collapse prevention: providing enough displacement capacity is adequate for a standard building. Design for continued functionality may be impossible without resorting to base isolation

set for bridge, considering for example traffic barriers or sound protection to limit deck accelerations and joints to limit relative displacements.

Designing for the collapse limit state requires the definition of an elastic spectrum associated with a much greater return period, say 1000 years, possibly like that shown in Figure 9. For a standard structure it could be sufficient to provide enough displacement capacity (possibly 100 mm, as shown in Figure 9) associated to an adequate energy dissipation capacity enhancing it equivalent displacement capacity (to 160 mm in the figure).[6] However, if the building should be designed for functional continuity in case of extreme events, the intersection points of the grey lines defining the limit levels of floor acceleration and equivalent displacement will be incompatible with the design spectra. The structure could be made strong enough or capable of an adequate displacement or flexible enough to avoid the accumulation of floor accelerations, but not both. Resorting to some kind of supplemental system like seismic isolation or dampers may appear to be the only viable solution at a design outset. Thus, it does not surprise that in Turkey all hospitals have to be base-isolated. Another option may be to focus on increasing the acceleration capacity of the non-structural elements to mitigate damage, as is typically done in the nuclear power plants.

5 | PROCEDURAL APPLICATION

The previous sections have touched some pertinent aspects facing earthquake engineering and its implementation in practice. Some conceptual solutions have been discussed briefly and, in this section, a potential workflow for how this may be implemented is given for both the case of design and assessment.

5.1 | Design procedure

A loss-based design procedure aims to design for a selected level of EAL, including indirect losses, set as the design target. A summary of the procedure is as follows.

1. Set conventional damage onset, say for example 2% of the replacement cost (R_c) at the 20-year return period event (T_{R0}).
2. Set collapse T_{RC} or P_{eC}; possibly $T_{RC} = 1000$ years ($P_{eC} = 0.001$) for standard constructions.
3. Calculate ratio between maximum possible indirect losses and R_C ($R_{I/D}$, possibly 1 to 5 times) based on the types of considerations made in Section 3.
4. Set the design EAL_{D+I}, which considers both the direct and indirect loss. This could be in the range of 0.1% for essential buildings and in the range of 2% for standard buildings; however, any value could be set depending on the anticipated magnitude of $R_{I/D}$, building importance and seismicity of the region.
5. Calculate the α value corresponding to $R_{I/D}$ and $EAL_{D+I,\,design}$ from Equation 6.
6. Set the direct damage ratio (or loss, L_{Df} possibly 20%) to be associated with the damage control return period (T_{Rf}) or probability of exceedance (P_{ef}) and find P_{ef} from Equation 1.

7. Associate a maximum floor acceleration and equivalent displacement to be respected in order to ensure that the direct damage ratio L_{Df} will not be exceeded (e.g.,: 32).
8. For the given seismic intensity to be considered, associate a combined acceleration-displacement spectrum associated to P_{ef} (e.g.,: 38, 39).
9. Identify a structure whose initial secant-to-yield period falls with the feasible range according to the combination of points 7 and 8 (Figure 8).
10. Associate a combined acceleration-displacement spectrum corresponding to P_{eC} (e.g.,: 38, 40) and design for collapse (Figure 9).

While the approach described above is largely conceptual in its application, it shows how the considerations made in previous sections can be sequenced together to form a novel design approach for structures than considers the direct and indirect losses in a structure directly. These considerations are then translated to tangible structural design parameters that engineers can use in practice. As can be seen, it is a notable deviation from the approach of other design codes in that it focuses much more on the actual performance of the building during and after an earthquake as opposed to some structural verifications at specified intensities that try to limit such outcomes indirectly.

5.2 | Assessment procedure

While the discussions up to this point and illustrated in the previous section have largely been oriented towards design, the same considerations could also be made in an assessment approach for existing buildings. A loss-based assessment procedure aims to calculate the annual probability of exceedance of the ground motions inducing collapse (and thus 100% of direct losses and the maximum associated indirect losses) and the EAL of the construction, including direct and indirect losses.

The first item is traditional, is summarized in points 1–3 of the procedure below and is fundamental to take decisions about possible strengthening measures, motivated by the needed of protecting human life. This objective can generally be pursued increasing the structure displacement capacity and has little influence on the EAL. The second parameter is essential to take decision about measures that could reduce the expected losses and assure function continuity for lower probability of exceedance ground motions. These measures may include interventions focused to modify the structure stiffness of equivalent yielding strength, but also focused on improving the response of non-structural elements. A summary of the procedure is as follows.

1. As in the case of design, set conventional damage onset, say for example 2% of the replacement cost (R_c) at the 20-year return period event (T_{R0}).
2. Calculate structure strength and dissipation and displacement capacity of the structural system (e.g.,: 23).
3. Associate a combined acceleration-displacement spectrum passing through the collapse point and calculate the associated P_{eC} (e.g.,: 38, 39).
4. Define a damage control point. This could be based on floor acceleration or drift limits or both. The issue is to correlate it to a potential loss level, say $L_{\mathrm{Df}} = 20\%$.
5. Associate a combined acceleration-displacement spectrum passing through the damage control (f) point and calculate the associated P_{ef} (e.g.: 35, 36).
6. Enter Equation 1 with P_{ef} and L_{Df} and obtain α.
7. Enter Equation 2 with α and calculate $\mathrm{EAL_D}$.
8. Estimate the ratio between maximum indirect and direct loss $\mathrm{R_{I/D}}$, based on the construction use.
9. Enter Equation 6 with α and $R_{\mathrm{I/D}}$ and calculate $\mathrm{EAL_{I+D}}$.
10. Design possible strengthening measure to decrease P_{eC} and recalculate both EALs, repeating steps 6–9.
11. Design possible measures to decrease P_{ef} and recalculate both EALs, repeating steps 6–9.
12. While P_{eC} may need to be taken below typical code values, in order to ensure a sufficiently low collapse risk to protect human life,[24] the value of P_{ef} depends on economic considerations; therefore, it will be appropriate to calculate the breakeven time, considering the cost of intervention and the EAL reduction.

As can be seen by the simple rearranging of these steps described previously, the assessment and identification of suitable retrofitting measures can be tentatively outlined via such an approach. While the detailed design and verification would no doubt come afterwards, these initial steps are intended to help engineers along a better path towards the identification of more resilient systems.

6 | CONCLUSIONS AND CLOSING REMARKS

Seismic design and assessment have long been focused on the structural performance of buildings, especially in its building implementation. This has always been in the name of protecting lives and limiting losses to buildings due to excessive damage. However, the direct link between some of the consequences and actual performance metrics that can be used in design has been missing for some time. The formulations derived and presented in this paper have the essential purpose of allowing a quick determination of the average expected annual loss, related to direct or indirect losses, as a function of a few simple parameters; thus, allowing a practical application of loss-based approaches to seismic design. The equations derived could be equally applied to a new design or an assessment context, as briefly described above.

Given the conceptual nature of the paper, such an approach would need further detailed examination many different contexts for different structural typologies, seismic hazards and national contexts but it is clear that it represents a shift in thinking from past building codes and is argued to be a more rational approach towards a more resilient society.

ACKNOWLEDGEMENTS

The work presented in this paper has been developed within the framework of the project “Dipartimenti di Eccellenza”, funded by the Italian Ministry of Education, University and Research at IUSS Pavia.

DATA AVAILABILITY STATEMENT

Data sharing not applicable to this article as no datasets were generated or analysed during the current study.

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How to cite this article: Calvi GM, O'Reilly GJ, Andreotti G. Towards a practical loss-based design approach and procedure. *Earthquake Engng Struct Dyn*. 2021;1–13. https://doi.org/10.1002/eqe.3530.