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Reliability Assessment of Masonry Infilled RC Frame Building's Earthquake Performance through Accidental Torsion Consideration

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Abstract

Accidental torsional behaviour induced by horizontal loading is difficult to predict, being a complex phenomenon governed by many variables. This problem gains an additional dimension of complexity when nonlinear responses with imperfections need to be considered. Therefore, evaluation and understanding the influence of accidental torsion are fundamental in seismic reliability estimation. This study offers vital insights based on the results of a 1/2.5 scale three-story masonry infilled reinforced concrete frame building's test on a shaking table. The building was tested under ten consecutive ground motions with increasing ag/g, recorded at Herzeg Novi station during the 1979-M6.9 Montenegro earthquake. The accidental eccentricity, considered a random variable, resulted from unsymmetrical masonry infill wall damage in an otherwise regular building. Its effect, in relation to that of other random (design) variables, was evaluated utilising weight factors and, in addition, assessed through various building code provisions and state-of-the-art research findings. The analysis revealed that the accidental eccentricity, as compared to other random variables considered, could, under certain conditions, reach values higher than those prescribed by the building codes. This unacceptable seismic reliability clearly warns that accidental torsion of masonry-infilled reinforced concrete frames in in-situ conditions must be considered even in regular buildings.

Keywords: Earthquake Behavior; Masonry Infill Wall; Reinforced Concrete Frame; Accidental Torsion; Reliability Analysis.

1. Introduction

Contemporary building codes or guidelines cannot accurately predict the building's accidental, i.e., unintended, torsional behaviour. Accidental torsion can occur when the building is subjected to horizontal loading, e.g., strong earthquake ground motion. It can arise due to: the non-uniform distribution of masses within the building, uneven changes, i.e., distribution in the stiffness of the structural elements and the structure as a whole, the torsional component of ground motion, etc. Additionally, uncertainty in determining centres of mass and rigidity can be significant.

Anagnostopoulos et al. [1] made a comprehensive state-of-the-art review as a continuation of the review done by De Stefano and Pintucchi [2], relating to the impact of induced torsion on buildings during earthquakes. They showed that this research area is accompanied by many publications, attributing that to the importance of torsion's adverse effects. Only in this century were more realistic models introduced, based on simplified one-story models [3], emphasising that parameters needed to be matched with real buildings [3]. The previous research on torsional effects was mainly based on the causes of torsion, such as the eccentric distribution of stiffness [4, 5], damping [6, 7], building mass [8, 9], and spatially uneven or torsional ground motion [10, 11]. It should be noted that because of the large number of parameters, significant simplifications and assumptions needed to be introduced, which often limits the applicability of given results. One of the most common simplifications is the omission of partition walls from the

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analysis, which is an important aspect of seismic design with its own set of research problems: size, type, and position of openings [12, 13], intentional interventions on the infill panel [14–16], out-of-plane collapse under in-plane action, or direct out-of-plane failure [17–19].

Accidental torsion is included in most building codes, such as Eurocode, ASCE, NBCC, the New Zealand building code, and the Model Building Code of Mexico [20–24], through the introduction of appropriate eccentricity. The eccentricity usually varies from 5 to 10 % of the observed building's floor plan dimension perpendicular to the direction of the earthquake action. Mohamed and Mehana [25] showed that even the method of analysis (static or dynamic), in line with codes, can influence the results of a symmetric floor plan building. Fronteddu et al. [26] defined the effect of different design principles for accidental torsion in NBCC, showing that the method can be adequate but depends on the ductility demand. De-la-Colina et al. [27] used code values of accidental eccentricities and Monte Carlo values in combination with nonlinear dynamic analysis on a symmetrical steel building, concluding that various methods of applying accidental torsion are not equivalent—shifting masses is simpler than applying floor torsion moments. Lin et al. [28] similarly concluded that adding accidental eccentricity does not equal the effects of using the torsional amplification factor. These conclusions point to the fact that there is still room for improvement of the building codes, and although clear progress has been made in research, it can also be said that the progress is not gaining as much attention and development as in the 70s and 80s of the 20th century [1]. This is especially true when partition walls are often not considered per code treatment, nullifying their effect. Namely, in other research studies, various possibilities of the influence of masonry infill walls on the behaviour of the reinforced concrete frame building have been observed. In some of them, it has been recorded that masonry infill walls have a positive effect [12, 29, 30], while in others, it has been observed that the masonry infill wall potentially damages the structure when horizontal force is applied [13, 31, 32]. There is indeed a general stance that infill can have positive and negative effects [33, 34].

Most of the existing research in this domain is related to even more complex issues, such as accidental torsion in irregular, asymmetric buildings, nonlinear range of behaviour, reliability analysis, and similar advanced problem areas. Each research study adds a new perspective and insight to the problem, simultaneously signalling that more is to be done. Kohiyama & Yokoyama [35] showed that even the most basic assumptions could be further improved by expanding the theory of structural eccentricity and concluding that there is an overlooked phenomenon called the Q-Δ effect (Q represents lateral force), accounting for large displacements. De-la-Colina & Valdés-González [36] continue with this thought process by proposing a design procedure that would not require accidental torsion but an amplification of design parameters, and Khatiwada & Lumantarna [37] propose a simplified method for determining torsional stability. Some research deals with the occurrence of torsion in regular symmetric structures in the nonlinear range [38–42], all pointing to the importance of this occurrence in regular buildings and specific issues regarding the material in question – RC or steel. Several more authors [43–47] have researched the subject of accidental eccentricity; however, with those utilising the nonlinear range, there is a lack of model verification with experimental data and considering partition walls, as earlier mentioned. On that note, Guéguen et al. [48, 49] highlight that the torsional response of structures has been mainly analysed with numerical studies due to a lack of experimental data with translational and rotational sensors. Although irregular buildings seem more important when studying accidental torsion effects, symmetric buildings' structural response to accidental torsion effects is increased compared to unsymmetrical buildings [50]. The variation of the strength of elements resistant to horizontal forces can result in considerable accidental torsion in the case of regular buildings [38]. Reliability analysis adds another layer of complexity to the analysis of accidental torsion, but regardless of the underlying methods, research agrees that considering accidental torsion impacts the reliability of a building. Chang et al. [51, 52] state that in the case of steel structures, consideration of accidental torsion significantly influences the probability of failure. Mortezaei & Mohsenian [53] deal with the reliability of symmetric structures and lay out the essential variables affecting reliability based on a box system building. Lin et al. [54] tie reliability to the analysis method, defining the differences in probabilities resulting from deflection and torsional amplification factors.

Based on the presented research, it is evident that the influence of accidental torsion is rarely considered in conjunction with experimental data, and research with the inclusion of infill walls in the influence of horizontal behaviour is sparse. When combined with research that systematically deals with this phenomenon from the aspect of reliability, there is a need for more data and research, especially as reliability is the basis for building code provisions. The framework of the FRAMA project (FRAme-MAsonry Composites for modeling and Standardisation) enables such essential insights as a 1/2.5 scale three-story masonry infilled reinforced concrete frame building was tested on a shaking table [55] (see Figure 1). Namely, the test results point to accidental torsion at an intensity significantly higher than recommended in mentioned building codes. Consequently, the basic idea of this paper is to show how the eccentricity e, as a measure of random torsion, in this case for RC frames with masonry infill walls, influences reliability. In this sense, a probabilistic analysis of the first-order reliability method (FORM) was carried out on predefined properties of random variables, and the reliability indices β and influence factors α_i were calculated for the building. The workflow is summarised in Figure 2. Based on the results, accidental torsion can significantly influence regular buildings because of unsymmetrical damage patterns connected to the nature of infill behaviour during cyclic loading.

Figure 1. A view on the 1/2.5 scale three-story masonry infilled reinforced concrete frame building before testing

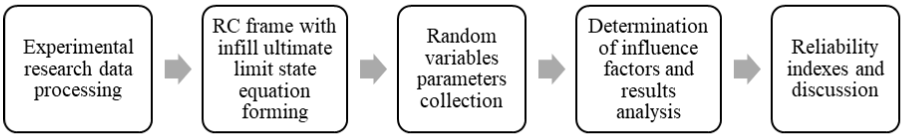


Figure 2. Methodology flowchart

2. Background Experimental Research

The 1/2.5 scale three-story masonry infilled reinforced concrete frame structure was designed and constructed in compliance with EN 1992-1-1:2004 and EN 1998-1:2005 provisions for moment-resisting frames by considering the medium ductility form of seismic construction detailing (see Figure 1). Masonry infill walls were built after the frame had hardened and were regularly positioned in plan and elevation. All openings were centred in the bays. In the first series of tests, masonry infill walls were built out of clay block masonry units laid with class M5 general-purpose mortar (see Figure 3).

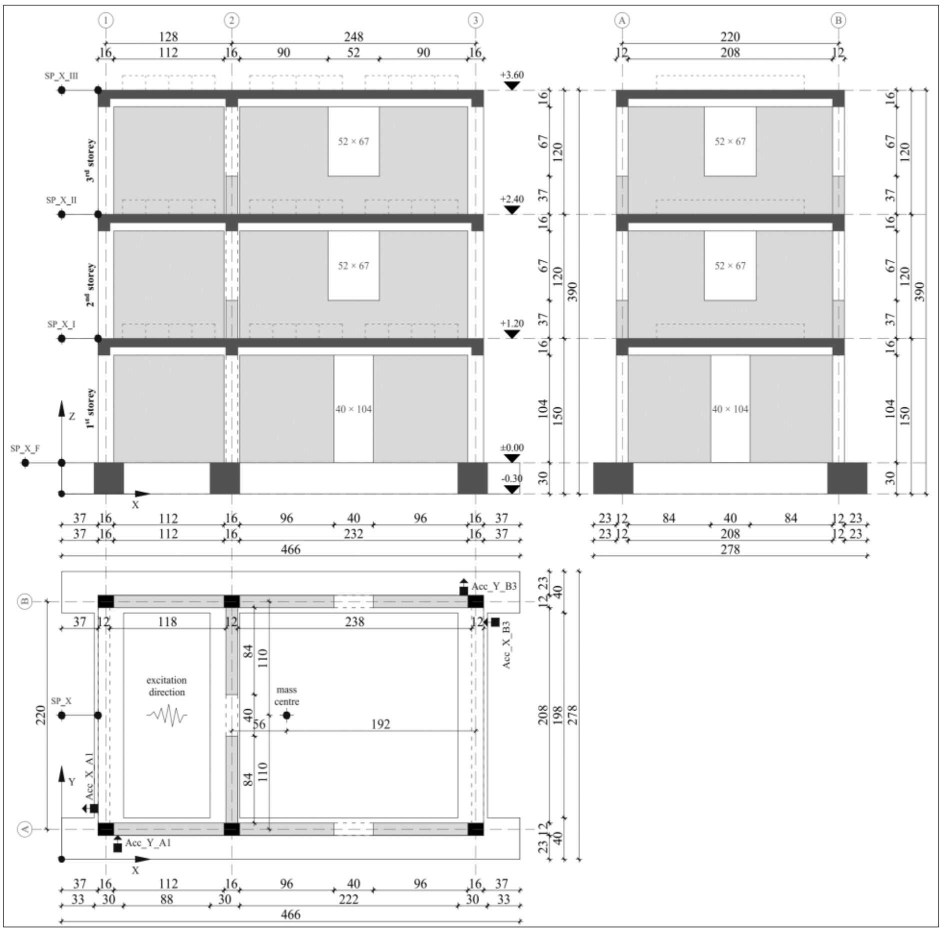


Figure 3. Longitudinal (top left), transverse (top right) and instrument plan views of the Series 1 build. (All dim. in cm) [25]

In the second series of tests, the masonry infill walls in the first and second stories were replaced by solid clay brick masonry, including reinforced concrete confining elements along vertical opening edges (see Figure 4). Opening sizes and joint mortar thicknesses were kept the same. The reinforcement plan for the RC frame is given in Figures 5 and 6. The properties of the utilised materials are provided in Table 1.

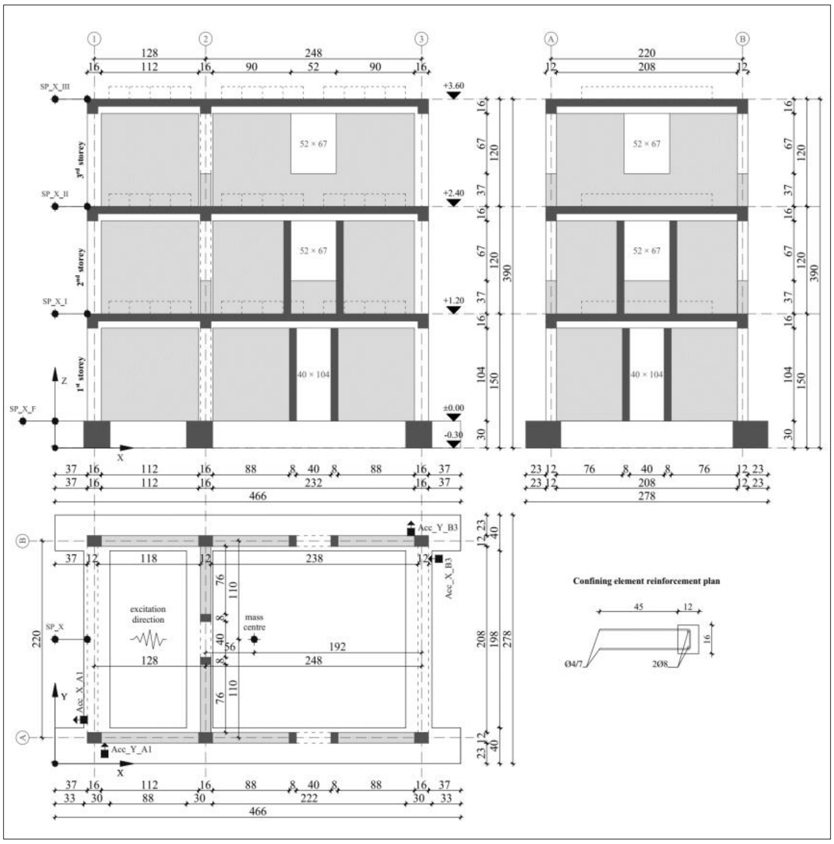


Figure 4. Longitudinal (top left), transverse (top right) and instrument plan views of the Series 2 build. (All dim. in cm) [25]

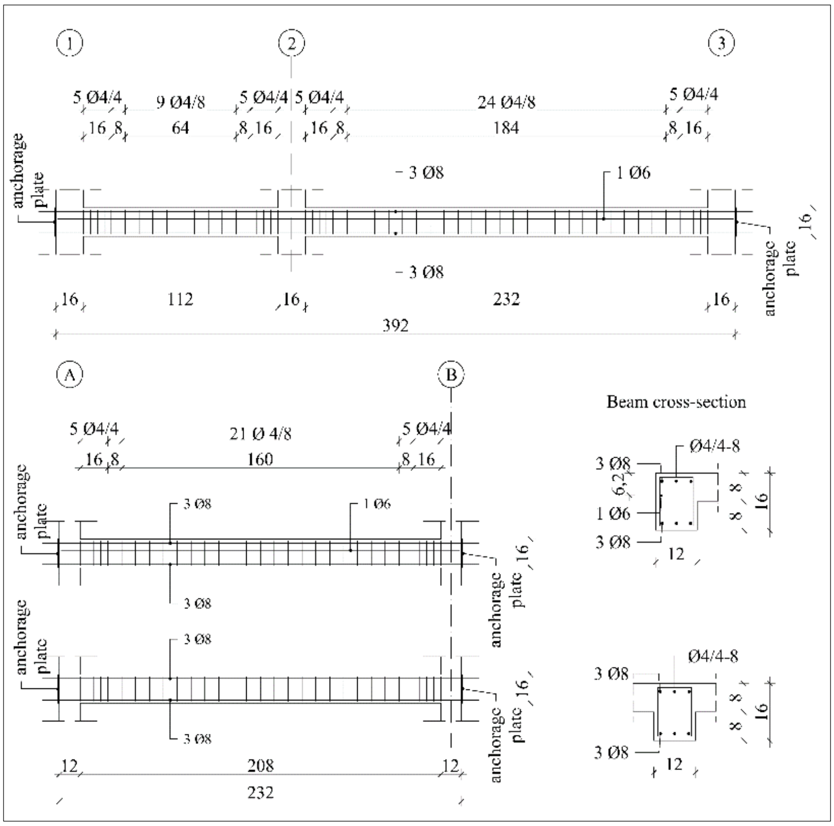


Figure 5. Beam reinforcement details (units in cm except for bar sizes)

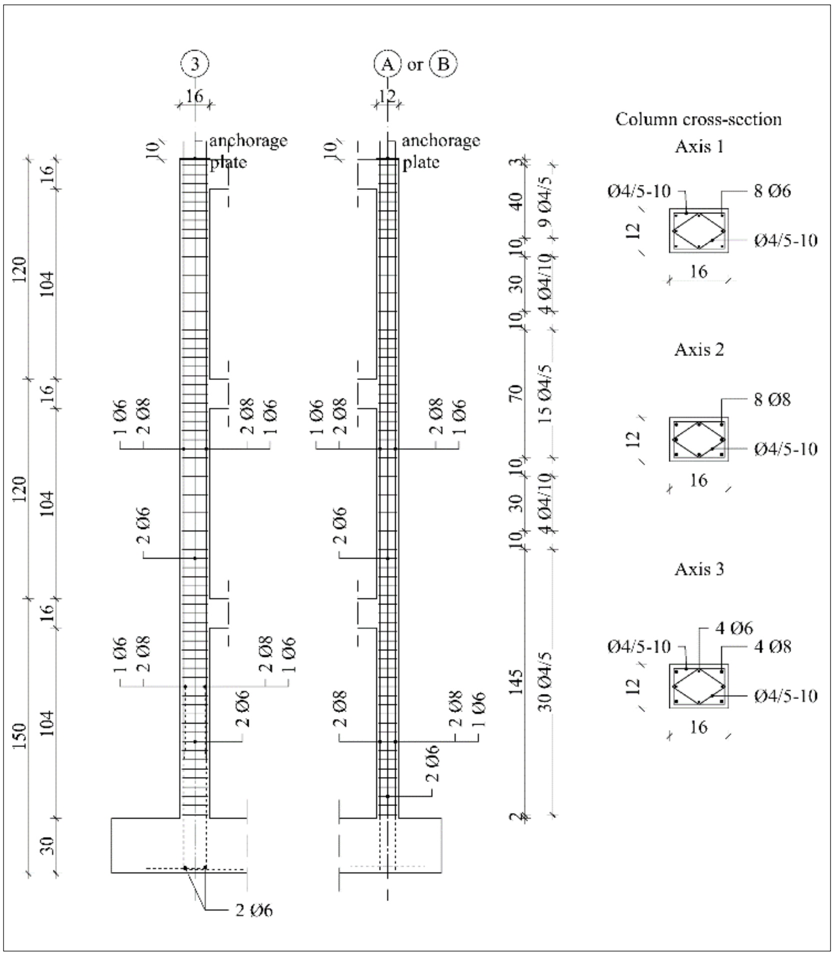


Figure 6. Column reinforcement details (units in cm except for bar sizes)

Table 1. Mean values of material mechanical properties

Property	Value		Units
Concrete cylinder strength	36.6		MPa
Secant modulus of elasticity of concrete	38840		MPa
Reinforcing steel yield / ultimate tensile strength	Ø 4 mm	753 / 780	MPa
	Ø 6 mm	564 / 589	MPa
	Ø 8 mm	591 / 621	MPa
Masonry Units & Mortar	Series 1*	Series 2*	
Masonry unit net compressive strength	31.2	20.0	MPa
Masonry mortar compressive strength	10.6	10.6	MPa

*Note: Series 1- clay block masonry, Series 2- Solid clay brick masonry

The test structure appearance, dimensions, cross-sections, and reinforcement plans for the beams and columns are given in Figures 1 and 3 to 6. The concrete was defined as class C25/30, with a nominal cylinder compressive strength of 25 MPa (nominal cube strength of 30 MPa) and a maximum aggregate size of 8 mm. The reinforcement was of grade B500B with a nominal yield stress of 500 MPa.

The north-south part of the record obtained at Herceg-Novi station during the 1979 M6.9 Montenegro Earthquake was used as input ground motion and applied in longitudinal, i.e. x direction of the building (see Figures 3 and 4). To comply with common similitude practices, the duration of the excitation was reduced by dividing the time step by $\sqrt{2.5}$. The excitation amplitude was scaled down to different peak ground accelerations (a_g / g). The base excitation was applied to the structure along its longitudinal (or x) direction in increasing intensity a_g / g: 0.05 g, 0.1 g, 0.2 g, 0.3 g, 0.4 g, 0.6 g, 0.7 g, 0.8 g, 1.0 g, 1.2 g (and additionally 1.4 g in the second series). Additional story masses were installed in steel ingots to increase the period of the specimen by doubling its mass and to comply with Cauchy-Froude similitude rules.

The test buildings experienced negligible-to-slight damage in the first story walls before 0.4 g. At 0.4 g in both test series, the separation of the first-story masonry infill wall and the RC frame was observed, with cracks around the perimeter of the infill walls with crushing at the corners. The wall with the door opening positioned in frame A of the

Series 1 building did not further participate in the seismic resistance, while the same in frame B did, as observed by the cracking of the wall. At 0.8 g in both test series, in the first and second story, the walls experienced moderate damage – the infill wall next to the door opening collapsed. The cracking was formed in both directions. The third story in both series had negligible damage during the testing. While also providing lower drift demands to the building of Series 2 compared to the ones of Series 1, the vertical confining elements that were added during the repair caused the equal distribution of earthquake damage and prevented the out-of-plane collapse of the walls. The main RC frame was not noticeably damaged in both test series, meaning the vertical structural capacity was not compromised. The shake table tests are described in detail in Guljaš et al. [55].

3. Reliability Analysis

The reliability analysis was performed for the whole masonry infilled RC frame building by considering ground story RC frames (both sides) with masonry infill walls (see Figure 3). This frame was found to be critical for earthquake behaviour and the cause of the accidental torsion occurrence (w.r.t. damage evolution of the building).

3.1. Earthquake Resistance Verification Model

According to Chandler and Hutchinson [8], the ultimate shear resistance of a single RC frame with a masonry infill wall to the horizontal shear (seismic) force V_{Ru} can be determined from the expressions given in Table 2.

Table 2. Expressions for determining the horizontal shear resistance of an RC frame with masonry infill wall [34]

Description	Expression	
Ultimate resistance of the RC Frame with masonry infill wall	$V_{Ru} = V_{Rf} + V_{Re}$	(1)
Ultimate resistance of the RC Frame without masonry infill wall	$V_{Rf} = \dfrac{3 \times M_R}{\left(h + \frac{h_b}{2}\right)}$	(2)
RC frame ultimate bending moment	$M_R = A_s \times f_y \times \left(\frac{l_c}{2} - d_1\right) + f_{ck} \times t_c \times \left(\frac{l_c}{2} - \frac{x}{2}\right)$	(3)
The effective width of the compression area of a column cross-section	$x = \dfrac{N_f + A_s \times f_y}{f_{ck} \times t_c}$	(4)
Ultimate yield force of a masonry infill wall under simultaneous horizontal and vertical loading	$V_{Re} = C_R \dfrac{A_m \times f_t}{C_I \times b}\left\{1 + \sqrt{C_I^2\, x\, \left(1 + \frac{\sigma_d}{f_t}\right) + 1}\right\}$	(5)
Coefficient of the frame-infill interaction; Coefficient of aspect ratio	$C_I = 2 \times \alpha \times b \times \dfrac{l}{h}$; $\alpha = \dfrac{(x_1 - x_2) \times h}{y_1 \times l}$	(6,7)
x_1, x_2 (see Figure 6)	$x_1 = \dfrac{l - w}{6\, sin\rho}$; $x_2 = \dfrac{w}{6\, sin\rho}$	(8,9)
y_1, w (see Figure 6)	$y_1 = \dfrac{h - w}{6cos\rho}$; $w = \dfrac{1}{4}\sqrt{l^2 + h^2}$	(10,11)
Vertical stress in the wall	$\sigma_d = a \times \dfrac{N}{A_m}$	(12)

Note: h_b is total beam height above the infill wall; A_s is the area of the reinforcement in a column; f_y is characteristic reinforcement yield strength; f_{ck} is characteristic concrete compressive strength; l_c is column width/depth parallel to wall direction; t_c is column width/depth perpendicular to wall direction; d_1 is the distance between the centroid of the tensile reinforcement and the closer cross-section's edge; N_f is the vertical force in the column; C_R is coefficient of the quality of masonry workmanship with the recommended value of 0.9; b is the shear strength ratio, i.e. b=1.1 for walls without opening and b=1.5 for walls with openings [8]; a is coefficient of the transmission of the vertical load to walls, i.e. a=0.3 for undamaged masonry and a=0 for damaged masonry; N is vertical force taken by the masonry; A_m is the area of the horizontal wall cross-section (A_m=$t.l$ see Figure 7).

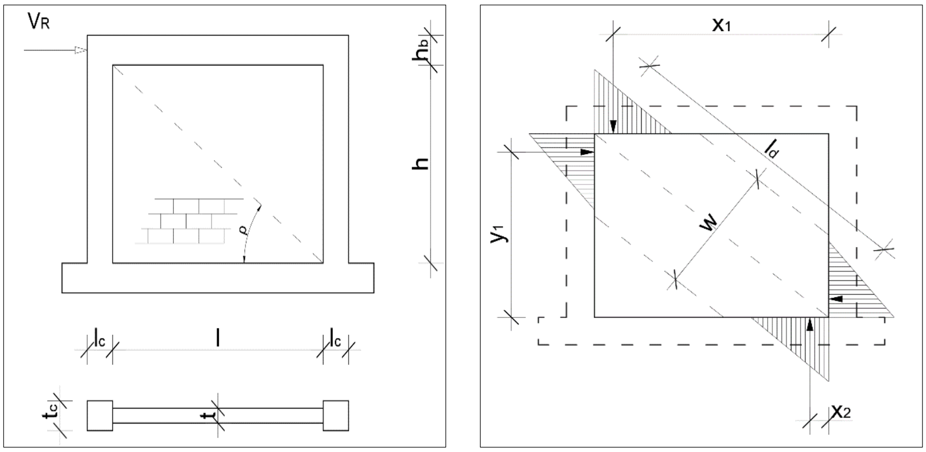


Figure 7. (a) RC frame with a masonry infill wall scheme; (b) idealised compressive stresses in the wall corners (ends of the diagonal strut) [56]

3.2. Observed Actions

Given that the structure's resistance to the shear force is to be verified, the relevant action is the horizontal force. The most important part of that same force is caused by (earthquake) acceleration on the structure's mass.

$$F_b = m \times a \tag{13}$$

where F_b is the total earthquake force, m is the mass, and a is the ground acceleration caused by the earthquake. Considering that due to the imperfection of the construction, the non-uniformity of the material properties, in some cases of non-symmetrical construction, etc., the centre of mass and the centre of rigidity do not match. As a result, additional forces appear in the structural members caused by balancing the moment that arises due to the action of the earthquake forces on the eccentricity arm.

$$F_{b,e} = \frac{F_b \times e}{X} \tag{14}$$

where X is the distance between the elements of the first frame and the elements of the second frame.

3.3. The Capacity Equation in General Form

To calculate the influence factor and the reliability index, the ultimate limit state equation was used in its general form

$$Z=R-E \tag{15}$$

where Z is the safety zone, R is resistance (section 3.1), and E is the effect of action (see section 3.2).

The final equation for the limit state, i.e., capacity, was obtained when its outcome reached a constant value.

3.4. Random Variables

Random variables defined through the relevant expressions are listed in Table 3, where the symbol m is used for the mean value and σ for the standard deviation. The calculation of the reliability index β and the influence factor α using random variables was made with the help of the Comrel 8.1 Symbolic computer programme (https://comrel-symbolic.software.informer.com/8.1/).

Table 3. The list of random variables [57-59]

Variable symbol	Variable description	*m*	*σ*	Units	Distribution type
A_{s1}	see Table 2 and Figure 6	85	1.7	mm^2	normal
f_y		500	30.0	N/mm^2	lognormal
l_c		160	6.4	mm	normal
d_1		19	1.7	mm	normal
f_{ck}		25	1.8	N/mm^2	lognormal
t_c		120	3.6	mm	normal
h		1040	31.2	mm	normal
h_b		160	4.8	mm	normal
l_1	wall length in frame 1	1120	44.8	mm	normal
l_2	wall length in frame 2	2320	92.8	mm	normal
t	wall thickness	120	4.8	mm	normal
f_t	wall tensile strength	0.23	0.01	N/mm^2	lognormal
h_z	wall height	1040	41.6	mm	normal
m	mass	29.200	876.0	kg	normal
e	eccentricity	1052.4	63.1	mm	normal
X	structure width	2200	88	mm	normal
$C_{I,1}^2$	coefficient of the frame-infill interaction 1	0.64	0.03	mm	normal
$C_{I,2}^2$	coefficient of the frame-infill interaction 2	8.70	0.35	mm	normal

When defining variable e, it is necessary to consider several variables influencing it. The variable e refers to the eccentricity that occurs due to certain elements' non-uniform stiffness and the distance between the centre of mass and the centre of rigidity. The position of the mass in the direction perpendicular to the direction of movement of the shaking table in the observed case does not change with time, making it a constant.

From the above, it can be concluded that the eccentricity e depends directly on the variation of stiffness of the RC frame (with masonry infill wall), so the standard deviations of the variable are determined concerning the stiffness. The mean value of the variable e was taken from the Series 1 tests at a load of 0.8 g (see Section 2) when it reached an average value of 35 cm per floor. The stiffness of the RC frame (with masonry infill wall) before deformation (see [56]) is described by the following expression:

$$K_i = \frac{1}{\frac{h^3}{3EI_e} + \frac{1,2h}{G_i A_e}} \tag{16}$$

The Equation 16 was broken down and analysed in Comrel 8.1 Symbolic.

The coefficient of variation for the independent variable e is obtained from the coefficients of variation of the wall cross-sectional area, and concrete shear modulus and wall shear modulus were multiplied by the influence factor ratios - the same is shown in Table 4.

Table 4. Influence factor for the eccentricity e

Random variable	Influence factor α
h	0.00
E	0.00
I	0.00
E_f	0.00
I_r	0.00
A_f	0.05
I_c	0.00
I	0.00
G_i	-0.24
A_m	0.94
G_f	0.24
$\Sigma\alpha^2$	1.00

The CV coefficient is obtained from the results of the variables that have the most significant influence on it: wall cross-sectional area A_m - CV=0.06; α_{Am}=0.94; concrete shear modulus G_f - CV=0.06; α_{Gf}=0.24 and wall shear modulus G_i - CV=0.16; α_{Gi}=0.24. Given that the dominant factor of influence is the cross-sectional area of the wall, the eccentricity distribution is taken to be equal to that of the cross-sectional area of the wall, i.e., the normal distribution. After the calculation, the obtained coefficient of variation for variable e is 0.064.

Due to the extensiveness and a considerable number of non-deterministic variables and limitations of the Comrel 8.1 Symbolic software package, the original limit state equation had to be simplified. The reduction of the subject equation was achieved through two steps: separate calculation for $C_{I,1}^2$ and $C_{I,2}^2$ and determination of corresponding statistical parameters, respectively. For the random variables $C_{I,1}^2$ and $C_{I,2}^2$, the influence of the random variables of which it consisted (h_z and l) was calculated.

Figure 8 shows the influence factors of the independent variables h_z and l within the C_I^2 coefficients.

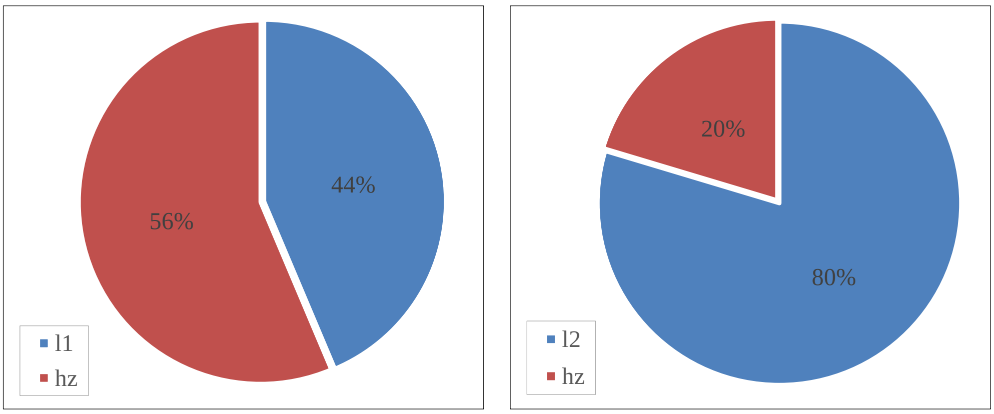


Figure 8. Influence factors for (a) $C_{I,1}^2$; (b) $C_{I,2}^2$

3.5. The Ultimate Limit State Equation

As the acceleration is directly related to the earthquake and it is difficult to determine the statistical parameters unambiguously, here, the acceleration of the earthquake is taken as a constant of 8 m/s^2 (0.8 g). Namely, at that level of horizontal acceleration, the first severe damage was observed on the infill wall (see section 2). The influence of accidental torsion was also noticed during the analysis of the test records.

The ultimate limit state equation is obtained when Equations 1 and the sum of Equations 13 and 14 are equalised. The equations are written until they are reduced to the independent variables listed in section 3.3. The gravitational acceleration, the masonry quality coefficient and the shear strength ratio parameter in the wall are introduced into the equation as constants. The gravitational acceleration g is taken as 9.81 m/s^2.

In the case of high masonry quality, the coefficient of masonry quality strives for unity. Its practical value is between 0.5 and 1.0, and the suggested value of 0.9 was taken [56]. For infill walls without openings, the value of the shear strength ratio parameter b is 1.1, while for walls with openings, b=1.5 [56]. N_1 and N_2, the vertical forces in the wall, were equal to zero since the wall serves as an infill and carries only its weight.

4. Results

Equation 16 was used to calculate the stiffness of frames 1 and 2 (see Figure2). It was found that frame 1 (see Figure 2) participated with 63% of the total stiffness, while frame 2 participated in the total stiffness with 37%. Since there is an opening in frame 2, its stiffness is reduced using the diagram in Figure 9 [60] - stiffness reduction coefficient of λ=0.4 (the area of the opening w. r. t. the area of the wall is 17%).

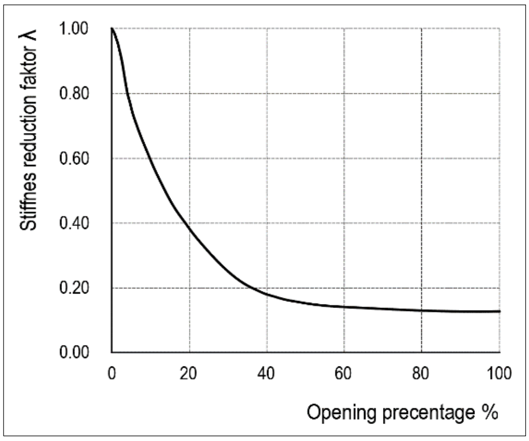
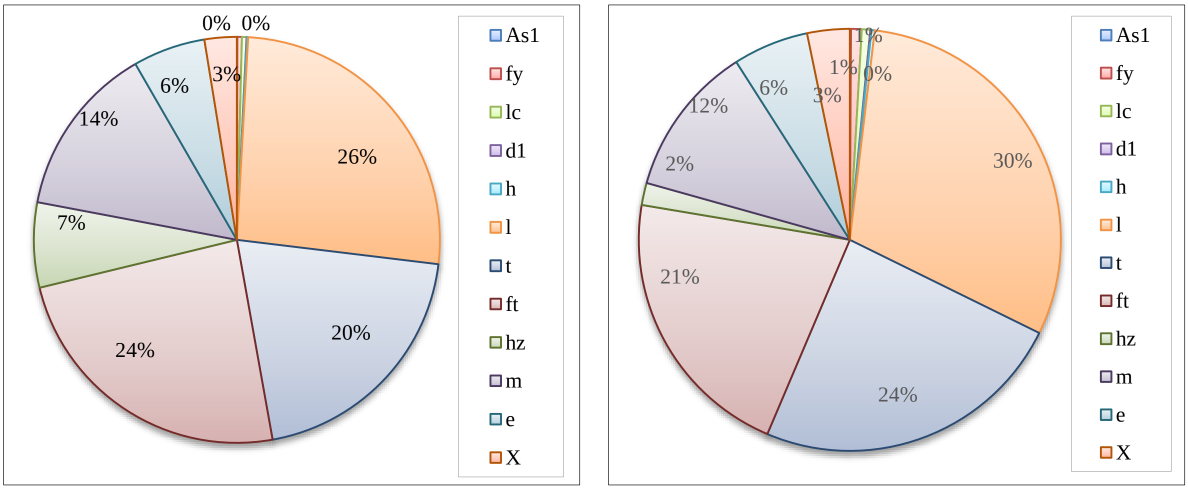


Figure 9. Influence factors for (a) $C_{I,1}^2$; (b) $C_{I,2}^2$

4.1. Reliability Index and Influence Factor for Frame 1

Reliability indices were calculated using the Comrel 8.1 Symbolic program, and the graphical representation of the obtained influence factors is shown in Figure10-a.



(a) (b)

Figure 10. Influence factors for (a) frame 1 (“shorter frame”); (b) frame 2 (“longer frame”)

In the case of frame 1, the obtained reliability index is β=-0.339. Given that the size of $C_{I,1}$ depends exclusively on l_1 and h_z parameters (see Table 2), in further comparisons of the influence factors, the quantity that corresponded to the influence factor of the random variable $C_{I,1}$ in the overall equation was divided into l_1 and h_z:

$$\alpha(l_1) = 0.66;\ \ \alpha^2(l_1) = 0.44 \tag{17}$$

$$\alpha(h_z) = -0.75;\ \ \alpha^2(h_z) = 0.56 \tag{18}$$

From the overall equation:

$$\alpha(C_{I,1}) = -0.30;\ \ \alpha^2(C_{I,1}) = 0.09 \tag{19}$$

The final values of the random variables l_1 and h_z were determined by adding up the influence factors from the basic equation and the derived equation, especially for $C_{I,1}$, taking into account the sign and the sum of squares equal to 1.

4.2. Reliability Index and Influence Factor for frame 2

The reliability index was calculated in the computer program Comrel 8.1 Symbolic. The graphical presentation of the results can be seen in Figure 10-b. The reliability index for the given frame is β=6.398 (mainly related to the masonry infill wall).

4.3. Overview of Influence Factors

Table 5 shows a tabular presentation of the influence factors of all random variables for both frames, with an indication of influence.

Table 5. Overview of influence factors

Variable (see Table 2)	Frame 1	Frame 2	Influence on
As1	0.02	0.03	r. c. frame
fy	0.06	0.09	r. c. frame
lc	0.06	0.08	r. c. frame
d1	-0.02	-0.03	r. c. frame
h	-0.03	-0.05	r. c. frame
l	0.51	0.55	masonry infill wall
t	0.45	0.49	masonry infill wall
ft	0.49	0.46	masonry infill wall
hz	-0.26	-0.13	masonry infill wall
m	-0.37	-0.34	action on structure
e	-0.24	-0.24	action on structure
X	0.16	0.18	structure

5. Discussion

From Figures 10-a and 10-b, it is evident that, of those related to the structure's resistance, the most significant random variables are l_2, the wall length in frame 2, t the wall thickness, f_t the wall tensile strength, and m mass of the structure. It is important to note that the first three listed variables refer to the masonry infill wall (earthquake resistance of the structure), and the fourth is related to the structure's mass and refers to the earthquake action on the structure.

The independent variables that are related to the structural capacity of the RC frame structure have little influence on the limit state equation – the probability of failure. As for the variables related to actions, it is essential to note that the random variables m and e have a significant influence. Acceleration does not induce variability as it is taken as a constant (well-defined in building codes), making the influence of accidental torsion high. According to the presented estimation of the influence factor, the influence of eccentricity is significant and cannot be disregarded. This needs to be considered with the fact that the test building was designed so that the frame structure can take horizontal shear (seismic) loads by itself. Even though it is a regular symmetrical building (along the axis of the ground motion direction), and the earthquake simulation was performed in one direction, excluding torsional effects caused by the ground, eccentricity is a crucial variable. The appearance of eccentricity is caused by stiffness changes in the masonry infill wall resulting from its damage. The damage is often unsymmetrical and hard to predict, even with symmetrical walls, due to imperfections in erection, material properties variation, and geometry imperfections. Even though the masonry infill wall has the role of a partition member, it has considerable stiffness when horizontal shear (seismic) forces appear, making it an essential part of the response to seismic loading.

With the increase in the influence of random variables m and e, the reliability index β decreases. This was to be expected as these variables drive up the variability to which the probability of failure is sensitive. Nevertheless, it is important to note that the reliability index of $\beta = -0.339$ for frame 1 corresponds to the observations from the tests, which showed that the first severe damage occurred at a horizontal load of 0.8 g. In contrast, the reliability index $\beta = 6.398$ for frame 2 indicates sufficient reliability in the observed situation. This distribution of reliability indexes can be attributed to different geometry and stiffness participation in the response, especially as the wall length plays a considerable role in its influence on the limit state equation. It should be noted that the reliability indices were considered as if each frame were operated separately and that the total reliability index would be between those values if the construction were viewed as a whole.

Accidental eccentricity occurs even in regular symmetrical buildings with actions and footing that do not lead to torsional effects. The results indicate that it is necessary to determine how to include the influence of accidental torsion due to stiffness changes of partition elements (structural masonry infill wall, in this case). When determining the influence of individual earthquake-resistant design parameters, it is also necessary to consider the properties of the partition itself (size, type, and position of openings [12, 13], intentional structural weaknesses [14, 16, 61]). Especially where it will cause an early failure even under smaller drifts (or forces), leading to the removal of the wall from the building's earthquake-resistant system and the possible torsion appearance. The shake-table tests performed on a large-scale, three-story RC frame building with URM infill walls, regular in plan and elevation indicated the appearance of "accidental" torsion due to the unpredictable behaviour of these walls. The building described in this work is analysed based on four ground-story frames, so it is crucial to conduct further tests on structures having more structural members. In addition to increasing the load-bearing capacity of the structural members, research can also go in the direction of how to exclude the partition elements from the earthquake resistance system and prevent them from falling out when horizontal forces occur perpendicular to their plane.

The presence of confining elements along the opening edges (Figure 4) seems beneficial for the earthquake behaviour of the building in terms of damage control and reliability w. r. t. building's design and prevention of infill wall out-of-plane failure ("walking out" collapse under in-plane action or direct out-of-plane collapse) [17-19].

6. Conclusion

Accidental torsional behaviour is still the subject of many research papers due to its importance during seismic loading. It is a significant source of damage, difficult to predict as many variables govern it. Evaluation and understanding of the influence of accidental torsion are fundamental in seismic reliability estimation. This study offers vital insights based on the results of a 1/2.5 scale three-story masonry-infilled RC frame building's test on a shaking table. The accidental eccentricity resulted from unsymmetrical masonry infill wall damage in a regular building. Weight factors revealed that the accidental eccentricity, compared to other random variables considered, could reach values higher than those prescribed by the building codes and cannot be disregarded. This unacceptable seismic reliability clearly warns that accidental torsion of masonry-infilled reinforced concrete frames in in-situ conditions must be considered even in regular buildings. The appearance of eccentricity is caused by stiffness changes in the masonry infill wall resulting from its damage. The damage is often unsymmetrical and hard to predict, even with symmetrical walls, due to imperfections in erection, material properties variation, and geometry imperfections. Even though the masonry infill wall has the role of a partition member, it has considerable stiffness when horizontal shear (seismic) forces appear, making it an essential part of the response to seismic loading. The results offer a clear pathway to understanding the changes in the reliability index based on the example provided. However, they are also limited regarding the number of structural members and specific ratios of strength and stiffness. This presents opportunities for future research, along with a similar study on such infilled steel systems, as they are fundamentally different in terms of ductility and the limitations of that ductility in the infill system.

7. Declarations

7.1. Author Contributions

Conceptualisation, D.B., D.M., T.D., and D.P.; methodology, D.B., D.M., and T.D.; software, D.B., D.M., and T.D.; validation, D.B., D.M., and T.D.; formal analysis, D.B., D.M., and T.D.; investigation, D.B., D.M., and T.D.; resources, D.B., D.M., T.D., and D.P.; data curation, D.B., D.M., T.D., and D.P.; writing—original draft preparation, D.B., D.M., T.D., and D.P.; writing—review and editing, D.M., T.D., and D.P.; visualization, D.B., D.M., T.D., and D.P.; supervision, D.B., D.M., T.D., and D.P.; project administration, D.P. and D.M.; funding acquisition, D.P. and D.M. All authors have read and agreed to the published version of the manuscript.

7.2. Data Availability Statement

Data sharing is not applicable to this article.

7.3. Funding

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7.4. Conflicts of Interest

The authors declare no conflict of interest.

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