Article

Numerical Analysis of an Earthen Masonry Structure Subjected to Blast Loading

Demiana Tse , João M. Pereira and Paulo B. Lourenço

ISISE, Department of Civil Engineering, Campus de Azurém, University of Minho, 4800-058 Guimarães, Portugal; jpereira@civil.uminho.pt (J.M.P.); pbl@civil.uminho.pt (P.B.L.)

* Correspondence: tse.demiana@gmail.com

Abstract: Historic monuments and construction capture the knowledge of civilizations of the past and are a source of pride for people of the present. Over the centuries, these buildings have been at risk from natural and man-made causes. The Alhambra, a UNESCO World Heritage Site in Granada, Spain, is one of such places. This paper aims to evaluate the structural performance of the *Torre de la Vela*, a tower in the Alhambra, under blast loads. The loads were based on historical records of barrels of gunpowder and were modeled as simplified pressure profiles using existing empirical equations. The effect of impulsive loading on the material properties was accounted for using dynamic increase factors, determined experimentally by previous authors. The model was created using finite element methods (FEM) and the problem was solved using explicit dynamic analysis available in Abaqus/Explicit. Using the failure volume damage index, a blast load applied outside and inside of the building would create a low damage level, which should be treated with caution given the occurrence of localized damage. The removal of elements exceeding a given damage threshold led to more visible damage patterns than the Concrete Tension Damage option in Abaqus.

Keywords: masonry; finite element modeling; blast analysis; dynamic increase factors; out-of-plane behavior

Citation: Tse, D.; Pereira, J.M.; Lourenço, P.B. Numerical Analysis of an Earthen Masonry Structure Subjected to Blast Loading. *CivilEng* **2021**, *2*, 969–985. <https://doi.org/10.3390/civileng2040052>

Academic Editors:
Anastasios Drougkas and
Savvas Saloustros

Received: 19 September 2021
Accepted: 3 November 2021
Published: 9 November 2021

Publisher's Note: MDPI stays neutral with regard to jurisdictional claims in published maps and institutional affiliations.

Copyright: © 2021 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC BY) license (<https://creativecommons.org/licenses/by/4.0/>).

1. Introduction

The vulnerability of masonry envelopes under blast loading is critical due to the potential loss of lives. Blasts near or inside a building can damage and destroy parts of the building and can produce both local and global responses related to different failure modes within structural members. Particularly, the out-of-plane response of masonry under dynamic loading is critical due to the risk this poses on the loss of life.

In this paper, damage induced by blast loading on an earthen masonry structure is studied using the finite element method (FEM). This study demonstrates the damage distribution and the level of damage when the structure is subjected to external and internal blast explosions. The accuracy of the FEM model is ensured by comparing the eigenvalue analysis results to a previous model created and calibrated to the building using dynamic identification [1]. Under dynamic loading conditions, materials can have different behavior when compared with quasi-static conditions [2–5]. Despite this, static material properties have previously been used in research work on the structural response and damage under impact and blast loading, which can lead to an inaccurate prediction of the damage [6,7]. Few numerical studies have been conducted regarding the effect of blasts on earthen masonry structures. Thus, it is important to better understand the behavior of masonry structures under these conditions. A simplified approach using analytical equations was used in this work to determine the pressure profiles of the blast [8–14]. The damage from the loading conditions was also characterized using existing failure criteria [9,15,16].

2. Blast Loading Definition and FEM Modeling

Blasts are defined as sudden releases of energy, leading to gas expansion and rapid increases in volume, classified as physical, chemical, or nuclear [17]. During an explosion, the blast wave is formed in front of the gas volume with the most energy released. This is defined as the side-on overpressure, where the pressure increases above the standard atmospheric pressure, P_0 . The peak side-on overpressure, P_{so} , relates to the free air pressure waves that have yet to encounter an obstacle [18]. The energy released in equivalent mass of TNT is the generally accepted reference standard for explosive yield and can be calculated in (1), where W_{TNT} is the equivalent charge in TNT, W_x is the mass of a given explosive, Q_x is the mass specific energy of the given explosive, and Q_{TNT} is the mass specific energy of TNT [19].

$$W_{TNT} = \frac{Q_x}{Q_{TNT}} W_x \quad (1)$$

During the Peninsular War of 1812, many towers in the Alhambra were destroyed by means of explosives by Napoleonic troops [20]. In 1590, a gunpowder factory located below the *Torre de la Vela* exploded [21]. These events led to developing the scope of the work within a historical context. For this reason, the type of explosive used in this research was black powder, the commonly used gunpowder in Europe until the early 1800s, where 1 kg of black powder is equivalent to approximately 0.60 kg of TNT [22]. Four “full-barrels” of gunpowder were used in the exterior blast and one “quarter-barrel” was used in the interior blast, corresponding to 110 kg of TNT and 6.8 kg TNT, according to standard sizes of gunpowder barrels used in the 19th century [23]. The first loading scenario was based on an assumption of the number of barrels that could be transported at the same time for storage along a possible route of transportation. This charge was placed at 4 m from the entrance of the tower, which is the approximate location of the stair landing leading to said entrance. The location of the second loading scenario was based on assumptions as to a possible storage location that would minimize turning to arrive at that location and would not impede the flow of traffic. The smallest standard barrel size was selected to allow an understanding of the building response without encountering significant numerical errors during simulation. Scaling laws allow for parametric correlations between a defined explosion and a standard charge of the same substance to predict the effects of large-scale explosions using a smaller scale [24]. The Hopkinson-Cranz scaling laws are the most widely accepted (2), where R is the standoff distance from the center of the detonation, in m, and W is the charge mass in kg of equivalent TNT.

$$Z = R/W^{\frac{1}{3}} \quad (2)$$

Blast loading on a structure can occur externally or internally. At a given distance, R , from the blast source, the overpressure increases almost instantaneously to the peak overpressure, P_{so} , at the arrival time, t_a (Figure 1). This is the beginning of the positive phase, which lasts for a duration of t_d . During the positive phase, when the time is greater than the arrival time ($t > t_a$), the overpressure decreases exponentially (Figure 1). When the overpressure reaches below zero, the positive phase is finished and the load has entered the negative phase, which lasts for a duration of t_{d-} , where the overpressure asymptotically approaches the initial value of the ambient pressure. For all explosions, the peak side-on overpressure applied to the front façade is the reflected overpressure, P_r . For an incident angle of zero degrees, and by setting the specific heat ratio of air, γ , to the known value of 1.4, the reflected pressure is obtained [13]. Two different approaches exist to simplify the applied pressure of the blast as a triangular-shaped load. The conservative approach considers the maximum value of the pressure and the entire duration of the positive phase. The less conservative approach keeps the same maximum pressure value as the original curve, but the duration of the load is recalculated such that the impulse of the original load and the simplification are equivalent [25]. Both of these approaches assume the effects of the negative phase on the structure are negligible. This simplification of the loading profile also

assumes the blast loading is acting on a rectangular structure positioned above-ground and that the structure is being subjected to a plane wave shock front [9]. Though the negative phase of the pressure profile can affect the dynamic response of the structure [26,27], this can be neglected when the duration of the negative phase is much smaller than the natural period of a structural component.

Figure 1. Time-history and spatial evolution of a blast adapted from [18].

Explosive devices placed near or on the ground can be classified as surface bursts, and equations for hemispherical surface bursts can be used to determine the peak side-on overpressure, reflected overpressure, arrival time, and duration of the pressure profile. Ullah et al. [8] compared the numerous empirical formulations proposed over the years to the charts proposed by the US Army Corps [9]. The equations proposed by Wu and Hao for the arrival time [10] and the equation by Sadovskyi for the positive phase duration [11] were closest to these charts [8]. Newmark and Hansen's equation for the peak side-on overpressure [12] was used in a previous case study [2]. The reflected overpressure was calculated based on previously determined formulations [13]. These conclusions were used to justify the methods in this research.

Blasts occurring in an internal or confined environment have increased effects on the structure, as the peak reflected pressure will be amplified to be greater than the magnitude of an unconfined explosion, and the blast wave will have a longer duration [28–30]. As a result, the impulse will be enhanced, as it is defined by the area of the overpressure history, the increase in the peak overpressure, and the length of the blast. Unlike initial reflected shock wave parameters (P_r , i_r), the magnitude of re-reflected waves is more difficult to quantify, and idealizations of the shock pulse as triangular loads can be done to approximate the internal pressure time-histories. Baker et al. suggested an approach where the peak pressure is reduced by half for each re-reflection and the duration of each pulse constant, consequently decreasing the impulse by half for each re-reflection [14]. This approach also accounts for the amplification of the reflected pressure and duration of the blast wave [13]. After three reflections, the pressure of the reflected wave is assumed to be zero (Figure 2). When a blast occurs in a partially vented structure, quasi-static pressures are expected to build up as the re-reflections decrease [13,31]. However, to simplify the loading profile input into the modelling software, these quasi-static pressures were neglected from the analysis.

The exterior blast loading scenario corresponded to 110 kg of TNT placed at 4 m from the south façade of the building. All loads acting on the south façade were calculated using the reflected pressure, and the remaining loads were calculated using the peak side-on overpressure. No pressure was applied on the bell gable and on the dark gray locations in Figure 3, which correspond to boundary conditions. The interior blast scenario corresponded to 6.8 kg of TNT placed in the north room of the building (Figure 4), at 0.4 m from the north wall and 0.2 m above the ground. The loads acting on the floor were neglected as the base of the model was pinned in the vertical direction in the simplified model. For both scenarios, the standoff distance between the explosive device and the center of the surface was determined, and the scaled distance was calculated.

Figure 2. Simplification of the internal blast wave reflections adapted from [13].

Figure 3. Locations of the blast on the exterior of the *Torre de la Vela*.

Figure 4. Zone locations for the interior blast.

The accuracy of the FEM model is ensured by comparing the eigenvalue analysis results to a previously created model of the *Torre de la Vela* calibrated to the building using dynamic identification [1]. Though the modeling strategy was not calibrated experimentally within this work, this strategy has been previously validated experimentally by other authors, in which the numerical and experimental results were found to be consistent [18,32–35].

When acting on a structure, blast loads can produce very high strain rates in the range of 10^2 to 10^4 s^{-1} , which alter the dynamic mechanical properties of the structure [17]. Because the dynamic mechanical properties of the material are vastly different than the static properties, dynamic increase factors (DIFs) can be used to account for this increase of strength. This work calculated DIFs for the Young's modulus, E , and compressive strength, σ_c , of brick masonry according to Pereira's equations [2]. Pereira's equations for the DIFs of the mortar were used for the Young's modulus and compressive strength of the rammed earth and infill because the dynamic behavior of the later materials was unavailable, and their static behavior is similar to mortar [36]. The DIFs for tensile strength are also unavailable and were assumed to be the same as the compressive strength. The DIFs were calculated assuming a strain rate of 100 s^{-1} (Table 1), and the material properties with and without accounting for the dynamic increase factors can be seen in Table 2. The resulting DIFs are also in line with the DIFs stated by other authors [3–5]. As the magnitude of these factors is high, neglecting these factors in the work would result in a large underestimation of the material strength, which provides an intrinsic safety estimation of the building's structural resistance, but does not accurately represent the behavior of the building.

Table 1. Dynamic increase factors for the brick masonry, the rammed earth, and the infill under blast loading.

Material Property	DIF	
	Brick Masonry	Rammed Earth and Infill
E	1.98	2.65
σ_c	1.97	1.76
σ_t	1.97	1.76

Table 2. Material properties with and without considering the dynamic increase factors (DIFs).

Properties	Material							
	Rammed Earth				Brick Masonry		Infill	
	Level 1		Other Floors		Original	DIF	Original	DIF
	Original	DIF	Original	DIF				
E (kN/m ²)	5.90×10^6	1.57×10^7	1.10×10^6	2.92×10^6	1.60×10^6	4.25×10^6	0.60×10^6	1.59×10^6
ν (—)	0.30	0.30	0.30	0.30	0.25	0.25	0.30	0.30
ρ (t/m ³)	2.25	2.25	2.25	2.25	1.60	1.60	2.25	2.25
σ_c (kN/m ²)	2500	4390	2500	4390	4000	7020	2500	4390
G_{fc} (kN/m)	4.00	7.02	4.00	7.02	6.40	11.24	4.00	7.02
σ_t (kN/m ²)	300	527	300	527	200	351	300	527
G_{ft} (kN/m)	0.017	0.026	0.017	0.026	0.013	0.019	0.017	0.026

The constitutive model adopted in Abaqus was the Concrete Damaged Plasticity (CDP) model, which is a modification of the Drucker-Prager model made by Lubliner et al. [37] and Lee and Fenves [38]. This model represents the inelastic and fracture behavior of the material by the concept of isotropic damage evolution combined with isotropic tensile and compressive plasticity. The CDP model also assumes that damage plasticity characterizes the failure for tensile cracking and compressive crushing of the material and allows the definition of the strain softening in tension and strain hardening in compression. Previous studies have used the CDP model for masonry buildings under blast loading [32,33], concrete members under high strain rate [39], and rammed earth constructions under seismic events [40]. The dilation angle, flow potential eccentricity, ratio of initial equibiaxial compressive yield stress to the initial uniaxial compressive yield stress, and viscosity parameter were defined according to a previous study [41]. Stress-strain diagrams were used to define the compressive and tensile behavior in Abaqus. To ensure consistency of the material properties compared to previous work on the *Torre de la Vela* [1], $1 \times 1 \times 1$ cm³ elements, corresponding to each material, were meshed with a single element and compared in both compression and tension. The parabolic stress-strain law was selected for compressive behavior, and the exponential stress-strain law was selected for tensile behavior. A damage criterion was defined for all the materials in tension, and a damage parameter of 0.95 was assigned to the maximum tensile strain of each material.

Four different geometries were modelled to reduce computational time for different loading scenarios, and to better understand the local effects of the load:

1. Full building model (Figure 5a);
2. Simplified model of the south façade (Figure 5b);
3. Model of Level 2 (Figure 5c);
4. Model of a single interior wall (Figure 5d).

Throughout the different models, different mesh sizes and element types were used. In model 1, general purpose tetrahedron pyramidal elements (C3D10) were used, with the element size ranging from 0.3 m in the vaults to 0.8 m in the exterior walls, for a total of 268,000 elements and 1,228,503 degrees of freedom. Models 3 and 4 also used C3D10 elements, where Model 3 had 180,187 elements in total. The size of the elements in Model 4 varied between 0.1 m, 0.2 m, and 0.3 m to observe the effects of the mesh size. In Model 2, C3D8R elements were used with 0.125 m elements around the door and 0.6 m elements for the remainder, for a total of 20,331 elements.

Figure 5. Models created for the loading scenarios: (a) full building model; (b) simplified façade model; (c) Level 2 model; (d) interior wall model.

Several approaches to numerically representing masonry structures exist, which include macro-modeling, detailed micro-modeling, and simplified micro-modeling [42]. Macro-modeling was used to model most of the structures in the work. In the simplified façade model of the *Torre de la Vela*, the simplified micro-modeling approach was used to represent the regions around the front entrance opening. Mohr-Coulomb was adopted to model the sliding failure on the frictional interfaces between the contact elements. As the interaction does not work in the plasticity framework, the model assumes non-associative flow, and the dilatancy angle in the model was automatically assigned a value of 0, in line with work by previous authors [43,44]. The cohesion was considered negligible for this problem because the cohesion using the Mohr-Coulomb failure criterion cannot be modeled in Abaqus. The normal stiffness and tangent stiffness (Table 3) were based on past studies [43]. Using the joint stiffness, the Young's modulus of the blocks was recalculated based on [45]. The friction coefficient (Table 3) was taken as the average value of past studies [43,46–48].

Table 3. Interaction properties.

Friction Coefficient	Normal Stiffness, $K_{n,joint}$	Tangent Stiffness, $K_{s,joint}$
0.73	$1.50 \times 10^8 \text{ kN/m}^3$	$7.50 \times 10^7 \text{ kN/m}^3$

The damage defined by the US Army Corps [9] is based on the damage on the maximum rotation at the support. This method could not be used to define the damage on the exterior rammed earth walls due to the slenderness of the walls resulting from their thickness. For this reason, this method of damage characterization was limited to the interior brick walls. For a global characterization of the damage, the Failure Volume Index was used to characterize the damage stage of the building, as this was the most accurate damage index according to Hoveidae et al. [15]. The damage index can be calculated according to (3), where V_{fail} represents the volume of elements with damage exceeding a certain threshold, and V_{total} represents the volume of all the elements in the structure. Subsequently, the damage indices can be used to define damage levels [16]. For the tension damage defined by Abaqus, a damage threshold of 0.77 was selected [15].

$$DI = \frac{V_{fail}}{V_{total}} \times 100 \quad (3)$$

3. Effects of Blast Loading on the *Torre de la Vela*

Two different scenarios were implemented to determine the blast loading effects on the *Torre de la Vela*. The exterior blast loading scenario considered 110 kg of TNT at 4 m from the south façade of the building, and the interior blast loading scenario considered 6.8 kg of TNT in the north room of the second level.

3.1. Exterior Blast

Following the nonlinear analysis in Abaqus/Explicit, the tensile strains in the *Torre de la Vela* at the end of the analysis were plotted to better understand which regions in the building experienced cracking (Figure 6). Cracking can be found around the entrance on Level 3, where the blast load is being applied. Some diagonal cracks can also be observed on the east façade, connected to a crack on the roof perpendicular to the blast. Additionally, significant cracking is present at the bell gable, which indicates a local failure. With respect to the interior of the building, shear cracks can be observed on the south vaults on Level 4, and radial cracking on the east vaults. These cracks correspond to the exterior cracks on the roof and the east façade. The outer brick masonry walls closest to the entrance of the building also show signs of cracking. The locations where the tensile damage in Abaqus exceeded 0.77 are plotted in Figure 7.

Two simplified models of the south façade were created to better understand the behavior of the façade closest to the blast loading—the first using a combined micro-macro modeling approach and the second using continuum elements. For the simplified model to be representative of the full building, springs were used where the façade intersected the remaining walls and slabs. These springs were calibrated using linear analysis on both models and comparing the displacement patterns. The south façade modeled using continuum elements used the removal of damaged elements feature in Abaqus, where elements reaching a maximum degradation can be removed from the model during the analysis. In Abaqus/Explicit, the element is removed from the mesh when the degradation reaches the maximum degradation at any single integration point in the element [49]. When using continuum elements, elements exceeding a damage threshold of 0.77 were removed from the model during the analysis [15].

Figure 6. Tensile strain at 0.62 s: (a) exterior of the building; (b) interior of the building.

Figure 7. Damage in tension greater than 0.77: (a) exterior of the building; (b) interior of the building.

In the micro-macro model, the largest displacements occurred on the lower portion surrounding the door (Figure 8a). Displacement in the north-south direction can also be observed growing radially from the door throughout the whole façade. Most of the displacement occurs above the base of the door, and not below, as the pressure profile from the blast is not applied below the door due to the presence of a bridge. The maximum displacement in the direction of the blast in the contact model is 2.55 cm. As seen in Figure 8b, damage is occurring around the opening where the blast is being applied, causing a hole in a circular pattern. Only the elements closest to the opening have exceeded the allowed damage, creating the appearance of a hole, but the damaged elements do not go through the entire thickness of the external wall. When comparing the two models, the locations of maximum displacement in the micro-macro model correspond closely to the locations where damaged elements have been removed.

Figure 8. Damage in tension greater than 0.77: (a) contact element model; (b) removal of damaged elements.

3.2. Interior Blast

The tensile damage in the regions of application of the blast is seen in Figure 9. A value of 0.77 was selected as the damage threshold, in line with the previous scenario [15]. When looking at the damage evolution, significant damage is noted on the wall Z1-S, just when the load is being applied. By 0.10 s, most of the elements on wall Z1-S have exceeded the tension damage threshold. In addition, the displacement on this wall is excessive, with a deflection of 2.4 m at the end of the analysis.

Figure 9. Time history of the damage to the interior walls of the building.

The wall Z1-S was modeled on its own to better understand the effect of the mesh size on the damage. Boundary conditions on this wall were selected so that the deformed shape of the wall matched the wall in the previous analysis. This wall was modeled using three different element sizes (0.1 m, 0.2 m, and 0.3 m) to understand if this impacted the damage in the wall. Figure 10 shows the displacement in all three walls occurring at a time of 0.06 s. In all three scenarios, the largest displacement occurs between the central and far-right arches, but the magnitude of the displacement is largest in the walls with a smaller mesh size. This difference in displacement directly affects the magnitude of the rotation at the center of the wall.

Figure 10. Displacement at 0.06 s for different sized meshes: (a) 0.1 m; (b) 0.2 m; (c) 0.3 m.

The previously defined explosive device was too high to properly compare the resulting damage patterns between mesh sizes. As such, an explosive device of 2.5 kg of TNT was selected. Continuum models with and without removal of damaged elements were created for the same three mesh sizes. In all three walls (Figure 11), the top of the column between the two largest arches is completely damaged, as no elements connect the base of the column to the rest of the wall. Similarly, all three walls show damage at the top of the wall, but the number of damaged elements decreases as the mesh size increases. In the walls with mesh sizes of 0.1 m and 0.2 m, significant damage is noted above the left arch, but this damage pattern is not visible in the wall with a mesh size of 0.3 m. In the model with a mesh size of 0.1 m, circular cracks can be observed between the two rightmost arches, starting above the left arch, and continuing to the top of the wall.

Figure 11. Cont.

Figure 11. Tension damage to wall Z1-S at 0.1 s: (a) 0.1 m mesh; (b) 0.1 m mesh with removed damaged elements; (c) 0.2 m mesh; (d) 0.2 m mesh with removed elements; (e) 0.3 m mesh; (f) 0.3 m mesh with removed elements.

4. Damage Quantification and Comparison

In the exterior blast scenario, a total volume of 5087 m³ existed, with 76.8 m³ of damaged elements. Using the damage index calculation, the estimated damage index in the tower is 1.51% (Table 4), making the damage level in the *Torre de la Vela* small for an explosive of 110 kg of TNT at 4 m from the building.

Table 4. Estimated damage level, damage index, and volume of failed elements for exterior blast scenario.

Damage Level	Damage Index	V_{fail} (m ³)	V_{total} (m ³)
SD	1.51%	76.8	5087.7

The compressive stresses in the south façade of the tower are ideal for comparing the differences between the models (Figure 12). In the whole model and the contact elements model, the highest compressive stresses occur at the base of the wall near the supports and gradually decrease through the height of the façade. The level of compressive stress is much greater in the whole building model than in the contact model, which, again, allows the blocks to slide. With respect to the distribution of compressive stresses in the simplified continuum model, higher values of compressive stress are seen at the base of the wall, near the supports, and around the door. This distribution is slightly different from those seen in the contact model and the whole building model, but more closely resembles the distribution seen in the whole building model as compressive stresses are present around the door opening. The difference between the stress distribution around the door of the simplified models is likely due to the dissipation of energy associated with the sliding of the blocks.

Figure 12. Compressive stresses on the south façade: (a) whole building model; (b) simplified contact model; (c) simplified continuum model.

For the interior blast, the damage index and damage level of each structural element were calculated as well as for the whole Level 2 model (Table 5). Overall, the Level 2 model has a small level of damage, suggesting minor cracking and no out-of-plane damage. However, when looking at individual structural elements, Wall Z1-S has a high damage level, which is expected due to the noticeable out-of-plane displacement in the wall. When looking at the other structural components individually, they all have small damage levels. Although the damage index may be appropriate when assessing the damage caused by seismic events [15], its use when analyzing the effects of localized loading conditions such as blast loading may not be appropriate, or at least some caution should be used when trying to apply these.

Table 5. Total volume, volume of damaged elements, damage index, and damage level for different structural components.

	Volume (m ³)	Damaged Volume (D > 0.77) (m ³)	Damage Index	Damage Level
Wall Z1-S	1430.65	33.39	53.14%	HD
Zone 1 Wall Left	42.57	0.52	2.29%	SD
Zone 1 Wall Right	12.21	6.49	0.00%	SD
Zone 2 3 Wall Left	3.02	0.07	0.00%	SD
Zone 2 3 Inner Walls	3.02	0.00	1.61%	SD
Vaults Zones 1 2 3	8.88	0.00	9.04%	SD
Outer Wall near Blast	27.64	0.45	7.98%	SD
Whole Level 2	822.48	13.69	2.33%	SD

For the interior wall modeled on its own, three different damage parameters were considered, the first being the displacement, the second being the rotation, and the third being the failure volume index. Some authors have defined the failure of a wall under blast loading as the point at which the wall reaches a displacement equal to its thickness [50,51]. For mesh sizes of 0.1 m and 0.2 m, the maximum displacement reached the wall thickness after 0.06 s (Figure 13). The wall with a mesh size of 0.3 m has a lower displacement and does not reach a maximum value of 0.6 m at the same time as the other two walls, thus not reaching failure at the same moment. However, the wall with a mesh size of 0.3 m does eventually reach failure (Figure 13). In general, the results are too different and not converging upon mesh refinement, so the subject requires more attention. The failure of a two-way masonry wall under blast loading is defined as reusable if the rotation at the supports is exceeding a value of 0.5° and is non-reusable if it exceeds a value of 2.0° [9]. Using this criterion, the wall fails very early in the application of the load, for all sizes of mesh (Figure 14). The rotation is calculated as a function of the displacement, so the same trends in the damage due to the displacement are expected in the damage due to the rotation with respect to the influence of the mesh size. It should be noted that the displacement results are expected to converge, although some differences may be justified when operating outside the assumption of small displacements. Consequently, this comparison between the effect of the mesh size on the damage requires further investigation. The failure volume index, which uses the tension damage available as a field output in Abaqus, seems to be a more appropriate approach when comparing the damage in the wall between models of different element sizes. Figure 15 further confirms the minimal influence the mesh size has on the damage index. When the load is first applied, the walls with mesh sizes of 0.1 m and 0.3 m both have a medium damage level, whereas the wall with a mesh size of 0.2 m has a small damage level. At the end of the analysis, all the walls of different mesh sizes experience high damage.

The removal of damaged elements feature allows for better understanding of what occurs at failure when the model is subjected to severe damage [49]. The advantage of this method is that highly distorted elements are removed from the calculation. In deleting damaged elements from the model during analysis, mass and energy are also removed from the system. Thus, in the case of substantial element deletion, the accuracy of the simulation will decrease. Because the removed elements occur when the degradation reaches the maximum degradation at a single integration point, the walls with removed elements are more likely to show more extensive damage patterns than the walls using banded contour plots to show the tension damage. This occurs because the values of tensile damage in Abaqus are extrapolated from the integration points to the nodes. Nevertheless, the distribution between the damage patterns is similar, as previously observed in Figure 11. Interestingly, the walls with damage plotted using contours show damage at the top of the wall, but at the front, whereas the removed damage elements at the top of the wall occur at the back. As the damage plotted in red is extrapolated from the integration point to the node, the contours represent locations where the node exceeds the given threshold of damage. However, the elements are removed when a single integration point exceeds

the damage threshold. Consequently, the difference between the locations of damage is likely caused by the way the data are interpreted in these locations.

Figure 13. Displacement at the center of the wall over time.

Figure 14. Time history of the rotation of the wall for different mesh sizes.

Figure 15. Damage index of the wall over time.

5. Conclusions

The aim of this work was to evaluate the structural response of the *Torre de la Vela* under blast loading, a rare case study applying a blast analysis on an earthen masonry construction within the context of cultural heritage. This work involved a literature review of the available concepts and theories related to blast loading, the preparation and validation of a numerical model, and the structural analysis of the tower under blast loading. The use of the failure volume index based on the tensile damage in Abaqus allows us to characterize the level of damage occurring in an earthen masonry structure after blast loading but should be used with caution as it does not always consider the local damage effects, as noted in the interior loading scenario of the tower. The work also utilized the disappearing damaged elements function in Abaqus to illustrate the damage, a feature rarely being utilized in the field of structural engineering. The use of contact elements causes more dissipation of energy in the facades. Finally, when characterizing the damage caused by blast loading, the mesh size did not affect the level of damage but did affect the distribution patterns.

Author Contributions: Conceptualization, D.T., J.M.P. and P.B.L.; methodology, D.T. and J.M.P.; formal analysis, D.T.; investigation, D.T.; resources, J.M.P. and P.B.L.; writing—original draft preparation, D.T.; writing—review and editing, J.M.P. and P.B.L.; supervision, J.M.P. and P.B.L.; project administration, J.M.P.; funding acquisition, P.B.L. All authors have read and agreed to the published version of the manuscript.

Funding: The financial support of the Fundação Millennium BCP provided by the University of Minho is acknowledged. This work was 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.

Data Availability Statement: Not applicable.

Acknowledgments: The authors would like to acknowledge the support of Javier Ortega and Annalaura Vuoto for sharing indispensable information on the *Torre de la Vela*.

Conflicts of Interest: The authors declare no conflict of interest.

References

1. Vuoto, A. Numerical Safety Assessment of Earthen Structures in La Alhambra, Granada, Spain. Ph.D. Thesis, Universidade do Minho, Guimaraes, Portugal, 2020.
2. Pereira, J.M.; Lourenço, P.B. Experimental Characterization of Masonry and Masonry Components at High Strain Rates. *J. Mater. Civ. Eng.* **2017**, *29*, 04016223. [[CrossRef](#)]
3. Hao, H.; Tarasov, B. Experimental Study of Dynamic Material Properties of Clay Brick and Mortar at Different Strain Rates. *Aust. J. Struct. Eng.* **2008**, *8*, 117–132. [[CrossRef](#)]
4. Burnett, S.; Gilbert, M.; Molyneaux, T.; Tyas, A.; Hobbs, B.; Beattie, G. The response of masonry joints to dynamic tensile loading. *Mater. Struct.* **2007**, *40*, 517–527. [[CrossRef](#)]
5. Asprone, D.; Cadoni, E.; Prota, A.; Manfredi, G. Dynamic behavior of a Mediterranean natural stone under tensile loading. *Int. J. Rock Mech. Min. Sci.* **2009**, *46*, 514–520. [[CrossRef](#)]
6. Baylot, J.T.; Bullock, B.; Slawson, T.R.; Woodson, S.C. Blast Response of Lightly Attached Concrete Masonry Unit Walls. *J. Struct. Eng.* **2005**, *131*, 1186–1193. [[CrossRef](#)]
7. Moreland, C.; Hao, H.; Wu, C. Response of retrofitted masonry walls to blast loading. In *Proceedings of the 6th Asia-Pacific Conference on Shock and Impact Load Structures*; CI-Premier: Singapore, 2005; pp. 405–412.
8. Ullah, A.; Ahmad, F.; Jang, H.-W.; Kim, S.-W.; Hong, J.-W. Review of analytical and empirical estimations for incident blast pressure. *KSCE J. Civ. Eng.* **2017**, *21*, 2211–2225. [[CrossRef](#)]
9. U.S. Army Corps of Engineers, Naval Facilities Engineering Command. *Structures to Resist the Effects of Accidental Explosions*, Air Force Civil Engineer Support Agency; UFC 3-340-02; U.S. Department of Defense: Washington, DC, USA, 2008.
10. Wu, C.; Hao, H. Modeling of simultaneous ground shock and airblast pressure on nearby structures from surface explosions. *Int. J. Impact Eng.* **2005**, *31*, 699–717. [[CrossRef](#)]
11. Sadovskiy, M.A. *Mechanical Effects of Air Shock Waves from Explosions According to Experiments, Selected Works: Geophysics and Physics of Explosion*; Nauka Press: Moscow, Russia, 1952.
12. Newmark, N.M.; Hansen, R.J. Design of blast resistant structures. In *Shock and Vibration Handbook*; Harris, C.M., Crede, C.E., Eds.; McGraw-Hill: New York, NY, USA, 1961.

13. Cormie, D.; Mays, G.; Smith, P. *Blast Effects on Buildings*; Thomas Telford Ltd: London, UK, 2011. Available online: <http://app.knovel.com/hotlink/toc/id:kpBEBE0001/blast-effects-on> (accessed on 17 January 2021).
14. Baker, W.E.; Kulesz, J.J.; Westine, P.S.; Cox, P.A.; Wilbeck, J.S. *A Manual for the Prediction of Blast and Fragment Loadings on Structures*; DOE/TIC-11268; U.S. Department of Energy: Washington, DC, USA, 1980.
15. Hoveidae, N.; Fathi, A.; Karimzadeh, S. Seismic damage assessment of a historic masonry building under simulated scenario earthquakes: A case study for Arge-Tabriz. *Soil Dyn. Earthq. Eng.* **2021**, *147*, 106732. [[CrossRef](#)]
16. Asteris, P.; Chronopoulos, M.; Chrysostomou, C.; Varum, H.; Plevris, V.; Kyriakides, N.; Silva, V. Seismic vulnerability assessment of historical masonry structural systems. *Eng. Struct.* **2014**, *62–63*, 118–134. [[CrossRef](#)]
17. Ngo, T.; Mendis, P.; Gupta, A.; Ramsay, J. Blast Loading and Blast Effects on Structures—An Overview. *Electron. J. Struct. Eng.* **2007**, *7*, 76–91.
18. Masi, F. Fast-Dynamic Response and Failure of Masonry Structures of Non-Standard Geometry Subjected to Blast Loads. Ph.D. Thesis, École Centrale de Nantes, Nantes, France, 2020.
19. Bangash, M.Y.H.; Bangash, T. *Explosion-Resistant Buildings: Design, Analysis and Case Studies*; Springer: Berlin, Germany; New York, NY, USA, 2006.
20. Encyclopedia Britannica. Alhambra. 19 December 2019. Available online: <https://www.britannica.com/topic/Alhambra-fortress-Granada-Spain> (accessed on 30 March 2021).
21. Jaquin, P.A. Analysis of Historic Rammed Earth Construction. Ph.D. Thesis, Durham University, Durham, UK, 2008. Available online: <http://etheses.dur.ac.uk/2169/> (accessed on 22 March 2021).
22. Reza, A.; Ibrahim, Z.; Vaitekunas, T. Investigation and analysis of an explosion at the goex black powder manufacturing facility. *Chem. Eng. Trans.* **2013**, *31*, 475–480. [[CrossRef](#)]
23. Griffiths, F.A. *The Artillerist's Manual and British Soldier's Compendium*; W.H. Allen: London, UK, 1856.
24. Brode, H.L. Numerical Solutions of Spherical Blast Waves. *J. Appl. Phys.* **1955**, *26*, 766–775. [[CrossRef](#)]
25. Pereira, J.M. Security Evaluation and Design of Structures Subjected to Blast Loading. Ph.D. Thesis, Universidade do Minho, Guimaraes, Portugal, 2014.
26. Masi, F.; Stefanou, I.; Vannucci, P.; Maffi-Berthier, V. Rocking response of inverted pendulum structures under blast loading. *Int. J. Mech. Sci.* **2019**, *157–158*, 833–848. [[CrossRef](#)]
27. Rigby, S.E.; Tyas, A.; Bennett, T.; Clarke, S.; Fay, S.D. The Negative Phase of the Blast Load. *Int. J. Prot. Struct.* **2014**, *5*, 1–19. [[CrossRef](#)]
28. Remennikov, A.; Rose, T.A. Modelling blast loads on buildings in complex city geometries. *Comput. Struct.* **2005**, *83*, 2197–2205. [[CrossRef](#)]
29. Liu, J.; Yan, Q.; Wu, J. Analysis of blast wave propagation inside tunnel. *Trans. Tianjin Univ.* **2008**, *14*, 358–362. [[CrossRef](#)]
30. Shi, Y.; Li, Z.-X.; Hao, H. Numerical investigation of blast loads on RC slabs from internal explosion. In *International Workshop on Structures Response to Impact and Blast*, [CD-ROM]; Israel Institute of Technology: Haifa, Israel, 2009.
31. Feldgun, V.; Karinski, Y.; Edri, I.; Yankelevsky, D. Prediction of the quasi-static pressure in confined and partially confined explosions and its application to blast response simulation of flexible structures. *Int. J. Impact Eng.* **2016**, *90*, 46–60. [[CrossRef](#)]
32. Pereira, J.; Campos, J.; Lourenco, P. Masonry infill walls under blast loading using confined underwater blast wave generators (WBWG). *Eng. Struct.* **2015**, *92*, 69–83. [[CrossRef](#)]
33. Pereira, J.; Lourenco, P. Risk Assessment Due to Terrorist Actions on Public Transportation Networks: A Case Study in Portugal. *Int. J. Prot. Struct.* **2014**, *5*, 391–415. [[CrossRef](#)]
34. Jablonski, J.; Carlucci, P.; Thyagarajan, R.; Nandi, B.; Arata, J. Simulating Underbelly Blast Events using Abaqus/Explicit—CEL. In Proceedings of the SIMULIA Customer Conference, Providence, RI, USA, 15–17 May 2012.
35. Zakrisson, B.; Wikman, B.; Häggblad, H.Å. Numerical simulations of blast loads and structural deformation from near-field explosions in air. *Int. J. Impact Eng.* **2011**, *38*, 597–612. [[CrossRef](#)]
36. Meyer, C.S. Numerical Simulations of the Mechanical Behaviour of Adobe. In *Dynamic Behavior of Materials, Volume 1: Proceedings of the 2012 Annual Conference on Experimental and Applied Mechanics*; Springer: New York, NY, USA, 2013. [[CrossRef](#)]
37. Lubliner, J.; Oliver, J.; Oller, S.; Onate, E. A plastic-damage model for concrete. *Int. J. Solids Struct.* **1989**, *25*, 299–326. [[CrossRef](#)]
38. Lee, J.; Fenves, G.L. Plastic-Damage Model for Cyclic Loading of Concrete Structures. *J. Eng. Mech.* **1998**, *124*, 892–900. [[CrossRef](#)]
39. Altaee, M.; Kadhim, M.; Altayee, S.; Adheem, A. Employment of damage plasticity constitutive model for concrete members subjected to high strain-rate. In Proceedings of the 1st International Multi-Disciplinary Conference Theme: Sustainable Development and Smart Planning, IMDC-SDSP 2020, Cyperspace, 28–30 June 2020. [[CrossRef](#)]
40. Bui, Q.-B.; Bui, T.-T.; Tran, M.-P.; Bui, T.-L.; Le, H.-A. Assessing the Seismic Behavior of Rammed Earth Walls with an L-Form Cross-Section. *Sustainability* **2019**, *11*, 1296. [[CrossRef](#)]
41. Mohamad, A.-B.A.E.; Chen, Z. Experimental and Numerical Analysis of the Compressive and Shear Behavior for a New Type of Self-Insulating Concrete Masonry System. *Appl. Sci.* **2016**, *6*, 245. [[CrossRef](#)]
42. Lourenço, P.B. Structural masonry analysis: Recent developments and prospects. In Proceedings of the 14th International Brick/Block Masonry Conference, Sydney, Australia, 13–20 February 2008.
43. Shrestha, K.C.; Aoki, T.; Miyamoto, M.; Wangmo, P. Pema In-Plane Shear Resistance between the Rammed Earth Blocks with Simple Interventions: Experimentation and Finite Element Study. *Buildings* **2020**, *10*, 57. [[CrossRef](#)]
44. Lourenco, P.; Ramos, L. Characterization of Cyclic Behavior of Dry Masonry Joints. *J. Struct. Eng.* **2004**, *130*, 779–786. [[CrossRef](#)]

-
45. Lourenco, P.; Oliveira, D.; Roca, P.; Orduña, A. Dry Joint Stone Masonry Walls Subjected to In-Plane Combined Loading. *J. Struct. Eng.* **2005**, *131*, 1665–1673. [[CrossRef](#)]
  46. Bui, T.-L.; Bui, T.-T.; Bui, Q.-B.; Nguyen, X.-H.; Limam, A. Out-of-plane behavior of rammed earth walls under seismic loading: Finite element simulation. *Structures* **2020**, *24*, 191–208. [[CrossRef](#)]
  47. Wangmo, M.P.; Shrestha, K.C.; Miyamoto, M.; Aoki, T. Assessment of out-of-plane behavior of rammed earth walls by pull-down tests. *Int. J. Arch. Heritage* **2019**, *13*, 273–287. [[CrossRef](#)]
  48. El-Nabouch, R.; Bui, Q.-B.; Plé, O.; Perrotin, P. Characterizing the shear parameters of rammed earth material by using a full-scale direct shear box. *Constr. Build. Mater.* **2018**, *171*, 414–420. [[CrossRef](#)]
  49. ABAQUS. *ABAQUS User's Manual*; Version 2019; Dassault Systèmes: Waltham, MA, USA, 2010.
  50. Doherty, K.; Griffith, M.C.; Lam, N.; Wilson, J. Displacement-based seismic analysis for out-of-plane bending of unreinforced masonry walls. *Earthq. Eng. Struct. Dyn.* **2002**, *31*, 833–850. [[CrossRef](#)]
  51. Zapata, B.J.; Weggel, D.C. Collapse Study of an Unreinforced Masonry Bearing Wall Building Subjected to Internal Blast Loading. *J. Perform. Constr. Facil.* **2008**, *22*, 92–100. [[CrossRef](#)]