

Article

Simplified Performance Based Nonlinear Static Evaluation of Existing Building—N2 Method

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Abstract

The nonlinear static (pushover) procedure for estimating target displacement is incorporated in Annex B of Eurocode 8 and is commonly implemented through the N2 method, originally developed at the University of Ljubljana. While advanced nonlinear dynamic analyses can provide detailed insight into seismic structural behavior, they require sophisticated modeling and significant computational effort. In contrast, simplified procedures are often preferred in engineering practice for preliminary assessment and design verification. This study evaluates the applicability of the N2 method by comparing analytically obtained target displacements with experimentally measured responses of a three-story model building subjected to seismic excitation. The nonlinear capacity curve derived from pushover analysis is transformed into the acceleration–displacement response spectrum (ADRS) domain and evaluated against the demand spectrum to determine the performance point. The experimentally obtained value, reported in a reference study, is used as a benchmark for assessment. The results indicate that the N2 method provides an approximate but rapid estimation of target displacement. The findings highlight both the practical usefulness and the inherent limitations of the N2 approach when applied to structures.

Keywords: performance-based analysis; N2 method; nonlinear static analysis; pushover analysis; Eurocode 8; structural health monitoring; target displacement

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1. Introduction

Over the past century, seismic structural design has undergone significant evolution, as evidenced by a series of case studies on other load cases or actions, such as gravity, wind and traffic, etc. Initially, seismic design involved simple mass-proportional lateral forces resisted by elastic structural action, particularly after key earthquakes in the early 20th century, such as the Kanto earthquake, the 1933 Long Beach earthquake in the USA, and the 1932 Napier earthquake in New Zealand, which caused considerable structural damage [1]. Consequently, certain design codes began to incorporate seismic considerations, providing fundamental guidelines for designing structures in high-seismic areas. These codes initially offered a rough estimate for seismic stresses at 10% of a building's weight, without accounting for the structure's natural period. Subsequently, this force

was distributed across the building's height in proportion to the mass at each story [2]. The distribution of force across the building height is illustrated in Figure 1 [3].

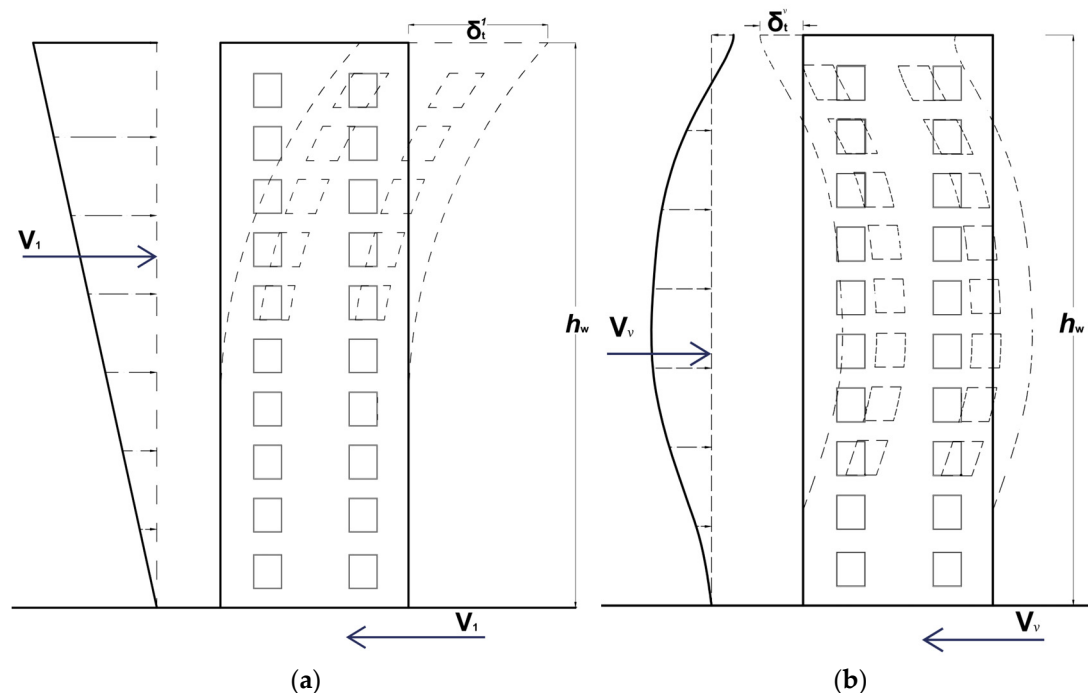


Figure 1. Lateral force across the building height for equivalent lateral force method according to mode shapes: (a) first mode and; (b) higher mode shape.

During the 1940s and 1950s, structural analysis still relied on the elastic structural response, but it began to consider the structural period when adjusting the intensity of inertia forces in design [1]. It was not until the 1950s that the dynamics of structures became abundantly clear, prompting researchers to start identifying the dynamic properties of each building, such as the period and overall stiffness.

Ductility considerations were first introduced in the 1960s and 1970s, prompted by practical and empirical evidence showing that well-detailed structures could endure ground shaking levels that generate inertia forces several times greater than those predicted by elastic analysis. To assess predicted performance, ultimate strength considerations were employed, using force levels that were arbitrarily deduced from elastic values in accordance with design codes from various seismically active regions [1]. When design lateral forces were determined based on the structure's period in the 1960s, inelastic time-history analysis emerged as an accurate method for estimating the structure's behavior under seismic loads. The development of the ductility concept, which illustrated that a building transitioning from elastic to inelastic behavior can dissipate seismic energy by enhancing its capacity to withstand large displacements, was reinforced by the finding that structures designed with elasticity in mind can endure inertia forces many times greater than those corresponding to the structure's strength [2,4].

In the 1980s, researchers pursued to determine precise ductility capacity values for various building types with different connection details for use in design codes. They relied on experimental research for each structural form and compared these findings with theoretical inferences. One notable example is the SPEAR building, as illustrated in Figure 2. The maximum safe displacement of the building, which could occur without causing significant damage to its structural components, was also established [2]. This gradually led to a growing awareness in the 1980s and 1990s that strength was important only insofar as it helped reduce displacements or stresses, which directly correspond to damage

potential. Consequently, deformations, rather than strength, should be the primary indicator of structural vulnerability [1]. In the mid-1980s, the N2 method was first introduced that is a nonlinear static procedure combining pushover analysis with an equivalent single-degree-of-freedom representation [5,6]. The fundamental concept was derived from the Q-model by Saiidi and Sozen in 1981 [7]. Over time, a more advanced version of this approach was developed [8], and its applicability was extended to include bridges [9].

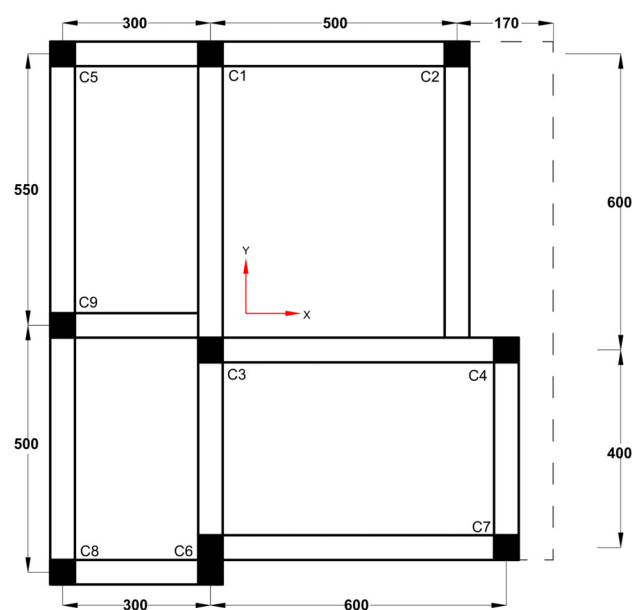


Figure 2. Plan view of the SPEAR building (all dimensions in cm).

As a consequence of this research, various seismic design philosophies have been formulated, many of which prioritize deformation capacity over strength. These design concepts are often termed “performance-based”, and numerous scholars have emphasized displacement-based evaluation rather than force-based approaches, as structural damage and performance degradation are more closely associated with deformation measures (dissipate seismic energy) than with force [2].

In this displacement-based design approach, the base shear force is calculated using the displacement response spectrum as the initial reference. This method also involves analyzing the structure while accounting for its inelastic phase. Force-based design (FBD) has been shown to present several issues, such as an incorrect R-factor and inaccurate estimates of the stiffness of the structure’s components. Consequently, it became essential to adopt an alternative design strategy to FBD, leading researchers to consider performance-based design (PBD) [10].

Performance-Based Design (PBD) stands as one of the most pivotal contemporary concepts in earthquake engineering. Unlike FBD, which often relies on empirical formulas, PBD seeks to analyze a building’s behavior under seismic forces using precise methodologies. Direct displacement-based design (DDBD) is a fundamental approach within PBD-based methodologies. This straightforward procedure is applicable to a wide range of structures, including masonry buildings, bridges, towers, dual wall-frame buildings, wall buildings, retaining walls, and reinforced concrete frame buildings. It is regarded as an accurate and well-equipped method that highlights the limitations of traditional FBD [10]. DDBD is one of the simplest design methodologies for analyzing multi-degree-of-freedom structures. In this approach, the structure is characterized by its secant stiffness and the corresponding elastic damping of a single degree of freedom structure. The core of this design lies in achieving a specified displacement limit state, determined either by

material strain limits or non-structural drift limitations as outlined in design codes under the design-level seismic intensity. By representing the structure with a replacement structure, many issues associated with FBD, which is currently used in most building regulations, are circumvented. Initial stiffness is employed to define an elastic period [2]. This paper thoroughly discusses the N2 approach, which utilizes a DDBD methodology.

The N2 approach was developed as a straightforward nonlinear method for the seismic analysis of structures. It combines the pushover analysis of a multi-degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) system. Presented in an acceleration-displacement response spectrum (ADRS) format, this method aids readers in graphically understanding the processes and connections between the key elements influencing seismic reactions. Inelastic spectra are used instead of elastic spectra with equal damping and period, highlighting the primary distinction between this method and the capacity spectrum approach. Additionally, demand quantities can be obtained directly without the need for iteration. When the structure primarily oscillates in the first mode, the N2 method generally provides reliable estimates; however, variations may still arise due to idealization and modeling assumptions, particularly when higher-mode contributions become significant.

Despite the extensive development and codification of the N2 method, most existing studies focus on analytical applications or numerical benchmarks, with limited direct comparison against experimentally obtained performance points. The present study contributes by evaluating the N2 method through comparison with experimentally measured seismic responses of a three-story model building. Nonlinear static analysis was performed using the ETABS V19 (CSI, Walnut Creek, CA, USA) to obtain the force–displacement (capacity) curve, which forms the basis for subsequent N2 evaluation. While the N2 procedure can be implemented within the software environment, the detailed step-by-step calculations for determining the target displacement in accordance with Eurocode 8 are explicitly presented to ensure methodological transparency and consistency. The model building was tested under nine peak ground acceleration (PGA) levels and for clarity, the complete target displacement calculation is demonstrated for a representative PGA of 0.4 g, noting that the same procedure applies to the remaining intensity levels. For each PGA level, the ADRS transformation, bilinearization, and performance-point determination were carried out independently using the same EC8-based assumptions after pushover analysis. The study focuses on the EC8-based target displacement evaluation to isolate and assess the methodological performance and limitations of the N2 approach.

2. Simplified N2 Method

It is widely recognized that the current seismic design process outlined in codes needs to improve. The structural engineering community has developed a new generation of design and rehabilitation methods, including performance-based design concepts. It is acknowledged that damage management must become a more explicit consideration in design [9]. Achieving this goal necessitates the integration of some form of nonlinear analysis into the seismic design technique. In the short term, combining the response spectrum technique with nonlinear static (pushover) analysis seems to be the most appropriate strategy. The capacity spectrum method, as used in ATC 40 [11], and the nonlinear static procedure, as employed in FEMA 273 [12], exemplify this strategy. The Displacement Coefficient Method, which is the latter approach, is an alternative procedure used in ATC 40 [11]. Another method is the N2 technique, developed at the University of Ljubljana, which stands for nonlinear analysis and two mathematical models.

In the mid-1980s, the N2 technique was initially developed [5,6]. Its fundamental concept was inspired by the Q-model created by Saiidi and Sozen in 1981 [7]. Over time, a more advanced version of this approach has been developed [8], and its applicability

has been extended to include bridges [9]. The N2 approach was presented by Fajfar in 1999 in the ADRS format, based on ideas from Bertero [13] and Reinhorn [14]. This version merges the visual representation benefits of Freeman's capacity spectrum method [15,16] with the solid physical foundations of inelastic demand spectra. Goel and Chopra have also utilized inelastic spectra in this context [17]. Essentially, the N2 approach is a variation in the capacity spectrum method, now based on inelastic spectra. A typical smooth elastic design spectrum is used to derive inelastic demand spectra, which can be related to the fundamental elastic spectrum using reduction factors consistent with the elastic spectrum. In pushover analysis, the assumed displacement form is linked to the lateral load pattern, allowing for a transparent conversion from a system with MDOF to an equivalent SDOF system.

The suggested method closely aligns with, and in some cases is even comparable to, the nonlinear static procedure outlined in FEMA 273 when a simple substitution for the reduction factor's spectrum is applied. The proposed method notably differs from the one developed by Reinhorn in 1997 primarily due to its simplicity. Reinhorn's approach is more open-ended and less restrictive. Several simplifications have been incorporated into the proposed N2 technique, imposing additional constraints. However, these allow the method to be formulated in a straightforward and user-friendly manner, which is beneficial for both the development of future design standards and practical design applications. The proposed N2 method could theoretically be considered a specific instance of the basic guideline described by Reinhorn. This study utilizes the description of the simple N2 method's steps applicable to planar models as outlined by Fajfar 2000 [18].

3. Case Study

3.1. Model Building

Buildings must perform as intended during earthquakes, considering various earthquake source characteristics such as severity and direction. Although crucial for predicting earthquake resilience, the mathematical modeling of framed-masonry buildings' behavior is often inadequate or unavailable. Consequently, the design relies on methodologies outlined in research literature [19–21], which may or may not align with specific design codes or regulatory requirements. Nonetheless, some design codes offer empirical formulas derived from extensive prior research [12,22]. Additionally, the impact on a structure can vary significantly due to differences in ground movement characteristics (earthquakes), which are influenced by geological conditions [23].

The model building selected for analysis in this study is compared with the simplest approach specified in Eurocode 8. This building was designed, cast, assembled, and tested at the IZIIS lab in Skopje, Macedonia. Both the scaled prototype and the model were constructed using the same materials, adhering to the Cauchy-Froude similitude principles, which are comprehensively discussed in other studies [24,25] and are beyond the scope of this work. To accurately simulate inertia forces, the total model mass was increased by adding steel ingots, with 4800 kg added to the top of each floor, resulting in a combined mass of 29,200 kg. According to the Cauchy-Froude similitude criteria, the specimen's period was extended by doubling its mass, and additional story masses in the form of steel ingots were added as previously mentioned. Considering the medium ductility form as outlined in Eurocode-8 for seismic construction detailing, the three-story, framed-masonry structure at a 1/2.5 scale was planned and built in accordance with EN1992-1-1:2004 and EN1998-1:2005 [21,26] specifications as a type of structure to be moment-resisting frames. The structure comprised two longitudinal frames with two bays, having center-to-center span lengths of 128 cm and 248 cm. One bay with a 220 cm center-to-center span length was present in the three transverse frames. The building's overall dimensions, including the foundation, were 460 cm by 280 cm by 390 cm (length by breadth by height).

The columns and beams were 16 cm deep and 12 cm wide. The reinforced concrete slab was 8 cm thick, and each level was 120 cm tall. The upper and lower regions of the slabs were reinforced with reinforcement meshes of type Q139 (4.2 mm diameter bars spaced 100 mm apart in both directions) and Q196 (5 mm diameter bars spaced 100 mm apart in both directions), respectively.

Hollow block masonry, constructed from clay units measuring $12 \times 25 \times 6.5$ cm (width, length, height) with 6 mm thick joints and a void volume of 68%, was used for the masonry infill walls, adhering to earthquake-resistant design standards for unreinforced structural masonry. The concrete utilized had a maximum aggregate size of 8 mm, and the C25/30 specification required a nominal cylinder compressive strength of 25 MPa (nominal cube strength of 30 MPa). The reinforcement, classified as Grade B500B, had a nominal yield stress of 500 MPa. Table 1 presents the characteristics of the construction materials, derived from experimental results, which have been simplified for computer model analysis. The first-story door openings in the masonry infill wall measured $40 \text{ cm} \times 104 \text{ cm}$, while the second- and third-story window openings were $53 \text{ cm} \times 67 \text{ cm}$, with each opening centered according to the bays. Figures 3 and 4 illustrate the test structure's appearance, measurements, cross-sections, and reinforcement plans for the beams and columns.

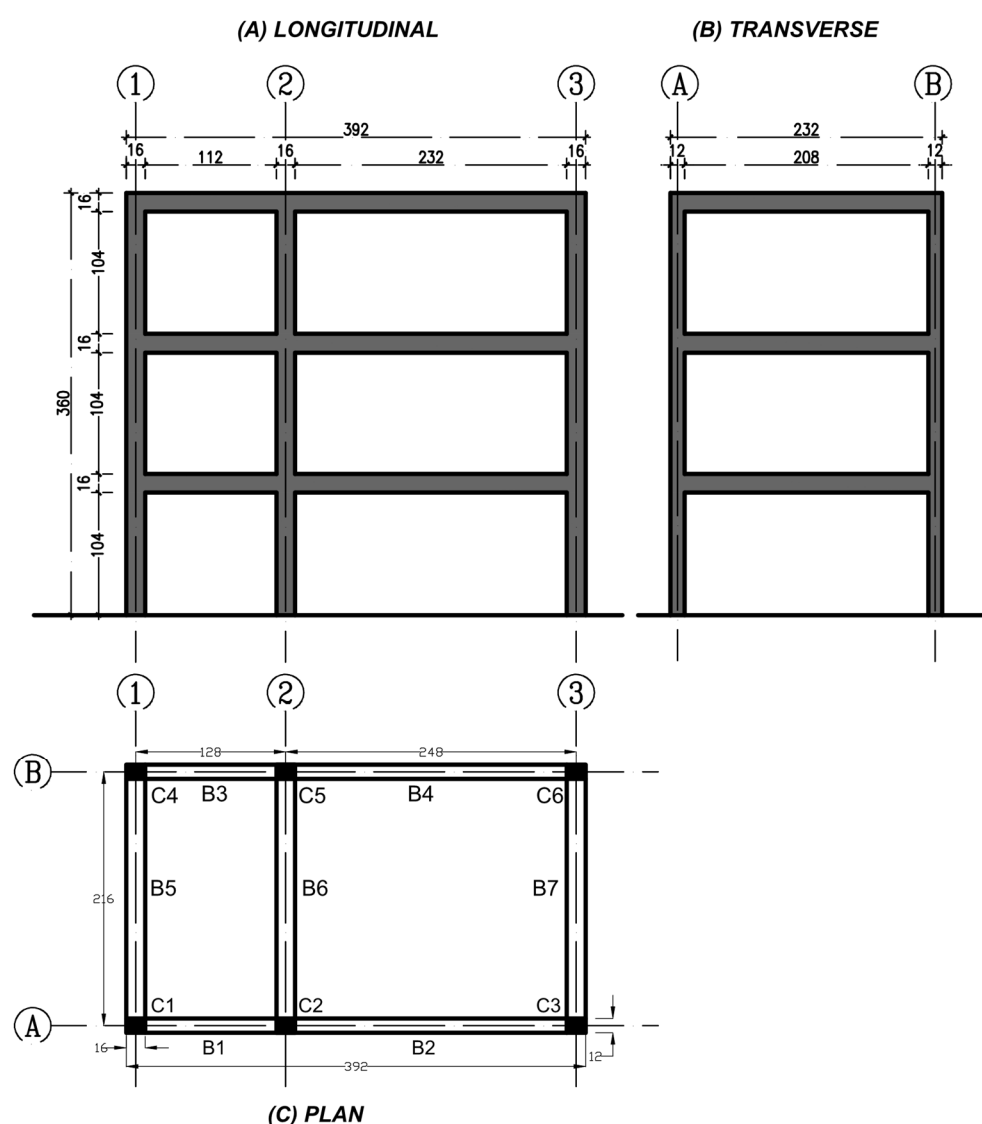


Figure 3. Structure views in the transverse, longitudinal, and plan directions (all dimensions in cm).

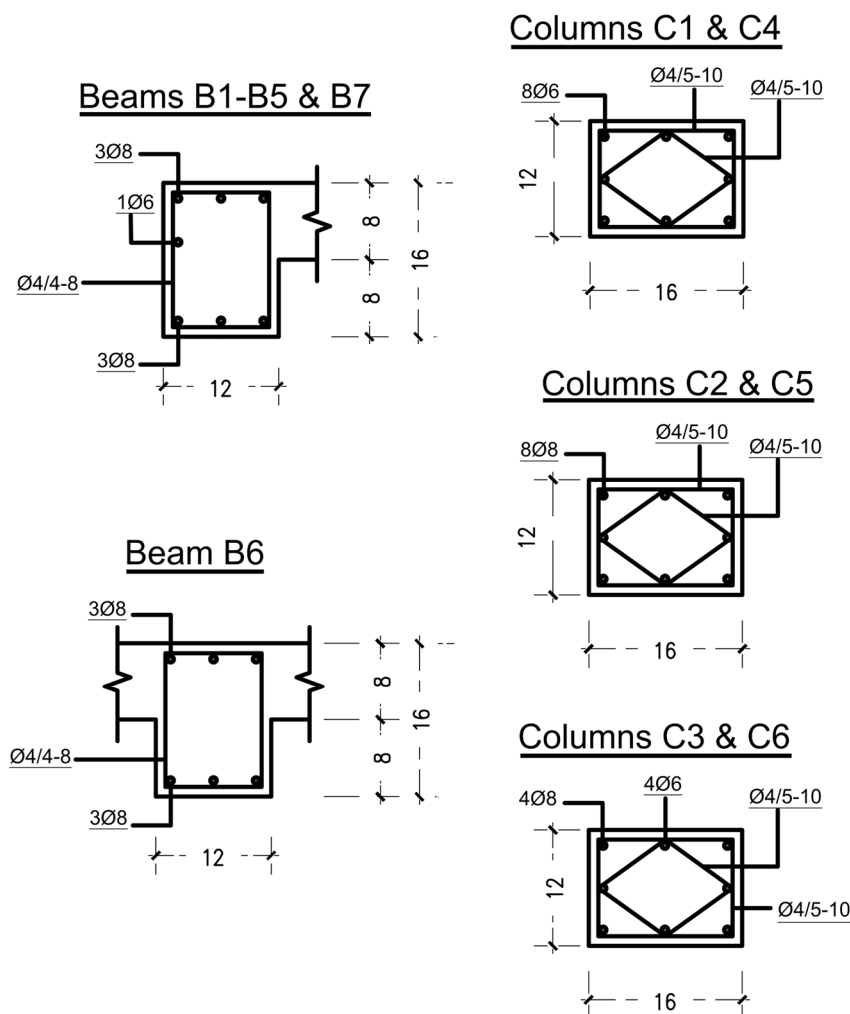


Figure 4. Details of beam and column reinforcement (units in cm except for bar sizes).

Table 1. Summary of model's data definitions and assignments.

Description		Selection	Remarks
Concrete Design code		Eurocode 2-2004	
Steel Design Code		Eurocode 3-2005	
Seismic Resistance Design Code		Eurocode 8-2005	
Concrete Grade	EN 1992-1-1:2004+ AC2:2010 (Clauses 3.1.2, 3.1.3, 9.2.1.1(1)) [26]	C25/30	$f'_c = 36.6 \text{ MPa}^*$
Concrete Weight		24.99 kN/m ³	
Concrete Modulus of Elasticity		31,476 MPa	38,839 MPa [*]
Concrete Poisson's Ratio		0.2	
Steel Grade	EN 1993-1-1:2005+ AC2:2009 (Clauses 3.2.1, 3.2.6) [27]	B500B	
Steel Weight		76.97 kN/m ³	
Steel Modulus of Elasticity (4 mm),		210,000 MPa	204,253.6 MPa [*]
(6 mm),			188,128.5 MPa [*]
(8 mm)			198,620.5 MPa [*]
Steel Minimum Yield Strength (F_y) (4 mm),	BS EN 10080-2005 [28]	500 MPa	753.5 MPa [*]
(6 mm),			563.5 MPa [*]
(8 mm)			590.5 MPa [*]

Steel Minimum Tensile Strength (Fu)	540 MPa	
Steel Expected Yield Strength (Fye)	550 MPa	
Steel Expected Tensile Strength (Fue)		
(4 mm),		780.4 MPa *
(6 mm),	590 MPa	588.9 MPa *
(8 mm)		620.8 MPa *
Beam Depth × Width	160 mm × 120 mm	
Beam Area of steel (−ve and +ve)	200 mm ² and 150 mm ²	Refer Figure 4
Beam Cover (top and bottom)	15 mm and 10 mm	
Column Depth × Width	160 mm × 120 mm	
Column steel	8, #10 Bars	Refer Figure 4
Column Confinement steel	#4 @ 50 mm	
Column Clear Cover	7 mm	
Slab Thickness	80 mm	
Slab Type	Shell thin	
Diaphragm	Rigid	
Mass Source	Self weight + superimposed dead load	Refer Section 3.2.1
Support Type	Fixed Support	
Modal Load case Type	Eigen	
Nonlinear dead load case type	Nonlinear static	
Nonlinear dead loads Applied	Mode shape in x-direction	Initial condition of pushover analysis
Pushover case type	Nonlinear static	Displacement control
Response Spectrum	PGA, ag = 1 g Damping = 5% Ground Type = D Spectrum Type = 2 Importance Class = II Behavior Factor, q = 1.5 Lower Bound Factor = 0.2	Refer Figure 5b, Spectral Acceleration (User defined to match with previous studies)
Time History	1979 Montenegro Earthquake's N-S component recorded at Herceg-Novi station	Time duration divided by $\sqrt{2.5}$ and matched to response spectrum applied in X-direction of model building.
PGA	0.4 g	This case serves as the reference for subsequent calculations
Beam Hinges Location (w.r.t length)	0 and 1	
Beam Hinges Type	Lumped, M3	
Beam Hinges Acceptance criteria	Auto Generated	
Column Hinges Location (w.r.t length)	0 and 1	
Column Hinges Type	Fiber, P-M2-M3	

* Laboratory-tested material properties [24,25].

3.2. Loading Conditions

3.2.1. Gravity Loads

To accurately simulate the inertia forces, the total model mass was incorporated using steel ingots, with 4800 kg placed atop each story of the three-story, framed-masonry building at a 1/2.5 scale. This approach adhered to the Cauchy-Froude similitude criteria. The building's total mass, including the foundation, was 29,200 kg. Steel ingots served as additional narrative masses to double the specimen's weight and extend its time period.

To replicate the aforementioned criteria in the computer model, an additional dead load of 540 kg/m² was applied to the surface area (~9 m²) of each floor as a superimposed dead load, alongside the structural self-mass (see Table 2). The mass source is defined as the structural self-weight plus the superimposed dead load, as summarized in Table 1, ensuring consistent mass participation in the dynamic analysis.

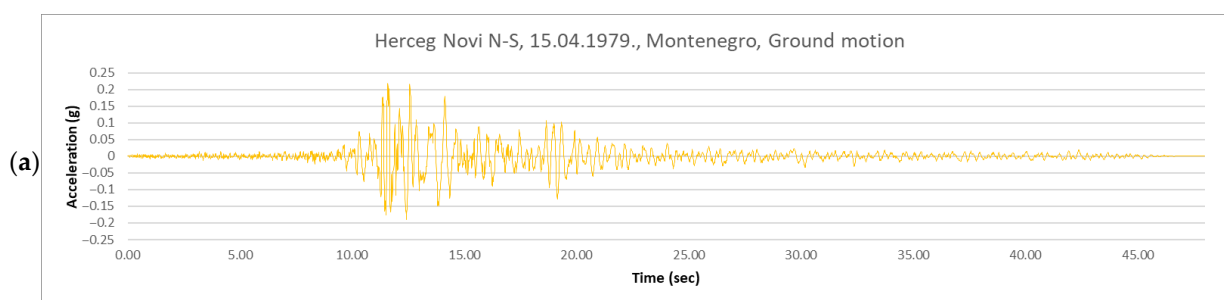
Table 2. Summary of masses of model building and Required Weight of Planar Computer Model [25,29].

Structural Element	Mass of Model Building (kg)	Required Mass of Computer Model (2D) (kg)	Mass of Computer Model (2D) (kg)
Foundation (<i>not required for the analysis</i>)	4700	-	
Mass of the frame structure	7400	7400/2	
Mass of the masonry infill	2700	-	
Mass of the superstructure	10,100	7400/2	10,900
Additional masses (12 steel ingots per floor, 400 kg each)	14,400 ¹	14,400/2	
Total mass	29,200	10,900	

¹ Steel ingots totaling 4800 kg per story (12 ingots × 400 kg) are distributed over the slab area of each floor of the three-story structure (see Section 3.2.1).

3.2.2. Earthquake Load

The N-S component of the 1979 Montenegro earthquake, as recorded at the Herceg-Novi station, served as the input excitation (see Figure 5a). To comply with standard similitude procedures, the duration of the excitation was shortened by reducing the time step by a factor of $\sqrt{2.5}$. The excitation's amplitude was scaled using the peak ground accelerations (ag/g). For comparison with available results, the model building received base excitation in its longitudinal (or x) direction at an intensity of $a_g = 0.4$ g. The graph shown in Figure 5b was recreated using the properties listed in Table 1 for use with a computer model.



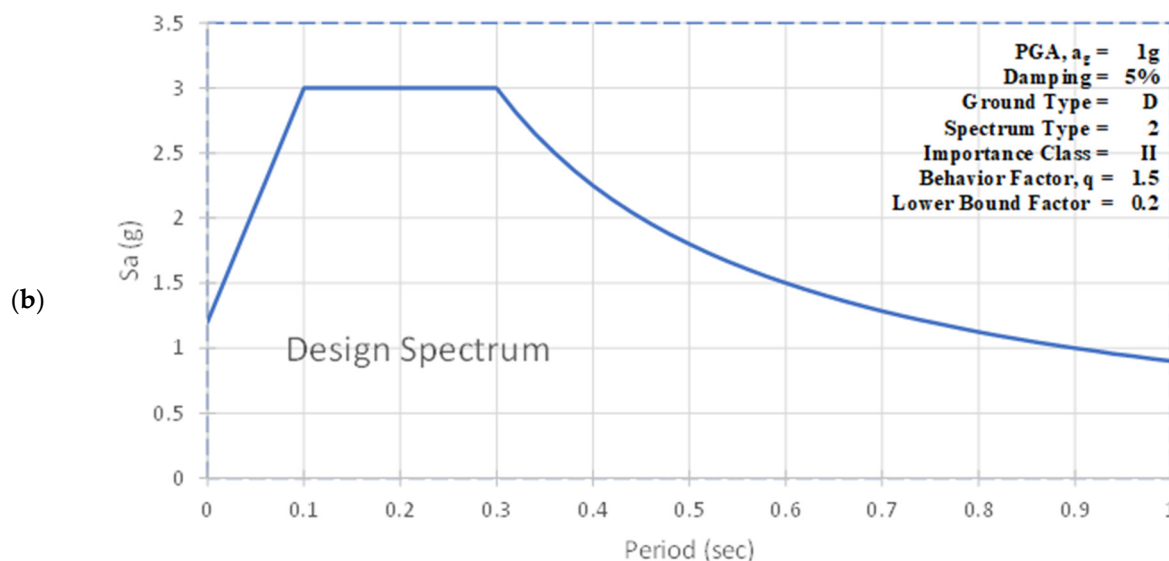


Figure 5. (a) The data recorded from the Herceg-Novi station during the 1979 Montenegro earthquake's north–south component before dividing the time step by $\sqrt{2.5}$; (b) Linear response spectra and its properties according to Eurocode 8 for Computer model.

4. Modeling Parameters

The following data are essential for input into ETABS, while the remaining information is automatically generated by the software based on the relevant codes and guidelines. Experimentally obtained material properties adopted from large-scale laboratory testing are used for performance-based nonlinear analysis. Detailed testing procedures and specimen information are documented in the referenced studies and are beyond the scope of the present work.

Furthermore, the condition of a structure can be evaluated through nonlinear analysis by examining the status of its hinges. ETABS offers various types of hinges, such as flexural, shear, axial, and axial plus flexural hinges, which can be specified and assigned different roles as needed. For instance, in the following case study, axial hinges are used for struts, while flexural hinges are applied to columns and beams. Generally, coupled axial-flexural (P-M) hinges are recommended for modeling columns. To simplify the modeling approach, shear hinges are omitted from beams and columns. Shear verification is not performed in this study and is instead identified as a post-processing consideration for designers, should it be required by specific performance objectives. Different hinge formulations were adopted to reflect the expected inelastic behavior of frame members. Beams were modeled using lumped plasticity M3 hinges, as their response is dominated by flexural yielding with localized plastic rotations. Columns were modeled using fiber-based P-M2-M3 hinges to capture the combined effects of axial force and biaxial bending, which are critical for accurately representing column behavior in nonlinear analysis.

The fiber P-M2-M3 hinge is considered the most realistic representation, although it may demand more processing time and memory. Each material within the hinge has several available hysteresis models. In our model building case, the hinge will adhere to the hysteresis models designated for concrete and steel.

5. Results of the Analysis

While the N2 method is designed for 2D planar models, it is beneficial to first construct a 3D spatial model and then validate the 2D planar model against it. The 2D planar model can be developed by selecting the necessary plane and transferring the load. This

is achieved by dividing the surface load of the corresponding bay in half and adding the slab's self-load, resulting in a line load distributed along the frame's beam line. The compatibility between planar and spatial models is demonstrated by comparing base reactions, ensuring the correct distribution of loads. The modal analysis of the planar model confirms that the natural frequency remains unchanged, exhibiting the same dynamic behavior as the spatial model. The results can then be compared for further analysis.

5.1. Dynamic Properties of Structure

The modal periods and mass participation ratios for both the 2D planar and 3D spatial models are shown in Table 3 for comparison. By examining the time periods of the 2D planar model and the 3D spatial model along the X-axis, or the longitudinal direction of the model building, it is evident that both structures share the same natural period. This parameter ensures that the 2D model possesses the same dynamic characteristics as the 3D model before performing the N2 method calculation. Due to the geometric symmetry and uniform mass–stiffness distribution, torsional modes exhibit low mass participation and do not significantly influence the global response.

Table 3. Comparative table for modal periods and mass participating ratios (UX) for 2D planar model and 3D spatial model.

Mode	Period (S)	UX	Mode	Period (S)	UX
2D Planar Model			3D Spatial Model		
1	0.221 ¹	0.8695	2	0.227 ¹	0.8667
2	0.071	0.106	5	0.072	0.1056
3	0.042	0.0244	6	0.06	0.0006

¹ First translational mode.

5.2. Push-Over Curve

The comparative graph illustrating base shear versus monitored displacement for the 2D planar and 3D spatial models is shown in Figure 6. Notably, the 2D planar model in the graph demonstrates more ductile behavior than the 3D spatial model, which is one of the graph's most striking features. This characteristic significantly influences the calculation of the performance point and may explain why the N2 method, in its simplest form, is applicable to planar models. Furthermore, the maximum base shear of the 2D planar model is only half that of the 3D spatial model. Since the plane under consideration represents exactly half of the model building, this confirms that the stiffness of 2D planar structures accurately reflects the model building.

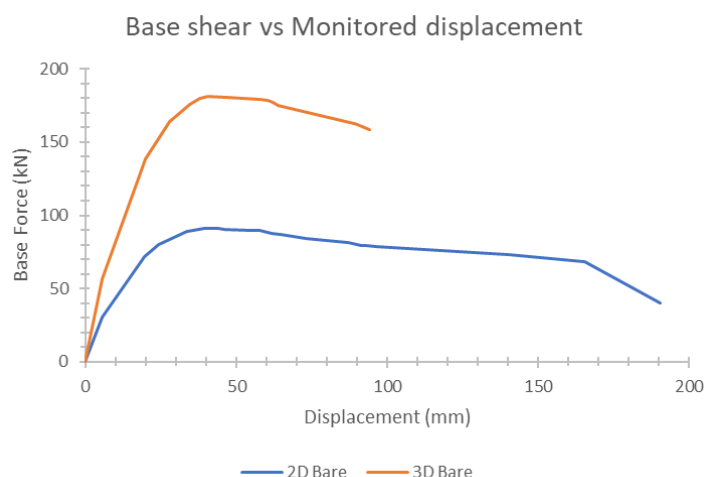


Figure 6. Base shear versus monitored displacement for the 2D planar and 3D spatial models.

5.3. Story Masses

The comparison of story masses between the results of a 2D planar and a 3D spatial model is presented in Table 4.

Table 4. Story masses of 2D planar and 3D spatial model.

Story	Diaphragm	Story Mass of 2D Bare Frame (kg)	Story Mass of 3D Bare Frame (kg)	Difference (%)
Story1	D1	3662.8	7418.6	50.1
Story2	D2	3662.8	7418.6	50.1
Story3	D3	3574.73	7002.45	50.1
Sum of Masses		10,900.33	22,259.65	50.1

As shown in Table 4, the comparison of the sum of story masses between the 2D planar and 3D spatial models reveals that the loads on the 2D planar model are exactly half of those on the 3D spatial model. This also ensures that the load is evenly distributed across the stories. Furthermore, Table 2 compares the overall mass of the model building with that of the 2D flat model. The difference, as observed, is negligible.

6. Application of the N2 Method to the Selected Model Case Study

In the realm of seismic analysis for structures, the N2 approach stands out as a relatively straightforward nonlinear method. It combines the pushover analysis of a multi-degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) system. Presented in an acceleration-displacement response spectrum (ADRS) format, this approach allows for a clear visual understanding of the steps and the interrelationships among the key factors affecting seismic response. Unlike the use of elastic spectra with equal damping and period, this method employs inelastic spectra, marking a significant distinction from the capacity spectrum method. Moreover, it eliminates the need for iteration to determine demand numbers. When a structure predominantly oscillates in the first mode, the N2 method typically yields reliable results, though variations may still arise due to idealization and modeling assumptions. However, there are additional constraints to consider. The N2 method is applied during the following phases of calculation.

6.1. Input Data, Pushover Analysis, Seismic Demand in AD-Format

A planar MDOF structural model is utilized, requiring nonlinear force-deformation relationships for structural components under monotonic loading, as illustrated in Figure 6, in addition to the data necessary for standard elastic analysis. To simulate the inertial forces experienced during ground shaking, the structure undergoes a sequence of lateral forces that increase monotonically, a process known as a nonlinear static (pushover) study. Throughout this process, gravity loads remain constant. As lateral loads gradually increase, different structural components yield sequentially, resulting in a progressive loss of stiffness in the structure. The beam element with concentrated plasticity at both ends is the most commonly used element model and has been applied in this case. The prevalent moment-rotation relationship is either bilinear or trilinear. Traditional definitions of seismic demand are expressed in terms of an elastic (pseudo)-acceleration spectrum S_{ae} , where spectral accelerations are presented as a function of the structure's natural period T , as shown in Figure 7.

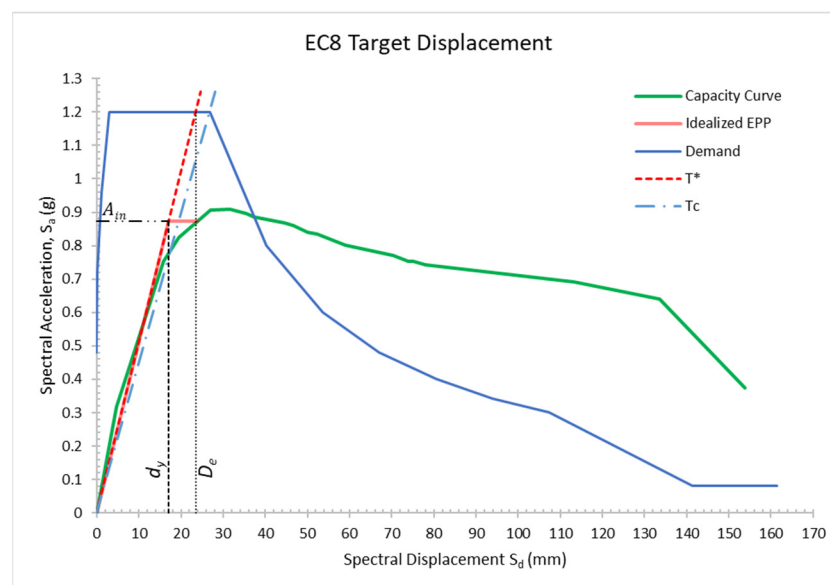


Figure 7. Acceleration Displacement Response Spectrum (ADRS) and Bi-Linear Graph.

Figure 7 represents the essential core of the N2 method and is presented in the acceleration displacement response spectrum (ADRS) format, in which the relevant response quantities are expressed in terms of spectral acceleration and spectral displacement. The green curve represents the capacity spectrum, while the red curve is the equivalent bilinear idealization of the capacity spectrum. This system interacts with the blue curve, which is the demand spectrum, to calculate the performance point. The bilinear idealization shown in Figure 7 is derived from the pushover capacity curve following the Eurocode 8 Annex B bilinearization rules.

Summarized Data

$$g = 9.8167 \frac{\text{m}}{\text{s}^2}$$

$$a_g = 0.4 \text{ g}$$

$$m_1 = 3.663 \text{ tonne}, \quad m_2 = 3.663 \text{ tonne}, \quad m_3 = 3.575 \text{ tonne}$$

Define 1st Modal Displacement Shape and Define Mass Matrix:

$$\Phi = \begin{bmatrix} 0.3455 \\ 0.7484 \\ 1 \end{bmatrix} \quad M = \begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} = \begin{bmatrix} 3.663 & 0 & 0 \\ 0 & 3.663 & 0 \\ 0 & 0 & 3.575 \end{bmatrix} \text{ tonne}$$

For the reference case (PGA = 0.4 g), the governing bilinear parameters used in the N2 evaluation are illustrated in Figure 7 as follows: yield displacement of the idealized SDOF system $d_y = 17.106$ mm, elastic displacement demand $D_e = 23.518$ mm, elastic spectral acceleration demand $A_e(T^*) = 1.20$, and inelastic spectral acceleration demand $A_{in} = 0.873$. These parameters were obtained from the pushover capacity curve following the Eurocode 8 Annex B bilinearization rules.

6.2. Equivalent SDOF System

By employing inelastic response spectra, the N2 approach determines seismic demand, explicitly accounting for inelastic behavior. Consequently, the structure is conceptually represented as an SDOF system. The transformation factor Γ (also called modal participation factor) is calculated such that its value at the roof equals 1.0. Γ facilitates the conversion between the MDOF model and the SDOF model, allowing both displacements and forces to be transformed using the same Γ value.

$$\Gamma = \frac{m^*}{\Phi^T \cdot M \cdot \Phi} \quad (1)$$

where m^* is the analogous SDOF system's mass represented by the numerator:

$$m^* = \Phi^T \cdot M \cdot 1 \quad (2)$$

So, using Equations (1) and (2),

$$m^* = \Phi^T \cdot M \cdot \begin{bmatrix} 1 \\ 1 \\ 1 \end{bmatrix} = 7.582 \text{ tonne}$$

$$\Gamma = \frac{m^*}{\Phi^T \cdot M \cdot \Phi} = 1.25$$

$$f_y = (A_{in} \cdot g) \cdot m^* = 64.977 \text{ kN}$$

The period T^* of the idealized SDOF system is computed as:

$$T^* = 2 \cdot \pi \cdot \sqrt{\frac{m^* \cdot d_y}{f_y}} \quad (3)$$

$$F_{t@yielding} = f_y \cdot \Gamma = 81.244 \text{ kN} \quad (4)$$

$$D_{t@yielding} = d_y \cdot \Gamma = 21.388 \text{ mm} \quad (5)$$

6.3. Seismic Demand for the SDOF System

Elastic demand spectra can be used to calculate inelastic demand spectra in AD format.

Comparing Figure 8 with the graph in Figure 7 illustrates the intersection between the elastic demand spectrum and the line representing the equivalent SDOF system's infinite elastic period ($T^* = 0.281$ s).

$$A_e(T^*) = 1.2$$

$$D_e = 23.518 \text{ mm}$$

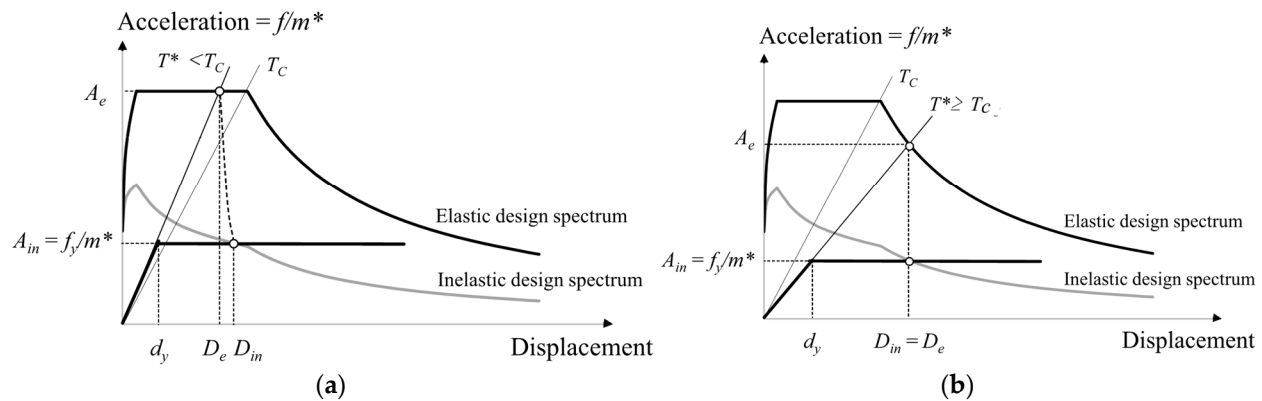


Figure 8. (a) Determination of the seismic demand for the equivalent SDOF system with the period in the short-period range; (b) Determination of the seismic demand for the equivalent SDOF system with the period in the medium/long-period range.

The reduction factor R_μ can be determined by calculating the ratio of forces between the elastic and inelastic systems.

$$R_\mu = \frac{A_e(T^*)}{A_{in}} \quad (6)$$

If the time T^* exceeds or matches the characteristic period of the ground motion T_c , the ductility demand becomes equal to the ductility-related reduction factor ($\mu = R_\mu$), and the inelastic displacement demand D_{in} matches the elastic displacement demand D_e , thus making the equal displacement rule applicable.

Since, $T^* = 0.281 \text{ s} < T_c = 0.3 \text{ s}$,

So, the ductility demand can be determined if the system's period is less than T_c from:

$$\mu = (R_\mu - 1) \cdot \frac{T_c}{T^*} + 1 = 1.4 \quad \text{Fajfar, 2000} \quad (7)$$

The definition of ductility can be used to derive the inelastic displacement demand D_{in} :

$$D_{in} = \frac{D_e}{R_\mu} \cdot (1 + (R_\mu - 1) \cdot \frac{T_c}{T^*}) = 23.958 \text{ mm} \quad (8)$$

6.4. Seismic Demand for the MDOF System

The displacement demand of the equivalent SDOF system, $D_{in} = d$, is multiplied by the transformation factor Γ to derive the displacement demand for MDOF systems, also known as the target displacement, d_r . The structure is then pushed to at least this target displacement (d_r) when subjected to lateral stresses that increase monotonically following a fixed pattern. The determination of relevant local quantities, such as story drifts, chord rotations, and internal forces corresponding to d_r , is based on the assumption that the distribution of deformations throughout the structure in the static (pushover) analysis closely resembles that which would be obtained in dynamic analyses.

$$d_r = D_{in} \cdot \Gamma = 29.956 \text{ mm} \quad (9)$$

6.5. Performance Evaluation

The target displacement for the roof, determined in previous steps, is 29.96 mm under the 0.4 PGA of the anticipated response spectrum. The next step involves repeating the pushover analysis up to this target displacement and extracting all necessary information for future design use. Below are some important aspects to consider.

6.5.1. Location and Response of Hinges

One of the most significant outcomes of nonlinear modeling is identifying the location of hinge formation. Under the anticipated load of 0.4 g PGA in the X-direction, all beams on the first and second floors are stressed and have only reached the damage limitation (DL) limit state, as shown in Figure 9a. While the columns of two stories lying opposite to the load application direction have reached the DL limit state, those under compression load or in the direction of force have not reached any limit states. Overall, according to the N2 method's results, the model is considered safe under the anticipated load of 0.4 g PGA in the X-direction. Under this load, Figure 9c illustrates the plastic hinge response of column C11 at the 1st story, and Figure 9d illustrates the plastic hinge response of column C13 at the 3rd story. The plastic hinge response of the first story column clearly demonstrates yielding, leading to the creation of the limit state warning, while the third story column's response is smooth. For instance, the location and response of hinges related to target displacement for the roof have been determined from previous steps to be 57.001 mm under the 0.8 g PGA of the anticipated response spectrum. Column C21, as shown in Figure 9b, has reached the near collapse (NC) limit state, whereas column C11 has reached the significant damage (SD) limit state. The figure illustrates the plastic hinge response of column C21.

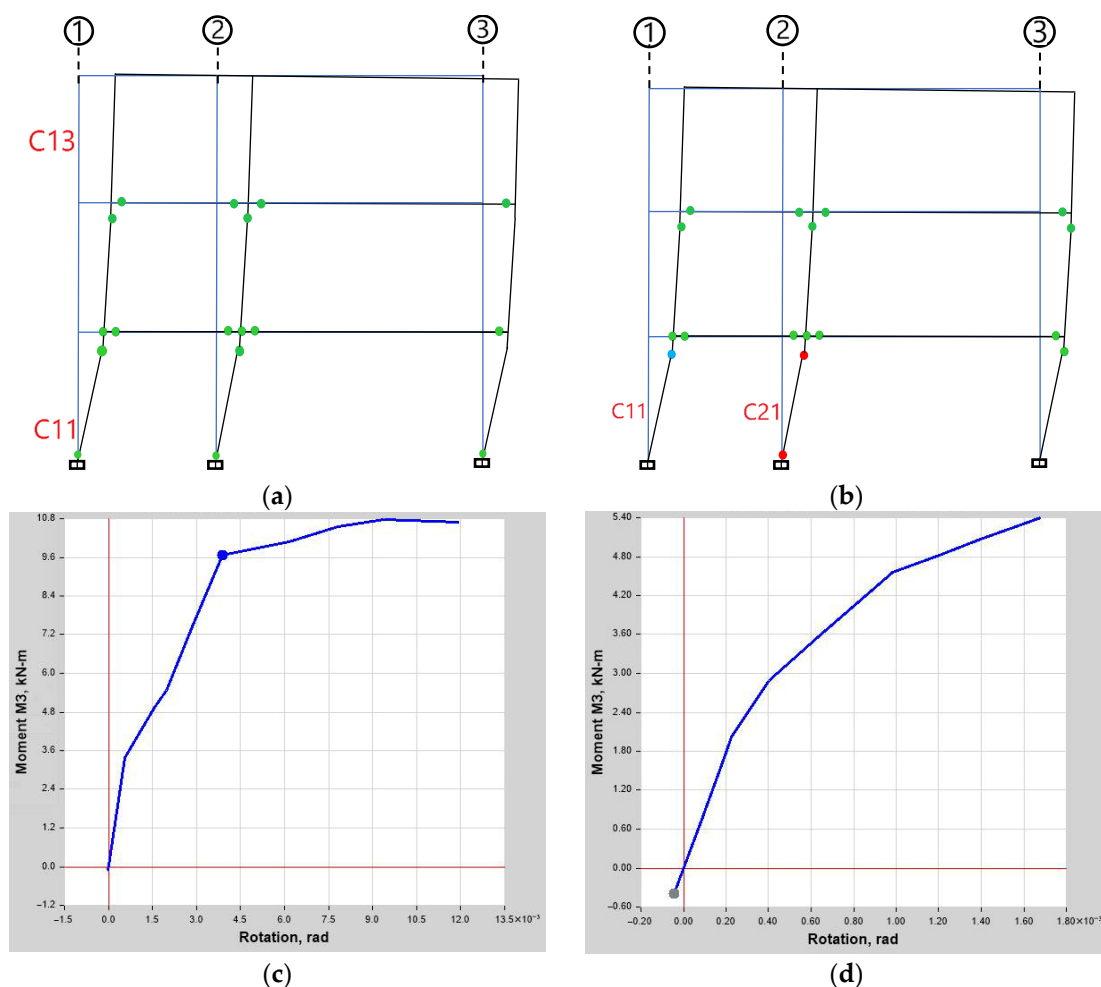


Figure 9. (a) Location of hinges formation under loading of 0.4 g PGA; (b) Location of hinges formation under loading of 0.8 g PGA; (c) plastic hinge response of column C11 at 1st story under 0.4 g PGA; (d) plastic hinge response of column C13 at 3rd story under 0.4 g PGA.

6.5.2. Member Forces

To confirm the validity of the frame design according to design guidelines, it is essential to assess it at the target displacement. To verify the frame structure's design, utilize the member stresses obtained from the pushover analysis at the desired displacement, either alone or in combination with other loads. While explicit member demand–capacity ratios are not reported herein, the procedure outlined allows such checks to be performed as a post processing step when required by design objectives.

7. Evaluation and Comparison of the Results

The table below presents the in-plane drift data for model building, derived from string potentiometers (SP) and x -axis accelerometers (ACC-X), based on research conducted at the IZIIS laboratory in Skopje, Macedonia. For comparison, the table also includes the target displacements estimated using the N2 approach against the anticipated loading. For each peak ground acceleration (PGA) level reported in Table 5, the target displacement was obtained by independently applying the same N2 procedure. A nonlinear static (pushover) analysis was performed to derive the structure's capacity curve, which was transformed into the ADRS format for each case and idealized following the Eurocode 8 Annex B provisions. The performance point was then determined using the scaled demand spectrum for the respective PGA. While the detailed calculation is illustrated for $a_g = 0.4$ g, the same workflow was consistently repeated for all remaining intensity levels.

Table 5. Evaluation and comparison of the results.

Intensity a_g/g (g)	Max. Displacements in X-Direc- tion of Model Building [24].	Target Displacement Calcu- lated by N2 Method
	(cm)	(cm)
0.1	0.176	0.54
0.2	0.289	1.23
0.3	0.371	2.07
0.4	0.57	2.96
0.6	0.846	4.53
0.7	1.701	5.14
0.8	1.938	5.89
1	3.361	6.92
1.2	4.632	7.89

8. Conclusions

In this study, nonlinear static (pushover) analysis was performed on a planar nonlinear model of a three-story building, and the resulting force–deformation relationship was used to estimate the seismic performance point through the simplified N2 method. The analytically obtained target displacements were compared with experimentally measured responses of the model building as represented in Table 5 and Figure 10. The comparison demonstrates that the N2 method is significantly more time efficient than detailed nonlinear dynamic analyses. However, it systematically overestimates the target displacement for the investigated structure, resulting in conservative predictions. Although the structure exhibits predominant first-mode behavior, variations between the N2 estimates and experimental results can be attributed to inherent simplifications of the N2 method, including idealized bilinearization and fixed displacement–shape assumptions. The results further show that modeling assumptions such as planar idealization and nonlinear hinge representation can influence the derived capacity spectrum and, consequently, the

estimated performance point [30]. Therefore, careful modeling and verification of dynamic characteristics are essential when applying the N2 method. Overall, the study confirms that the N2 approach is suitable as a rapid preliminary assessment tool, but its applicability should be confined to structures with predominant first-mode dominance, and its limitations should be recognized in performance-based evaluations. Moreover, it is recommended for future studies that analytical predictions obtained using the N2 method be validated against experimental results or structural health monitoring data, particularly for taller structures.

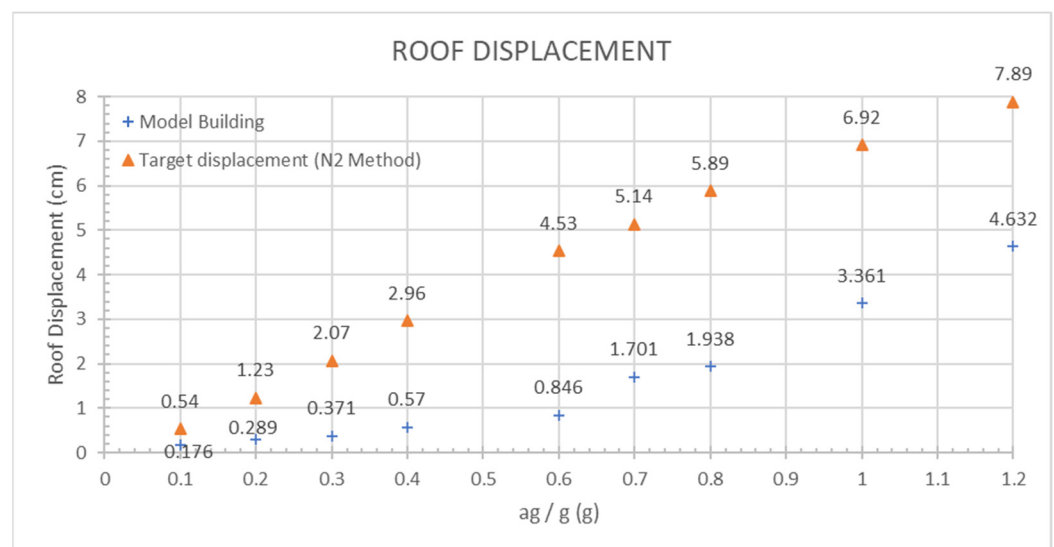


Figure 10. Comparison of roof displacement of model building vs. target displacement (N2 method).

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Abbreviations

The following abbreviations are used in this manuscript:

f'_c	Concrete compressive strength
g	Acceleration due to gravity
a_g	Peak Ground Acceleration
m_1, m_2, m_3	Story Masses
f_y	Yield strength of the idealized SDOF system
d	Displacement equivalent of SDOF system of the f - d relationship

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