

## DERIVATION OF FRAGILITY CURVES FOR TRADITIONAL TIMBER-FRAMED MASONRY BUILDINGS USING NONLINEAR STATIC ANALYSIS

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**Abstract.** *Recent earthquakes and two experimental campaigns on timber-framed masonry walls have shown that timber-framed masonry buildings possess a good displacement capacity and hence can withstand severe earthquakes without collapse. In the present paper, timber-framed masonry panels with diagonal braces are studied. Using a simplified model based on non-linear (NL) lumped plasticity strut elements, NL analyses are carried out of typical traditional buildings in Lefkas (Greece) with diagonally braced timber-framed masonry walls in their lateral load resisting system. Furthermore, an investigation is carried out regarding the foundation of the buildings. The key feature of the Lefkas buildings is their dual structural system. The primary system consists of a stone masonry ground floor and all upper floors are made of timber-framed masonry walls. Timber posts in the ground floor, a few centimetres apart from the stone masonry, constitute the secondary structural system which is connected to the upper floors. This latter system is activated once the ground floor stone masonry piers fail. Two different structural models are developed to simulate each system. Pushover curves are derived from the NL analyses of the buildings and are then converted into capacity curves assuming the fundamental mode dominates. On these curves four damage states (slight damage, moderate damage, heavy damage and collapse) are defined on the basis of criteria related to the actual response of the building. The first three damage states are defined on the capacity curve of the primary system, whilst the ultimate damage state is related to the response of the secondary system. Then, fragility curves in terms of spectral displacement are generated, adopting a log-normal statistical distribution of the probability of damage.*

## 1 INTRODUCTION

Timber-framed Masonry (TFM) is a traditional structural system that in seismic areas has proved to effectively resist earthquakes. It is well known that un-reinforced masonry (URM) is a system that in general cannot resist strong earthquakes; its high vulnerability, documented during several recent earthquakes (e.g. [1]), originates from its low tensile and shear strength, and the low ductility of its components. On the other hand, the combination of wood braces and masonry can provide adequate lateral load resisting capacity. Buildings made in some form of TFM are found in Ancient Greece from the Bronze Age (16<sup>th</sup> century B.C.), and since then TFM has a continuous history, as well as widespread occurrence.

A relatively recent development in TFM is the inclusion of diagonal timber members in the frame. A very advanced, from the structural engineering point of view, TFM construction is found in Lefkas Island in the Ionian Sea (western part of Greece). This structural system was produced by local builders in the region at least since the 17<sup>th</sup> century AD; it is reported that after the destructive earthquake that hit Lefkas in 1825 the local authorities decided to rebuild collapsed buildings of the island using this system [2]. Erecting such a structural system which comprises wood and masonry in harmonious proportions is a challenging task that could be properly carried out only by highly-skilled builders.

The development of this structural system, which is to some extent follows the principles of displacement-based design (a very recent trend in earthquake engineering), has strongly benefitted from the experience gained by frequent, yet moderate intensity, earthquakes in the area, that permitted observations and inspired improvements with the course of time. In its historical context, the system is a pioneer construction, which is the key reason why it has been listed as a monument of world heritage by the United Nations [3]. However, construction of these traditional buildings came to an abrupt end during the last century; hence nowadays most of them date back at least a century ago. TFM buildings have been given a considerable amount of attention by architects and archaeologists. On the other hand very little has been done so far to assess their seismic capacity and vulnerability/fragility.

Empirical fragility curves have been published for TFM buildings using data from the 2003 Lefkas earthquake. After this 2003 earthquake that had an epicentre very close to the NE end of the island, a detailed survey of damage and collapse has been carried out by Karababa [4,5]. Processing this data Karababa & Pomonis generated fragility curves for the TFM buildings of Lefkas [6]. To the authors' best knowledge, no analytically derived fragility curves exist for this type of building; such curves are developed in the remainder of this paper.

## 2 ANALYSIS OF THE STRUCTURAL SYSTEM

### 2.1 'Dual' structural system of buildings with more than one storey

The ground floor of TFM buildings in Lefkas when they have more than one storey is constructed in URM, typically stone masonry. Walls in the upper floors are made in TFM. Stones of the URM ground floor, usually calcareous limestone, are extracted from local quarries or those of the neighbouring Epirus coast [3]. The ground floor height is on average 3 m and the thickness of the walls may vary from 0.6 to 1.0 m. The external façade is made of ashlar-work, while the internal one is made of roughly dressed and worked stones or even rubble walls in irregular courses.

TFM of the upper floors consists of timber elements forming a 3D timber-frame, and masonry infills. Timber sections are on average  $10 \times 10 (\pm 2)$  cm<sup>2</sup>. The timber frame comprises spans that vary from around 1.0 up to 2.2 m. Diagonal members join opposite corners but sometimes one of them may miss or be halved. An important feature of the TFM is the use of

curved timber elements to stiffen the connection of timber posts to beams. These angles are L-shaped for one-sided connections (Figure 1) or T-shaped for two-sided connections. It is apparent that local builders had comprehended the substantial lateral load capacity of timber elements and tried to exploit it as much as possible.

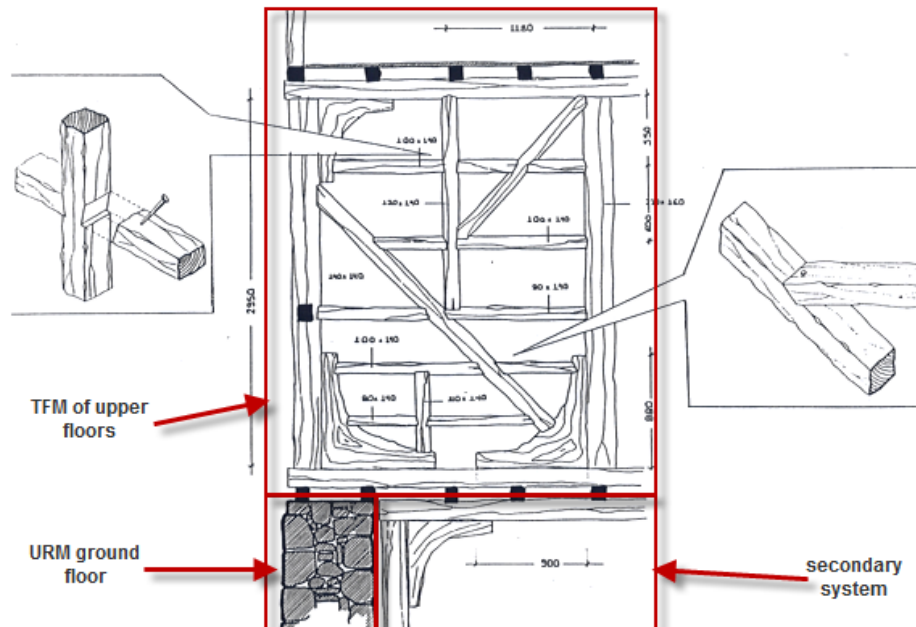


Figure 1. Connection of TFM super-structure with the URM ground floor and the secondary system (after [2]).

Nevertheless, the key structural feature of Lefkas buildings is their dual system. In fact, apart from the URM ground floor there is a secondary structural system on the internal side of the walls comprised of timber posts. Normally to every pier corresponds a single timber post a few centimetres apart the walls (Figure 1). This small distance of the timber posts from the walls results from the plinth foundation of the posts which is usually on a compact massive stone appropriately curved and usually equipped with thin metal plates serving that clamp the base of the column (Figure 2). These timber posts are connected to the beams, as well as the TFM walls, of the upper floor. Hence, the beams of the floors and the TFM walls are supported by both the primary and the secondary system. This sophisticated connection is realised by embedding floor beams into the URM ground floor walls and at the same time joining them to the timber columns of the secondary system.

Timber columns are not acted by gravity loads under normal service conditions in the building and support to the upper floors gravity loads is provided by the primary system (i.e. the URM ground walls). During an earthquake, if its intensity is so high as to drive the URM ground floor walls to partial or full collapse the secondary system will be activated. Due to the much higher deformability of the timber columns compared to the URM walls, this secondary system has displacement reserves able to accommodate the increased displacement demand of the earthquakes, at the expense of significant damage. Collapse of the URM walls of the ground floor may be classified as 'severe damage' state. Consequently, the building will be able to avoid collapse but damage will be high or possibly irreparable due to permanent displacements after seismic shaking; however, adequate life safety is ensured.

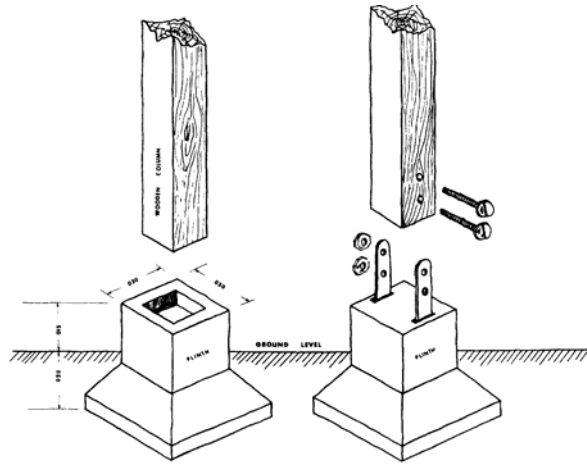


Figure 2. Stone foundations of timber posts of the secondary system of Lefkas' buildings (after [7]).

## 2.2 Typical building studied

A TFM building with the previously described dual system, situated in the central square of Lefkas city, called the 'Berykiou' building, is selected for further analysis as its geometrical and mechanical characteristics are typical of those found in the island (Figure 3). The ground floor is made of limestone with stonework texture in courses and walls 0.8 m thick. The TFM of the upper storeys is covered with thin galvanized iron sheeting for their protection against weathering. The configuration of the diagonals of the TFM is not exactly known due to the presence of the opaque metal sheathing. It is assumed here that every TFM panel consists of two diagonals as illustrated in Figure 3b, and a previously developed model for TFM with two diagonal braces [8] is utilised.



Figure 3. Berykiou building in the centre of Lefkas city: (a) façade geometry and (b) timber elements of TFM walls and the ground floor secondary system (shown in red).

The timber frame of the upper storeys is supported by both the ground floor URM and the timber posts of the secondary system shown in Figure 3. A typical section of timber elements of TFM is assumed (10×10 cm) and the thickness of TFM is also assumed as 10 cm. Timber posts of the secondary system have a section 15×15 cm<sup>2</sup>. Referring to Figure 2, two bounding cases are considered for column end restraint, i.e. fully clamped or free to rotate.

	Masonry	Wood
Compressive strength $f_{c,x}$ [MPa]	3.50	18.9
Compressive strength $f_{c,y}$ [MPa]	3.50	4.77
Tensile strength $f_{t,x}$ [MPa]	0	18.9
Tensile strength $f_{t,y}$ [MPa]	0	4.77
Modulus of elasticity $E_x$ [MPa]	1750	11000
Modulus of elasticity $E_y$ [MPa]	1750	370
Shear modulus $G$ [MPa]	673	690
Weight [kN/m <sup>3</sup> ]	20	3.5
Poisson ratio $\nu$	0.2	0.3

Table 1. Mechanical properties for the materials used in the model in the longitudinal (x) and transverse (y) direction.

Mechanical properties of wood correspond to timber class C24 and service class II according to EC5 (see Table 1). Experiments in traditional stone masonry specimens made of two outer wythes of ashlar stones and an inner core of rubble have shown that these walls present relatively low strength compared to that of stones [9]. In this regard a conservative value of compressive strength is adopted (Table 1). Masonry is considered isotropic here, as the strength differences in the two axes are not substantial [10]. Finally, a simplified beam model is adopted.

### 2.3 NL static analysis

A 2D plane model of the façade is set up in SAP2000 [11]. TF walls are modelled by axial force vs. axial deformation (N- $\epsilon$ ) lumped plasticity hinges in the diagonals [8]. Modification of the axial stiffness of the diagonals is required to take into account their sliding:

$$k_s = \frac{(H^2 + L^2)^{3/2} + H^3}{EA} \cdot \frac{1}{L^2} \cdot \frac{V_y}{\delta_y} \quad (1)$$

In Equation (1)  $E$  is the Young's modulus of timber,  $A$  is the area of the section,  $\delta_y$  and  $V_y$  are the corresponding displacement and shear force at yielding and  $L$ ,  $H$  the panel dimensions.

The URM piers of the ground floor were modelled using moment vs. rotation (M- $\theta$ ) lumped plasticity hinges according to [12,13].

The primary and secondary systems are modelled separately for two reasons:

- (i) the considerable differences in modal characteristics of these two systems result in different shape of the pushover curve, and
- (ii) introducing a softening branch in the curve is prone to numerical instabilities that can cause errors and/or early termination of the analysis.

The failure mechanism of the primary system after its NL static analysis is depicted in Figure 6 where plastic hinges are indicated by a different colour associated to their damage state. ‘Collapse’ of the building is due to the development of a ground floor mechanism. However, upper floor TFM has suffered considerable damage as shown by the widespread

inelastic deformations of the TFM panels. Indeed, development of hinges in the ground floor is preceded by the occurrence of the TFM wall hinges. However, the evolution of plastic deformation in the ground floor is rather quick and finally dominates the failure mechanism.

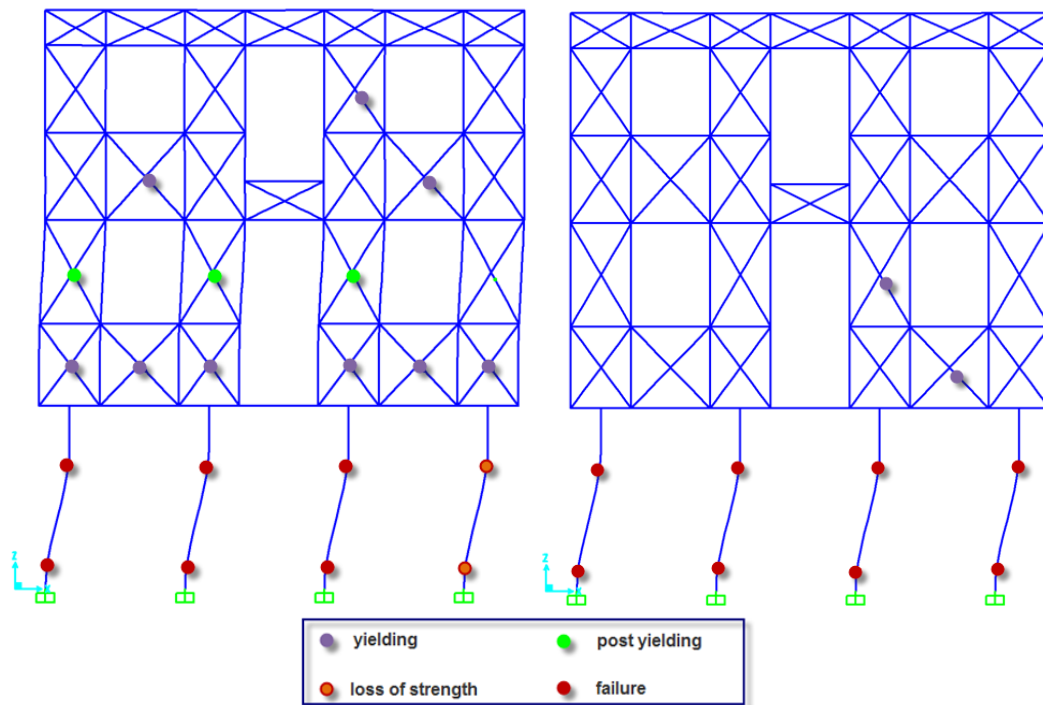


Figure 4. Failure mechanism of the primary (left) and the secondary (right) system of Berykiou TF building.

On the contrary, NL static analysis of the secondary system leads to collapse of the timber posts of the ground floor, with negligible inelastic deformation in the upper floors; the latter remain almost intact and the whole plastic deformation is concentrated in the ground floor.

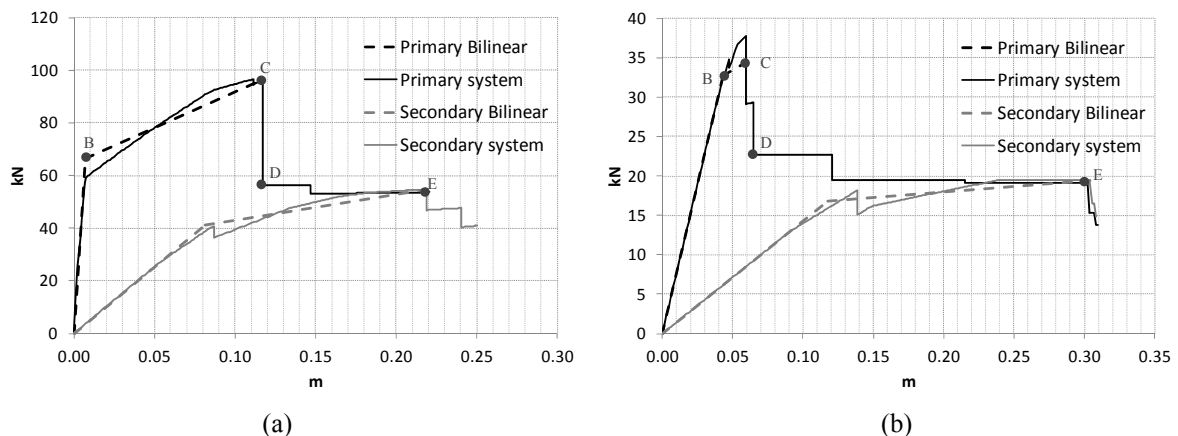


Figure 5. Pushover curves of the primary and secondary system of 'Berykiou' TFM building with: (a) full restraints and (b) flexible supports (SSI).

These two mechanisms in the real building coexist and interact; activation of the secondary system occurs after failure of the URM piers, but from then on it follows the deformation of the primary system. Consequently, combination of the pushover curves is in order; failure of

the ground floor piers constitutes the point wherein the two curves should be joined. Then, the resulting curve is idealised by a multi-linear one that allows definition of a yield point.

Previous analyses have been carried out without considering any interaction between the building and the soil as full restraints have been applied at the basis of the piers. However, the city of Lefkas is situated on soft alluvial deposits [14] which give rise to compliance of the URM foundations. These URM foundations are usually strengthened with a substructure made of layers of tree trunks to increase their stiffness [7,15]. Dynamic impedances for this kind of foundation and this type of soil have been estimated as  $k_{sdv}=2400 \text{ kN/m}^3$  for the vertical direction and  $k_{sdh}=1600 \text{ kN/m}^3$  for the horizontal one, to account for soil – structure interaction (SSI).

### 3 DERIVATION OF FRAGILITY CURVES

Fragility curves are derived based upon the key assumption [16] that the 'Berykiou' building represents a typical TFM building of this typology, displaying 'average' response. The seismic behaviour of this building type can then be expressed in terms of an 'average' pushover curve. The capacity spectrum method [17] based on inelastic demand spectra [18] is used here to relate pushover curves with seismic demand. So, pushover curves are transformed into capacity curves, i.e. the pushover curve of the corresponding SDOF (single degree of freedom) system. This transformation is implemented using properties of the predominant mode in the considered direction. The validity of this procedure depends on the contribution of the first mode, which in this case is over 90%, which ensures reliable results.

A capacity curve is generally characterized by two points; (i) the yield point B that represents the transition from elastic to inelastic deformations, and (ii) the maximum capacity point C which represents failure of the piers of the primary system. Nevertheless, in Lefkas' dual system buildings there are two more significant points: (iii) point D related to the yielding of the secondary system and (iv) point E representing collapse of the secondary and eventually of the entire building. This sequence of points specifies the main four representative stages of the seismic performance of TFM buildings with a dual system.

It is well documented in the current literature that fragility curves can be described by (cumulative) normal, lognormal, or beta distributions [19]. The lognormal distribution (eq. 2) is adopted here since it describes adequately the evolution of seismic damage [16].

$$P[ds \geq ds_i | A_g] = \Phi \left[ \frac{1}{\beta_{ds,i}} \ln \left( \frac{A_g}{\bar{A}_{g,dsi}} \right) \right] \quad (2)$$

In Equation (2)  $\Phi$  is the cumulative distribution function of the lognormal distribution,  $\beta_{ds,i}$  is the standard deviation of the natural logarithm of the damage state  $i$  and  $\bar{A}_{g,dsi}$  the mean value of the appropriate measure that represents seismic intensity (here spectral displacement) at which the building reaches the threshold of damage state  $i$ .  $\bar{A}_{g,dsi}$  is computed according to the previous discussion of the damage stages through the following expressions:

$$\bar{S}_{d1} = \delta_y = \delta_B \quad (3a)$$

$$\bar{S}_{d2} = 0.4 \times (1.5\delta_y + \delta_c) \quad (3b)$$

$$\bar{S}_{d3} = \delta_c \quad (3c)$$

$$\bar{S}_{d4} = \delta_u = \delta_E \quad (3d)$$

In the absence of other information, the following assumption is made in order to calculate the standard deviation  $\beta_{ds,i}$  [20,21]; first the binomial distribution (eq. 4) is adopted since it depends only on one parameter, i.e. the mean value of the spectral displacement:

$$P_i = \frac{n!}{i!(n-i)!} \cdot (\overline{S_{d,i}})^i \cdot (1 - \overline{S_{d,i}})^{n-i} \quad |_{i=1-n} \quad (4)$$

Consequently, for the mean spectral displacement of each damage state  $i$  defined from (3), the probability of exceedance of all other damage states may be computed using Equation (4).

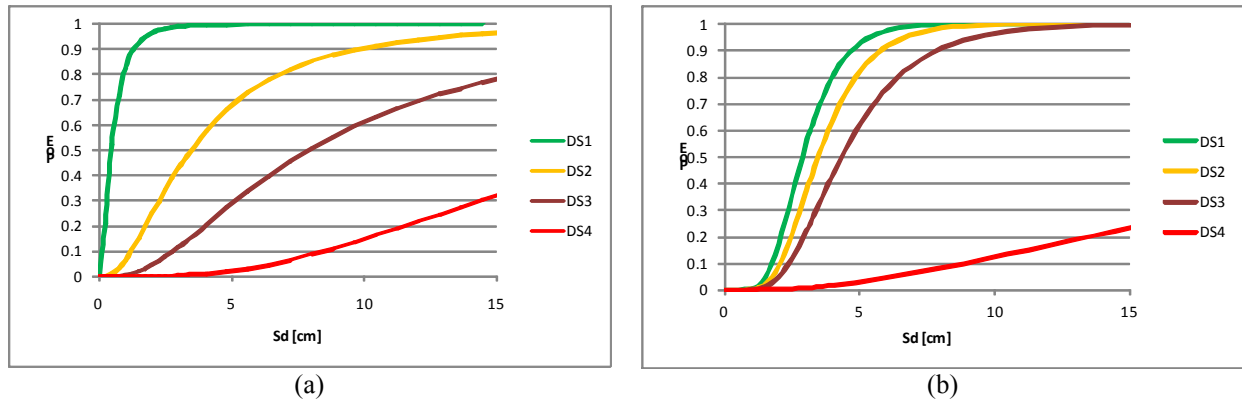


Figure 6. Fragility curves for 3storeys TFM buildings in Lefkas island on: (a) fully restrained and (b) flexible foundation (SSI).

Figure 6 shows the fragility curves for three-storey TFM buildings with a dual system. It is seen that DS1 (light damage) will be reached for rather low earthquake intensity. Taking into account the deformability of soil, DS1 shifts to higher displacements, which in general would mean higher intensities (Figure 6b); however, this depends on the type of response spectrum used for estimating demand. A similar trend is noted with respect to DS4, whose probability of exceedance decreases for the same values of spectral displacement. On the contrary, intermediate damage states (DS2 and DS3) shift towards lower intensities which leads to the conclusion that there is lower probability for a building on soft soil to reach DS1 and DS4 compared to a building on rock; the opposite holds for DS2 and DS3. Clearly, these issues have to be verified with additional analyses.

#### 4 CONCLUSIONS

The key findings from this study can be summarised as follows:

- TFM buildings in Lefkas having a dual structural system (described in section 2) can be thought of as an early realisation of displacement based 'design', as they are expected to resist strong earthquakes through their displacement capacity rather than their strength.
- The primary system aims to support gravity loads and, under low to moderate intensity earthquakes, to prevent premature cracking and noticeable deflection of the building.
- The secondary system renders displacement capacity to the building and provides the necessary deformability to survive a severe seismic event.
- The study of the above two systems using a different model for each, is a reasonable analysis procedure since the presence of the secondary system has a negligible effect on the response when the primary system is still intact.



- Pushover curves of TFM buildings are best idealised by four points on a multi-linear curve representing the transition of the building through four discrete response states.
- Damage states were related to the response phases of the building, defined in terms of displacements. So, three (DS1, DS3 and DS4) out of the four damage states are related directly to the response phases, establishing a robust definition.
- The derived fragility curves for TFM buildings show that although these may suffer cracks and damage for low intensity earthquakes, they are expected to avoid collapse due to their high displacement capacity. These findings agree with observed response of TFM buildings in past earthquakes in Greece.

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