

Collapse fragility analysis of historical masonry buildings considering in-plane and out-of-plane response of masonry walls.

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ARTICLE INFO

Keywords:

Historical masonry buildings
Incremental dynamic analysis
Out-of-plane response
Real ground motion records
Record-to-record variability
Fragility curves

ABSTRACT

This paper explores the seismic fragility assessment of historical masonry buildings in the context of performance-based earthquake engineering, using multiple-record incremental dynamic analyses (MR-IDA). The methodology is applied to two case studies, the first being a stiff monumental structure, and the second a tall and slender masonry building. Both case studies are modelled in the OpenSees software using three-dimensional macroelements that account for the in-plane (IP) and out-of-plane (OOP) response of masonry walls. Simultaneously, the numerical models account for the influence of non-linear connections in floor-to-wall interfaces and wall-to-wall interlocking. The record-to-record variability induced by the random nature of the ground motion is addressed through earthquake selection consistent with the variability of European hazard. Ground motion records are normalised at a uniform level of seismic intensity to be sequentially increased for the computation of IDA curves. Once IDA curves and fragility functions are derived, the failure results are thoroughly discussed. The fragility functions accounting for the uncertainties arising from the record-to-record variability are contrasted against the results of the seismic assessments affected by multiple sources of uncertainty in material and modelling parameters from a previous study. Finally, the fragility curves derived from the methodology presented herein are directly compared with fragility functions from probabilistic seismic demand analysis (PSDA). Overall, IDA-based fragility functions are found to provide more conservative predictions than the ones obtained from the cloud-based approach using unscaled records.

1. Introduction

Throughout Europe, historic urban centres are mostly composed of masonry constructions, some of them being landmarks of cultural heritage [1]. Masonry structures are known to be highly vulnerable to earthquake damage. On top of that, a large portion of the European building stock is located in regions of significant seismicity [2,3]. Depending on the age of construction or the structural typology, masonry buildings might exhibit a very different behaviour during major earthquake events, as observed after the Faial (1998) earthquake in Portugal [4], the L'Aquila (2009) and Emilia (2012) earthquakes, both in Italy, the Lesvos (2017) earthquake in Greece [5–7], and more recently the Kahramanmaraş Earthquakes (M_w 7.7 and M_w 7.6) in Turkey [8]. It is paramount to adopt efficient procedures for assessing the seismic vulnerability of historical masonry constructions, which may be a cumbersome task when dealing with epistemic and aleatory

uncertainties [9–11]. In this sense, that deterministic assessment of such buildings may lead to unrealistic results given the significant sources of uncertainties in their structural characterization. Conversely, the probabilistic framework of Performance-based earthquake engineering (PBEE) [12,13] emphasizes several issues at the system level, such as collapse risk, repair costs, fatalities, and post-earthquake loss of functionality. Thus, this methodology denotes a useful tool for determining important outcomes and findings in the assessment of historical masonry construction. Besides, expected annual loss and collapse risk have been integrated lately in line with modern PBEE requirements [14]. In the context of PBEE, several methodologies have been implemented to tackle the collapse safety investigation of engineering structures, including single- and multi-record incremental dynamic analysis (IDA) [15], endurance time analysis (ETA) [16], and multiple stripe analysis (MSA) as a particular case of IDA with dynamic analyses performed at discrete levels of intensity [17,18].

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<https://doi.org/10.1016/j.engstruct.2024.118804>

Received 9 May 2024; Received in revised form 1 August 2024; Accepted 13 August 2024

Available online 24 August 2024

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IDA stands as one of the most efficient approaches for evaluation of the seismic resilience of the structural systems [19,20]. The fundamentals of IDA were rigorously established in [15], and its first practical application was depicted by Vamvatsikos et al. [21] on a medium-height older reinforced concrete (RC) structure. These contributions focused on aspects of structural behaviour revealed by IDA, such as the large record-to-record variability, the hardening and softening, the flatline, and the structural resurrection (further discussion on these concepts is presented in Chapter 2). Later on, Dolsek [22] introduced a set of models of a four-storey RC frame to address modelling uncertainties using the Latin hypercube sampling (LHS) method. It was found that including modelling uncertainty in extended IDA increases the computational cost and may have significant implications on the collapse capacity. Likewise, IDA methodology has been applied to seismic risk assessment of bridges, leading successfully to the identification of damage states (DSs) and derivation of fragility curves [23,24]. For the case of bridges, the PBEE probabilistic framework has also been adopted to estimate repair costs in recovery investments [25]. More recently, Repapis and Zeris [26] contrasted the results of static pushover analyses and IDA procedures for the case of reinforced concrete buildings with clay brick masonry infill walls, while He and Lu [27] focused on seismic fragility assessment of a super tall building with hybrid control using IDA methodology. Kazemi et al. [28,29] employed IDA techniques to study the seismic vulnerability of steel moment-resisting frames (SMRFs) and showed the improvement of performance levels when infill masonry is adopted as a retrofit strategy for the steel-damaged buildings. Other engineering structures have been examined in the context of PBEE through IDA such as concrete dams and isolated nuclear power plants [30,31].

The literature survey reveals that there is a scarcity of application of IDA techniques for the fragility assessment of masonry buildings, mainly attributed to the extensive computational demand in large-scale modelling of them [32,33]. For instance, Abo-El-Ezz et al. [34] approached the fragility assessment of low-rise stone masonry buildings through a displacement-based damage model accounting for the characteristics of existing stone masonry buildings in Eastern Canada. Vanin and Beyer [35] proposed a logic tree approach to incorporate the epistemic uncertainty in the seismic assessment of masonry building and pointed out the displacement capacity as a major source of bias compared to record-to-record variability. Chieffo et al. [36] estimated the vulnerability of an isolated masonry building in terms of expected damage using macroelements and non-linear static analyses. The variability of structural and geometrical parameters in the fragility assessment of masonry building aggregates, through non-linear static analyses, was accounted for in the works of Battaglia et al. [37] and Angiolilli et al. [38]. Thanks to the advances in the macroelement modelling strategy introduced in [39], Tomić et al. [40] successfully accounted for the randomness in material properties of historical masonry buildings within the single-record IDA framework. Recently, Bernardo et al. [41] provided analytical fragility curves, supported by non-linear static analysis, for representative building archetypes accounting for the uncertainty in materials properties, geometry, and seismic demand for several levels of seismicity in Portugal. While the focus of these investigations has been on uncertainty scenarios arising mainly from geometric and material properties the effects of ground motion variability have been left aside, and there is a clear need for their investigation.

This paper adapts the multi-record IDA procedure to evaluate the seismic fragility of historical masonry buildings in the context of PBEE. The proposed framework is applied to two historical masonry constructions: (1) the Holsteiner Hof building, as an example of stiff monumental heritage structures; and (2) the Lausanne Malley, representative of tall and slender residential masonry buildings. The OpenSees software [42] is adopted for the numerical modelling of the case studies, in which three-dimensional macroelements, proposed recently by Vanin et al. [39], are used to account for IP and OOP effects of

masonry walls. Moreover, the effects of non-linear connections in floor-to-wall interfaces and wall-to-wall interlocking are considered by implementing simplified frictional and tension-damage one-dimensional constitutive laws, respectively [43]. A set of accelerograms consistent with the European hazard is selected to cover the record-to-record variability, and non-linear time history analyses are conducted subsequently. Within this process, ground motion records are normalised first at a uniform level of intensity and scaled up sequentially for the computation of IDA curves. The fragility curves derived from this process, which account exclusively for uncertainty stemming from the record-to-record variability, are contrasted against the results of the fragility analysis for the same buildings accounting for modelling uncertainties through single-record IDA methodology [40]. In addition, a final comparison between the fragility functions computed by means of the framework proposed herein and the ones obtained from probabilistic seismic demand analysis (PSDA) [44,45] is presented.

2. Theoretical background

The main goal of this investigation is to implement multiple-record IDA for the collapse fragility assessment of historical masonry buildings in the context of PBEE. Hence, it is needed to predict the structural performance of the buildings under analysis for a full range of credible earthquake ground motion intensities [46]. Through probabilistic seismic response analysis, one can determine the mean annual rate of exceedance of a structural response parameter, regarding and engineering demand parameter (EDP) exceeding certain level of edp [47]. Mathematically, this process can be expressed by means of the total probability theorem as:

$$\nu(\text{EDP} > edp) = \int_0^\infty (1 - P[\text{EDP} < edp | \text{IM}]) \left| \frac{d\nu(\text{IM})}{d\text{IM}} \right| d\text{IM} \quad (1)$$

where $P[\text{EDP} < edp | \text{IM}]$ denotes the probability that the EDP is smaller than a certain level of edp at the ground motion intensity measure (IM), and $\nu(\text{IM})$ is the mean annual rate of exceedance of the ground motion intensity IM.

The following subchapters explain the concepts of fragility functions as a mean to quantify the probability of reaching a particular level of DS—or collapse ultimately—at a certain level of IM, as well as MR-IDA as the adopted procedure for its computation.

2.1. Analytical collapse fragility functions

Conceptually, fragility functions were introduced in the field of earthquake engineering by Kennedy et al. [48]. Fragility curves state the conditional probability of a structural system reaching or exceeding a DS or a predefined level of EDP given an IM parameter [49]. Mainly, these functions can be grouped into three types, empirical, analytical, and heuristic [50]. Empirical fragility curves are reproduced after earthquake damage observations [51,52] while heuristic functions are based on experts' opinions [53]. Conversely, the analytical methodology requires structural modelling and numerical simulations since they are constructed based on non-linear dynamic analysis [54–57]. The analytical collapse fragility function is defined in Eq. (2).

$$P[C | \text{IM} = x] = \Phi\left(\frac{\ln(x/\eta)}{\beta}\right) \quad (2)$$

where $P[C | \text{IM} = x]$ denotes the probability of a structural system to collapse under a ground motion with an intensity level x , Φ is the standard normal cumulative distribution function, η is the median of the fragility function, and β is the logarithmic standard deviation, also referred to as dispersion.

Eq. (2) assumes that the values of IM of ground motions producing the collapse of the structural system follow a log-normal distribution. Previous investigations have confirmed the validity of this assumption

[49,58–61]; nonetheless, alternative distributions can be implemented. Furthermore, the implementation of Eq. (2) requires the calibration of η and β from the results of structural analysis. The fitting of a fragility function to the analytical set of data points leads then to the estimation of $\hat{\eta}$ and $\hat{\beta}$. In this regard, Baker [18] exposed common statistical approaches for estimating these parameters from analytical data. Among them: (i) the method of moments (MM) in which the parameters $\hat{\eta}$ and $\hat{\beta}$ are found such that the resulting distribution has the same moments (median and standard deviation) as the data points; (ii) the sum square error (SSE) that minimizes the error between the observed fractions of collapse data and probabilities observed within the fragility function; and (iii) the maximum likelihood estimation (MLE) in which the $\hat{\eta}$ and $\hat{\beta}$ parameters in the resulting distribution have the highest likelihood of having produced the observed data. Prior to fitting, percentile IDA curves should be used to derive fragility curves depicted by single dots (i.e., each dot representing a ratio of the number of records that caused structural collapse to the total number of records at an IM level).

Former investigations have addressed the fragility assessment of unreinforced masonry buildings by measuring structural damage in terms of drift ratio [37,38,62,63]. The values of drift limits reported in these investigations will serve as a reference to establish the asymptotic values of EDPs for deriving analytical fragility functions at collapse DS.

2.2. Multiple-record incremental dynamic analysis MR-IDA

IDA methodology implies the sequential increment of intensity for each ground motion in a suite until they cause numerical collapse in the non-linear dynamic analysis, which is interpreted as structural collapse [15]. Overall, the methodology can be summarised as: (i) Develop a robust numerical model for the structural system under analysis able to capture potential sources of non-linearity (e.g., concentrated/distributed plasticity, second-order effects, etc.); (ii) Select an adequate suite of records to capture the ground motion variability [47]; (iii) For a given IM parameter and EDP, plot each single-record IDA (capacity curve) including the median and the 16 % and 84 % quantiles of the distribution, to study the seismic demand from low seismic intensity levels prior to yielding up to ultimate performance levels; (iv) Use the IDA data to analyse deeply and better understand the behaviour of the structure (e.g., identification of limit-state capacities, collapse fragility analysis, etc.).

It should be noted that because of the complex structural behaviour frequently observed in masonry structures, influenced by multiple sources of non-linearity (i.e., materials, second-order effects, floor-to-wall, and wall-to-wall connections, etc.), and large record-to-record variability, IDA curves generated by different earthquake motions often result in quite dissimilar responses that are difficult to predict a priori [15]. Thus, fractile curves including the median, 16 % and 84 % percentiles, are a suitable approach for summarising IDA data. These percentile curves are much smoother than individual IDA curves and can better represent the global behaviour of a structure [23]. Besides, limit states can be easily identified on these curves. Immediate occupancy (IO) is violated when the building exceeds a certain level of EDP still within the elastic range; collapse prevention (CP) is reached when the local slope on the IDA curve decays to 20 % of the initial elastic slope; and finally, the global dynamic collapse (GC) is clearly observed at the flatline, where the structure responds with practically infinite EDP to any IM increase [64]. Depending on the particular dynamic behaviour of the structural system, IDA curves may exhibit a softening response; minor hardening; severe hardening; and also, a weaving behaviour (see Fig. 1a). Additionally, Fig. 1b depicts the resurrection phenomenon which seems to be a premature failure at a given seismic intensity level, but a safe response at a higher one [30].

The hunt & fill algorithm [15] has been used to optimally select the scaling levels of IMs and minimize the number of runs by finding prematurely the level of GC and performing additional analyses at intermediate levels. While this strategy has been successful in different applications [65–67], the capacity curves of masonry buildings are expected to exhibit discontinuous behaviour marked by the incidence of structural resurrection, which turns the early identification of GC into a more cumbersome task. Thus, the hunt & fill algorithm is not used, and discrete levels of IMs are adopted to characterise the overall structural behaviour of the case studies from elasticity to yielding and finally collapse.

3. Numerical modelling of buildings and ground motion characterisation

As mentioned, IDA implies performing a series of non-linear dynamic analyses of a structure, using multiple records scaled up to higher levels of intensity, to cover the overall model's behaviour, from elastic to yielding and inelastic incursion, finally leading to global dynamic

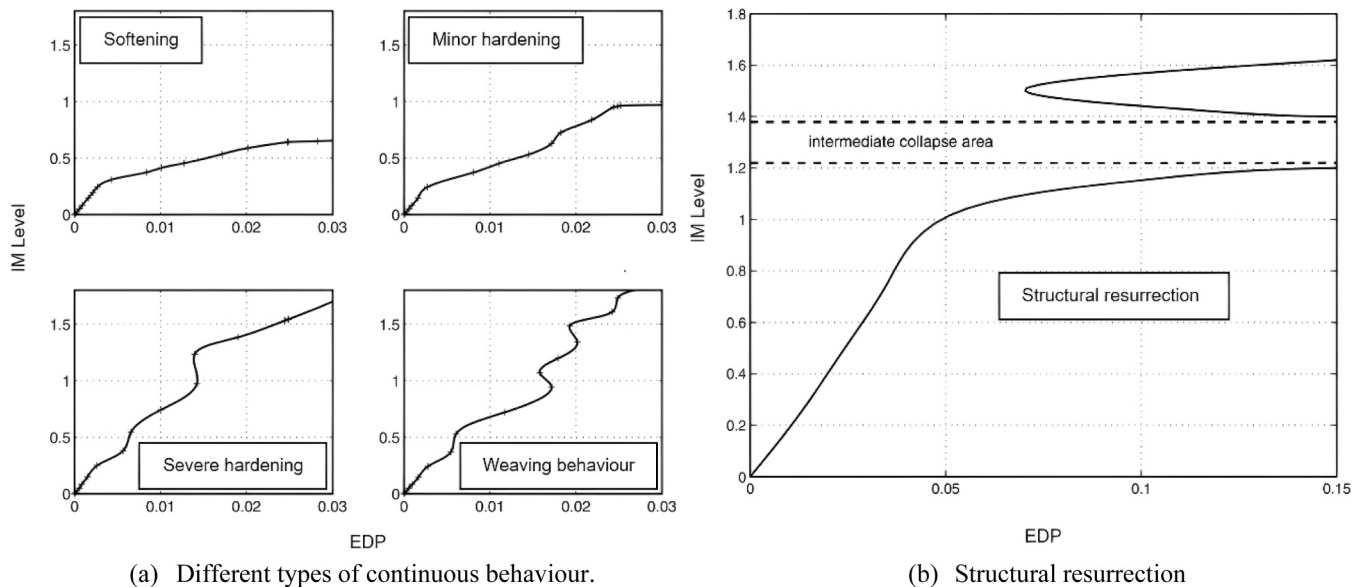


Fig. 1. IDA curves typologies
Source Vamvatsikos and Cornell [15].

collapse. Therefore, the structural models for IDA should be capable of simulating inelastic deformation, force redistribution due to sequentially deterioration in the inelastic range, contribution of higher modes, and structural collapse ultimately. Below, the description of the case studies and numerical modelling approach are presented, as well as the record selection strategy.

3.1. Numerical modelling of buildings

Two buildings in Switzerland, representative of typical stiff monumental buildings, and tall and slender masonry buildings, are selected as case studies. The first one corresponds to the Holsteiner Hof building located in the city centre of Basel and considered a cultural heritage landmark according to the Swiss Inventory of Cultural Property of National and Regional Significance. The second case study resembles a residential URM building located in the city of Lausanne. Table 1 provides a general description of the buildings' topology regarding the number of storeys, dimensions in plan, storey height, wall thickness, floor and roof system, and retrofitting. A more detailed description of the building can be found in [40,68].

Both structures are modelled in OpenSees [42] through the equivalent frame model (EFM) approach using a recently developed three-dimensional macroelement [39] to account for IP and OOP effects simultaneously. The macroelement is formulated as a one-dimensional element with two nodes at the element ends and one additional node at the midspan through which the non-linear shear/flexural response is accounted for [70]. The element can capture the IP and OOP response through three sectional models applied at the element ends and at the central section (P- Δ formulation is considered to capture the non-linear geometrical effects within compatibility equations). Drift values can be calculated individually for flexural and shear deformations by considering the rotations and lumped shear deformations at the central node. Exceeding the limits in drift values ($\delta_{c,flexure}$ or $\delta_{c,shear}$) will lead to the loss of lateral strength of the element.

The floor system is modelled using orthotropic elastic membranes with higher stiffness in the direction of the beam span and lower stiffness in the other direction (i.e., membrane definition is given by the two moduli of elasticity in the orthogonal directions, shear modulus, and thickness of the diaphragm). Nonetheless, floor-to-wall connections are modelled to account for non-linear behaviour and potential connection failure at the beam support, which can result in the OOP failure of a pier element. In this regard, zero-length elements are adopted for modelling the floor-to-wall connections and a simple material model for frictional interfaces is defined. The local direction "x" of this material is given by the direction in which the vertical load is acting. In the perpendicular plane, frictional sliding is allowed ruled by a friction coefficient, μ , and no cohesion. The material can fail and lead to the loss of load-bearing capacity if a maximum slip is exceeded in the positive local "y" direction, defined by the orientation of the beam. Furthermore, the material can model pounding of the beam when the slip in the negative direction "y" or any direction "z" exceeds a predefined "gap" value. Likewise,

Table 1
Description of the buildings' topology.

Building	Holsteiner Hof	Lausanne Malley
General	Stiff monumental heritage structure	Tall and slender residential masonry building
N° storeys	2	6
Dimensions in plan	26.00 m × 14.00 m	14.00 m × 12.00 m
Storey height	4.50 m	2.80 – 3.20 m
Wall thickness	60 – 30 cm	60 – 25 cm
Floor system	Timber beams and planks	Timber beams and planks
Roof system	Secondary wooden truss structure	Secondary wooden truss structure
Retrofit	Minor during 1976–1979.	Soundproofing and seismic performance[69].

wall-to-wall interfaces are modelled through zero-length elements to which a proper uniaxial material model is attached to simulate the formation of vertical cracks and separation of the orthogonal walls, which might lead to potential OOP failure of macroelements. The material features a linear elastic behaviour in compression, with no crushing, and a finite tensile strength with exponential softening. Fig. 2 describes the simplified frictional and tension-damage one-dimensional constitutive laws adopted for the modelling of floor-to-wall interfaces and wall-to-wall interlocking, respectively.

To perform the non-linear dynamic simulations, a 5 % proportional Rayleigh damping is assumed, and a secant stiffness damping model is adopted to avoid overdamping in the OOP failure mechanism, and to capture the full OOP rocking response. Additionally, it should be mentioned that the dynamic simulations conducted herein do not take into account the role of soil deformability or non-linear soil-structure interaction (SSI). Table 2 summarises the modelling parameters adopted as the mean or median values reported by Tomić et al. [40].

The symbol (*) over the values in the third column of Table 2 denotes the median value taken from a lognormal distribution. The median values of 1 reported for k_{floors} , f_w , and μ_{f-w} , were implemented in [40] to vary the values of the floor stiffness, the stiffness of the interlocking interface, and the friction coefficient that governs the frictional sliding (see Fig. 2a). Nonetheless, their influence is neglected in this research since only the effect of record-to-record variability is investigated as main source of uncertainty. Fig. 3 illustrates the numerical models of the Holsteiner Hof and Lausanne Malley buildings, respectively, as well as their first three vibration periods, T_1 , T_2 , and T_3 .

3.2. Ground motion characterisation

One of the main aspects of IDA and collapse capacity estimation lies in the utilisation of a proper record selection strategy for the inputs of inelastic time history analysis [23]. For Eq. (1) to be valid, the seismic inputs should be selected in such a way that the main seismological features (e.g., Moment magnitude, M_w ; distance metrics, R_{JB} ; shear wave velocity, V_{s30} ; and site conditions; etc.) affecting the levels of IMs, are adequately represented in the selected suite of motions. To this end, a large database of accelerograms is collected covering the hazard of the most relevant seismic-prone areas in Europe (i.e., Italy, Greece, Turkey, Portugal, etc.) [79–82]. A methodology based on unconditional selection, not dependent on structural periods, by Jayaram et al. [83] is implemented for the definition of the suite of motions, composed of 21 recordings whose response spectra target the mean and variance pre-defined by the confidence interval of the European dataset (i.e., means and covariances are determined based on the predefined target spectrum). Within the selection process, the seismological parameters are set as: $4.5 \leq M_w \leq 7.8$; $90 \text{ m/s} \leq V_{s30} \leq 1050 \text{ m/s}$; $R_{JB} \leq 185 \text{ km}$; and no pulse-like records [84,85]. Table 3 shows the 21 selected motions and provides their description regarding the name of the earthquake, event ID, latitude and longitude of the epicentre (Lat. epi and Long. epi), depth of the focus defined by the focal depth (FD), M_w , R_{JB} , and V_{s30} . Lastly, Fig. 4 shows the 5 % damped geometric mean spectral acceleration (S_a) of the selected records, including the mean, median, and 95 % confidence interval.

Table 3 shows different soil conditions, mainly soil types C and D according to Eurocode 8 classification [86]. It should be remarked that the methodology adopted herein for ground motion selection does not target a specific soil type but is based on the mean target defined from the collected dataset, where the predominance of recorded accelerograms in stations with soil types C and D is notorious. Thus, the effect of soil conditions on fragility analyses of historical masonry constructions remains a pending issue and requires further investigation.

4. Analysis of results

This section presents the discussion of IDA results in regard to the

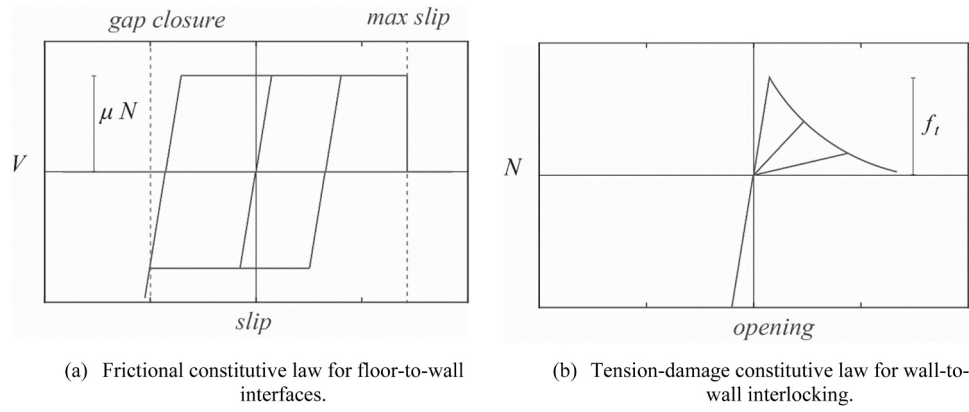


Fig. 2. One-dimensional constitutive laws to model non-linear connections [43].

Table 2
Masonry and modelling parameters.

Parameter	Definition	Mean value
<i>Masonry Parameters</i>		
E_m [Pa]	Modulus of elasticity[71–73]	3.5×10^9
G_m [Pa]	Shear modulus[71–73]	1.5×10^9 *
f_{cm} [Pa]	Compressive strength[71–73]	1.3×10^6
c_m [Pa]	Cohesion[71–73]	2.33×10^5 *
μ_m [-]	Friction coefficient[71–73]	0.25 *
<i>Modelling Parameters</i>		
k_{floor} [-]	Floor stiffness factor[74,75]	1 *
f_w [-]	Wall-to-wall connection factor[76]	1 *
μ_{f-w} [-]	Floor-to-wall friction coefficient[43,77]	1 *
$\delta_{c,flexure}$ [-]	Drift capacity in flexure[78]	1.04 %*
$\delta_{c,shear}$ [-]	Drift capacity in shear[78]	0.70 %*
ζ [-]	Damping ratio[43]	5 %

Holsteiner Hof and Lausanne Malley buildings. To accomplish this task, ground motion records are normalised at a uniform level of intensity, and non-linear dynamic analyses are conducted increasing gradually the level of intensity at each new analysis. The large number of analyses provides ample collapse data which is examined thoroughly to characterise the phenomena. Subsequently, capacity curves from single IDA are derived and summarised using smooth percentiles (16 %, 50 %, and 84 %, respectively) in which performance points are identified to study the behaviour of structures throughout different DSs. The results computed through the methodology proposed herein account exclusively for the effects of record-to-record aleatory randomness while material properties are assumed to be deterministic (see Table 2). However, the investigation by Tomić et al. [40] dig into the effects of the uncertainty stemming from limited data in material and modelling properties by reproducing 400 combinations of 11 modelling parameters through the Latin hypercube sampling (LHS) [87]. Hence, the results of collapse fragility analyses accounting independently for both sources of uncertainty are compared. A final comparison between IDA and PSDA approaches for historical masonry buildings is presented

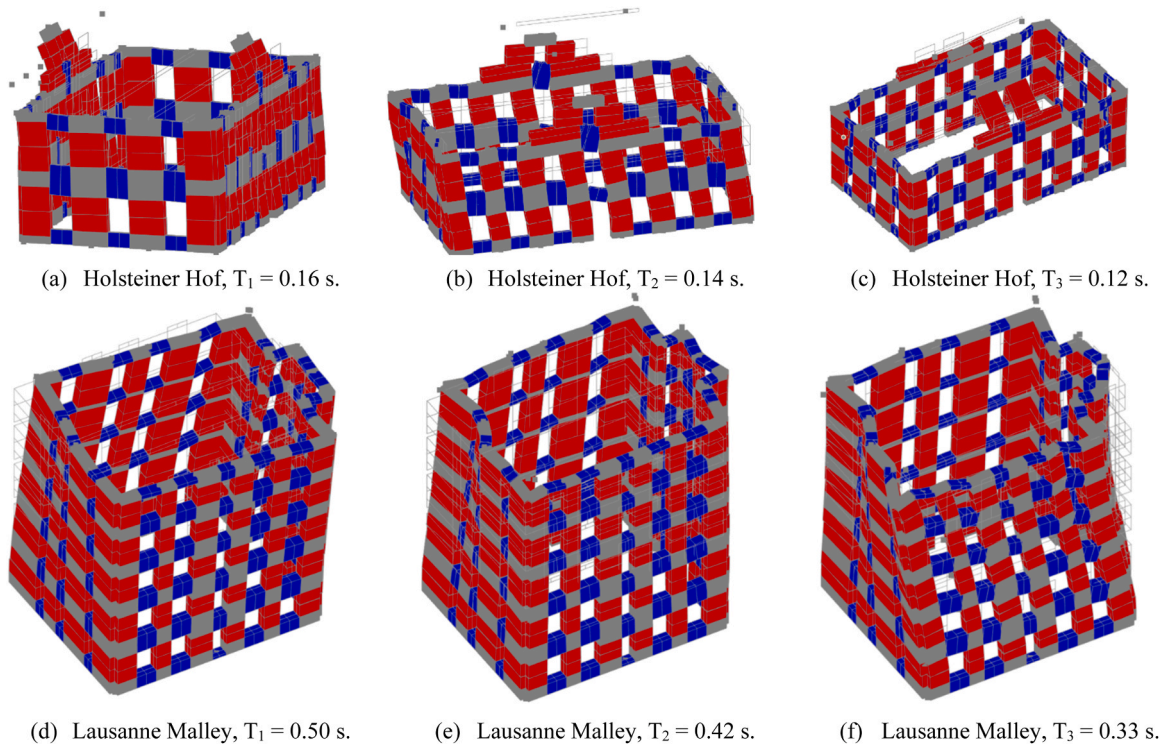


Fig. 3. Numerical models and fundamental vibration periods.

Table 3
Selected ground motions.

Record No.	Name	Event ID	Lat. epi [km]	Long. epi [km]	FD [km]	M _w	R _{JB} [km]	V _{s30} [m/s]
1	TK.3805. HNX.D.INT-20230206_0000008. ACC.MP.ASC	INT-20230206_0000008	37.17	37.08	20.00	7.8	175.45	384.00
2	IT.MSCT.00. HGX.D.EMSC-20161026_0000095. ACC.MP.ASC	EMSC-20161026_0000095	42.90	13.09	9.60	5.9	38.80	652.00
3	HL.NAX1. HNX.D.EMSC-20201030_0000082. ACC.MP.ASC	EMSC-20201030_0000082	37.91	26.84	10.00	7.0	134.88	461.26
4	IT.SDM.00. HGX.D.EMSC-20160824_0000006. ACC.MP.ASC	EMSC-20160824_0000006	42.70	13.23	8.10	6.0	45.23	752.00
5	IT.CPS.00. HGX.D.EMSC-20161030_0000029. ACC.MP.ASC	EMSC-20161030_0000029	42.83	13.11	10.00	6.6	64.34	732.00
6	IT.ASG.00. HNX.D.IT-1976-0002. ACC.MP.ASC	IT-1976-0002	46.26	13.30	5.70	6.4	127.85	960.00
7	IT.CPC.00. HNX.D.IT-2012-0011. ACC.MP.ASC	IT-2012-0011	44.84	11.07	8.10	6.0	54.31	174.00
8	HL.MSLA.00. HNX.D.GR-1993-0027. ACC.MP.ASC	GR-1993-0027	38.20	21.77	12.40	5.6	32.93	255.40
9	IT.MSC.00. HGX.D.EMSC-20160824_0000013. ACC.MP.ASC	EMSC-20160824_0000013	42.79	13.15	8.00	5.5	31.10	652.00
10	IT.FMG.00. HNX.D.IT-2009-0009. ACC.MP.ASC	IT-2009-0009	42.34	13.38	8.30	6.1	19.26	790.00
11	IT.SSU.00. HNX.D.IT-2012-0008. ACC.MP.ASC	IT-2012-0008	44.90	11.26	9.50	6.1	51.02	489.00
12	TK.3146. HNX.D.INT-20230206_0000222. ACC.MP.ASC	INT-20230206_0000222	38.11	37.24	10.00	7.5	180.33	641.30
13	IT.TRL.00. HGX.D.EMSC-20170118_0000034. ACC.MP.ASC	EMSC-20170118_0000034	42.53	13.28	9.60	5.5	23.88	380.00
14	IT.RTL.00. HGX.D.EMSC-20170118_0000037. ACC.MP.ASC	EMSC-20170118_0000037	42.50	13.28	9.40	5.4	35.18	170.00
15	IT.NOR.00. HGX.D.EMSC-20170118_0000119. ACC.MP.ASC	EMSC-20170118_0000119	42.47	13.27	9.50	5.0	36.51	423.00
16	HL.VAS2. HNX.D.EMSC-20151120_0000014. ACC.MP.ASC	EMSC-20151120_0000014	38.47	20.49	12.00	4.7	19.50	260.70
17	IT.CTL.00. HNX.D.INT-20221109_0000046. ACC.MP.ASC	INT-20221109_0000046	43.98	13.32	5.00	5.5	43.88	208.00
18	IT.GBP.00. HNX.D.IT-1998-0063. ACC.MP.ASC	IT-1998-0063	43.18	12.77	10.00	4.8	20.11	224.00
19	IT.CSC.00. HNX.D.IT-1997-0004. ACC.MP.ASC	IT-1997-0004	43.02	12.89	5.70	5.7	27.34	698.00
20	IT.AQK.00. HNX.D.IT-2009-0174. ACC.MP.ASC	IT-2009-0174	42.50	13.38	9.00	5.0	15.41	705.00
21	IT.CLF.00. HGX.D.EMSC-20161103_0000003. ACC.MP.ASC	EMSC-20161103_0000003	43.03	13.05	8.10	4.7	9.25	145.00

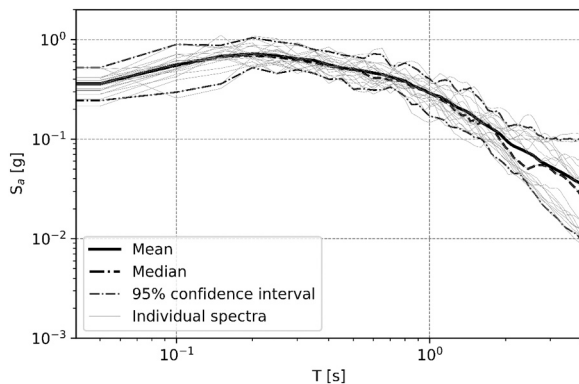


Fig. 4. 5 % damped geometric mean S_a of selected records.

using the results reported in Caicedo et al. [68].

4.1. Holsteiner Hof

A total of 399 analyses were conducted with intensity levels distributed as [0.05 0.10 0.15 0.20 0.25 0.30 0.35 0.40 0.45 0.50 0.55 0.60 0.65 0.70 0.80 0.90 1.00 1.10 1.20] out of which 271 led the building to global collapse; all of them at intensity levels higher than 0.20 g of peak ground acceleration (PGA). Collapse is observed mainly in two scenarios: (i) when P – Δ effect triggers the loss of the global equilibrium after reaching excessive levels of OOP deformation, and (ii) induced by a series of IP failures which makes it impossible to reach convergence and local equilibrium. Moreover, when the macroelement reaches either $\delta_{c,flexure}$ or $\delta_{c,shear}$ limits, the lateral stiffness and strength are set to zero, but the pier retains the ability to transfer axial load. When 50 % of the piers in the same direction of one storey reach their drift limits, the IP failure is labelled either as flexure or shear. Moreover, mixed IP failure can be identified when approximately the same amount of piers exceed the $\delta_{c,flexure}$ and $\delta_{c,shear}$ limits. Analogously, mixed IP-OOP failure may be identified as well. On this occasion, 44 % of the failure observations correspond to OOP, while the remaining 56 % are IP failures. Within the IP observations, flexure IP is predominant with 58 % of occurrences, followed by 25 % of mixed IP failures, and merely 17 % by pure shear IP (88, 38, and 26 collapse cases, respectively). Approximately, 60 % of the IP failures are evenly distributed between the second

floor 2 in the “x” and “y” directions (F2 x-dir and F2 y-dir). 14 % of IP are mixed located while the remaining 29 % is distributed among first floor “x” and “y” directions and the third one in “x” direction (F1 x-dir, F1 y-dir, and F3 x-dir). 54 % of the OOP occurs at F1 y-dir while 24 % and 19 % occur at F1 x-dir and F2 x-dir, respectively. Only a small portion of the failures is observed at F2 y-dir which correspond exclusively to 3 analyses. In general, all OOP failures correspond to overturning of the main façade and central node overturning. Results of failure analyses for the Holsteiner Hof building are summarised by means of the statistics portrayed in Fig. 5.

4.1.1. IDA curves and fragility functions

Capacity curves from IDA are plotted in Fig. 6 showing the maximum values of EDP, adopted as the maximum values of average roof displacement ($\max(\bar{\Delta}_r)$). The equivalent levels of global drift ratio are shown as well at the secondary axis on top. Because of the high levels of non-linearity, severe hardening, and weaving behaviour are predominant within individual curves. Structural resurrection is also detected—sometimes more than once within the same curve—and it is attributed simply to problems with convergence in the non-linear dynamic analyses. At least 11 individual IDA curves are necessary to accurately estimate the quartile curves. 16th, 50th, and 84th percentiles are useful for interpretation of these capacity curves, and to better capture the full performance of the structure through different DSs, from linear elastic to inelastic behaviour, until collapse at last. For this purpose, performance points are identified. For the Holsteiner Hof building, the limit for IO is specified approximately at the end of the elastic branch at $\max(\bar{\Delta}_r)$ lower than 5 mm or global drift ratios lower or equal to 0.05 %. CP, on the other hand, is recognized when the slope of the IDA curve reduces to 20 % of the initial elastic slope; all CP performance points are identified at drift ratios lower than 0.15 %. Finally, the GC is revealed by the appearance of the flatline at $\max(\bar{\Delta}_r) = 40$ mm, or global drift ratios approaching 0.45 %. It should be noted that these observations are coherent with the findings of previous investigations on the global drift ratio limits of unreinforced masonry buildings [37,38,62, 63] for the derivation of fragility functions at different levels of DS, as stated in Section 2.1. Furthermore, recent investigations [88–90] identified, numerically and experimentally, drift-based DSs for historical masonry structures, showing consistency with the numerical results attained herein. Additionally, Fig. 6 illustrates how the dispersion induced by the record-to-record variability (β_R) influences MR-IDA curves when considering different IMs. The values of β_R are reported

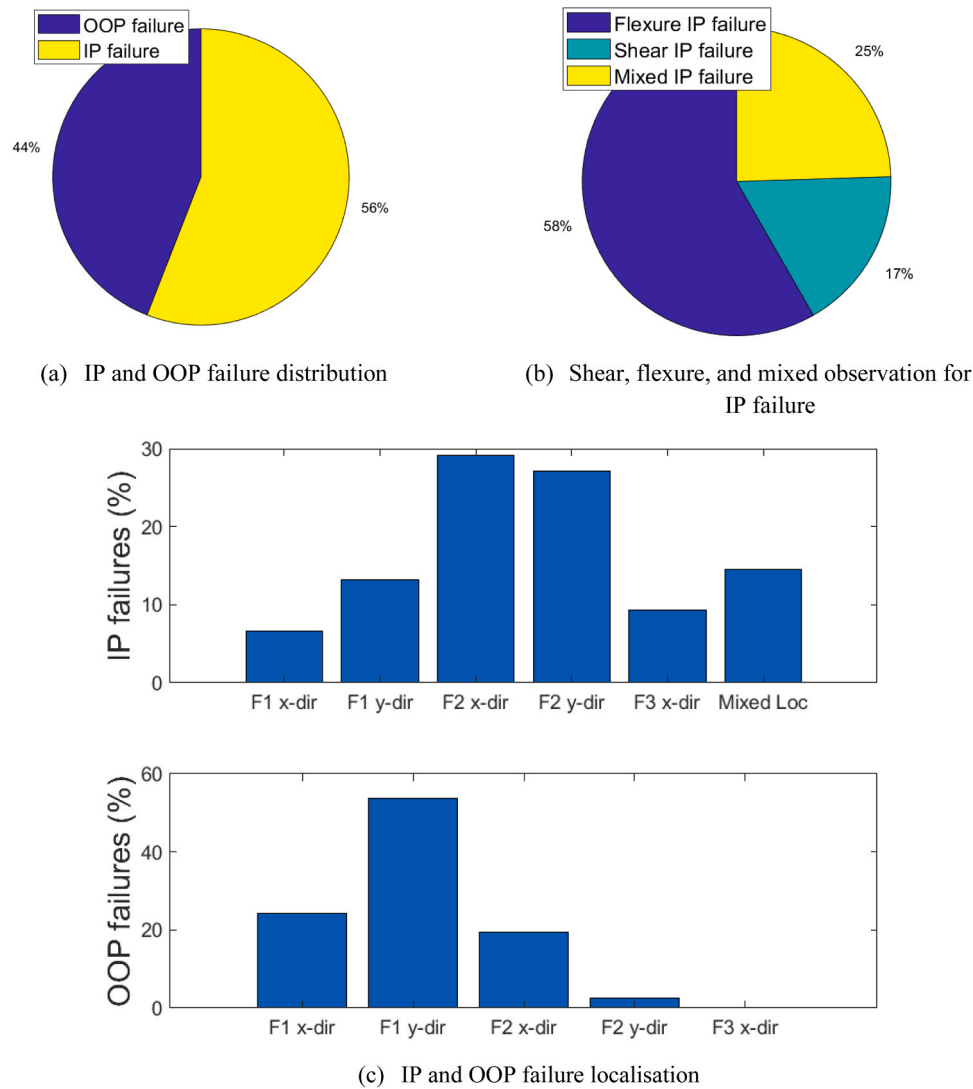


Fig. 5. Failure statistics — Holsteiner Hof.

in Table A1 for a full list of IMs identified previously in the research by Caicedo et al. [68]. Besides PGA, which defines the level of intensity within the performed IDAs, the acceleration spectrum intensity (ASI) is depicted in Fig. 6b since it shows the lowest value of β_R , being outperformed just by the composed metric I_{comp} (See Fig. 6c) proposed specifically for the dynamic assessment of masonry buildings [68]. Interestingly, through the values of β_R the same IMs as in [68] are identified as the optimal ones to study the seismic dynamic behaviour of the Holsteiner Hof building. The best ranked IMs (i.e., lowest β_R) include PGA; ASI; Effective Peak Acceleration, EPA; Improve Effective Peak Acceleration, IEPA; Cordova intensity, S_a^* ; and Vamvatsikos intensity, either $\bar{S}_a$ or $\bar{S}_a$.

Now, results from IDA are employed to build collapse fragility curves to measure the probability of failure at different levels of seismic intensity. Collapse observations represent the ratio of the number of records that induced GC to the total number of records at an IM level, to which the best distribution is fit using the SSE method, which exhibits the highest computationally efficiency among all the three methods reported in Section 2.1 and leads to practically same results [30]. The values of $\hat{\eta}$ and $\hat{\beta}$ and corresponding analytical functions fitting the collapse data points through a lognormal cumulative distribution (LogCDF) are shown in Fig. 7 using the same IMs as for IDA curves. Alternatively, collapse fragility could be defined as the likelihood of

exceeding the global drift ratio limit of 0.45 % defined in Fig. 6 for the GC. A deeper discussion of these functions is presented in the following subsections when contrasting the fragility analysis accounting for uncertainty in modelling parameters vs. record-to-record variability, and the IDA approach vs. PSDA with unscaled records.

4.1.2. Material and modelling uncertainties vs. record-to-record variability

While the uncertainty induced by the record-to-record variability (i.e., random nature of the ground motion) is quantified by the $\hat{\beta}_R$ parameter in this research, modelling parameters are assumed to be deterministic, thus, neglecting them as a source of uncertainty. Nonetheless, the investigation by Tomić et al. [40] accounted for the uncertainty of material and modelling parameters by performing LHS to generate 400 variations of the 11 parameters reported in Table 2. As in the current research, collapse fragility curves were generated, assuming PGA as IM. In this regard, the fitting parameters for the Holsteiner Hof building accounting for modelling uncertainties are $\hat{\eta}_U = 0.3592$ and $\hat{\beta}_U = 0.0873$. Such parameters showed consistency with the set of fitting parameters exposed in Fig. 7a for PGA. Ideally, both sources of uncertainty should be considered simultaneously. Nevertheless, accounting for both in the seismic assessment of historical masonry buildings can be extremely expensive from a computational standpoint, and it is the main reason for its consideration separately. Alternatively, Cornell et al. [91]

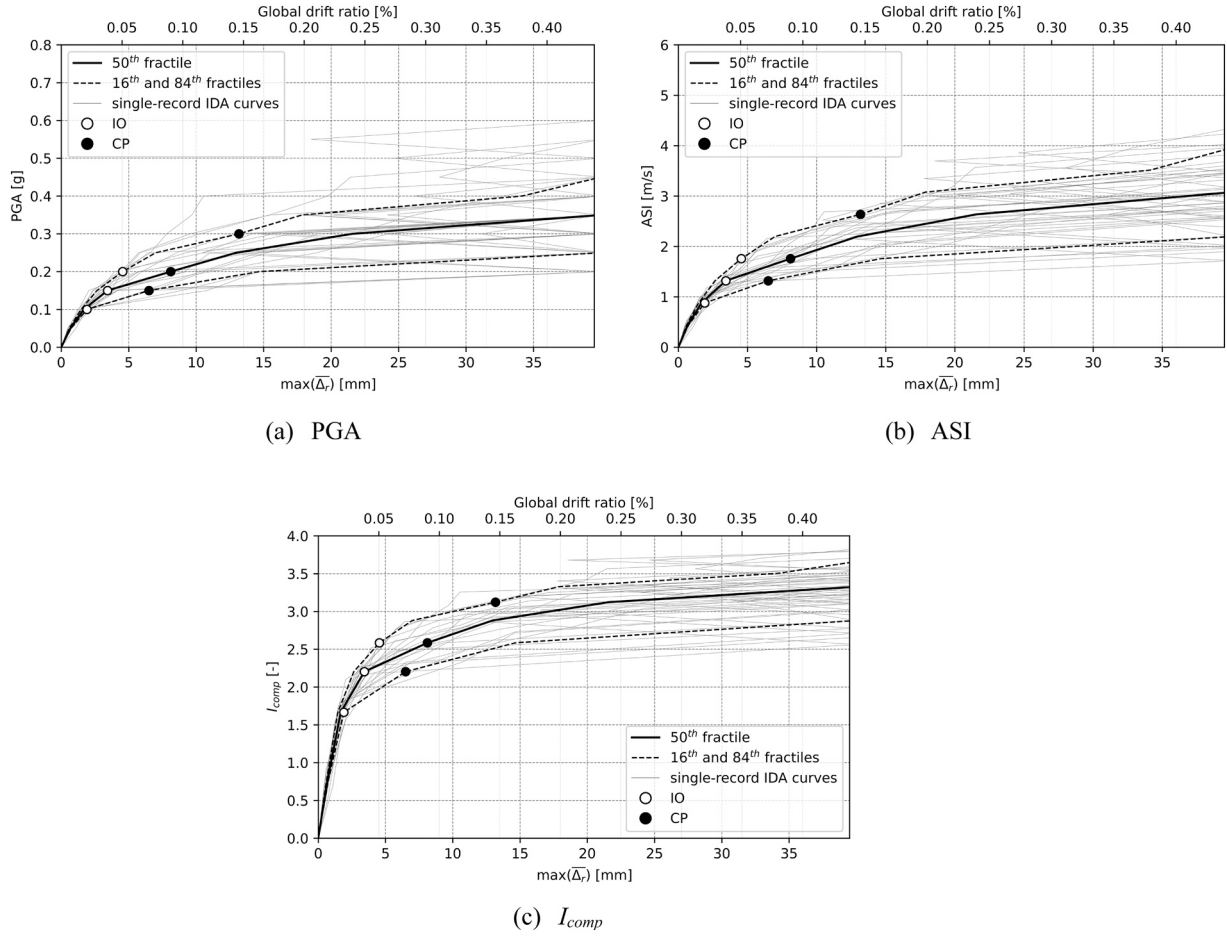


Fig. 6. IDA curves displaying maximum values of EDPs — Holsteiner Hof.

proposed a simplified method to combine analytically both sources of uncertainty by means of Eq. (3):

$$\beta_{RU} = \sqrt{\beta_R^2 + \beta_U^2} \quad (3)$$

Hence, Fig. 8 illustrates the impact of modelling uncertainties and record-to-record variability on the collapse fragility of the Holsteiner Hof building, plus an additional curve accounting for both effects through Cornell et al. approach [91]. The median for the mixed effect was fixed as $\hat{\eta} = 0.3259$ (shown in Fig. 7a), however, adopting $\hat{\eta}_U$ leads to practically indistinguishable results because of the proximity in the median values. Although similar, the effect of record-to-record variability leads to a slightly more brittle response, reaching, for instance, a collapse probability near 80 % at PGA = 0.4 g, while at the same level of intensity the probability is lower than 70 % when accounting solely for modelling uncertainties. On the other hand, for the combined effects it is shown how, in contrast to the other curves, higher collapse probabilities are obtained up to an intensity level close to 0.4 g. Contrastingly, lower collapse probabilities are obtained after this threshold.

4.1.3. MR-IDA vs. PSDA

Cloud-based fragility curves can be obtained after the aggregation of the results from probabilistic seismic demand models (PSDMs). Within this methodology, the structural system is examined under the action of unscaled ground motion records, in which the EDP vs. IM cloud response delivers the conditional probability of an EDP reaching or exceeding a certain level of *edp*, at a given level of seismic intensity, i.e., $P(\text{EDP} \geq \text{edp} | \text{IM})$ [92]. Fig. 9 compares the fragility functions at collapse DS, obtained after MR-IDA in the current study, and the cloud-based approach using the results reported in the research by Caicedo et al.

[68]. Clearly, the MR-IDA approach provides much more conservative results that, as previously shown, match entirely the actual distribution of numerical collapse observations reproduced herein. On top of that, the suite of non-linear dynamic analyses conducted in [68] led the Holsteiner Hof building to global collapse only in 9 cases which, evidently, are not enough to characterize the collapse distribution in the whole range of seismic intensity. Moreover, within the cloud-based approach, the moments of the distribution (i.e., median and standard deviation) are estimated using the sample moments from a set of data, and further considerations should be accounted for including numerical failures, leading to inaccurate results when dealing with lack of data [30, 93,94].

4.2. Lausanne Malley

For the second case study the bins of intensity are distributed as [0.05 0.10 0.15 0.20 0.25 0.30 0.35 0.40 0.50 0.60]. Different than for the first case study, at PGA = 0.40 g mostly collapse is observed, thus, bins of 0.5 and 0.6 were utilised merely to discard any further structural resurrection. In this regard, 172 out of 210 analyses led the building to global instability.

Fig. 10 presents the statistics of the failure analysis for the Lausanne Malley building. These failure observations show similarity with the results of the Holsteiner Hof building being distributed as 55 % OOP and 45 % IP. Also, similar to the first case study, flexure failure is predominant within IP collapse with 68 % of occurrences, followed by 29 % of mixed failures, and only 3 % (i.e., 2 collapse observations) by pure shear IP. Moreover, IP failures are predominantly observed at upper stories, actually, more than 70 % are concentrated at the last storey, either F5 x-

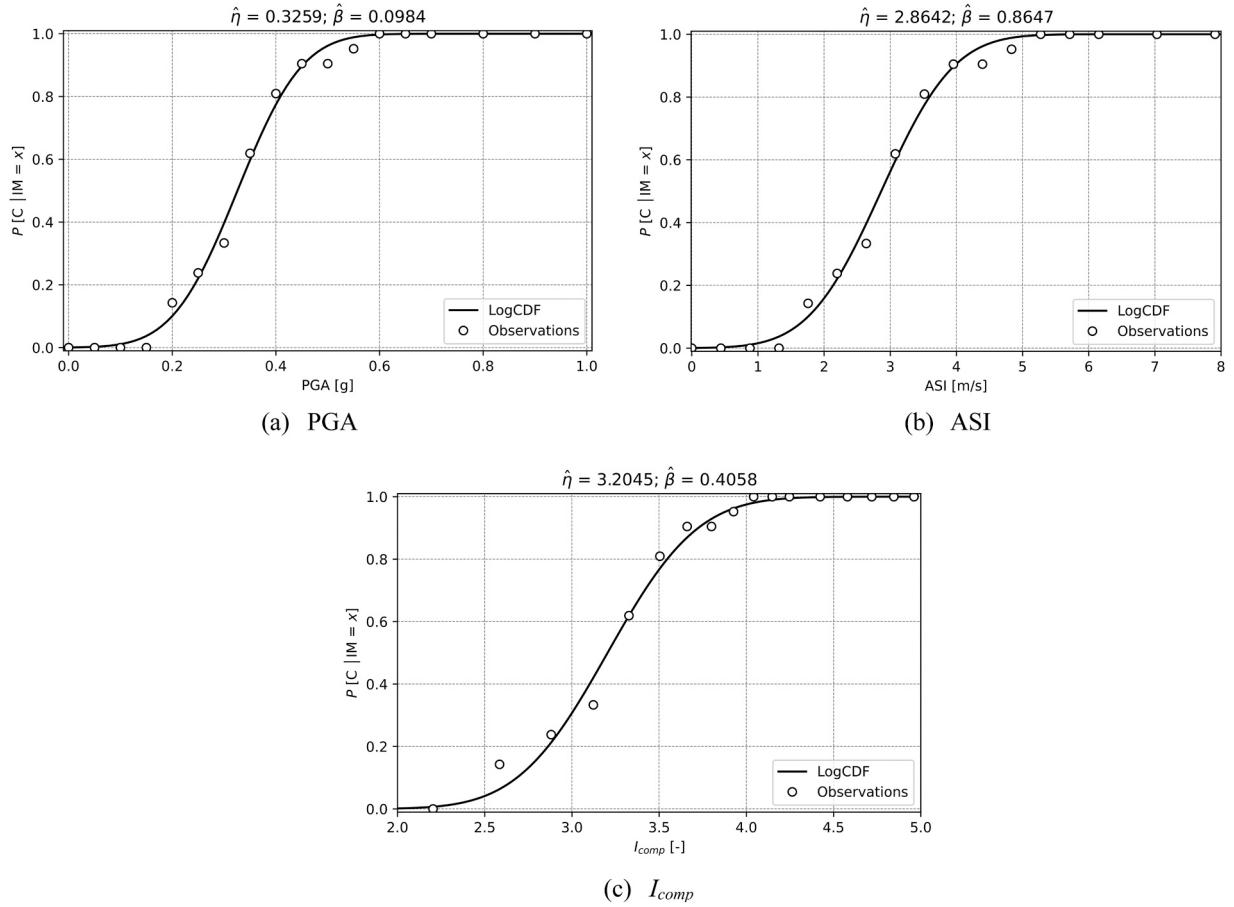


Fig. 7. Fitting of analytical collapse fragility functions considering different IMs — Holsteiner Hof.

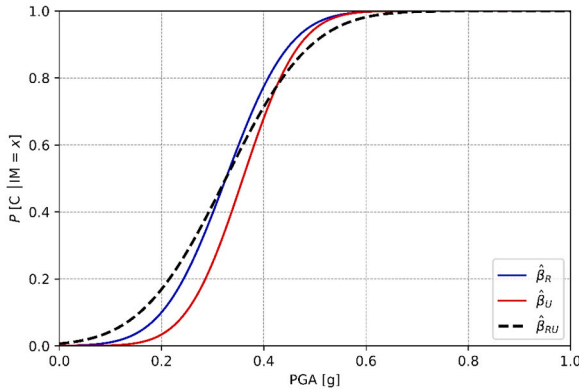


Fig. 8. Impact of modelling uncertainties and record-to-record variability on collapse fragility curves — Holsteiner Hof.

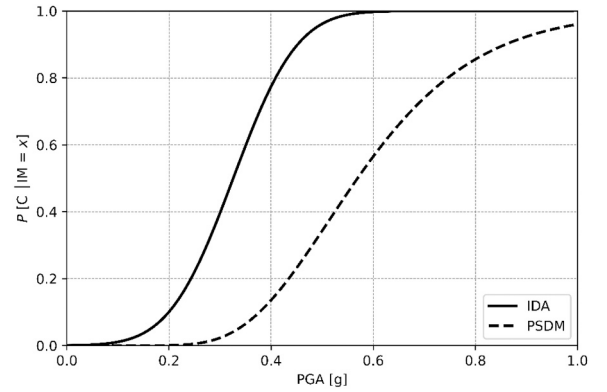


Fig. 9. Comparison of fragility curves from IDA and PSDA methodologies — Holsteiner Hof.

dir or F5 y-dir, but mostly in the “y” direction with almost 50 % of the counts. On the other hand, OOP failures induced by the overturning of the main façade and central node overturning are, as expected [95], observed at upper stories, with almost 50 % at F5 x-dir. Approximately, 30 % of OOP collapses are evenly distributed among F4 x-dir, F4 y-dir, and F5 y-dir; while less than 20 % are sparsely distributed among F1, F2, and F3 in both directions “x” and “y”.

4.2.1. IDA curves and fragility functions

For this case study, the IO limit is established in the fractile curves at $\max(\Delta_r)$ lower than 10 mm which is around 0.05 % of the equivalent global drift ratio. According to all three curves (i.e., 16th, 50th, and 84th

percentiles) this limit is expected to be reached at a 0.05 g fraction of PGA. On the other hand, CP points are identified for $\max(\Delta_r)$ values in the range of 20 mm to 40 mm, or global drift ratios ranging from 0.15 to 0.25 %, respectively. In terms of PGA, CP is reached between 0.10 to 0.20 g as appreciated in Fig. 11a. GC is observed when the structure approaches values of $\max(\Delta_r)$ in the order of 90 mm or 0.55 % global drift. In this case, stable quartiles are estimated using 14 individual capacity curves. The observations of global drift limits to be used subsequently for deriving the analytical collapse fragility are once again consistent with the findings reported in the literature for building structures with similar features [88–90]. Moreover, Fig. 11b and c show the effect of dispersion, β_R , on the capacity curves in terms of alternative

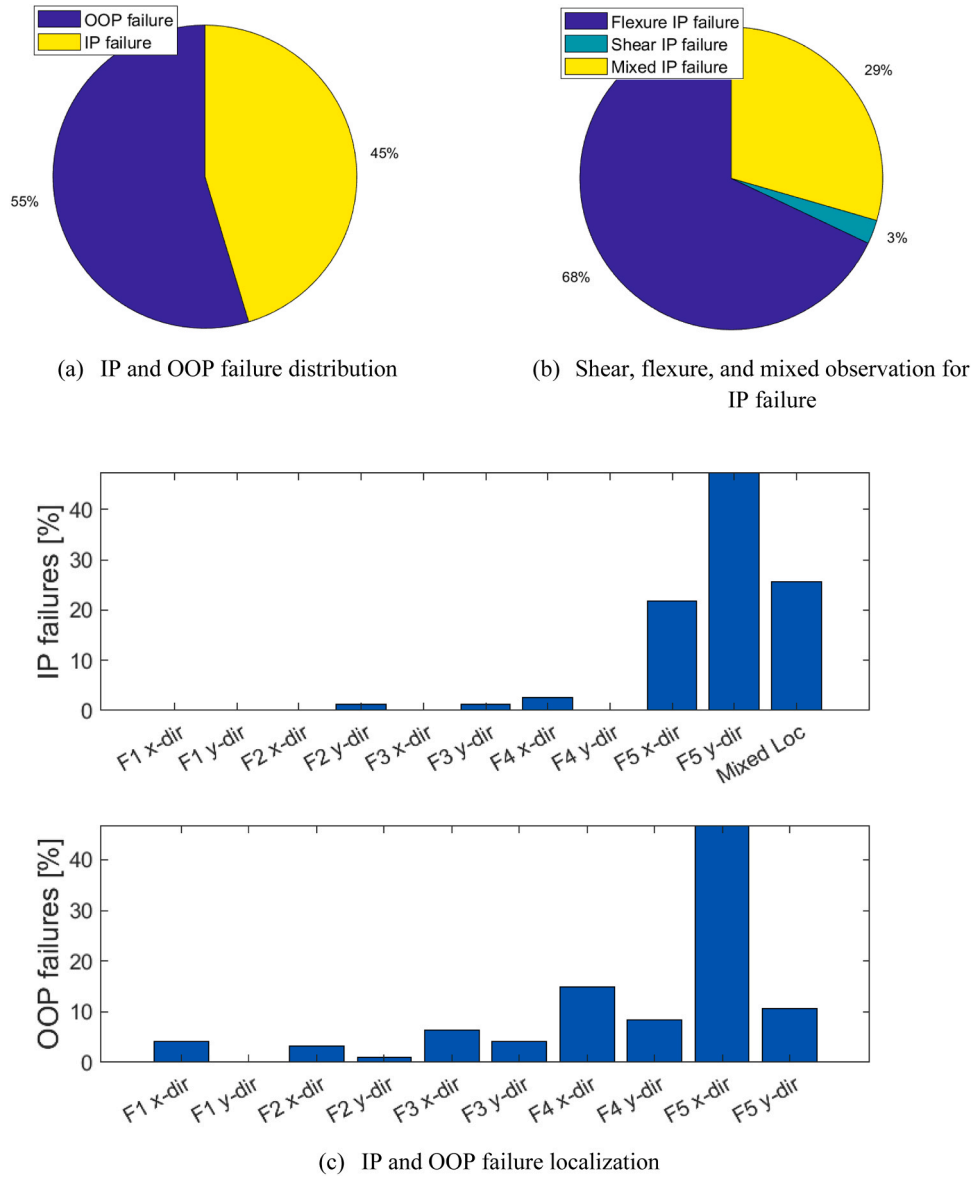


Fig. 10. Failure statistics — Lausanne Malley.

IMs. The modified spectral acceleration intensity (ASI^*) is ranked as the best individual metric (i.e., lowest β_R) right after I_{comp} which as in the previous case exhibits the lowest rate of dispersion induced by the record-to-record variability (See Table A1). Through the values of β_R , the best IMs are identified and combined into the I_{comp} as suggested by Caicedo et al. [68]. The best ranked IMs, in descending order, include ASI^* , PGA, $\bar{S}_a$, $\bar{S}_a$, Shaking intensity rate (SIR), EPA, ASI, velocity spectrum intensity (VSI), Housner intensity (HI) and S_a^* . Five of these metrics (ASI^* , VSI, S_a^* , $\bar{S}_a$, and $\bar{S}_a$) coincide with the optimal IMs identified through PSDMs for the same building [68]. Subsequently, fragility functions are derived by fitting a LogCDF to the collapse observations. The fitting is depicted in Fig. 12 besides the $\hat{\eta}$ and $\hat{\beta}$ values calculated for the LogCDF in terms of PGA, ASI^* , and I_{comp} . Fragility functions are discussed in the next sub-sections when comparing the results of uncertainties arising from the record-to-record variability vs. modelling parameters, and MR-IDA vs. PSDA.

4.2.2. Material and modelling uncertainties vs. record-to-record variability

Fig. 13 compares fragility curves accounting for the effects of record-to-record variability and modelling uncertainties independently, as well

as the mixed effect by employing the Cornell et al. [91] approach. As in the previous case, the median is fixed as $\hat{\eta} = 0.1833$ (See Fig. 12a) for the mixed effect. The moments of the distribution capturing the uncertainty in material and modelling parameters are $\hat{\eta}_U = 0.1743$ and $\hat{\beta}_U = 0.0440$ leading to slightly higher probabilities of collapse after the 0.15 g of PGA threshold when compared with the distribution accounting for record-to-record variability. Additionally, the mixed effect of both sources of uncertainty, $\hat{\beta}_{RU}$, indicates a higher likelihood of collapse until 0.20 g of PGA approximately, and leads to less conservative estimations after that level of intensity.

4.2.3. MR-IDA vs. PSDA

The last comparison is made considering the two different approaches based on MR-IDA and PSDA, respectively, for the computation of fragility functions. On this occasion, the peak ground velocity (PGV) is selected for comparison since it was identified as the optimal one for the Lausanne Malley building by means of PSDMs in [68]. Besides, this metric, as PGA, is relatively familiar to most practitioners and researchers in the fields of earthquake and structural engineering. It is depicted in Fig. 14 how both functions seem to match reasonably well up

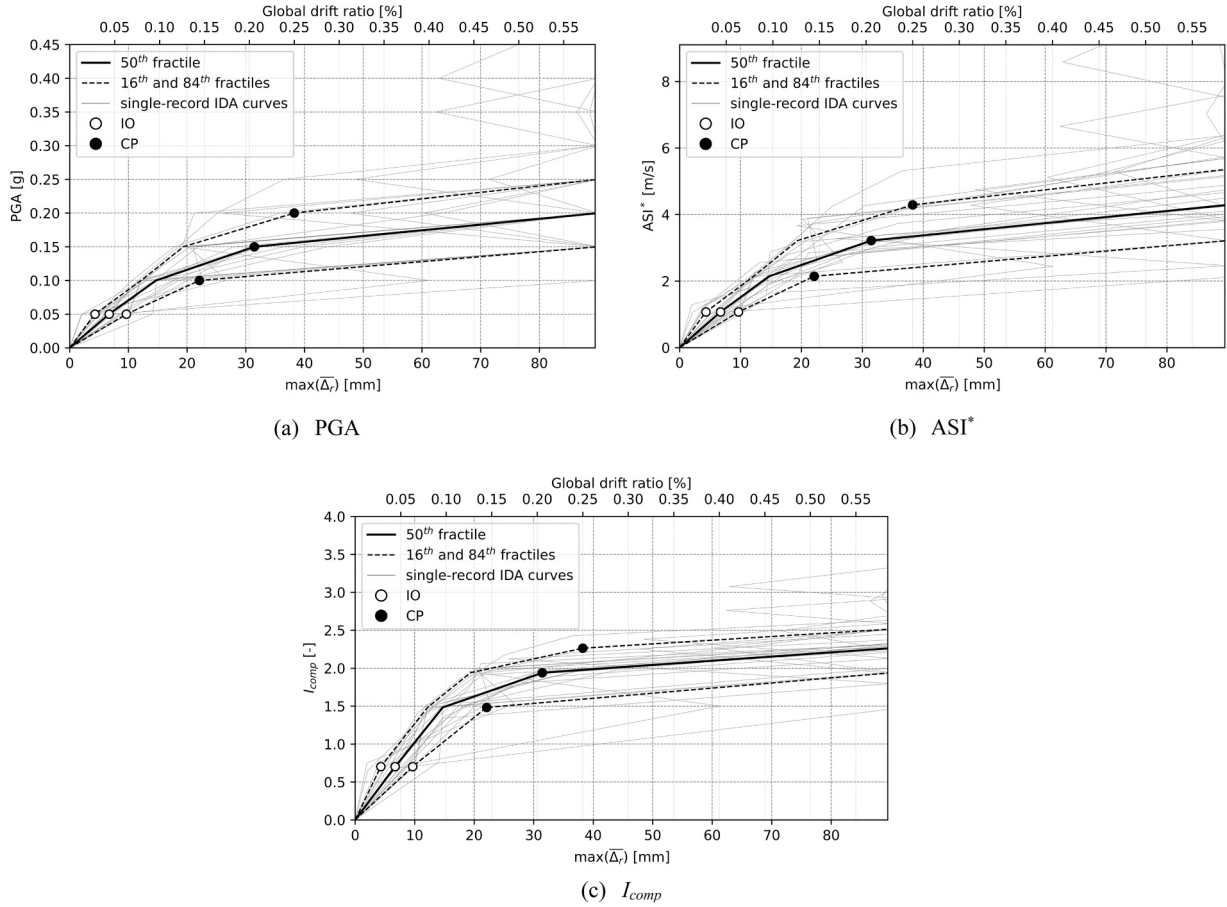


Fig. 11. IDA curves displaying maximum values of EDPs — Lausanne Malley.

to an intensity of 0.15 m/s in terms of PGV. Yet, the two curves diverge for higher values of PGV, leading the cloud-based approach to less conservative results (i.e., lower probabilities of failure). Here, it should be noted that the MR-IDA approach matches the actual distribution of collapse observations, leading, in consequence, to more reliable results. Thus, the distribution of collapse probability by Caicedo et al. [68] using PSDMs delivers an acceptable level of precision for lower values of intensity but fails at capturing collapse observations at higher intensity levels. Moreover, it is worth mentioning that the fragility function obtained by PSDMs is based solely on 28 collapse observations [68] in contrast to 172 collapse data points in the MR-IDA methodology studied herein. This is also the reason for observing better correspondence between the two approaches for the case of the Lausanne Malley rather than for the Holsteiner Hof building in Fig. 9 whose fragility through PSDA was built just on 9 collapse cases. Overall, the IDA-based approach stands out as a more reliable methodology for the derivation of realistic fragility functions matching the actual distribution of collapse observations.

5. Conclusions

A methodology based on multiple-record incremental dynamic analysis (IDA) to evaluate the probabilistic seismic response of historical masonry buildings in the framework of performance-based earthquake engineering (PBEE) was investigated. The OpenSees software was implemented for the numerical modelling of two case studies using three-dimensional macroelements to account for in-plane (IP) and out-of-plane (OOP) effects of masonry walls. Additionally, non-linear connections for floor-to-wall frictional interfaces and wall-to-wall interlocking were considered using innovative simplified constitutive laws

within the numerical models. The uncertainty induced by the random nature of earthquake motions (i.e., record-to-record variability) was addressed through selection consistent with the European seismic hazard. Non-linear dynamic analyses were performed increasing gradually the level of intensity every new analysis. The large number of analyses provided lots of collapse data which was carefully examined to characterise the phenomena. Capacity curves from IDA were presented alongside smooth percentiles (16 %, 50 %, and 84 %, respectively) and performance points to study the structural behaviour at different damage states. Results of collapse fragility accounting specifically for record-to-record variability were contrasted against previous findings of seismic fragility considering the uncertainty linked to material and modelling parameters. Further comparison was presented regarding IDA and cloud-based probabilistic seismic demand analysis (PSDA) methodologies to approach the collapse fragility assessment of historical masonry buildings.

Particular to the Holsteiner Hof building, representative of stiff monumental heritage structures, performance points were identified as immediate occupancy (IO) at the end of the elastic branch with $\max(\bar{\Delta}_r)$ lower than 5 mm, or global drift ratios lower or equal to 0.05 %; collapse prevention (CP) at values of drift ratio in between 0.05 to 0.15 %; and global collapse (GC) at $\max(\bar{\Delta}_r) = 40$ mm, or equivalent global drift ratios approaching 0.45 %. Utilising the dispersion induced by record-to-record variability (β_R), optimal intensity measures (IMs) for the Holsteiner Hof building were identified, including: the peak ground acceleration (PGA); acceleration spectrum intensity (ASI); effective Peak Acceleration (EPA); improved effective peak acceleration (IEPA); Cordova intensity (S_a^*); and Vamvatsikos intensity ($\bar{S}_a$ and $\bar{\bar{S}}_a$), which coincide with the findings presented in [68]. Furthermore, the effect of uncertainty stemming from record-to-record variability was found to

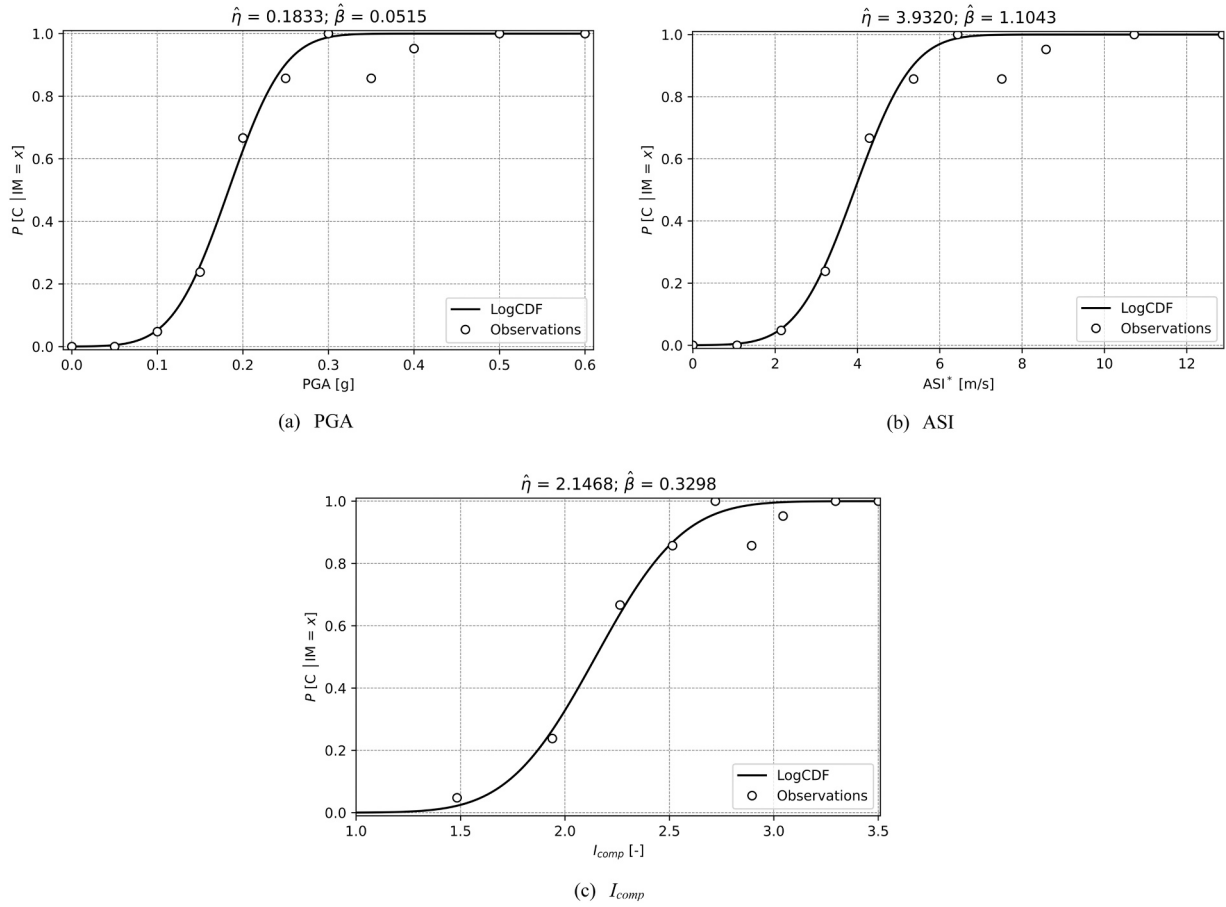


Fig. 12. Fitting of analytical collapse fragility functions considering different IMs — Lausanne Malley.

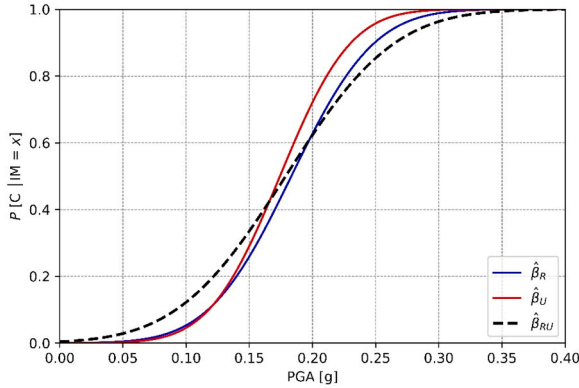


Fig. 13. Impact of modelling uncertainties and record-to-record variability on collapse fragility curves — Lausanne Malley.

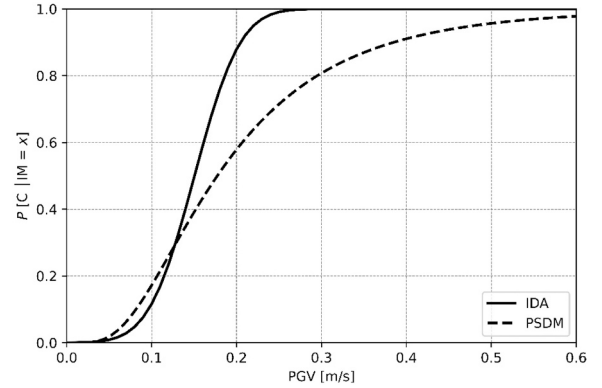


Fig. 14. Comparison of fragility curves from IDA and PSDA methodologies — Lausanne Malley.

lead to more brittle response than when accounting merely for modelling uncertainties. For the Lausanne Maley, representative of tall and slender residential masonry buildings, the IO limit was identified at levels of $\max(\Delta_r)$ lower than 5 mm, or global drift ratios near 0.05 %; CP points were observed for global drift ratios oscillating between 0.15 % and 0.25 %; and GC was reached at values of $\max(\Delta_r)$ in the order of 90 mm or 0.55 % global drift. In addition, the IMs leading to the lower β_R induced by the record-to-record variability included: the modified acceleration spectrum ASI^* , PGA, $\overline{S_a}$, $\overline{S_a}$, Shaking intensity rate (SIR), EPA, ASI, velocity spectrum intensity (VSI), Housner intensity (HI) and S_a^* , agreeing five of them (ASI^* , VSI, S_a^* , $\overline{S_a}$, and $\overline{S_a}$) with the findings of

optimal IMs identified through PSDMs for the same structure [68]. Different than for the first case study, the Lausanne Malley structure was found to be more sensitive to the uncertainty in material and modelling parameters rather than record-to-record variability.

In general, it was demonstrated how the record-to-record variability and the consideration of modelling uncertainties lead independently to practically identical values in the moments of the lognormal distribution (i.e., median and standard deviation) fitting the collapse observations. Further, the effectiveness of the composed measure, I_{comp} , proposed originally by Caicedo et al. [68] for the dynamic assessment of masonry buildings, was shown to lead to the lowest dispersion within the performed IDAs. Finally, for both case studies, it was demonstrated how the

MR-IDA outperformed the PSDA approach in generating reliable fragility functions fitted to the numerical collapse observations reproduced herein. Through the MR-IDA methodology, that was comprehensively examined in this research, it was possible to match the actual distribution of collapse observations, resulting in more truthful fragility functions than the ones obtained from the cloud-based approach using unscaled records. Future work should focus on analysing different structural typologies and understanding the effect of soil conditions on fragility analyses of historical masonry constructions.

CRedit authorship contribution statement

Daniel Caicedo: Writing – original draft, Visualization, Validation, Software, Methodology, Investigation, Formal analysis. **Igor Tomić:** Writing – review & editing, Visualization, Supervision, Software, Methodology, Investigation, Formal analysis, Conceptualization. **Shaghayegh Karimzadeh:** Writing – review & editing, Supervision, Resources, Methodology, Conceptualization. **Vasco Bernardo:** Writing – review & editing, Visualization, Supervision, Conceptualization. **Katrin Beyer:** Writing – review & editing, Supervision, Resources, Project administration. **Paulo B. Lourenço:** Writing – review & editing, Supervision, Resources, Project administration.

Declaration of Competing Interest

The authors declare that they have no known competing financial

interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

The numerical models of buildings developed in OpenSees and the selected ground motions to perform IDAs are shared openly through the repository <https://doi.org/10.5281/zenodo.12531709>.

Acknowledgments

This study has been partly funded by the STAND4HERITAGE project that has received funding from the European Research Council (ERC) under the European Union's Horizon 2020 research and innovation program (Grant agreement No. 833123), as an Advanced Grant. This work was also partly financed by FCT/MCTES through national funds (PIDDAC) under the R&D Unit Institute for Sustainability and Innovation in Structural Engineering (ISISE), under reference UIDB / 04029/2020 (doi.org/10.54499/UIDB/04029/2020), and under the Associate Laboratory Advanced Production and Intelligent Systems ARISE under reference LA/P/0112/2020. This work is partly financed by national funds through FCT - Foundation for Science and Technology, under grant agreement 2023.01101. BD attributed to the first author. The second author was supported by the Swiss National Science Foundation through grant 200021_175903/1 Equivalent frame models for the in-plane and out-of-plane response of unreinforced masonry buildings.

Appendix

Table A1

β_R for the whole set of IMs.

N°	Parameter	Definition	Mathematical formation	β_R	
Ground motion dependent scalars IMs				Holsteiner Hof	Lausanne Malley
1	PGA	Peak ground acceleration	$\max(\ddot{u}_g)$	0.3552	0.3976
2	PGV	Peak ground velocity	$\max(\dot{u}_g)$	0.7603	0.5598
3	PGD	Peak ground displacement	$\max(u_g)$	1.1127	0.7644
4	a_{RMS}	Root-mean-square of acceleration[96]	$\sqrt{\frac{1}{t_d} \int_0^{t_d} \ddot{u}_g^2 dt}$	0.5016	0.4672
5	v_{RMS}	Root-mean-square of velocity	$\sqrt{\frac{1}{t_d} \int_0^{t_d} \dot{u}_g^2 dt}$	0.8137	0.5845
6	u_{RMS}	Root-mean-square of displacement	$\sqrt{\frac{1}{t_d} \int_0^{t_d} u_g^2 dt}$	1.1215	0.7678
7	I_A	Arias intensity[97]	$\frac{\pi}{2g} \int_0^{t_d} \ddot{u}_g^2 dt$	0.4525	0.4471
8	CAV	Cumulative absolute velocity[98]	$\int_0^{t_d} \dot{u}_g dt$	0.7627	0.6094
9	CAD	Cumulative Absolute Displacement	$\int_0^{t_d} u_g dt$	1.0284	0.7276
10	SED	Specific energy density	$\int_0^{t_d} \dot{u}_g^2 dt$	0.9128	0.6524
Ground motion dependent compound IMs				Holsteiner Hof	Lausanne Malley
11	PGV/PGA	Velocity to acceleration ratio	—	1.2217	0.8634
12	PGV ² /PGA		—	1.0335	0.7120
13	I_a	Riddell–Garcia intensity[99]	$(PGA)(D_{5-95})^{1/3}$	0.4256	0.4445
14	I_v		$(PGV)^{2/3}(D_{5-95})^{1/3}$	0.8999	0.6507
15	I_d		$(PGD)(D_{5-95})^{1/3}$	1.1321	0.7791
16	I_F	Fajfar intensity[100]	$(PGV)(D_{5-95})^{0.25}$	0.8295	0.6059
17	I_Z	Cosenza and Manfredi intensity[101]	$\int_0^{t_d} \dot{u}_g^2 dt / (PGA)(PGV)$	1.2207	0.8594
18	I_C	Characteristic intensity[102]	$(a_{RMS})^{1.5}(D_{5-95})^{0.5}$	0.5276	0.4799
19	SIR	Shaking intensity rate[103]	I_{A5-75}/D_{5-75}	0.3691	0.4096
Structure-independent spectral IMs				Holsteiner Hof	Lausanne Malley
20	EPA	Effective peak acceleration[104]	$\frac{1}{2.5} \int_{0.1}^{0.5} S_a(\zeta = 0.05, T) dT$	0.3015	0.4097
21	EPV	Effective peak velocity[104]	$\frac{1}{2.5} \int_{0.8}^{1.2} S_v(\zeta = 0.05, T) dT$	0.5108	0.4517

(continued on next page)

Table A1 (continued)

N°	Parameter	Definition	Mathematical formation	β_R	
Ground motion dependent scalars IMs				Holsteiner Hof	Lausanne Malley
22	IEPA	Improved effective peak acceleration[105]	$\frac{1}{2.5} \int_{T_p^{-0.2}}^{T_p^{+0.2}} S_a(\zeta = 0.05, T) dT$	0.3909	0.4441
23	IEPV	Improved effective peak velocity[105]	$\frac{1}{2.5} \int_{T_p^{-0.2}}^{T_p^{+0.2}} S_v(\zeta = 0.05, T) dT$	0.5517	0.4618
24	HI	Housner intensity[106]	$\int_{0.1}^{2.5} PS_v(\zeta = 0.05, T) dT$	0.5258	0.4312
25	ASI	Acceleration spectrum intensity[107]	$\int_{0.1}^{0.5} S_a(\zeta = 0.05, T) dT$	0.3015	0.4097
26	ASI*	Modified acceleration spectrum intensity[108]	$\int_{0.1}^{2.0} S_a(\zeta = 0.05, T) dT$	0.3869	0.3863
27	VSI	Velocity spectrum intensity	$\int_{0.1}^{2.5} S_v(\zeta = 0.05, T) dT$	0.4667	0.4142
28	DSI	Displacement spectrum intensity	$\int_{2.0}^{5.0} S_d(\zeta = 0.05, T) dT$	0.9954	0.7010
Structure-dependent spectral IMs				Holsteiner Hof	Lausanne Malley
29	$S_a(T_i)$	S_a at the i^{th} vibration mode	$i = 1$	0.5063	0.4617
30			$i = 2$	0.5063	0.4554
31			$i = 3$	0.5278	0.4332
32			$i = 4$	0.5278	0.4632
33			$i = 5$	0.5278	0.4632
34			$i = 6$	0.5278	0.4632
35			$i = 7$	0.5278	0.4632
36			$i = 8$	0.5278	0.4632
37			$i = 9$	0.5278	0.4632
38			$i = 10$	0.5278	0.4681
39	$S_v(T_i)$	S_v at the i^{th} vibration mode	$i = 1$	0.5481	0.4722
40			$i = 2$	0.5481	0.4490
41			$i = 3$	0.6000	0.4368
42			$i = 4$	0.6000	0.4714
43			$i = 5$	0.6000	0.4714
44			$i = 6$	0.6000	0.4714
45			$i = 7$	0.6000	0.4714
46			$i = 8$	0.6000	0.4714
47			$i = 9$	0.6000	0.4714
48			$i = 10$	0.6000	0.4639
49	$S_d(T_i)$	S_d at the i^{th} vibration mode	$i = 1$	0.5053	0.4615
50			$i = 2$	0.5053	0.4555
51			$i = 3$	0.5277	0.4331
52			$i = 4$	0.5277	0.4631
53			$i = 5$	0.5277	0.4631
54			$i = 6$	0.5277	0.4631
55			$i = 7$	0.5277	0.4631
56			$i = 8$	0.5277	0.4631
57			$i = 9$	0.5277	0.4631
58			$i = 10$	0.5277	0.4679
59	$S_a(T = 0.1 \text{ s})$	—	—	0.5278	0.5102
60	$S_a(T = 0.2 \text{ s})$	—	—	0.4069	0.4473
61	$S_a(T = 0.5 \text{ s})$	—	—	0.4435	0.4617
62	$S_a(T = 1.0 \text{ s})$	—	—	0.5476	0.4766
63	S_a^*	Cordova intensity[109]	$(S_a(\zeta = 0.05, T_1))^{\alpha} (S_a(\zeta = 0.05, cT_1))^{\beta}$	0.3634	0.4315
64	$\overline{S_a}$	Vamvatsikos intensity[110]	$(S_a(\zeta = 0.05, T_a))^{\alpha} (S_a(\zeta = 0.05, cT_b))^{\beta}$	0.3641	0.4026
65	$\overline{\overline{S_a}}$		$(S_a(\zeta = 0.05, T_a))^{\alpha} (S_a(\zeta = 0.05, T_b))^{\beta} (S_a(\zeta = 0.05, T_c))^{\gamma}$	0.3316	0.4091
66	S_a^{1-to-N}	Multiple period intensities[111,112]	$\prod_{i=1}^N S_a(\zeta = 0.05, T_i)^{\alpha_i}$	0.5063	0.4538
67				0.5059	0.4538
68				0.5008	0.4516
69				0.5006	0.4514
70				0.4938	0.4514
71				0.4937	0.4514
72				0.4937	0.4514
73				0.4937	0.4503
74				0.4937	0.4451
75	I_{comp}	Composed measure[68]	$I_{comp} = \prod_{i=1}^m IM_i^{\theta_i}$	0.3006	0.3671

Note: for a better understanding of nomenclature and equations, please refer to the references provided in the definition column.

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