

Review

Historical Evolution of the Impact of Seismic Incident Angles on the Safety Assessment of Various Building Construction Typologies

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Abstract: In the existing building stock, typically characterised by a high degree of irregularity, the effects of earthquakes are strongly dependent on the epicentre–structure direction and the angle of incidence of the seismic motion. However, the scientific community has not yet reached a unanimous consensus on the evaluation of the effects of seismic incidence angles. Therefore, this paper conducts an extensive investigation of the international literature on current methods to consider seismic directionality, systematically reviewing more than 80 publications on this topic. Following a brief overview of the problem and an analysis of the initial developments of the multidirectionality concept of seismic input, a state-of-the-art review is presented based on the considered analysis methods, specifically response spectrum analysis, nonlinear static analysis, and nonlinear response history analysis. Moreover, the adoption of multidirectional seismic input in popular codes and standards is presented and discussed. This study provides the first comprehensive synthesis of research on the seismic incidence angles across diverse building typologies, offering crucial insights for future code revisions and highlighting significant gaps in current analytical methods and standards, thereby setting a new direction for subsequent empirical investigations. Specifically, the extensive state-of-the-art review revealed that, until now, the evaluation of the angle of incidence was primarily conducted on existing reinforced concrete buildings with a limited number of storeys, analysed with nonlinear response history analysis. This underscores the need for future research to extensively investigate the impact of the angle of incidence on other types of construction typologies.

Keywords: building engineering; seismic angle of incidence; multidirectionality of seismic action; building construction typologies; state-of-the-art review; existing building stock



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1. Introduction

In most cases, when carrying out the seismic analysis of a three-dimensional structural model, the angle of seismic incidence (ASI), i.e., the direction along which to apply the seismic motion, is unknown. Over the past decades, this issue has received significant attention from the research community, which demonstrated the importance of this angle on the structural demand. However, despite copious research and achieved improvements, there is still a lack of consensus about the need to take into account the ASI in the structural analysis.

Since the actual ground motions have a three-dimensional nature with two components in the horizontal plane and one vertical component, the seismic response of ordinary structures not affected by the vertical component is highly dependent on the horizontal components of ground motion. Since the structure may have the highest demand along a direction that does not necessarily align with the reference axes, exploring the ASI of orthogonal components of the earthquake is crucial to ensure the structural safety of the building.

The angle of seismic incidence (ASI) plays a crucial role in accurately assessing the seismic response of existing buildings. However, the abundance of research on this topic necessitates a comprehensive synthesis of past findings. This paper aims to synthesise past and recent research on the impact of ASI on existing building stock, fostering a common understanding among researchers and practitioners regarding seismic directionality effects. The work begins with a discussion on the development and definition of the ASI. Then, a state-of-the-art review of prior studies that sought to evaluate the effect of the ASI is presented. To clarify the key findings, these studies have been categorised based on the adopted analysis method: response spectrum analysis (RSA), linear response history analysis (LRHA), nonlinear static analysis (NSA), and nonlinear response history analysis (NRHA). The paper continues with a survey of the adoption of the ASI by different building codes, standards, and guidelines. Finally, conclusions, recommendations, knowledge gaps, and suggestions for future research are presented.

2. Definitions and First Developments

2.1. Definitions

Since the beginning of seismic engineering studies, the ASI at which an engineering demand parameter (EDP) attains its maximum value (i.e., the critical angle) and the corresponding EDP (i.e., the critical response) were recognised as fundamental issues in the seismic analysis of structures. The generic response R in a point of a structure, intended as any component of structural stress, deformation, or displacement, varies according to the ASI, which is determined by the earthquake's epicentre and the building's structural orientation. The problem can be defined considering the reference system of the seismic input (that includes both the position of the epicentre and the recording station of the accelerograms) and the reference system of the structure, each oriented in a different way. The components of the seismic input (a_r, a_s, a_t) are provided by accelerograms recorded according to three orthogonal directions (r, s, t), which are generally different from those providing the maximum structural response.

In the literature, the discussion about the critical angle of seismic incidence starts with Penzien and Watabe [1], who observed that, since the position of the epicentre of the earthquake is not predictable, the direction along which the earthquake propagates with respect to the orientation of the structure is unknown. According to their work, the recorded components can be rotated to the principal directions along which the correlation between the three components is null, using orthogonal transformations identical in form to those used in the transformation of stress. These directions correspond to the maximum, intermediate, and minimum quadratic intensities of the accelerations, among possible directions.

More specifically, the uncorrelated components of the ground motion records are obtained considering that the interdependence degree of the recorded seismic components can be measured using the matrix of quadratic intensity μ of the motion recorded [2] at a given station along axes (r, s, t):

$$\mu = \begin{bmatrix} \mu_{rr} & \mu_{rs} & \mu_{rt} \\ \mu_{sr} & \mu_{ss} & \mu_{st} \\ \mu_{tr} & \mu_{ts} & \mu_{tt} \end{bmatrix} \text{ with } \mu_{rr} = \frac{1}{2} \int_0^d a_i(t) a_j(t) dt; \quad i = r, s, t; \quad j = r, s, t \quad (1)$$

where d is the duration of the earthquake. Non-null values for cross-terms are an indication of the correlation between the recorded components. The components of the recorded accelerograms along directions (r, s, t) may be rotated along the principal directions of the earthquake (p, w, v) using the following transformation of coordinates:

$$\begin{Bmatrix} a_p \\ a_w \\ a_v \end{Bmatrix} = \Phi^T \begin{Bmatrix} a_r \\ a_s \\ a_t \end{Bmatrix} \quad (2)$$

where Φ is the matrix whose columns are the three eigenvectors of matrix μ . The new coordinate system, the quadratic intensity matrix μ_p , of the principal components (a_p, a_w, a_v) is given by

$$\mu_p = \begin{bmatrix} \mu_{rr} & 0 & 0 \\ 0 & \mu_{ss} & 0 \\ 0 & 0 & \mu_{tt} \end{bmatrix} \quad (3)$$

The cross-terms of matrix μ_p equal to zero indicate that the components (a_p, a_w, a_v) are uncorrelated to each other. Analysing different ground motion records, Penzien and Watabe [1] showed that principal directions are reasonably stable during the ground motion. In most cases, the major principal axes (p) connect the epicentre to the recording station (Figure 1), the intermediate principal axis (w) is orthogonal to the major principal axis (p) in its plane, and the minor principal axis (v) is vertical.

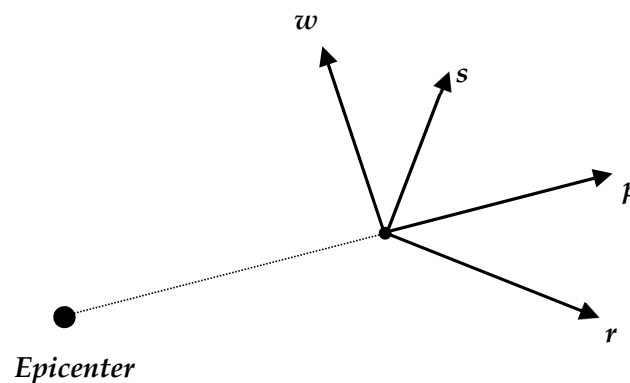


Figure 1. Recording directions (r, s) and principal directions (p, w) of the ground motion record in the horizontal plane.

The orientation of the principal axes (p, w, v) with respect to the reference system of the structure (x, y, z) defines the angle of incidence (θ), which is in general unknown and for this reason, it varies between 0 and 360 degrees (Figure 2). Note that the direction generating the maximum structural response R (i.e., the critical response) could act along a direction different both from (p, w, v) and from (x, y, z), along a direction between 0° and 360° [3].

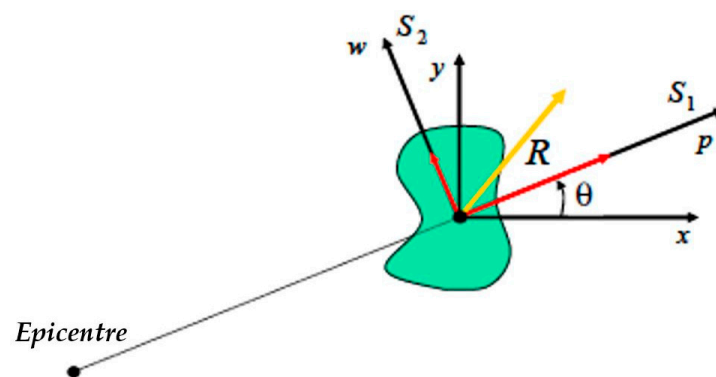


Figure 2. Principal directions (p, w), reference system of the structure (x, y), and generical structural response R .

Today, as in the past, the discussion about the directionality and multidirectionality of seismic input has always overlapped with those of the simultaneity of the action of the two horizontal seismic components. For this reason, before discussing the origin of the concept of multidirectionality, it is important to remember the following:

- The simultaneity concept considers that the three seismic components simultaneously act during the earthquake;
- The directionality considers that the main seismic component acts along the axis that connects the epicentre to the structure;
- According to the multidirectionality idea, the position of the epicentre of the design earthquake is not known a priori nor is the direction along which the main earthquake component could act, and for this reason, the structure should be analysed using different angles of seismic incidence.

The concepts of simultaneity, directionality, and multidirectionality were historically analysed mostly using RSA. More recently, these studies were extended to other types of seismic analyses, such as LRHA, NSA, and NRHA.

Combination rules were historically introduced to solve the problem of the simultaneous action of the seismic components. The introduction of these rules has focused attention on the variation of the seismic angle of incidence with respect to the seismic principal axes, opening the way to studies about the multidirectionality of the seismic input.

2.2. Combination Rules in Linear Analyses

In seismic linear analysis, the simultaneous action of the seismic components is addressed by combining the structural demands obtained from response spectrum analysis in the two principal horizontal directions (x and y). One of the most common approaches for combining structural demands is the Square-Root-of-the-Sum-of-the-Squares (SRSS) method, introduced by Newmark and Rosenblueth [4]. This method assumes uncorrelated seismic components and estimates the total response (r^2) by summing the squared responses (r_k^2) from each direction (k). The SRSS method finds its roots in the work of Goodman et al. [5], who applied it to combine modal contributions in response spectrum analysis, assuming well-separated modes. However, for closely spaced modes, Der Kiureghian [6] and Wilson et al. [7] proposed the Complete Quadratic Combination (CQC) rule. Other researchers, including Newmark [8] and Rosenblueth and Contreras [9], explored the application of multicomponent earthquake excitation in linear analysis. They proposed the Percentage Rule, which offers an approximation by combining the response from one direction with a scaled percentage (λ) of the responses from other directions. More specifically, Newmark [8] and Rosenblueth and Contreras [9] suggested λ to be 40% and 30%, respectively. More recently, Wilson et al. [10] concluded that the Percentage Rules are dependent on the user's selection of the reference system while the SRSS combination produces a seismic demand independent from the reference system. Moreover, they observe that the Percentage Rules could underestimate the design forces in certain members with respect to the SRSS combination.

Smeby and Der Kiureghian [11] proposed the CQC3 rule, an extension of the CQC rule used to combine modal responses to the three seismic components. Subsequently, Menun and Der Kiureghian [12–14] further applied CQC3 to determine the generical response quantity in the function of the seismic orientation angle θ and identify the most critical orientation of the ground motion components for each response quantity of interest, taking into account for the correlation between individual seismic components. Note that the CQC3 rule is a general approach that encompasses the more commonly used 30%, 40%, and SRSS rules as special cases [15,16]. Camata et al. [17] demonstrate that SRSS generally yields higher structural response values compared to the critical response obtained with CQC3. Additionally, the 30% rule can underestimate responses by up to 9% compared to SRSS, as shown in their research. For further details on the evolution of combination rules, refer to Wang et al. [18].

Current seismic codes address the combined effects of multiple seismic components in linear analysis requiring the use of either the SRSS rule or, alternatively, the 30% rule [19]. Some codes prioritise the 30% rule with SRSS as an alternative [20,21], while others solely allow the 30% rule [22–25].

2.3. Combination Rules in Nonlinear Static Procedures

While originally developed for linear analysis, current seismic codes extend the application of combination rules to PushOver (PO) analyses as well. For instance, EC8-Part 1 [19] requires the use of SRSS or the 30% rule when employing a spatial model within a Nonlinear Static Procedure (NSP). Equations (4) and (5) illustrate the formulations of these two rules, considering the signs of the structural responses.

$$R_{SRSS} = \sqrt{|E_{dx}^2| + |E_{dy}^2|} \quad (4)$$

$$R_{30\%} = \max\left\{\left(\pm E_{dx} \pm 0.3E_{dy}\right); \left(\pm 0.3E_{dx} \pm E_{dy}\right)\right\} \quad (5)$$

where E_{dx} and E_{dy} represent the forces and deformations due to the application of the target displacement in the x and y directions, respectively.

The main objective of these combination rules is to consider the simultaneous action of the seismic horizontal components in NSPs of 3D structures. Despite the large research on PO analysis of irregular 3D structures subjected to bidirectional ground motions (e.g., [26–30]), the application of combination rules in NSP as prescribed by EC8-Part 1 [19] remains currently under-investigated.

Although there are many studies in the literature on the application of pushover analysis on irregular 3D structures subjected to bidirectional ground motions (for example, [26–30]), only a few authors apply the combination rules to NSP according to the formulation of the EC8-Part 1 [19]. This is probably because, as indicated in Avramidis et al. [31], these rules have been analytically derived and numerically verified in the published scientific literature assuming linear structural behaviour. In fact, the SRSS spatial combination rule provides exact peak results by assuming that (a) the horizontal ground motion components have equal intensities and are applied along the angle of incidence of 0 or 90 degrees; (b) they are statistically uncorrelated; and (c) linear response spectrum analyses are used to derive peak structural responses along each direction of the seismic action [11,12,15]. The 30% combination rule is an empirical rule obtained by considering the same assumptions of the SRSS rule and by applying a percentage parameter (equal to 0.3) on the seismic effects in order to minimise the difference with the SRSS rule. Therefore, EC8-Part 1 [19] appears not to consider the derivation of the spatial combination rules, and their application in NSP to verify if they lead to conservative results has not yet been adequately investigated in the literature.

Several authors consider the combination rules in NSP without applying the superposition of effects in nonlinear analysis. Reyes and Chopra [32] use the SRSS combination rule to extend the modal pushover analysis (MPA) procedure to a three-dimensional analysis of buildings subjected to two horizontal components of ground motion, simultaneously. The modal pushover analysis (MPA) was developed to include the contributions of all “modes” of vibration that contribute significantly to the seismic demand [33]. In this procedure, first, the natural frequencies, ω_n , and modes, Φ_n , are calculated. Then, for the n^{th} mode, the pushover curve is developed in the X direction using a lateral load pattern proportional to Φ_n . Gravity loads are applied before the lateral forces. Each pushover curve is then converted into a bilinear curve and the target displacement due to the n^{th} “mode” alone is calculated using the selected NSP [34]. All these steps are then repeated for as many modes as required for sufficient accuracy. The total responses for the x -component of the ground motion are obtained by combining the displacement demands using the CQC rule, i.e., $r_x = (\sum_i \sum_n \rho_{in} r_{ix} r_{nx})^{1/2}$. The same procedure is repeated to calculate the response r_y due to the y -component. The total response r is calculated by combining the responses r_x and r_y with the SRSS combination rule, i.e., $r = (r_x + r_y)^{1/2}$.

Cimellaro et al. [35] proposed a PO analysis approach, termed Bidirectional Pushover Analysis (BPA), specifically for irregular 3D reinforced concrete buildings. This method addresses the combined effects of bidirectional ground motions, taking into account for

the torsional effects using the Extended N2 Method [36,37]. The BPA procedure includes several steps. The initial step involves an eigenvalue analysis of the 3D building model in each principal direction (x and y) to determine natural frequencies ω_n , vibration modes Φ , and modal participation factors Γ in the two orthogonal directions x and y . The next stage involves defining monotonic lateral load patterns proportional to the modal profiles with the highest mass participation ratios in their principal structural directions (x and y). The load patterns are then simultaneously applied to the 3D model using a force pattern equal to $X = m_i \Phi_i$ in the x direction and $Y = \gamma m_j \Phi_j$ in the y direction, where γ is a load factor whose value is greater than zero and less than unity, i.e., $0 < \gamma \leq 1$. The capacity curves are then transformed into equivalent single-degree-of-freedom (SDOF) systems, and the displacement demands in the X direction are obtained through their intersection with response spectra [38]. Subsequently, the characteristic nonlinear base shear-roof displacement curves at the centre of mass for the multi-degree-of-freedom (MDOF) system are obtained in both directions (x and y). To assess the accuracy of the proposed method, the research compares the median floor displacements and rotations obtained from BPA with those from NRHA. By applying BPA with various γ values (0.2 to 1.0), Cimellaro et al. [35] identify the optimal lateral load distribution ratio (γ) that minimises the sum of the differences between BPA and NRHA at each floor. According to their results, the optimal value of γ ranges between 0.6 and 1. The median rotations obtained at each floor using the NRHA are finally compared with the structural demands and with the BPA applied using $\gamma = 0.6$. BPA showed better results with respect to the Extended N2 Method, especially at the lower storeys. Based on their findings, Cimellaro et al. [35] suggest, therefore, a 100 + 60% combination rule for their BPA, deviating from the standard code-defined 100 + 30% rule. Finally, they correlate this alternative rule with the application of seismic forces along a specific incident angle of around 31° .

Cantagallo et al. [37] carried out two separate PO analyses, pushing the structures in the x and y directions and combining the results using the 100:30 directional combination rule, as indicated in EC8-Part 1 [19]. Their study demonstrates that the directional combination rule (and specifically the 100:30 rule) does not improve the prediction of the seismic response of the N2 method.

3. Evaluation of Directionality Effects Based on Analysis Method

This section describes the state of the art of the studies on the directionality of the seismic input, dividing them according to the type of analysis carried out in determining the critical response. This categorisation partly reflects the historical evolution of such works, and, therefore, was chosen in order to accurately narrate the development of the studies on the directionality effects. However, the numerous works analysed could be divided according to different parameters. To this purpose, Table 1 summarises the content of the analysed works, defining for each of them the type of analysis, number of storeys, structural material, structural model, and engineering demand parameter (EDP) of the case-study buildings.

Table 1. List of the works analysed by the authors with the description of the type of analysis, type of case study, structural model, and EDP used in each work.

Reference N.	Author(s)	Analysis Type	Case-Study Structure(s)	Structural Model(s)	EDP(s)
[39]	Wilson and Button (1982)	RSA	Three-storey steel building. Steel columns differently oriented.	Linear	Maximum stress
[11]	Smeby and Der Kiureghian (1985)	RSA	Two irregular-in-height buildings having a maximum number of storeys equal to one and four, respectively. Elements having pre-defined inertia.	Linear	Displacements

Table 1. Cont.

Reference N.	Author(s)	Analysis Type	Case-Study Structure(s)	Structural Model(s)	EDP(s)
[10]	Wilson et al. (1995)	RSA	Simplified one-storey building.	Linear	Local Moments
[40]	Lopez and Torres (1997)	RSA	One-storey reinforced concrete (RC) building.	Linear	Torsional moment, referred to as the centre of mass
[12]	Menun and Der Kiureghian (1998)	RSA	RC bridge.	Linear	Local Moments
[41]	Anastassiadis K. et al. (2002)	RSA	Six-storey RC building.	Linear	Stress result (or sectional forces)
[42]	Fernandez-Davila et al. (2000)	LRHA	Five-storey RC building.	Linear	Axial force in the columns $MRV = \frac{\max r - \min r}{\min r} \cdot 100$
[43]	Athanatopoulou A.M. (2005)	LRHA	Five-storey RC building.	Linear	Internal forces N (axial), Mx (bending moment) and Vx (shear force). Maximum displacement at a specific joint
[44]	Kostinakis et al. (2011)	LRHA	Three single-storey RC buildings.	Linear	Maximum normal stresses (axial forces and bending moments in two orthogonal directions)
[45]	Marinilli and Lopez (2008)	LRHA	One-storey RC building.	Linear	Axial forces
[46]	Alam et al. (2020)	LRHA	18-storey RC building.	Linear	Axial force N , bending moment M , shear force V , Inter-storey Drift Ratios IDRs
[47]	Kalkan and Kwong (2014)	LRHA	Six-storey RC building.	Linear	Axial force, bending moment, shear force, normalised first-storey drift
[48]	Cannizzaro et al. (2017)	NSA	Three-storey historic masonry building.	Macro-Element	Three-dimensional capacity dominium (PO curves for different ASIs). Ductility demand
[49,50]	Chácará et al. (2019)	NSA	1-storey brick specimen.	Macro-Element	Three-dimensional capacity dominium (PO curves for different ASIs)
[51]	Kalkbrenner et al. (2019)	NSA	Two-storey historic masonry building.	Shell Elements	Local and Global Displacements
[52]	Ghayoumian and Emami (2020)	NSA	4-, 8-, and 12-storey RC buildings.	Fiber-base model for columns and walls and concentrated plastic hinge model for beams	Inter-storey drift ratio (IDR), ductility and damage indices
[53]	Cantagallo et al. (2022)	NSA	Two one-storey and one five-storey RC buildings.	Force-based fibre section model for beams and columns	Displacement demand and shear demand
[54]	Cantagallo et al. (2023)	NSA	Two five-storey RC buildings.	Beam-with-Hinges elements with ends modelled with force-based fibre section models	Base Shear, Roof Displacement, and IDRs
[55]	MacRae and Mattheis (2000)	NRHA	3-storey steel structure.	Fibre hinge model for column and elastic model for beams	Drifts
[56]	Rigato and Medina (2007)	NRHA	2 RC one-storey structures with various degrees of inelasticity.	Plastic hinges with a bilinear hysteretic model for columns. No beams. Rigid diaphragms	Column displacement, ductility ratios, slab rotations and column drift ratios

Table 1. Cont.

Reference N.	Author(s)	Analysis Type	Case-Study Structure(s)	Structural Model(s)	EDP(s)
[3,57]	Lagaros (2010)	NRHA	Two six-storey and two three-storey RC (regular and irregular) buildings.	Force-based fibre elements. Nonlinear shear (V - γ) law	Maximum Inter-Storey Drift Ratio (MIDR)
[58]	Reyes and Kalkan (2015)	NRHA	30 single-storey symmetric and asymmetric single-storey steel buildings.	Buckling Restrained Braces (BRBs) with a simplified trilinear model	Displacements, floor accelerations, member forces and plastic deformations
[59]	Kalkan and Reyes (2015)	NRHA	Nine-storey steel building.	Trilinear plastic hinges at the ends of beams and columns. Four rigid links hinged at the corners with a rotational spring	Storey drifts, floor total accelerations, member chord rotations, and beam and column moments
[60]	Kostinakis, K. G. et al. (2013)	NRHA	3D single-storey RC buildings.	Plastic hinges both for columns (with PMM interaction) and beams	Park and Ang damage index for elements
[61]	Magliulo G. et al. (2014)	NRHA	One four-storey and two five-storey RC buildings	Beams and columns with lumped plasticity model	Top displacements, ratio between demand rotation and capacity rotation
[62]	Cantagallo et al. (2015)	NRHA	Two one-storey, one two-storey, and one three-storey symmetric and asymmetric RC structures.	Force-based fibre elements	MIDR
[63]	Emami and Halabian (2015)	NRHA	4-, 8-, and 12-storey RC moment-frame archetypic structures.	Fibre model for columns and lumped plasticity model for beams	Roof drift index, normalised inter-storey, storey ductility demands, storey damage indices
[64]	Kostinakis et al. (2015)	NRHA	Four double-symmetric and four asymmetric in-plan five-storey RC buildings, with and without structural walls.	Plastic hinges, which are located at the column (PMM interaction diagram) and beam	Park and Ang Damage Index
[65]	Fontara et al. (2015)	NRHA	Single-storey RC asymmetric building.	Not indicated	Park and Ang Damage Index
[66]	Sun et al. (2016)	NRHA	3-storey RC school building with upper space truss.	Fibre elements for beams and columns, link for upper space truss, shells for slabs	Stresses in the elements
[67]	Amarloo and Emami (2019)	NRHA	4-, 8-, and 12-storey RC moment-frame structures with typical L-shaped plans.	Lumped plastic hinges	Drift, ductility and damage indices
[68]	Kostinakis et al. (2018)	NRHA	Two 2-storey RC buildings with equal and unequal stiffness in the two orthogonal directions.	Plastic hinges, which are located at the column (P-M m interaction diagram) and beam	Park and Ang Damage Index (DI) modified by Kunnath et al.
[69]	Giannopoulos and Vamvatsikos (2018)	NRHA	SDOF system and 6-storey steel moment-resisting frame building with an L-shaped plan.	Elastic-perfectly plastic model with no cyclic degradation.	MIDR and Maximum Peak Floor Acceleration (MPFA)
[70]	Pavel and Nica (2019)	NRHA	6-, 8-, and 10-storey doubly symmetrical RC wall structures.	Nonlinear flexural hinges in beams with trilinear hysteretic models, multi-spring (MS) model with nonlinear axial springs at both ends and bidirectional nonlinear shear springs in the middle for columns and walls	IDR, maximum displacement at the top of the building, and maximum shear force at the base of the structural walls.

Table 1. Cont.

Reference N.	Author(s)	Analysis Type	Case-Study Structure(s)	Structural Model(s)	EDP(s)
[71]	Skoulidou et al. (2019)	NRHA	Six RC buildings (3-, 4-, and 5-storey) with infilled frame systems with and without in-plan irregularities	Lumped plastic hinges for beam elements, truss for infills	Collapse Fragility curves
[72]	Skoulidou et al. (2020)	NRHA	Six RC buildings (3-, 4-, and 5-storey) with infilled frame systems with and without in-plan irregularities	Lumped plastic hinges, truss for infills	MIDR, MPFA, and maximum roof drift
[73]	Bugueño et al. (2022)	NRHA	Four 5-storey RC buildings	Nonlinear fibre elements	Inter-storey drift and the roof displacement

3.1. Response Spectrum Analysis

Response Spectrum Analysis (RSA) is the most widely used method for seismic analyses and the design of buildings. The first applications of the concepts of multicomponent seismic input and ASI were developed for this type of analysis in the early 80s. In this section, an accurate overview of the applications of the multidirectional seismic input using RSA is described.

Wilson and Button [39] indicated that “a structure must resist a major earthquake motion of magnitude S_1 for all possible angles θ and, at the same point in time, resist earthquake motions of magnitude S_2 , at 90° to the angle θ ” (Figure 3).

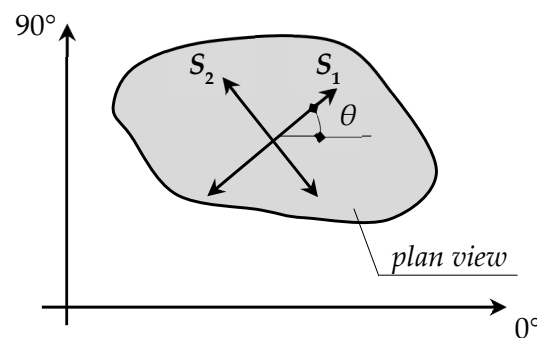


Figure 3. Earthquake spectra input according to [39].

For this purpose, they evaluated the resulting internal force f_1 due to S_1 applied at an angle θ , considering the contributions of the force f_0 due to S_1 applied at an angle of 0° and the force f_{90} due to S_1 applied at an angle of 90° .

$$f_1 = \pm \cos \theta f_0 \pm \sin \theta f_{90} \quad (6)$$

Then, they evaluated the resulting internal force f_2 due to S_2 (applied at an angle $\theta + 90^\circ$), assuming that the minor input spectrum is some fraction of the major input spectrum $S_2 = \alpha S_1$:

$$f_2 = \pm \alpha \sin \theta f_0 \pm \alpha \cos \theta f_{90} \quad (7)$$

In the above equations, the $\pm$ signs are included because the terms f_0 and f_{90} have lost their signs in the modal combination calculations. The authors calculated the critical angle θ_{cr} , i.e., the angle for which these forces are maxima, applying the formulation $\partial f / \partial \theta = 0$ to the two above equations, where all + signs were considered:

$$\tan(2\theta_{cr}) = \frac{2f_0f_{90}(1 + \alpha^2)}{(f_0^2 - f_{90}^2)(1 - \alpha^2)} \quad (8)$$

Wilson et al. [10], in an effort to demonstrate that the 100–30% and 100–40% combination rules were dependent on the user's selection of the reference system, determined the critical angle with the same procedure used in [39]. The authors provided a closed-form solution to determine the critical angle of response for an elastic structure and for a seismic load represented by two orthogonal and statistically independent ground motion spectra, S_1 and S_2 , applied at an arbitrary angle θ , where $S_1 \neq S_2$. Based on these assumptions, they proposed the following formula for the calculation of the critical angle:

$$\tan(2\theta_{cr}) = \frac{\pm 2f_0 f_{90}}{f_0^2 - f_{90}^2} \quad (9)$$

Equations (8) and (9) differ in the term α related to the ratio between S_2 and S_1 . In Equation (9), θ_{cr} is, therefore, independent from the spectral ratio.

Smeby and Der Kiureghian [11] demonstrated that the formulation of θ_{cr} obtained from [39] had certain limitations as it did not account for cross-terms between the input components. Starting from the CQC (Complete Quadratic Combination) modal combination rule, they formulated the following relation for the critical angle θ_{cr} :

$$\theta_{cr} = -\frac{1}{2} \tan^{-1} \frac{2 \sum_i \sum_j \rho_{0,ij} \Psi_i^{(1)} \Psi_j^{(2)} \bar{S}_i^{(1)} \bar{S}_j^{(1)}}{\sum_i \sum_j \rho_{0,ij} [\Psi_i^{(1)} \Psi_j^{(1)} - \Psi_i^{(2)} \Psi_j^{(2)}] \bar{S}_i^{(1)} \bar{S}_j^{(1)}} \quad (10)$$

where $\Psi_i^{(1)}$ represents the effective participation factor associated with mode i and input component (1); $\bar{S}_i^{(1)}$ is the mean response spectrum for the principal direction (1), representing the mean maximum response of an oscillator of frequency ω_i and damping ζ_i ; and $\rho_{0,ij}$ is the correlation coefficient between responses in modes i and j . Similarly, $\Psi_j^{(1)}$ and $\bar{S}_j^{(1)}$ are associated with the mode j and $\Psi_i^{(2)}$, and $\Psi_j^{(2)}$ to the response spectrum applied along the principal direction (2).

Menun and Der Kiureghian [12] determined θ_{cr} and the corresponding maximum response with the CQC3 rule. The authors first assumed that the shapes of the two horizontal directions were proportional with the following formulation:

$$S_{2i} = \alpha S_{1i} \quad (11)$$

where $0 \leq \alpha \leq 1$ is a constant and S_{1i} and S_{2i} are the spectral ordinates for mode i , oriented along axes 1 and 2, respectively. Therefore, they derived the total structural response in function of the orientation of the seismic input:

$$R = \left[(R_1^2 + R_2^2 + R_3^2) - (1 - \alpha^2) \left(R_1^2 - \frac{1}{\alpha^2} R_2^2 \right) \sin^2 \theta + 2 \left(\frac{1 - \alpha^2}{\alpha} \right) R_{12} \sin \theta \cos \theta \right]^{1/2} \quad (12)$$

where R_1 , R_2 , and R_3 represent the contribution to the response of the structure from the ground motion components 1, 2, and 3, and R_{12} represents a cross-term between the contributions to the response from the ground motion components 1 and 2. Equation (12) contains three distinct terms. The first term represents the response if the principal directions of the ground motion coincides with the structure axes, i.e., if $\theta = 0$. The second and third terms, which involve the angle θ , account for the non-coincidence between the principal directions of the ground motion and the structure axes. Differentiating Equation (12) with respect to θ and setting the resulting expression equal to zero [12] obtained the following relation of θ_{cr} :

$$\theta_{cr} = \frac{1}{2} \tan^{-1} \left[\frac{\frac{2}{\alpha} R_{12}}{R_1^2 - \frac{1}{\alpha^2} R_2^2} \right] \quad (13)$$

Lopez and Torres [40] determined the critical incident angle and the associated maximum structural response for the general case of three ground motion components having different spectral shapes. The authors defined S_1 , S_2 , and S_3 as the response spectra for the three ground motion components (1, 2, and 3), R as the peak response due to the simultaneous actions of the three spectra, and θ as the angle between the x structural axis and the input spectrum S_1 (Figure 4).

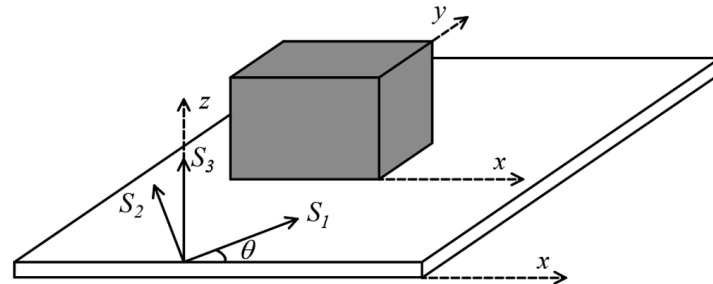


Figure 4. Horizontal acceleration response spectra (S_1 and S_2) applied along any arbitrary directions with respect to the structure reference axes according to [40].

According to Lopez and Torres [40], the peak modal response (R_i^1) and (R_i^2) due to the spectra S_1 , and S_2 , in the i th mode of vibration, are identified via Equations (14) and (15), respectively.

$$R_i^1 = R_i^{1x} \cos \theta + R_i^{1y} \sin \theta \quad (14)$$

$$R_i^2 = R_i^{2y} \cos \theta - R_i^{2x} \sin \theta \quad (15)$$

where (R_i^{1x}), (R_i^{1y}) and (R_i^{2x}), (R_i^{2y}) are the peak modal responses acting along the x and y reference axes, due to the response spectra S_1 and S_2 , respectively.

The peak response, R^1 , due to S_1 , is obtained by combining the peak modal responses as indicated in Equation (16), where C_{ij} is the correlation coefficient between responses in modes i and j . Similar equations were used for the seismic components 2 and 3.

$$R^1 = \left[\sum_i \sum_j C_{ij} R_i^1 R_j^1 \right]^{1/2} \quad (16)$$

Assuming the ground motion components act independently, the total peak response (R) due to all three components is determined using the SRSS method as shown in Equation (17).

$$R = \left[(R^1)^2 + (R^2)^2 + (R^3)^2 \right]^{1/2} \quad (17)$$

The authors finally obtain the peak response R in the function of θ , combining Equations (14)–(17).

$$R(\theta) = \left\{ \left[(R^{1x})^2 + (R^{2y})^2 \right] \cos^2 \theta + \left[(R^{1x})^2 + (R^{2y})^2 \right] \sin^2 \theta + 2 \sin \theta \cos \theta \left[\sum_i \sum_j C_{ij} R_i^{1x} R_j^{1y} - \sum_i \sum_j C_{ij} R_i^{2x} R_j^{2y} \right] + (R^3)^2 \right\}^{1/2} \quad (18)$$

where

$$R^{1x} = C_{ij} R_i^{1x} R_j^{1x}; R^{1y} = C_{ij} R_i^{1y} R_j^{1y}; R^{2x} = C_{ij} R_i^{2x} R_j^{2x}; R^{2y} = C_{ij} R_i^{2y} R_j^{2y} \quad (19)$$

Equation (18), as presented in [40], expresses the peak response (R) of the structure for any ASI. When ASI is zero ($\theta = 0^\circ$), the ground motion spectra act along the principal structural directions (x , y , and z), and the peak response is obtained using the SRSS method.

The critical angle, θ_{cr} , corresponds to the specific incidence angle (θ) that maximises the peak response (R) described in Equation (18) from [40]. This critical angle can be found by differentiating R with respect to θ and setting the result equal to zero.

$$\theta_{cr} = \frac{1}{2} \tan^{-1} \left\{ \frac{2 \sum_i \sum_j C_{ij} [R_i^{2y} R_j^{2x} - R_i^{1x} R_j^{1y}]}{(R^{1y})^2 + (R^{2x})^2 - (R^{1x})^2 - (R^{2y})^2} \right\} \quad (20)$$

Assuming that $S_2 = \alpha S_1$, θ_{cr} is obtained from [40] via Equation (21).

$$\tan 2\theta_{cr} = \left\{ \frac{2 \sum_i \sum_j C_{ij} R_i^{1x} R_j^{1y}}{(R^{1x})^2 + (R^{1y})^2} \right\} \frac{(1 - \alpha^2)}{(1 - \alpha^2)} \quad (21)$$

Equation (21) indicates that for $\alpha = 1$, critical angles are undefined. Therefore, for $\alpha \neq 1$, θ_{cr} can be calculated as indicated in Equation (22).

$$\tan 2\theta_{cr} = \left\{ \frac{2 \sum_i \sum_j C_{ij} R_i^{1x} R_j^{1y}}{(R^{1x})^2 + (R^{1y})^2} \right\} \quad (22)$$

The critical angle expressions derived from [40] and shown in Equations (20) and (22) align with the findings presented in Equation (10) derived from [11]. However, it differs from those proposed by [10,39], which do not consider the effect of the two seismic components in each mode.

Anastassiadis et al. [41] developed the tensorial properties of an arbitrary response quantity for the general case of orthotropic seismic excitation, carrying out the direct calculation of the maximum responses without a preliminary determination of the critical incident angle. The papers present a design procedure for (a) determining the critical orientation of the seismic input, i.e., the orientation that gives the largest response, (b) calculating the maximum and the minimum values of any response quantity, and (c) identifying the most unfavourable combinations of several stress resultants (or sectional forces) acting concurrently at a specified section of a structural member.

This section details the pivotal role of RSA in evaluating how variations in seismic incident angles affect the structural behaviour of various building types. The discussion begins with foundational work by Wilson and Button [39] and progresses through significant advancements by researchers such as Smeby and Der Kiureghian [11] and Menun and Der Kiureghian [12], who refined calculations of the critical angle. Notably, the introduction of the Complete Quadratic Combination (CQC) rule by Smeby and Der Kiureghian [11] incorporated cross-term effects between seismic components, significantly enhancing the accuracy of critical angle estimations. Lopez and Torres [40] expanded this methodology to address scenarios involving three ground motion components, each with distinct spectral shapes, leading to a more generalised understanding of structural responses under varied seismic inputs. Further, Anastassiadis et al. [41] advanced the application of RSA by directly calculating maximum and minimum response values without a prior determination of the critical incident angle, refining the tools available for defining θ_{cr} . This evolution of RSA has been indispensable in the development of seismic design standards and in enhancing structural resilience against earthquakes, particularly during periods when the capabilities of information technology limit advanced structural analyses.

3.2. Linear Response History Analysis

While response spectrum analysis (RSA) offers computational advantages, it can underestimate seismic demands in MDOF systems. This is because peak modal responses

often occur at different times and some approximations might be introduced when the peak modal responses are combined. As a result, various researchers have employed linear response history analysis (LRHA) to investigate building responses under bidirectional seismic loads.

The computational advantages of RSA are evident where multi-degree-of-freedom systems are considered because modal responses attain their peaks at different time instances. In these cases, some approximations might be introduced to combine the peak modal responses. For this reason, numerous researchers employed linear response history analysis (LRHA) to examine building structures and specifically θ_{cr} and the corresponding seismic demand.

Fernandez-Davila et al. [42] determined the seismic demand of a 3D elastic five-storey structure subjected to one and two seismic components of a ground motion record applied with ASI at a step of 15° . The authors found that the maximum response in a structure might not occur when seismic components align with principal directions. Additionally, the commonly used 100–30% and SRSS combination rules can underestimate responses by up to 25% compared to a bidirectional LRHA.

Athanatopoulou [43] proposed analytical formulas to determine θ_{cr} and the corresponding maximum responses for two or three seismic components, assuming linear structural behaviour. These formulas were validated through parametric studies [74–76], revealing that (1) θ_{cr} can vary for different response quantities even with the same earthquake record, (2) the same response quantity can have different θ_{cr} depending on the specific earthquake record, (3) the maximum response at the critical angle can be up to 80% larger than the response obtained from applying seismic excitation along structural axes and (4) the response quantity can vary by up to 200% within the 0 – 180° incidence angle range.

Kostinakis et al. [44] investigated the internal forces required for longitudinal reinforcement design in RC elements using LRHA on 3D single-storey buildings. They compared four procedures: three procedures followed seismic code recommendations, applying accelerograms along structural axes, and the fourth procedure considered various ground motion component orientations. Their findings showed that applying accelerograms only along structural axes might not provide conservative results.

Marinilli and Lopez [45] compared critical responses and incidence angles obtained from LRHA and RSA for single-storey RC structures with varying natural periods. Their findings suggest that RSA can be adequate for estimating critical responses and corresponding angles for design purposes. However, LRHA might provide more refined values.

Alam et al. [46] investigated the influence of seismic excitation direction on response variability in a realistic, asymmetric RC structure. Their results indicate that the critical orientation leading to the maximum seismic demand can vary depending on the specific response quantity (EDP) and the characteristics of the ground motion. This implies the inexistence of a single critical orientation that maximises all seismic responses simultaneously.

Kalkan and Kwong [47] suggested that using ground motion components directly aligned with fault-normal (FN) and fault-parallel (FP) directions might lead to underestimating seismic responses. The authors analysed the 3D structural model of a six-storey symmetrical building with 20 Near-Fault (NF) ground motion records oriented along rotation angles varying from 0 to 180° , with a 10° interval. They showed that (1) the critical angle (θ_{cr}) corresponding to the maximum response can vary depending on the specific ground motion and the response quantity of interest and (2) applying FN/FP motions along the building's principal directions is unlikely to guarantee the maximum response. Their research emphasises the importance of considering bidirectional ground motions at various incidence angles to capture the full range of possible responses.

This section explores the application of LRHA to assess seismic responses in buildings under multidirectional seismic loads. While RSA is favoured for its computational simplicity, it often underestimates seismic demands in MDOF systems due to the asynchronous occurrence of peak modal responses. In contrast, LRHA offers a more accurate assessment by accounting for the full temporal progression of seismic forces.

The findings discussed in this section highlight how the maximum response of a structure may not occur when seismic components align with the principal structural directions and maximum seismic demand can vary depending on the specific EDP. This challenges traditional seismic analysis practices, which typically assume that principal directions coincide with the greatest seismic demands. By applying seismic forces at varied incident angles, LRHA enables engineers to identify critical angles where seismic demand is maximised, thus providing a deeper understanding of potential vulnerabilities within a structure's design, within the limits of a linear analysis.

3.3. Nonlinear Static Analysis

In recent years, different authors addressed the multidirectional NSA using different procedures. A detailed analysis of these procedures can provide an important starting point for clarifying eventual gaps, helping future research.

Cannizzaro et al. [48] analysed the behaviour of a masonry building damaged during the L'Aquila Earthquake, performing 12 nonlinear static analyses having input directions with angular steps of 30° . Because of the difficulties involved in identifying the fundamental vibration modes along seismic directions different from 0° and 90° , the authors considered only the mass proportional load pattern and a unique participation factor Γ for all the considered directions. The authors constructed a three-dimensional capacity dominium, plotting 12 pushover curves in the function of the considered seismic incident angles. The same procedure was applied by Chácara et al. [49,50].

Kalkbrenner et al. [51] performed a multidirectional PO analysis of an irregular historical masonry building without any box behaviour. They first developed a nonlinear 3D Finite Element Model (FEM) and then implemented multiple PO analyses to determine the seismic capacity of the structure along eight incident angles, defined for angles multiples of 45° . The horizontal load patterns were assumed proportional to the mass distribution and the control nodes were selected by analysing the most vulnerable structural members of the masonry building. The pushover curve of the MDOF system was transformed into the capacity curve of an equivalent SDOF system using the mass participation factor Γ according to the N2 method [34]. The authors assumed that, for each specific PO loading orientation, the direction for the calculation of Γ corresponds to the largest horizontal component (X or Y) of the control displacement. The capacity curve of the equivalent SDOF system was evaluated only along this direction. The other steps follow the pushover procedure suggested in the Eurocode 8 [19]. The comparison of the multidirectional NSA with a nonlinear response history analysis (NRHA) carried out, applying a seismic input along the transversal and the longitudinal directions, revealed a mean error equal to 16.2%, showing a proper approximation of the multidirectional NSA with the NRHA.

Ghayoumian and Emami [52] employed multidirectional PO analyses to investigate the seismic response of torsionally irregular modern buildings with a special dual-RC system. Their analysis included 4-, 8-, and 12-storey archetypes and compared responses along non-principal directions with those obtained along principal axes. By examining the distribution of inter-storey drifts, ductility, and damage indices, the authors revealed that the maximum seismic responses of these archetypes might occur along non-principal directions, highlighting the importance of considering these orientations in design.

Cantagallo et al. [53] investigated the effects of the multidirectional seismic load and the combination rules in the pushover analysis. The study was carried out on several 3D irregular RC structures. The seismic demand was calculated with the N2 method in terms of shear displacement demand [34], applying the seismic force both along the conventional structural directions X and Y and along incident angles θ_i , varying between 0° and 360° with steps of 22.5° . In order to validate the multidirectional PO approach and the code-conforming combination rules, the structural demands obtained from the different pushover analyses were compared with those computed using NRHA performed using suites of twenty pairs of both real and generated ground motion records applied at the reference structural axes. The research indicated that the combination rules do not

produce significant improvements to the conventional N2 method. Moreover, the authors found that displacement demands obtained from multidirectional NSA provide accurate predictions of floor rotations if compared with NRHA with seismic inputs applied at 0° and 90°. In single-storey structures, both multidirectional and conventional pushover procedures provide an overestimation of shear demand. Conversely, in a multi-storey structure, multidirectional NSA produce unconservative shear demands.

Cantagallo et al. [54] propose a systematic approach for performing multidirectional NSA. They validate this approach by comparing the results with multidirectional NRHA on two existing RC frame buildings. PO loads were applied to both structures at various incident angles, varying between 0° and 360° with 15° intervals. Their results demonstrate that the structural demands obtained from multidirectional NSA, while generally lower, exhibit a closer correlation with the multidirectional NRHA results compared to conventional NSA.

In this section, various studies that used NSA to assess structural demands obtained from multidirectional seismic loads were presented. Although there are several methods to apply PO load distributions, such as adaptive [77,78] or multi-mode methods [33,79] (among them, the generalised force vectors for multi-mode PO analysis [80,81]), the authors dealing with multidirectional PO analysis use generally triangular, modal or uniform load patterns, with the only exception of [52], which uses an adaptive pushover procedure. The application of these loads along different incident angles demonstrates that (a) the most significant seismic demands may not be aligned with the principal structural directions and (b) there is a substantial variability in seismic demand depending on the direction of load application. This highlights the necessity for a design approach that accounts for this variability. Comparisons between the results obtained from multidirectional NSA and NRHA revealed comparable results, emphasising the effectiveness of NSA in capturing the effects of variability in seismic incident angles with the advantage of simultaneously evaluating the effects of structural nonlinearity with limited analysis times.

3.4. Nonlinear Response History Analysis

NRHA is typically applied considering two horizontal seismic components acting simultaneously. The use of two seismic components stems from two key factors: (a) accelerometers commonly record seismic events using two horizontal time series, one corresponding to the north–south (NS) direction and the other to the east–west (EW) direction, and (b) applying both horizontal components simultaneously allows engineers to consider the simultaneous effect of the seismic event along the structure's two principal reference directions.

Since the direction of future earthquakes is unpredictable, researchers have rotated in the horizontal plane the recorded ground motion components employed for NRHA (a_x and a_y), using the coordinate transformation shown in Equation (23), where θ is the angle of rotation in the counterclockwise direction.

$$\begin{aligned} x_a &= a_x \cos \theta + a_y \sin \theta \\ y_a &= -a_x \sin \theta + a_y \cos \theta \end{aligned} \quad (23)$$

The main results of the research obtained from the multidirectional NRHA are described below in chronological order.

MacRae and Mattheis [55] were among the first researchers to investigate the influence of seismic incidence angle on multi-storey steel structures. Their study focused on the response of a nearly symmetrical three-storey building subjected to Near-Fault (NF) ground motions. The building model included rigid floor diaphragms using diagonal elements, elastic beams, and columns modelled with fibre hinge models. The structural model was subjected to three groups of earthquake records along various incident angles ranging from 0° to 90°. The authors observed that regardless of the method of analysis and the methods for estimating responses to bidirectional shaking (such as the 30%, 40%, and SRSS methods), building drifts under bidirectional shaking depend on the chosen reference axes.

When these axes coincide with the building's principal axes, the conventional methods for assessing the bidirectional horizontal shaking can underestimate the inelastic storey drifts in a non-conservative manner.

Rigato and Medina [56] investigated the response of two reinforced concrete (RC) structures with varying degrees of inelasticity under bidirectional seismic excitation. The first structure is an asymmetric, single-storey shear-type system with an eccentric mass distribution, while the second structure is the corresponding torsionally balanced structure. The study employed a total of 60 structural models by varying the structural period, element strength (affecting plastic hinge moments), and model type. A suite of 39 ground motion pairs recorded on stiff soil sites with epicentral distances between 13 km and 60 km was selected. The component with the higher peak ground acceleration (PGA), designated as the "major component", was applied at various incidence angles with 5° increments. The other component, the "minor component", was consistently applied at +90°. All major components were scaled to the same spectral acceleration $S_a(T_1)$ value, with the minor components proportionally scaled. The study evaluated various engineering demand parameters (EDPs), including (a) column displacement ductility ratio (normalised top displacement by yield displacement), (b) slab rotation, (c) column drift ratio (normalised top displacement by column height), and (d) average ductility (sum of individual column ductility ratios in one direction divided by four). The results revealed that maximum EDPs can occur at incidence angles other than 0° and 90°. Rigato and Medina [63] found that the ratio between the peak inelastic deformation at a specific angle and the peak values at 0° and 90° ranged on average from 1.1 to 1.6 for both balanced and unbalanced models. Similar trends were observed for ductility demands. The study by Rigato and Medina [56] emphasises the importance of considering bidirectional ground motions with varying incidence angles relative to the building's principal directions during the performance assessment of structures. This approach provides a more comprehensive understanding of potential seismic demands compared to analyses limited to 0° and 90° incidence angles.

Lagaros [57] investigated the influence of ASI on the seismic loss assessment of two RC buildings, one regular and one irregular. Their findings suggest that using five randomly chosen incidence angles is sufficient to capture the randomness of the earthquake's direction and its impact on seismic losses. This aligns with the observations reported in [3].

Reyes and Kalkan [58] and Kalkan and Reyes [59] investigated the effect of ASI on various EDPs provided via steel buildings subjected to NF ground motion records. Their studies included both symmetric and asymmetric buildings, with ground motions rotated along FN/FP directions. The authors employed 30 ground motion records, rotating them from 0° to 360° at 5° increments in the clockwise direction for a total of 72 incident angles. They observed that, for a given earthquake, the rotation angle leading to the maximum elastic and inelastic responses pair can differ. This implies that conclusions drawn from linear system analyses might not be directly applicable to nonlinear systems. Furthermore, the studies revealed two key findings: (a) there is not a single rotation angle that simultaneously maximises all EDPs and (b) NRHA analyses based on FN/FP-oriented ground motions can underestimate peak displacements compared to other incidence angles. The studies demonstrate that this underestimation might be less than 20% for a broad range of ground motions [59]. These findings emphasise the importance of considering a wider range of seismic input directions beyond just FN/FP orientations, particularly for asymmetric structures, typical of existing buildings.

Kostinakis et al. [60] investigated the impact of ASI on a single-storey RC building subjected to 16 bidirectional ground motions and 4 different seismic intensity levels. The horizontal components of each ground motion were applied along incidence angles ranging from 0° to 355° with 5° increments. The building was a rectangular plan with four parallel plane frames in each direction and a rigid roof deck. Material inelasticity was modelled using bilinear hysteretic plastic hinges at column and beam ends. The research considered the effects of the axial load-bending moment (P-M-M) interaction at column hinges through a specific P-M-M interaction diagram. The results demonstrated that applying accelerograms

only along the principal structural axes might not be sufficient for optimal performance evaluation, highlighting the importance of considering a broader range of incidence angles.

Magliulo et al. [61] explored the effect of ASI on the response of a multi-storey RC building with an L-shaped plan. The study employed three sets of seven earthquakes, each with two horizontal components. These ground motion records were incrementally rotated at 30° intervals along twelve different incident angles (from 0° to 330°). The selection process ensured the Eurocode 8 spectrum-compatibility criteria for three specific hazard levels. Beams and columns were modelled using lumped plasticity models that did not account for the axial load-bending moment interaction in columns. The authors selected the top displacements in both orthogonal directions and the vectorial top displacement (i.e., the combination SRSS of top displacements in the x and y directions at each NRHA step) as EDPs. The results revealed that the critical seismic angle can lead to increases in top displacements exceeding 15% compared to the scenario without ground motion rotation. In one instance, this increase reached nearly 40%. Additionally, the study observed variations in maximum column rotations of up to 30%. Furthermore, the authors quantified the uncertainty associated with the variability of ASI. They employed empirical cumulative distribution functions (CDFs) for the three considered EDPs measured at the three intensity levels. These empirical distributions were then fitted with a lognormal distribution, where the standard deviation (β) represents the overall uncertainty in the results. By comparing the obtained β values with those suggested by FEMA P695 [82], the study concluded that the uncertainty introduced via the seismic incidence angle is not negligible.

Cantagallo et al. [62] investigated the impact of ASI on the response of four symmetric and asymmetric RC structures. The study employed two sets of ground motion records: (a) 61 pairs of accelerograms, each consisting of two simultaneous horizontal components chosen based on a specific earthquake scenario with a 10% probability of exceedance in 50 years and (b) 20 pairs of spectrum-compatible ground motion records derived from the un-scaled records. Each ground motion record was applied at various incidence angles ranging from 0° to 180° with 22.5° increments. NRHA were carried out using a force-based fibre-section frame model [83]. The results obtained by the authors highlight that (1) the seismic demand on a perfectly symmetrical, single-storey RC structure exhibits minimal variation with changing ASI, (2) the maximum inter-storey drift ratios (MIDRs) for structures with plan irregularities show significant dependence on ASI, (3) ground motions applied along directions with the lowest structural capacity generate the highest demands, and (4) the influence of ASI on structural response decreases as the number of ground motion records used in the analysis increases. These findings highlight the importance of considering ASI, particularly for irregular structures. Using a broader range of ground motion directions can provide a more comprehensive understanding of potential seismic demands.

Emami and Halabian [63] investigated the influence of ASI on multi-storey RC frames. Their study employed a set of 3D moment-frame archetypes (4, 8, and 12 storeys) representing short-rise to medium-rise RC buildings. These structures were modelled using lumped plastic hinge models for beams and Multi-Spring (MS) fibre models for columns. Multiple ground motion sets were applied to each archetype. Each set included 14 records scaled to a target design spectrum as per ASCE/SEI 7–10 standards [25]. The spatial distribution of EDPs was evaluated, varying the incident angles from 0° to 90° with 5° increments. The results corroborated previous findings, indicating that bidirectional excitation along principal axes (x and y) can lead to lower seismic demands compared to specific non-principal orientations (at various θ angles). The authors emphasised that neglecting directionality effects can result in the underestimation of seismic demands. Ideally, design procedures and performance assessments should consider EDPs under non-principal orientations. However, acknowledging the impracticality of conducting numerous NRHA for a vast range of angles, Emami and Halabian [63] proposed the use of amplification factors. These factors account for the potential increase in EDPs due to varying ASI. The proposed factors

range from 1.08 to 1.39, depending on the considered EDP, ground motion type (NF-FF), and the considered archetype structure.

Kostinakis et al. [64] investigated the effect of ASI on the structural damage of eight five-storey RC buildings (four symmetrical and four asymmetric in plan), both with and without structural walls. The buildings were subjected to 100 bidirectional ground motions applied along 72 different ASIs. Structural damage was evaluated using the Park and Ang Damage Index [84]. The damage level in these buildings was significantly influenced by the ASI of the ground motion. This sensitivity to ASI also depends on the structural system (presence or absence of walls) and the distance of the ground motion record to the earthquake's fault rupture.

Fontara et al. [65] studied the impact of ASI on the response of single-storey asymmetric RC structures. Their study revealed that (a) the most unfavourable structural demands were not necessarily observed when the ground motion was applied along the principal structural axes and (b) the structure's sensitivity to ASI variations increased as the level of damage progressed into the nonlinear range. In other words, structures experiencing greater damage became more susceptible to the influence of ASI.

Sun et al. [66] evaluated the influence of ASI on the seismic response of a school building using multiple ground motion records rotated along various incidence angles. Their work delves deeper by analysing the variations in response spectra caused by different ASI values providing valuable insights into how the seismic demands on the building change depending on the angle of the ground motion.

Amarloo and Emami [67] investigated the effect of ASI on the seismic response of 4-, 8-, and 12-storey RC moment-frame structures with L-shaped plans. Each archetype model was subjected to a set of 20 strong ground motion records, consisting of 10 NF and 10 FF records. NRHA were performed by rotating the ground motions at 10° increments from 0° to 350°. The analysis of the inter-storey drift ratios (IDRs) reveals that (1) the most critical IDRs (indicating potential for structural damage) do not necessarily occur when the ground motion is aligned with the principal structural axes (x and y); (2) the angle leading to the maximum IDR varies depending on the specific ground motion record; and (3) the maximum IDR (MIDR) is significantly higher than the demands observed when the ground motion was applied along the x - or y -axis alone. Acknowledging the impracticality of conducting NRHA for a vast range of angles, the authors proposed using amplification factors for seismic demands. These factors range from 1.20 to 1.40 and were determined based on the upper bound of a 95% confidence interval. This approach offers a practical way to account for the potential increase in demands due to varying ASI during design. Finally, Amarloo and Emami [67] explored the impact of ASI on seismic performance-based assessments. Specifically, after having defined different performance levels based on specific damage index (DI) values, they found that the multidirectional seismic input can change the performance level in 7 out of 20 cases. This highlights the importance of considering ASI in performance assessments to ensure structures meet the intended safety objectives under earthquake loads.

Kostinakis et al. [68] investigated how variations in ASI affect the seismic demands of two RC buildings, one with equal stiffness in both directions and another with unequal stiffness. To assess the impact of ASI, the researchers used both LRHA and NRHA with 13 sets of uncorrelated horizontal ground motion records, where each set was applied at 72 different incidence angles ranging from 0° to 360°, with 5° increments. The study adopted a bilinear elastic-perfectly plastic moment-rotation relationship to model the nonlinear behaviour concentrated at the ends of frame elements. The structural model also considered the interaction between moment and axial force acting on the structural elements. Two EDPs were considered: the maximum values of the displacements at the top of a reference column and the Park and Ang [84] Damage Index (DI), as modified by Kunnath et al. [85]. An Overall Structural Damage Index (OSDI) was also evaluated as a weighted average of local damage indices (DIs) at the ends of each element, where dissipated energy serves as the weighting factor [86]. The most significant finding of

the research is that ASI has a considerable influence on the EDPs in most cases. The only exception observed was for perfectly symmetrical buildings with equal stiffness in both directions. For these structures, the maximum values of specific combined response quantities (e.g., the SRSS of displacements in orthogonal directions) were independent of the seismic excitation's orientation. This observation suggests that current engineering practices, which often apply ground motions only along principal structural axes (0° and 90° incidence), might be sufficient for symmetrical buildings. However, for structures with stiffness asymmetry, incorporating a wider range of ASI scenarios during the design and verification process is crucial to capture potential variations in seismic demands and ensure adequate structural performance.

Giannopoulos and Vamvatsikos [69] investigated the need for incorporating directionality in the record-to-record variability for seismic performance assessments in the case of limited computational resources. They employed single-degree-of-freedom (SDOF) and multi-degree-of-freedom (MDOF) systems for analysis. For this purpose, elastic and inelastic SDOF systems were analysed using a comprehensive set of 135 non-pulsive and 102 pulsed ground motion records. Each record was rotated through 60 different orientations (0° to 180° with 3° increments). Moreover, a more complex MDOF system was analysed, consisting of a 3D L-shaped steel frame subjected to 60 ground motion records (36 non-pulsive and 36 pulsed), oriented along incidence angles from 0° to 360° , within 10° increments. The study indicated that (1) utilising a broader range of ground motion records is more crucial than analysing each record at multiple orientations and (2) relying solely on the critical orientation for design or assessment can lead to overly conservative results as the actual critical angle depends on the relative orientation between the fault and the structure. The authors recommend, therefore, using the as-recorded orientation, particularly for Far-Field (FF) ground motions. Moreover, Giannopoulos and Vamvatsikos [69] found that employing a scalar intensity measure (IM) that combines spectral values from both horizontal components (e.g., geometric mean) can further mitigate the influence of ASI.

Pavel and Nica [70] investigated the effect of ASI on the seismic response of three RC wall structures (6-, 8-, and 10-storeys) designed according to Romanian seismic codes. Nonlinearity in the elements was modelled using nonlinear bending hinges at both ends and a shear hinge in the middle of the elements. Columns and walls were modelled with a Multi-Spring (MS) model with nonlinear axial and shear springs at both ends. NRHA were carried out using five pairs of NF ground motion records, where each pair was rotated clockwise at 5° increments, from 0° to 180° . The authors considered 3 EDPs: storey drift, bending moment, and shear force at the base of structural walls. The authors evaluated the impact of ASI on these EDPs, observing that (1) the discrepancy between the mean drift profiles obtained from multidirectional NRHA and the drift profile from elastic analysis increased with building height and (2) the use of unidirectional NRHA of the as-recorded horizontal components did not lead to the maximum possible structural response (with an underestimation of the maximum drift up to 50%). Based on their findings, Pavel and Nica [70] recommend performing NRHA with bidirectionally rotated ground motion components to account for the effects of ASI.

Skoulidou et al. [71] investigated the influence of two factors on the accuracy of seismic collapse risk assessments for RC buildings with infilled frames: (a) ground motion group size, i.e., the number of different ground motion records used in the analysis and (b) the number of ASIs, i.e., the number of orientations at which the ground motion is applied to the structure. The study employed NRHA on six low- and mid-rise RC buildings (with both regular and irregular configurations). These structures were subjected to multi-stripe analyses, using multiple FF ground motion records. The authors observed that increasing the number of ground motions used in the analysis leads to a reduction in the variability of the estimated collapse risk. Due to the significant uncertainties observed with a number of ground motions lower than 15, the authors recommend using a minimum of 20 ground motions for a reliable collapse risk assessment. Compared to the ground motion group size, the impact of ASI on the variation of collapse risk estimates is less significant. However, ASI

plays a role when a limited number of ground motions is used (i.e., smaller than 25). When only one ASI (typically 0°) is considered, the collapse risk estimate can be biased by up to 10%. This bias is reduced by employing just two ASIs (0° and 90°) for ground motion group sizes exceeding 15. Adding more ASI scenarios beyond two provides minimal additional benefit. Overall, the study suggests that using a group of 20 or more ground motions with two incidence angles (0° and 90°) offers an adequate assessment of collapse risk, regardless of the building's structural regularity.

Skoulidou and Romão [72] investigated the effect of ASI and ground motion group size on the evaluation of the seismic demands of multi-storey RC buildings with infilled frames. NRHA were carried out using structural models with a lumped plasticity without P-M-M interaction for both beams and columns. These models were subjected to groups of bidirectional ground motions of size n , ranging from 10 to 40, applied along ASI from 1 to 12 for four intensity levels. The study focused on three common EDPs: Maximum Inter-Storey Drift Ratio (MIDR), Peak Floor Acceleration, and Maximum Roof Drift. To assess the impact of ground motion group size and ASI, the researchers analysed specific statistics of the EDPs: mean, median, and standard deviation. They then quantified a “risk exceedance ratio” to measure how much these estimates deviate from reference values within a certain margin of error. The reference values were obtained using the largest group size (40 ground motions) and all 12 ASIs. The results showed that when only interested in the mean/median values of EDPs, using just one ASI might suffice for large ground motion groups (at least 30 ground motions, depending on acceptable risk levels). In most cases, two or three incidence angles provided slightly conservative results and using more than four offered minimal benefits. This suggests that for large datasets and acceptable risk tolerances, a limited number of ASI scenarios can be employed. If probabilistic analyses requiring information about the standard deviation are performed, using only one ASI is inadequate. In such cases, two to four ASIs are recommended, depending on the risk level and ground motion group size. For large groups (30–35 ground motions), two ASIs are sufficient, while smaller groups (20–25 motions) might require up to four ASIs. Finally, the authors concluded that the size of the ground motion group significantly impacts the risk exceedance ratios, recommending using at least 20 ground motions for reliable EDP estimates.

Bugueño et al. [73] studied four RC structural archetypes in order to evaluate the effect of ASI on inter-storey drift and roof displacement. A set of strong ground motions recorded in Chile was used, modifying the records with a matching process. For each record, the horizontal pair of components was applied on each archetype with incident angles between 0° and 360° , with a variation of 22.5° . The authors presented their results in terms of polar graphs. EDPs were obtained at the centre of mass of each storey, combining the results at the two reference axes with the SRSS combination. The results of the NRHA indicated that (1) there is not a unique incident angle that produces the highest values of all EDPs and (2) there is a relationship between the structural stiffnesses in each direction of analysis and the influence of ASI; that is, when the stiffness in the two reference directions is very similar, as in a double-symmetric frame, the polar graphs of the inter-storey drifts and the roof displacements are almost circular; conversely, in the other cases, these graphs have an oval shape.

There are many other studies that use NRHA for analysing a broader range of EDPs (such as input energy, residual displacement, and non-exceedance probability curves) and structural types (vertically regular and irregular structures) but they consider only single- or bidirectional recordings, neglecting the effect obtained from seismic incidence angle variability on structural demand; for this reason, their results were not included in the overall framework of this paper. For example, but not exhaustively, the works of [87–92] can be considered.

In this section, the application of multidirectional NRHA in evaluating the seismic demands of various building construction typologies was described. The studies reported here specifically highlighted a significant variability in EDPs such as displacement, drift,

and ductility when ground motion records are oriented along non-principal directions. This variability often indicates that maximum responses may occur at incident angles other than the conventional 0° and 90° . This is particularly true for structures with irregular layouts, where ground motions applied in non-principal directions can lead to significantly higher seismic demands, underscoring the importance of considering ASI in structural design and analysis. Comparative analyses between NRHA results and other analysis methods reveal that traditional approaches might underestimate seismic demands if they do not consider varied incident angles, leading to non-conservative results. Overall, these findings underscore the critical role of multidirectional NRHA in providing a more comprehensive understanding of the impact of seismic incident angles on structural demand.

4. Analysis and Discussion

In order to have a comprehensive understanding of the influence of seismic directionality on the seismic behaviour of various construction types, this study analysed numerous works in the literature on this topic. More specifically, initially, a brief review was conducted on 31 studies concerning combination rules in both linear analysis and nonlinear static analysis. Subsequently, 39 works focused on the influence of the angle of incidence of seismic input were analysed. The analysis of this second group of works revealed that all authors addressed the topic through the analysis of one or more case studies, each analysed using a different method and characterised by different structural materials, number of storeys, and type of (non)linear structural model. Figure 5 summarises these characteristics into four groups of histograms, which indicate as follows:

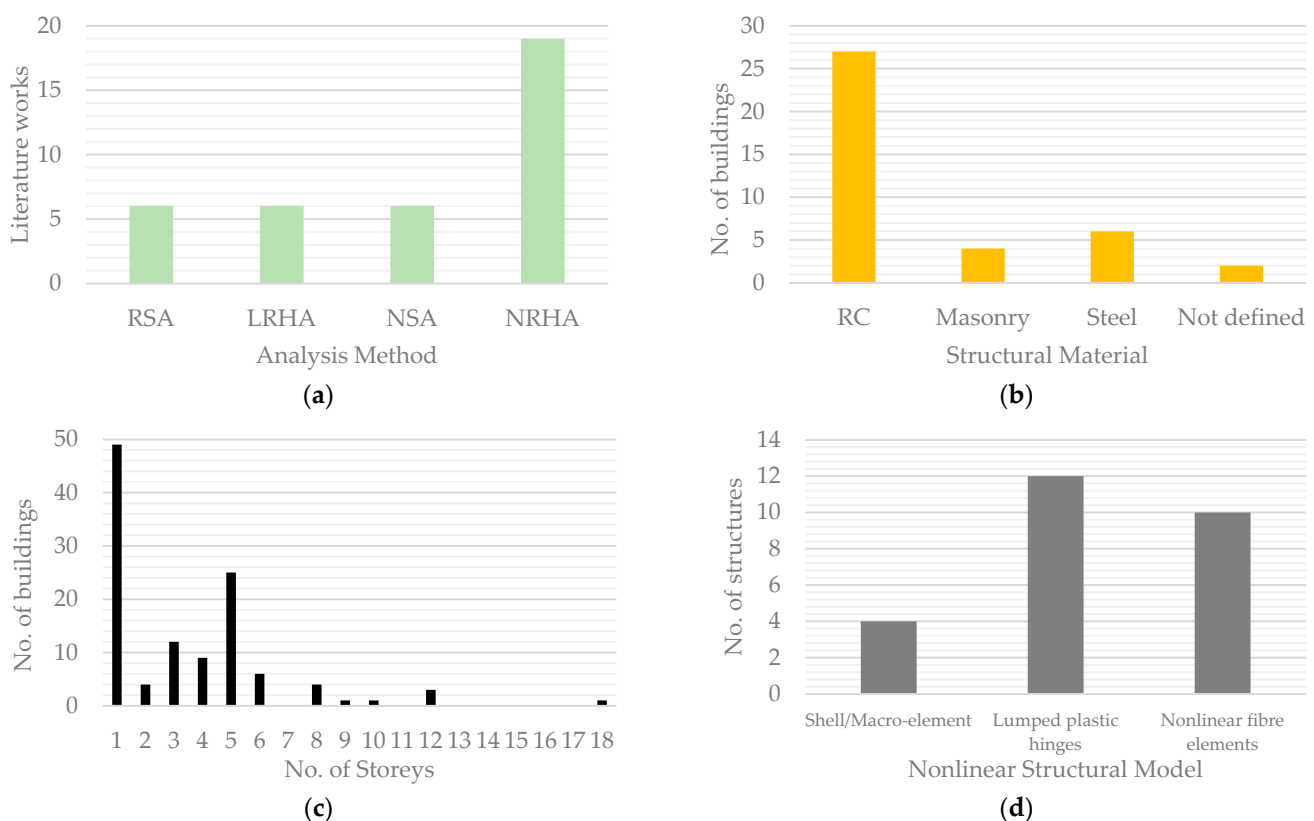


Figure 5. Summary of the analysis methods and the characteristics of the structural systems used in the scientific literature that analyses the effect of the ASI: analysis methods used in the different cited literature works (a), structural material (b), number of storeys (c) and type of nonlinear structural model used in the case-study structures described in the scientific literature (d).

The most used analysis method for demonstrating the influence of the directionality effect of the seismic input is NRHA (Figure 5a). The other analysis methods were examined

in a limited number of studies. While this result is predictable for linear analyses, where input directionality is addressed with combination rules, the small number of works using NSA demonstrates that greater attention should be given to this analysis method in evaluating the incidence of seismic input on structural behaviour.

Most of the case-study structures have RC construction systems (Figure 5b). This depends on several factors: (1) the topic had its initial development in the period of greatest development of RC construction; (2) the influence of the ASI has an effect especially on existing buildings, and multidirectional analyses of existing RC buildings (which are predominantly framed), are computationally less expensive than those of existing masonry buildings; (3) in today's construction, steel is still a material that is underutilised. The case-study buildings have a number of storeys ranging from 1 to 18, but most of them have only one storey (Figure 5c). Consistently with the prevailing construction type characterised by reinforced concrete structural systems, there are numerous buildings with three, four, and five floors. However, there is a lack of analysis of buildings with more than six floors. The 27 nonlinear structural models are carried out prevalently using lumped plastic hinges defined by phenomenological laws (Figure 5d). A significant number of structural models use fibre elements with distributed or concentrated nonlinearities. The limited number of structural models performed with shell or macro-elements reflects that of masonry construction systems, once again emphasising the need to analyse the effect of ASI on existing masonry buildings.

Recently, the number of ASIs was correlated with the number of ground motion records used in the analyses, showing that using a set of ground motion records at least equal to 20, the number of ASIs could be reduced to two or four [71,72]. The authors of this work suggest focusing future research on this topic using a larger number of building typologies.

5. Standards

Research on multidirectional seismic analysis presents a diverse range of conclusions and recommendations regarding the impact of ASI. There is no single agreed-upon approach for handling the influence of ASI and accounting for it in design and performance assessment. This inconsistency in research translates to a lack of clear guidelines in current earthquake engineering standards.

Regarding the reference directions, ASCE 41-06 §3.2.7.1 [93] and ASCE 41-17 §7.2.5.1 [24] specify that where concurrent multidirectional seismic effects must be considered, horizontally oriented, orthogonal X- and Y-axes shall be established. Where the NRHA is used as the basis for analysis with a 3D model, elements and components of the building have to be analysed for forces and deformations associated with the application of the suite of ground motions selected according to §16.2 of ASCE 7.

ASCE/SEI 7-22 [94] specifies that when NRHA are used to analyse structures located on NF sites, each pair of horizontal ground motion components has to be rotated along the FN and FP directions. At all other sites, away from the causative fault, ASCE/SEI 7-16 [88] requires the application of each pair of horizontal ground motion components in orthogonal orientations of the structure (typically 0° and 90°). According to [95,96], rotating ground motion pairs along FN/FP directions in the proximity of an active fault system represents a conservative strategy because this allows the designer/engineer to consider both the direction of rupture propagation and the effects of the seismic energy along a single long-period pulse of motion in the FN/FP directions [97–99].

NZS [100] requires the use of the component directions that produce the most unfavourable value for the chosen EDP. However, a crucial limitation of NZS [100] is the absence of specific guidance on how to determine this critical direction. This lack of clear instruction regarding the most critical axes for seismic input often leads engineers to apply the horizontal seismic components along the principal structural axes.

Eurocode 8 [19] §4.3.3.1 indicates that “Whenever a spatial model is used, the design seismic action shall be applied along all relevant horizontal directions (with regard to the structural layout of the building) and their orthogonal horizontal directions. For

buildings with resisting elements in two perpendicular directions these two directions shall be considered as the relevant directions". In summary, this standard indicates that the seismic input can only be applied in the two perpendicular directions of a building.

Chinese code GB 50011-2010 [101] in §5.1.1 states that "Generally, the horizontal earthquake actions should be considered and checked separately along the two major axial directions of the building structure; and the horizontal earthquake action in each direction should be borne by the lateral-force-resisting components in this direction". The code also suggests that "for structures with oblique lateral-force-resisting components, if the intersection angle is greater than 15° , the horizontal earthquake action along the direction of each lateral-force-resisting component shall be calculated". While this last statement provides a useful starting point for taking into account the multidirectionality of the seismic input, it does not offer any guidance on how to calculate the direction of the seismic input to be considered if the resisting elements have no unique direction.

The Italian Building Code [22] does not provide specific provisions about the directionality of the seismic input. However, according to the instructions for the application of this standard [23], a description of the seismic actions consistent with the event can be obtained by projecting each pair of ground motion records along the principal directions of the earthquake, as indicated in [1].

The Turkish Building Earthquake Code [102] takes into account the multidirectionality of the seismic input in NRHA. Specifically, according to this code, after having selected the ground motion pairs of accelerograms according to the spectrum-compatible criterion, these ground motion records have to be applied simultaneously in two directions (x and y) of the main axes of the structural system according to TBEC. Then, the axes of the ground motion records are rotated by 90° and the calculation is repeated. The same scaling factor is used for both horizontal components of ground motions. In the end, in order to take into account all possible input directions, the SRSS rule has to be applied for the combination of EDPs in both orthogonal directions.

Despite the significance of the orientation of ground motion extensively documented in the literature, this issue has not yet drawn the attention of many modern seismic guidelines. This leads current engineering practice to a significant underestimation of the seismic demand.

6. Conclusions

ASI indicates the direction along which seismic force acts on a 3D structural model. It is a critical factor that needs careful consideration during the seismic analysis of complex 3D structures. For this reason, the research community has devoted a significant amount of attention to ASI in recent decades. Despite extensive research and advancements, no definitive consensus has emerged on the most appropriate approach for handling ASI. This work aims to provide a comprehensive review of the existing scientific literature on ASI's influence on building response. This review encompasses 10 regulatory codes, 67 articles published in 20 different journals, and additional resources like books, book chapters, conference proceedings, and technical reports (14 in total). All sources are current, up to February 2024.

The review was carried out according to the type of analysis used to determine the structural demand of the building structure: response spectrum analysis (RSA), linear response history analysis (LRHA), nonlinear static analysis (NSA), and nonlinear response history analysis (LRHA). For each type of analysis, the sources were cited in chronological order to provide an historical evolution of the impact of ASI on the safety assessment of various building construction typologies.

Most research shows that (1) the critical angle, i.e., the angle providing the maximum seismic demand, depends on the specific ground motion and the EDP and (2) the application of the seismic input along the reference axes of the structure could lead to a non-conservative estimate of the seismic demand. The analysis of case studies addressed in the literature showed that, until now, this topic has been predominantly tackled on

existing RC buildings with a limited number of floors and with NRHA. This highlights the necessity for future studies to examine the influence of ASI through NSA on existing masonry buildings or tall RC buildings.

While advancements have been made in understanding ASI, current building codes have not yet fully incorporated these findings. In fact, only a few standards require NF recordings to be oriented along the FN/FP directions, and others specifically indicate the need to consider the directions of seismic input that provide the maximum response but without specifying how they can be identified. Future code revisions could benefit from literature studies on ASI to provide more comprehensive and robust seismic guidelines. This could involve specifying methodologies for considering ASI during analysis or requiring a minimum number of ground motion records and ASI scenarios for reliable seismic demand estimates.

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