

Article

The Influence of Numerical Model Refinement on the Prediction of the Spatial Formation of Plastic Hinges in a Moment-Resisting Frame

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Featured Application

This study guides structural engineers in selecting appropriate numerical models for accurate post-earthquake assessment of steel structures. The findings can be directly applied in anti-seismic design and to optimize sensor placement in Structural Health Monitoring (SHM) systems for detecting and monitoring plastic hinges.

Abstract

Ductile behavior of the materials applied in seismic structural design is highly appreciated. During damaging response, the structures shall form well controlled plastic hinges. The localization of these hinges is critical for ensuring the integrity of beam–column connections. However, solving this task creates a numerically exhausting problem, requiring various levels of precision modeling. This study investigates the variation in plastic hinge spatial formation using a single-story steel frame numerical model with three distinct numerical environments: SAP2000 (lumped plasticity), SeismoStruct (distributed plasticity), and Abaqus (3D solid modeling). Nonlinear static pushover and nonlinear time history analyses (NTHA) were performed across varying seismic intensities. The results indicate discrepancies between simplified engineering models and the 3D solid model and reveal that the plastic hinge center position (x_{PH}) is not stationary; its location fluctuates based on the intensity of excitation and the extent of local flange buckling. Furthermore, the inter-story Drift Ratio (IDR) analysis shows the lumped model tends to underestimate peak displacements in relation to the solid model by even 26%. This research emphasizes the need for a reasonable selection of a numerical model to perform the analyses required for SHM system design, as well as implications for damage indication and frame assessment after a seismic incident.

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1. Introduction

Steel moment resisting frames (SMRFs) are commonly used in seismically active regions due to the good ductility of their connections and their ability to dissipate energy [1–4]. Moreover, the connections used in such structures are also evaluated under cyclic

loading conditions, taking into account cases with steel and composite elements [5]. Current standards, e.g., the European standard EC8-1 [6] or the American standard AISC 341 [7], require the application of the ‘strong column–weak beam’ (SCWB) principle, which keeps the columns in an elastic state whilst forcing the formation of plastic hinges in the beams. In these zones, the cross-section has reached its design elastic capacity and allows further rotation of the joint within the plastic range. As the intensity of the loading increases, the hinges that form dissipate energy by increasing the plastic deformations of the structure [1].

The critical areas of steel moment frames are the connections, which determine the energy dissipation capacity of the entire structure. Beam-to-column connections are classified primarily according to the method and location of forced yielding. Non-reduced beam sections (nRBS), commonly used until the 1990s, connect a beam of constant cross-section directly to the column face. Following the Northridge earthquake, alternative solutions were introduced, such as reduced sections (RBS, in which local weakening of the flanges moves the hinge away from the column face) and pre-qualification connections, where the cross-section at the column is reinforced rather than weakened to induce yielding further from the weld [8]. Kim and Kim [9] point out that the type of connection—standard, RBS, or flange-reinforced—has little effect on the overall load-carrying capacity of the frame but does influence ductility and the failure mechanism under progressive collapse conditions. This paper focuses on nRBS connections, in which—unlike in RBS connections—the location and extent of the hinge are not predetermined by the cross-section geometry.

Connections of nRBS beams, due to the lack of a clear location for the plastic hinge, challenge the analysis of Structural Health Monitoring (SHM), where it is essential to determine the location, correctly identify, and, ultimately, assess the structure’s condition after a seismic event. After 1994 (the Northridge earthquake), numerous moment frames were equipped with accelerometers for dynamic response analysis and preliminary assessment of the structure’s condition [10]; however, as noted by Li et al. [11], global methods are insufficient for localizing damage in specific elements. Consequently, local methods have been developed based on strain sensors in the area of the expected hinge, such as piezoelectric sensors along the beam flange that detect local damage from the difference in strain amplitude before and after a shock [12] and methods for estimating the initial and final stiffness of members [11,13]. Similarly, Iyama et al. identified cracks in members primarily through measurements of strain and displacement [14] and secondly by using sensor arrays distributed across the structure to calculate the distribution of moments within the frame [15]. Estakhri and Nikkhoo [16] emphasize that the mechanisms of formation and the locations of plastic hinges in moment frames are key issues in SHM, as they determine which structural elements require priority monitoring. Therefore, preliminary analyses of the location of the plastic hinge and its behavior for a given system are necessary and should be carried out during the structural design stage. The design of the SHM system depends on these analyses, as the proper placement of sensors enables early damage detection and rapid response.

The type of the adopted numerical model has a significant impact on the accuracy of structural response representation and thus on the effectiveness of analyses carried out for SHM systems [10,14]. In practice, models based on beam, shell, and solid elements are used; while beam models based on Euler–Bernoulli theory, Timoshenko theory, or fiber-type cross-section discretization are commonly used in global analyses, shell and solid models are used to more accurately represent local deformations and plasticity zones. The studies comparing the impact of the type of elements used in numerical analyses, e.g., [17–19] mainly concern reinforced concrete structures rather than steel ones. In nonlinear modeling, a distinction is made between two approaches: the Concentrated Plastic Hinge

(CPH) approach and the distributed plasticity (DP) approach. While in the first approach many levels of modeling refinement can be applied, from the application of a rotational spring [20] up to the addition of, e.g., a Krawinkler panel zone [21], it simplifies damage localization to a single location. On the other hand, the DP approach allows the development of the plastic zone to be observed along the member [22,23]. From an SHM perspective, this means that the accuracy of damage localization increases with the level of detail in the model, but at the same time, the computational cost rises.

These differences are clearly illustrated by comparative studies carried out in various computational environments, although they mainly refer to reinforced concrete structures. Analyses carried out in OpenSees and SeismoStruct on a reinforced concrete frame have shown that models based on concentrated hinges can yield very similar results, whilst with fiber models, more noticeable discrepancies arise due to different material models, methods of cross-section definition, and damping schemes [24]. Similarly, comparisons of models carried out in SAP2000 and OpenSees have shown that different methods of modeling plasticity and section stiffness lead to different pushover curves and different predictions of load-bearing capacity degradation [25]. In reinforced concrete structures, analogous comparisons of CHP and DP models have shown that the level of idealization affects not only the values of forces and displacements, but also the mechanism of plasticity initiation and propagation, which is also relevant for steel structures [26–28]. Dides and de la Llera [29], in their studies on the response of asymmetrical buildings, note that CPH models perform better in steel than in reinforced concrete and that the level of discretization has the greatest influence on the accuracy of CPH and DP models. While the already mentioned studies (especially [27,29] and Alzaben et al. [30]) effectively compare modeling techniques, they focus on global response and capacity curves. None of the references addresses the impact of plastic hinge migration on SHM sensor placement guidelines.

Multi-purpose FEM software such as Abaqus is widely used in research on steel joints and connections, as it enables local phenomena to be modeled with high accuracy. This software is used in the analysis of joints subjected to cyclic loading, including welded and reinforced joints, where damage initiation, local stiffness degradation, and the development of plastic deformation are significant factors [31–37]. Among advanced engineering software packages, Adina [30] and Ansys [38] are also worth mentioning. Due to uncertainties in the material models and boundary conditions adopted, as well as the computational cost, 3D models primarily serve as a reference for calibrating simplified models, rather than as a standard tool for analyzing entire structures.

Despite the wide range of comparative studies, a significant research gap remains regarding the formation of plastic hinges in nRBS beams. Most of the available studies focus either on the global response of steel frames or on the detailed behavior of individual connections but rarely analyze how the choice of modeling level affects the identification of the location where a plastic hinge initiates. This is particularly important in SHM, as the location of sensors and the interpretation of warning signals differ between models designed with concentrated and distributed plasticity [10–13,15]. Furthermore, nRBS beams do not have a point of cross-sectional weakening; thus, the location of the plastic zone is a crucial element in the design of an SHM system, and the quality of the calibration of warning thresholds will depend on the level of model simplification in numerical simulations. Due to the different computational algorithms underlying the various types of plasticity models, element types, and the computational algorithms themselves in engineering software, the structural response may vary significantly between different computational environments.

Directly, the problem of analyzing and localizing plastic hinges in the steel moment resisting frames was investigated by Venezuela-Beltrán et al. [3], Grebowski et al. [22],

Elkady & Lignos [32], Montuori et al. [2]. Unlike these studies, further on in this article, detailed nonlinear solid modeling using the finite element method (FEM) will be compared with traditional modeling using ‘frame’ elements.

In summary, there is still a lack of studies that combine a comparison of numerical models with the problem of accurately locating the plastic hinge in nRBS beams and the requirements placed on SHM systems. In the opinion of the authors, the issue of properly monitoring the plastic response of steel structures through plastic hinge formation is still not enough covered in the health monitoring of steel structures. This study aims to fill this gap by (I) analyzing the influence of the level of detail in the numerical model on the predicted location and extent of the plastic hinge zone in nRBS joints, (II) identifying the level of modeling sufficient to obtain reliable results from such an analysis; and (III) assessing the implications of the above for sensor placement and the interpretation of results in SHM systems. The main novelty of this study is to use a very fine mesh of a 1-story steel frame as part of a large 5-story MRF to study early and advanced phases of plastic hinge formation under seismic excitations, to better plan future introduction of monitoring plastic hinge developments. Comparison with simplified approaches of popular software gives proper assessment of their departure from fine FEM modeling.

This is particularly important in light of the recent progress in the application of modern rotation rate sensors in SHM, see, e.g., [35–37], as these sensors, when properly positioned, can be used in the early warning systems of seismic damage.

The structure of the article includes this Introduction; Section 2, containing a description of models and assumptions applied to numerical analyses; Section 3, covering the results of dynamic simulations carried out for models with varying levels of reproduction of the real structure; and Section 5, containing conclusions and a summary of the research conducted.

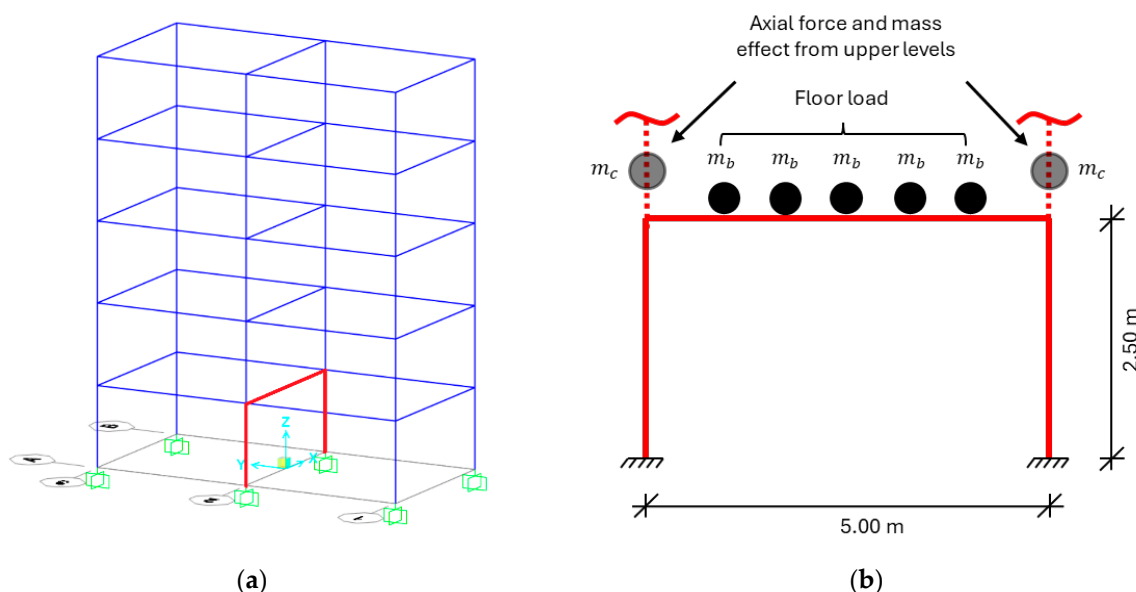
2. Materials and Methods

2.1. Description of the Structure

Analysis of the modeling approach to plastic hinge formation and its consequences for SHM systems requires a suitable parametric study on a simplified model to isolate the measured effect on the structure. Therefore, a two-dimensional steel moment resisting frame was designed, being an isolated part of a multi-level building. The geometry of the frame includes a height of 2.50 m and a span width of 5.00 m, as shown in Figure 1. Cross-sections of elements were defined with respect to the weak beam–strong column rule according to European standards Eurocode 3-1-1 and Eurocode 8-1 [6,39], along with industry practice guidelines [40]. Floor load is reduced to five mass points distributed along the beam, equal to $m_b = 3569.01$ kg each. The first mass is concentrated at a point 0.80 m from the column axis, the next at a distance of 0.85 m from the previous one. This slightly non-uniform arrangement resulted from modeling and meshing consideration in the 3D solid model. A geometric deviation of 5 cm is structurally negligible and has no effect on the distribution of internal forces, and thus on the area where a plastic hinge can form. Column and beam parameters are shown in Table 1. The material was designed as European construction steel, S235. To reduce computation time, a reduced single-story 2D frame was analyzed. To ensure proper axial forces in the columns, together with the inertia effect of upper levels, the total mass of the upper stories was added and modeled on top of each column as a concentrated mass ($m_c = 36757.22$ kg).

Table 1. Parameters of frame elements.

Parameter	Unit	Column Values	Beam Values
Height (h)	mm	250	250
Width (b)	mm	250	150
Web thickness (t_w)	mm	9	8
Flange thickness (t_f)	mm	14	10
Radius (r)	mm	0	0

**Figure 1.** Examined structure: (a) overall structure design and (b) frame isolated for analysis.

2.2. Numerical Modeling Approaches

Conducting a comprehensive analysis requires modeling of the steel frame with different levels of numerical idealization of the structure. Such a comparison gives the possibility to evaluate the impact of model simplicity on the accuracy of plastic hinge localization. For this purpose, three different FEA codes were selected—SAP2000 (version 26.2.0), SeismoStruct (version 2026 Build-2), and Abaqus (version 2025 FD04). Numerical models were additionally validated using the OpenSees code.

2.2.1. Concentrated Plasticity Approach—SAP2000 Beam Model

The first model was developed in SAP2000 [41], a common multi-purpose engineering software. Visualization of the model detail is shown in Figure 2a. The model was built with the use of frame elements (Timoshenko beams). The material of the structure was defined as European S235 steel, with an isotropic hardening constitutive model. The plasticity behavior of the model was designed as Concentrated Plastic Hinges, where the interaction of axial force and biaxial bending moments is considered at the hinge location (P-M2-M3 criteria). Beam plastic hinges were designed to be located within 250 mm from the column axes, with a relative length of 250 mm, as recommended by NIST GCR 17-917-46v2 standard [42]. The same CPHs were modeled near restraints to ensure iteration convergence during numerical analysis under heavy seismic load. The plastic deformation curve is defined according to FEMA 356 standard criteria [43], with respect to characteristic points A-B-C-D-E. Element discretization is done automatically, resulting in 12 finite elements per frame. This model serves as an engineering reference point, assuming that the joint is located in a strictly defined area, which eliminates the physical cause of variation in its location.

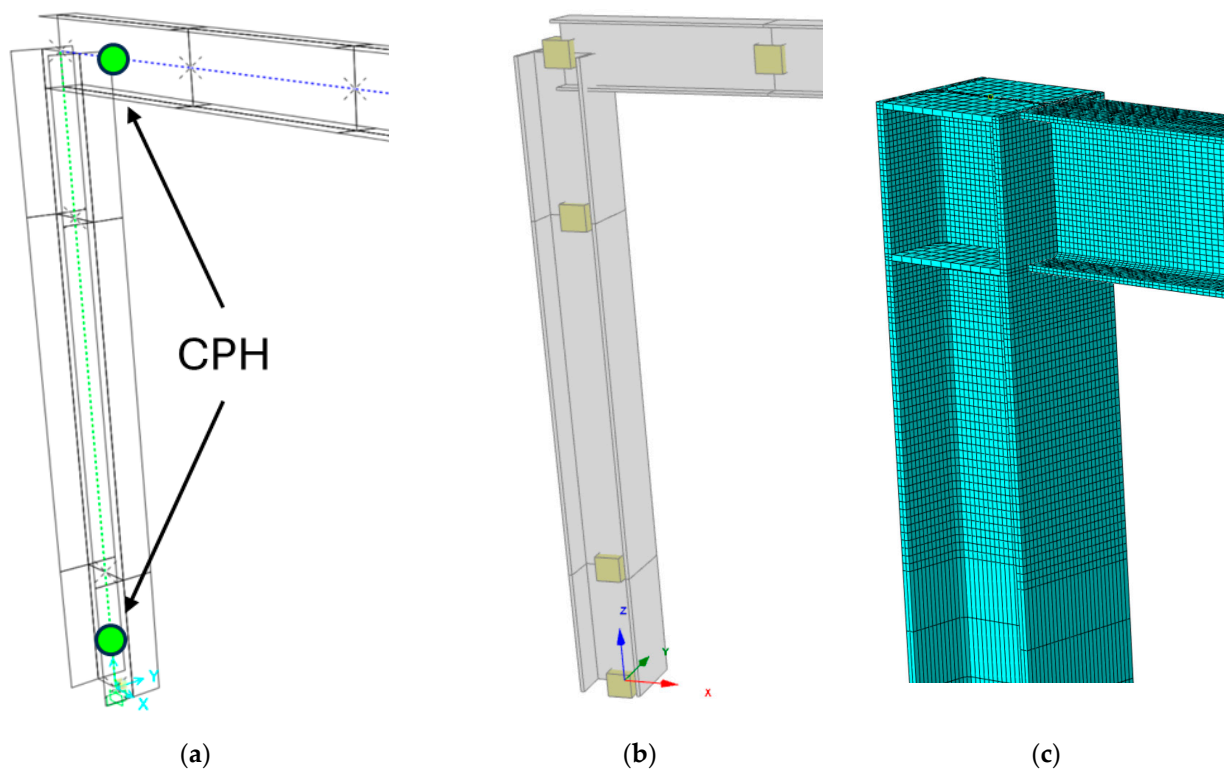


Figure 2. Comparison of model assembly in analyzed software: (a) SAP2000, (b) SeismoStruct, (c) Abaqus.

2.2.2. Distributed Plasticity Approach—SeismoStruct Fiber Model

The second model, built in the SeismoStruct software [44], was based on inelastic force-based frame elements. Visualization of the model detail is presented in Figure 2b. It represents a fiber approach to frame discretization, where the beam cross-section is divided into fibers to calculate inelastic behavior. Each fiber follows its own stress–strain relationship, integrated at section integration points. These elements do not account for shear deformations. The nonlinear response of the fibers in sections is integrated through the Gauss–Lobatto scheme along the beam. Moments and forces are calculated along the element, where the distribution of plasticity may be observed. The model was defined with 150 section fibers per cross-section, applied to 12 beam elements with nine integration sections each. The material constitutive model is bilinear with isotropic strain hardening with true yield strength of $f_{y,c} = 259$ MPa and a strain hardening parameter of 0.03. Values of the Young Modulus ($E = 210$ GPa), Poisson ratio ($\nu = 0.3$) and specific weight of steel ($\gamma = 76972.86$ N/m³) are adjusted to ensure compatibility between the models. The fiber discretization scheme allows tracking plasticity distribution along the element. That method may result in the development of a plastic hinge in areas of stress concentration, derived from the geometry of the structure and the applied load.

2.2.3. Continuum Solid Approach—Abaqus Model

The reference continuum model (with the highest idealization of the designed structure) was built using Abaqus software [45], a comprehensive engineering environment based on the FEA method. Solid elements were used, which led to a realistic simulation of structural behavior under applied load. Visualization of model details are presented in Figure 2c. An 8-node solid element with reduced integration (C3D8R) and enhanced hour-glass control were used. The mesh was densified in the predicted joint area (around 250 mm from the column faces) to an approximate size below $9 \times 9 \times 5$ mm. The mesh

convergence study showed that increasing the number of elements twice leads to similar (up to 4%) results in the stress and strain response of the structure. To increase the quality of the mesh, a chamfer on the web and flange with the thickness of the web was used. The material model implemented in Abaqus is consistent with models in the other software, with respect to true stress–strain representation. Yield strength and tensile strength of steel equal $f_{y,true} = 259$ MPa and $f_{u,true} = 445$ MPa, respectively. Geometric nonlinearity was taken into consideration by using large displacement theory. The assembly of continuum models requires interaction definitions between instances. All contact (joint area) was designed as a tie constraint, assuming a rigid connection of beam and column. To increase the representation level of real construction, column ribs with a thickness of beam flange were implemented to stiffen the model. Such a developed solid model in Abaqus software is able to indicate local buckling, calculate stress distribution, and recognize plastic deformation in every finite element in the model, which is crucial for plastic hinge localization analysis.

2.2.4. Verification of the Models with Modal Analysis

Comparison of different models requires calibration of base properties for further parametric studies. Three models with different levels of structural representation were compared to verify their compatibility with respect to global physical properties and natural frequencies. Table 2 presents descriptive parameters of the model, including the software used, element type, number of elements, total weight, and natural frequencies. Additional frame models were modeled using OpenSees software [46] to verify the impact of element type on eigenvalues. Models taking into account shear deformations (using Timoshenko beams and SAP2000 software solution) provide similar results compared to continuum models. The Euler beams together with fiber models act differently, resulting in a higher frequency for the same structure model due to higher stiffness as described in the element's stiffness matrix. Mass participation factors of the modes are consistent across the models, as follows: f_1 : 99% in horizontal direction; f_2 : 19% in vertical direction; f_3 : 1.2% in vertical direction; f_4 : 0.4% in horizontal direction. These results prove compatibility between the models in the linear regime, with differences explained by different element descriptions.

Table 2. Model parameters and their natural frequencies.

Model	Element Type	Element No.	Weight [kg]	Model Natural Frequencies			
				f_1 [Hz]	f_2 [Hz]	f_3 [Hz]	f_4 [Hz]
Beam (SAP2000)	beams	12	91,902	1.946	5.551	14.699	16.409
Beam (OpenSees)	ETBCE (Timoshenko) beams	12	91,902	1.945	5.547	14.68	16.409
Beam (OpenSees)	EBCE (Euler) beams	12	91,902	2.054	5.912	16.234	16.434
Fiber (Seismostruct)	IFBFE	150/c-s	91,875	2.052	5.909	16.228	16.434
Solid (Abaqus)	C3D8R	46,284	91,938	1.879	5.603	15.118	15.294

2.3. Simulation Procedures

Comparing the nonlinear behavior of SMRF numerical models built at different levels of idealization requires a common, model-independent reference. Nonlinear static pushover analysis is widely used as a global analysis method, characterizing the overall response of the frame, expressed through base shear versus inter-story drift, regardless of

the underlying modeling approach. Such an approach allows direct comparison of key nonlinear parameters across models—such as initial stiffness, yield strength, and displacement, and post-yield hardening ratio—even with different underlying plasticity models and element equations. In this regard, in a Displacement-Controlled Static Pushover Analysis, a monotonic horizontal force was applied to the frame node until a total inter-story drift of 0.3 m (12%) was achieved. Pushover analysis led to the determination of the backbone curve, with indication of elastic response, yield point, and hardening effect. This analysis was used to define the overall behavior of the examined SMRF.

The second step of analysis included the Nonlinear Time History Response of the structure under various intensities of seismic records. The motion equations were solved using the direct integration method (Hilber–Hughes–Taylor method for every software). Damping was defined as a Rayleigh model, scaled for the first and second modes of vibration, with a damping ratio $\zeta = 3\%$, as a compromise between 2% mentioned for raw steel frames and 5% used as a seismic analysis standard. The classic structural damping matters mostly when the structure vibrates in linear phase of the response. Since plastic deformations dominate response during damage, its role is of less importance in this study.

Nonlinear Time History Response Analysis requires an accurate seismic record. Literature review indicates that the seismic record of the 1940 El Centro earthquake is well known and widely used in seismic analysis of structures. Since it has a broadband spectrum and rather long duration, it is useful as a benchmark ground motion. Indeed, this is so, particularly for structures of short fundamental natural period (here, it is less than 0.55 s—see Table 2). This seismic record is characterized by strong acceleration in the initial phase and diminishing vibrations in the final phase. Due to the small vibration intensity in the later phase, the analyses were restricted to the first 35 s to reduce computation time. Figure 3 presents the El Centro East–West component acceleration and velocity signal (after drift correction) used in further analysis. The key parameters of the record are as follows: $PGA = 2.1 \text{ m/s}^2$ and $PGV = 0.488 \text{ m/s}$.

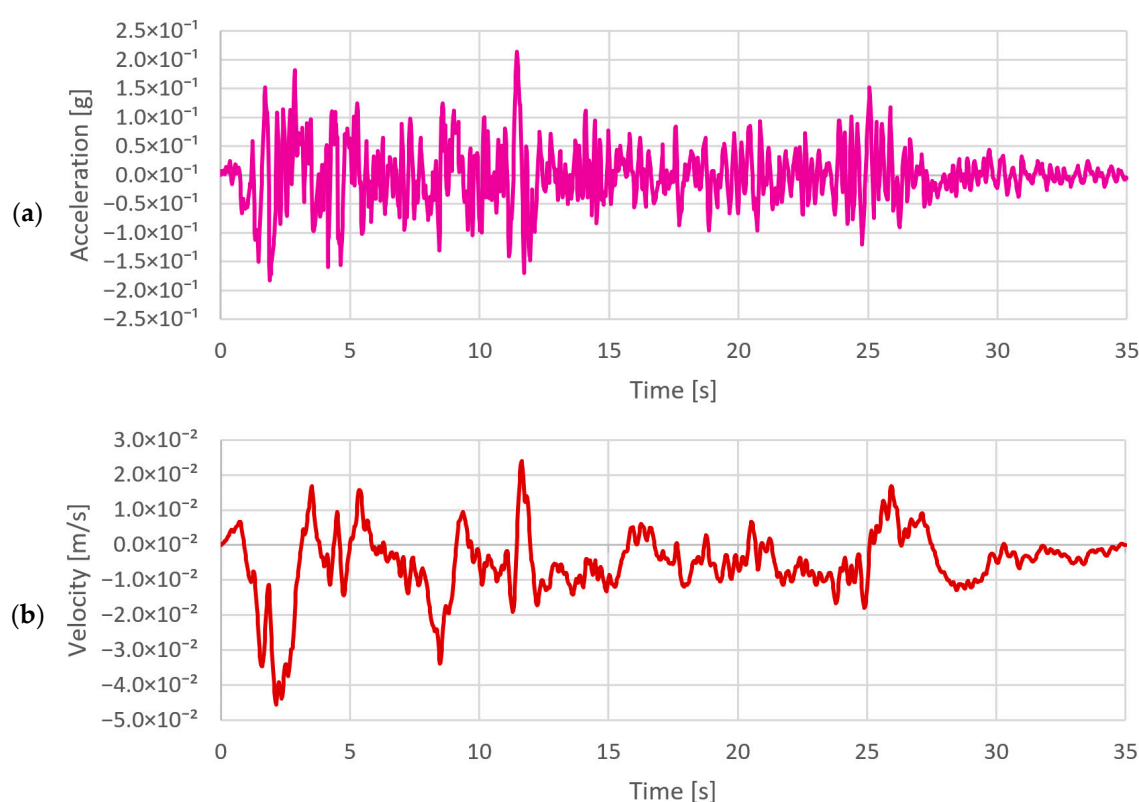


Figure 3. El Centro earthquake acceleration (a) and velocity (b) records.

Scaling of the seismic record is based on multiplying the gravity acceleration by the initial factor of the seismic record. Table 3 shows a summary of the intensities used in the analyses. Intensities below the original El Centro acceleration record may be considered as weak seismic motion with no critical impact on the structure. Intensities equal to or above the initial acceleration are considered highly likely to induce severe damage to the structure.

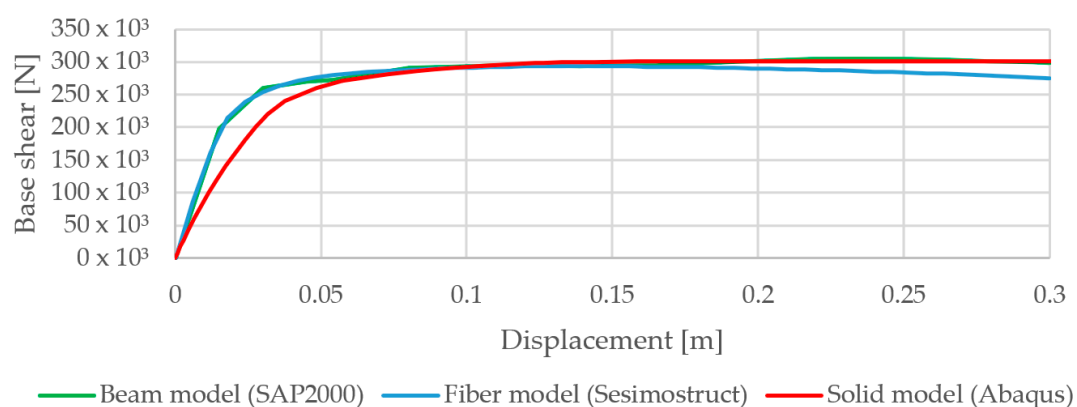
Table 3. Scaled intensities of initial seismic records used in further analysis.

Label	Intensity Factor	PGA [m/s^2]	PGV [m/s]
0K1	0.1	0.210	0.049
0K5	0.5	1.050	0.244
1K0	1	2.099	0.488
1K5	1.5	3.149	0.732
2K0	2	4.199	0.976

3. Results

3.1. Global Static Frame Response: Pushover Analysis

The static analyses carried out are illustrated in the form of pushover curves for each model. Figure 4 shows the relationship between the displacement of a node (where the elements axes cross at the corner of the frame) and the applied force, equivalent to the base reaction. The curves demonstrate a general convergence in the structural response within the elastic range. The yield point for all models lies within the range (base shear ~ 275 kN). The solid model shows slightly lower stiffness, as indicated by the steeper slope of the curve in the displacement range from 0 to 0.05 m. This is due to minor local plastic deformations occurring in the initial stages of loading in the connection area. Minor deformations of the system's geometry and a slight change in stiffness in the joint zone started to increase further under higher loads (base shear above 200.0×10^3 N).

**Figure 4.** Comparison of pushover curves for different models.

The strengthening effect can be observed in every model; the ratio of the force at the end of the strengthening section to the force at the start of the strengthening section for the beam, fiber, and solid models is 8.5%, 1.0% and 9.1% respectively. The fiber model shows a tendency for the material model to weaken in the final stage of the analysis. The beam and fiber models terminated the 'displacement-controlled' analysis at a

displacement of 0.3 m. The solid model continued until global instability occurred at a total node displacement of 0.36 m.

The simple beam model (SAP2000) presents the sharpest transition into the plastic phase due to the nature of the CPH joints, where displacements increase sharply once the limit values are exceeded. The fiber model (SeismoStruct) displays smoother, yet very similar, behavior. The slightly different behavior at large displacements can be attributed to different P-Δ effects across the software. The solid model reveals a smooth transition and slight changes in the monotonous strengthening, which may be caused by the progressive yielding of successive finite elements and the occurrence of local section buckling effects. Additional simulations of nonlinear static pushover analysis were performed with use of OpenSees models, as well as Euler-type and Timoshenko-type beams. Both curves are almost identical with the lumped plasticity model (SAP2000). Therefore, it is not necessary to carry out further analyses using OpenSees models. When analyzing the global response of the individual models, it can be concluded that, despite the use of different types of elastic-plastic elements and plastic hinges, the behavior of each is very similar, and any minor differences may stem from the element types themselves.

3.2. Dynamic Global Response: Nonlinear Time History Analysis

While static analyses allow the assessment of load-bearing capacity under controlled loading conditions, the actual response of a structure subjected to seismic loads requires consideration of dynamic effects and the cyclic nature of the excitation. In order to achieve this, nonlinear time history analyses were carried out, where global node displacements were measured. The primary measure is the inter-story drift ratio (IDR), represented by Equation (1):

$$IDR = \frac{Ux_{abs}}{h_{story}} [\%] \quad (1)$$

where horizontal displacement of the story ceiling is divided by the height of the exact story, expressed as a percentage. The IDR is widely used in seismic engineering. It allows the structure's susceptibility to excitation to be assessed, taking dynamic effects into account for each model, and serves as a good basis for comparison between models with varying degrees of structural representation.

The largest displacements and deformations (to be used in further analyses) were recorded at the top-left corner of the frame; therefore, detailed data will be presented for this node. Figure 5 shows a comparison of the models' responses for intensity 1K0 throughout the entire excitation, in terms of the node's displacements in the X-direction (along the beam). The response characteristics of the individual models are consistent. The responses of the SAP2000 simple beam and Seismostruct fiber models are consistent, whilst the solid model shows slightly smaller limit and final displacements but a greater vibration amplitude in the second half of the excitation.

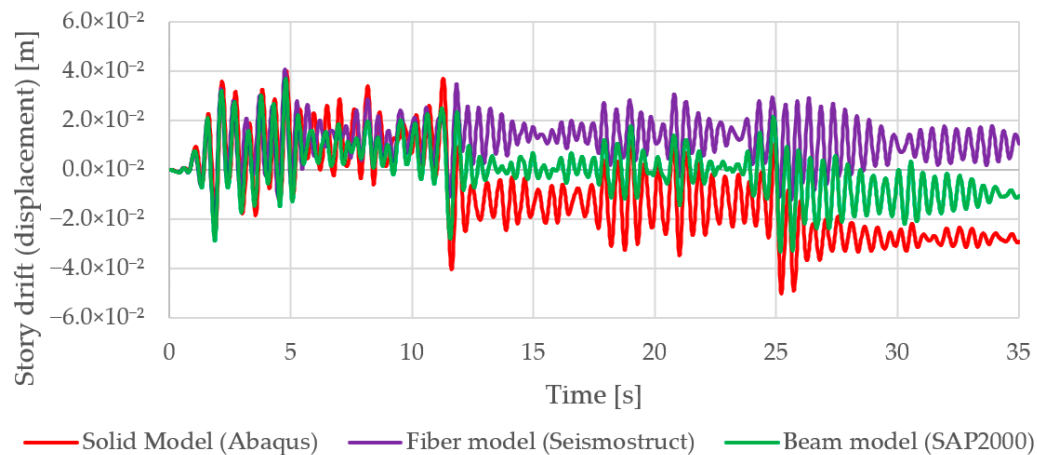


Figure 5. Node displacement response during time history analysis under 1K0 intensity seismic excitation for different models.

Peak inter-story drift for the intensity 1K0 for simple beam, fiber and solid models reached $U_{x_{abs,beam}} = 0.037$ m, $U_{x_{abs,fiber}} = 0.041$ m and $U_{x_{abs,solid}} = 0.050$ m, respectively. These values relate to following IDR values: $IDR_{beam} = 1.49\%$, $IDR_{fiber} = 1.64\%$ and $IDR_{solid} = 2.01\%$. A similar trend in the readings can be observed for all models in the initial moments of the analysis, but there are clear discrepancies from $t = 11.64$ s onwards, i.e., after a stronger pulse.

Peak drift indicates the maximum instantaneous displacements. However, when assessing a structure following seismic excitation, the residual drift is a key parameter, as it indicates permanent damage to the structure. A non-overlapping moving average was used to track permanent deformations. Figure 6 shows the progression of the moving averages during the excitation. A significant shift in the vibration axis is visible at around 2 s, 12 s and 24 s, at the points where high accelerations occurred. Residual displacements are defined as the moving average at the end of the analysis period. For example, for an intensity of 1K0, the residual drift (calculated in the same way as peak drift) for the simple, fiber and solid models was, respectively, $IDR_{residual,beam} = 0.39\%$, $IDR_{residual,fiber} = 0.40\%$ oraz $IDR_{residual,solid} = 1.11\%$. The discrepancies are significant, and the largest residual displacements were recorded in the solid model.

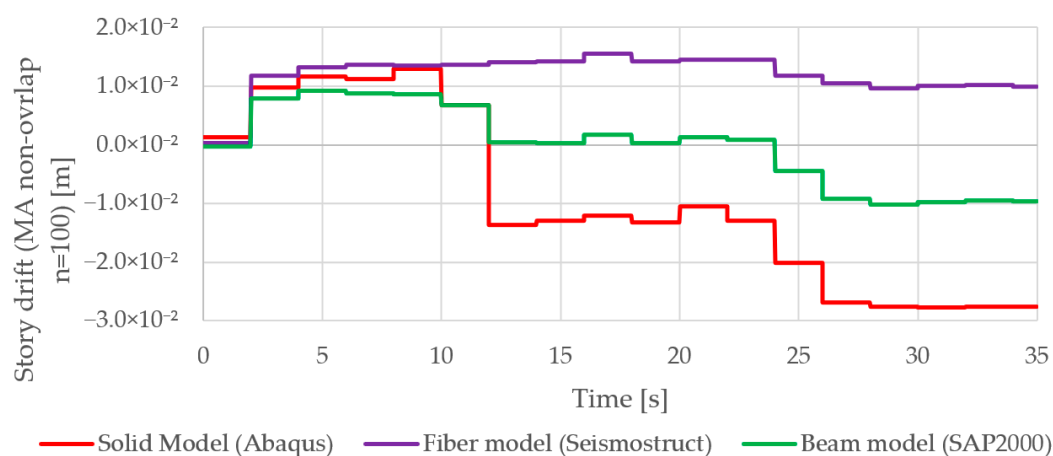


Figure 6. Comparison of non-overlapping moving averages of node displacement response during time history analysis under 1K0 intensity seismic excitation for different models.

The results for the peak IDR and final IDR for all analyzed intensities and models are summarized in Table 4. The solid model demonstrates significantly higher IDR values than the simple and fiber models, in terms of both extreme and final displacements. The solid model takes into account local buckling of members, which may lead to higher final IDRs. The simple and fiber models exhibit greater structural stiffness, resulting in smaller temporary and residual deformations.

Table 4. Peak Drift Ratio and residual permanent Drift Ratio for different models during time history analysis.

Intensity	Model					
	Simple (SAP2000)		Fiber (SeismoStruct)		Solid (Abaqus)	
	Peak Drift [%]	Residual Drift [%]	Peak Drift [%]	Residual Drift [%]	Peak Drift [%]	Residual Drift [%]
0K1	0.21	0.00	0.18	0.00	0.28	0.02
0K5	1.12	0.24	0.78	0.11	1.08	0.10
1K0	1.49	0.39	1.64	0.40	2.01	1.11
1K5	2.47	0.83	2.26	0.36	2.49	1.07
2K0	3.40	0.75	2.86	1.18	4.29	0.74

Dynamic analyses indicated that the maximum recorded node displacements in all models (expressed as IDR) show very good similarity for both low and high intensities, as illustrated in Figure 7. For medium intensities, the simple beam and fiber models exhibit a very similar trend, with the solid model showing values up to three times higher. At the highest intensity, the fiber model shows the highest readings, ending the analysis with residual displacements of 1.18%, while the solid and simple beam models show values of 0.74% and 0.75%, respectively. The solid model demonstrates the highest recorded deformations for medium intensities, which is crucial when assessing the feasibility of repairing damaged buildings.

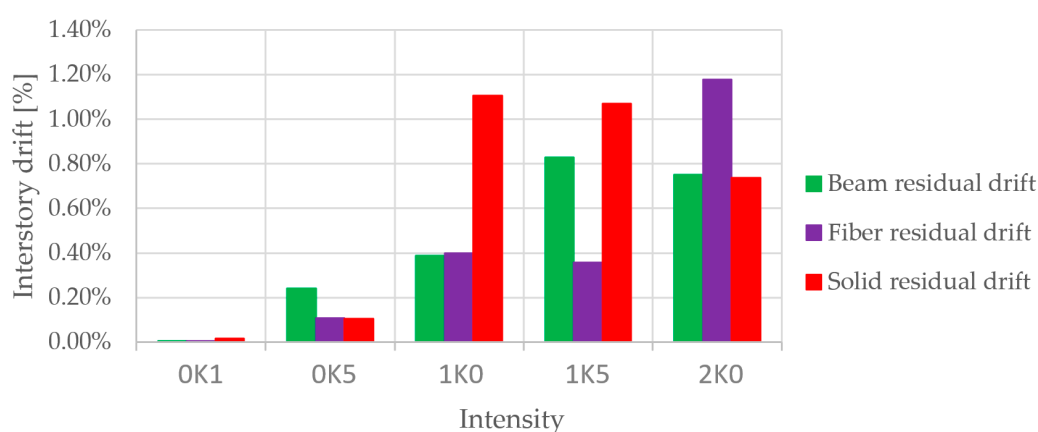


Figure 7. Residual drift for various intensities and models after time history analysis.

3.3. Spatial Variations of Plastic Hinge Position

Global parameters determine the overall condition of the structure, but a more detailed analysis of the plastic deformations in the cross-section is required to assess damage to the elements. Measurements of plastic deformations along the length of the beam are used to precisely determine the location, extent, and propagation of the plastic hinge zone in nBRS beams. A comparison of the maps showing the distribution of plastic yield along the beam will highlight the level of accuracy for each model, and accurate maps can serve

as a basis for determining the optimal locations for installing SHM system sensors and for interpreting warning signals.

The differences in the models' responses regarding the development of the plastic zone are strongly linked to the types of elements used. In the simple beam model created in SAP2000, it is not possible to determine the position and range of the plastic hinge, as these are predefined by the user. In order to analyze the impact of these parameters on the results, their effect on the residual displacements (IDR) was examined. The nominal value of the joint's position is 0.25 m from the node axis (in accordance with standard guidelines [42]), with a variation of $\pm 25\%$ of the element's length (equal 0.8 m). The nominal value of the relative length of the joint is $x_{PH} = 0.25$ m, with a variation of $\pm 25\%$. For reference, see Figure 8. Table 5 shows the values obtained from the beam model analysis (in SAP2000) for intensity 1K0. The residual IDR values tend to decrease as the relative length of the hinge is reduced. The position of the hinge along the beam axis has a minor impact on the results. The corresponding values for the fiber and solid models are 0.40% and 1.11%, respectively. It appears that a highly refined calibration of the beam model with CPH can be adapted to the response of the fiber model, but the values obtained from the solid models lie outside the scope of reasonable CPH parameterization values. The limitation of CPH models in reproducing the residual IDR value observed in Abaqus results from the unavoidable limitations of typical beam-based models. These simplified models do not account for neither local geometric instabilities (flange/web buckling) nor the coupled plasticity effect caused by shear forces (P-M-V interaction) occurring near the column face.

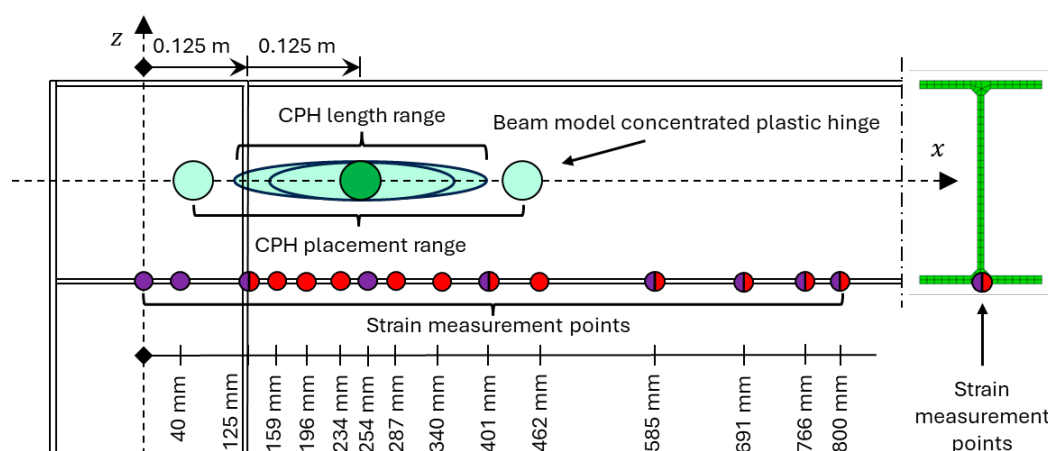


Figure 8. A diagram showing the locations of the strain measurement points, with ordinates given relative to the axis of the column for the fiber model (purple) and the solid model (red), as well as the location of the defined CPH joint in the simple beam model (green).

Table 5. Residual IDR values for different locations and relative length of CPH in the simple beam model (SAP2000 software) in the case of time history analysis under 1K0 intensity.

Residual IDR [%]		PH Location (From I-End of 0.8 m Element)		
		−25% = 0.05 [m]	0 = 0.25 [m]	+25% = 0.45 [m]
PH relative length	−25% = 0.19 [m]	0.31%	0.29%	0.29%
	0 = 0.25 [m]	0.40%	0.39%	0.44%
	+25% = 0.31 [m]	0.51%	0.47%	0.53%

In order to deeply analyze the response of the fiber and solid models in relation to the CPH standard guidelines, the plastic deformation values were extracted from the simulations. For the fiber model, the values are presented for the central fiber of the lower

flange, measured at the cross-section integration points. The solid model provides values from finite elements located at positions similar to those indicated by the fiber model, as well as at intermediate points selected by the authors to increase the density of the readings. Figure 8 shows a diagram of the strain measurement points.

Figure 9 shows the total deformations at the specified points from the SeismoStruct model. The values given are for maximum deformations (on the left) and residual deformations, i.e., at the end of the analysis (on the right). The fiber model shows the greatest strains in the integration section at the edge of the element, on the column face. For each intensity level, the strains decrease linearly to approximately 15% of their maximum values at a distance of approximately 545 mm from the column face. Permanent strains for intensities 1K0–2K0 on the column face do not exceed $\varepsilon_{fiber} = 0.035$, and for all intensities they decrease to zero at a distance of 545 mm.

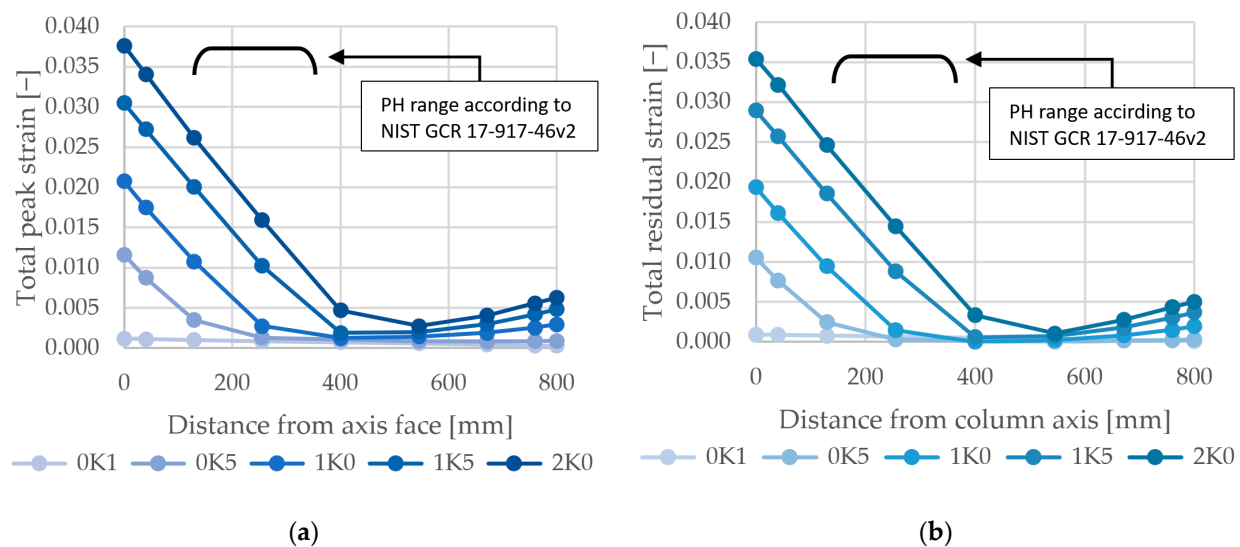


Figure 9. Strains obtained from the SeismoStruct fiber model time history analyses according to peak response (a) and residual strain (b).

The solid model also shows the distribution of plastic deformation along the beam. The peak logarithmic strain values (Hencky's strain criterion, which includes both elastic and plastic strains) indicate which finite elements were subjected to the greatest instantaneous increases in strain. Figure 10 shows the extreme values of logarithmic strain for the entire excitation history. The propagation of plastic deformation is visible as the intensity increases; at an intensity of 1K0, the zone is limited to a distance of ~ 275 mm from the column face, while for higher intensities it extends to approximately 600 mm, i.e., the more than double the beam's cross-section (equal 250 mm).

The quantitative analyses of peak and residual strains were performed between fiber and solid models to calculate the relative error of fiber model results with reference to more accurate solid model results. Table 6 contains ratios for peak strain and Table 7 contains ratios for residual strain. Results lead to the conclusion that the fiber model globally underestimates the calculation of strains in relation to the solid model but overestimate the strains near the connection surface.

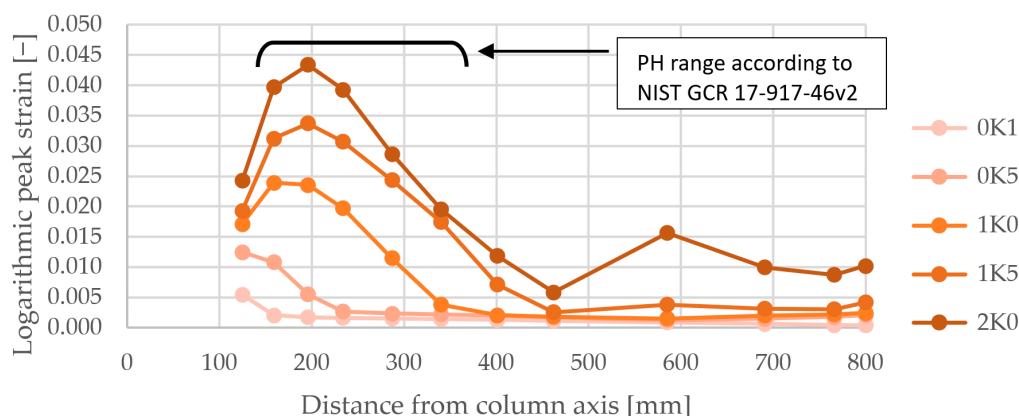


Figure 10. Peak logarithmic strain (Max. Principal (Abs)) obtained from time history analyses performed on solid models.

Table 6. Quantitative comparison of fiber and solid models' peak strain in key locations as relative error calculated with reference to solid results, obtained during time history analysis. Location expressed as a ratio of point distance from column face and the total end span length (800 mm). Result expressed as [%].

Intensity	Measurement Point Placement as End Span Length Ratio [%]								
	0	5	16	32	50	68	84	95	100
0K1	-78	-44	-36	-37	-35	-32	-29	-15	-28
0K5	-6	-19	34	-41	-36	-29	-43	-49	-56
1K0	21	-27	-45	-27	-30	-3	-4	16	24
1K5	58	-13	-35	-41	-25	-47	-3	40	16
2K0	55	-14	-33	-18	-19	-82	-59	-36	-39

Table 7. Quantitative comparison of fiber and solid models' residual strain in key locations as relative error calculated with reference to solid results, obtained during time history analysis. Location expressed as a ratio of point distance from column face and the total end span length (800 mm). Result expressed as [%].

Intensity	Measurement Point Placement as End Span Length Ratio [%]								
	0	5	16	32	50	68	84	95	100
0K1	-82	-50	-42	-43	-42	-40	-40	-23	-18
0K5	1	-22	44	-78	-82	-88	-67	-71	-77
1K0	29	-29	-49	-44	-98	-19	-10	29	36
1K5	73	-13	-37	-45	188	-72	-10	55	16
2K0	69	-16	-35	-20	38	-92	-67	-43	-45

Solid model analyses provide detailed stress and strain maps, with respect to multi-axial 3D strain tensor measures. Figure 11 and Figure 12 illustrates the moment of maximum increase in plastic strain in the X-direction (and thus associated with the tension in the flanges of the beam cross-section) and the von Mises stress map at the same time increment for each load intensity. As indicated by the deformation extreme values, the greatest damage occurs at approximately $t = 25.24$ s or $t = 11.6$ s. These are the time stamps when local PGA or PGV maximum values are reached.

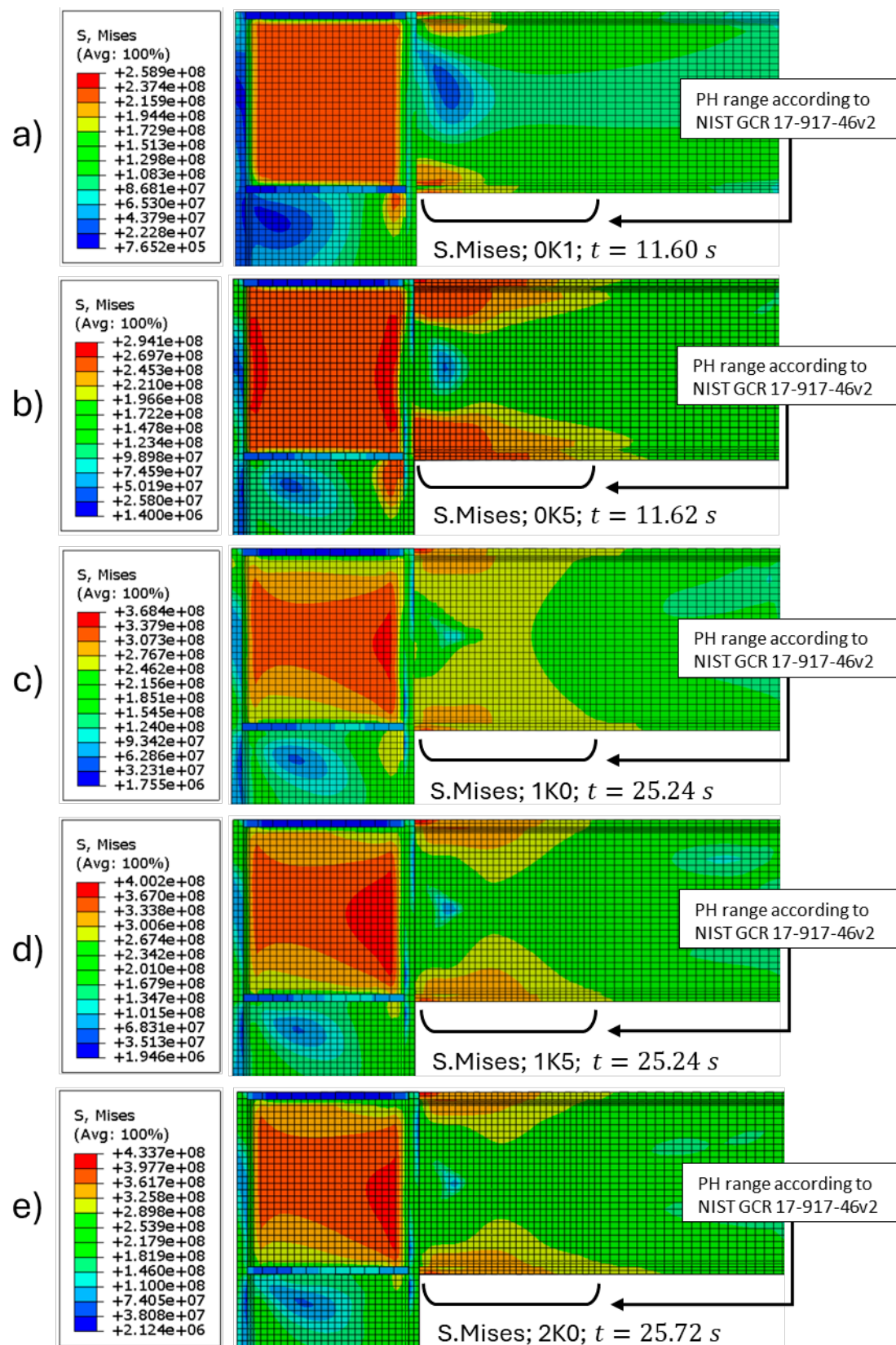


Figure 11. Von Mises stress maps of frame corner obtained from simulation in Abaqus software during stress extrema for various intensities: (a) 0K1, (b) 0K5, (c) 1K0, (d) 1K5, (e) 2K0.

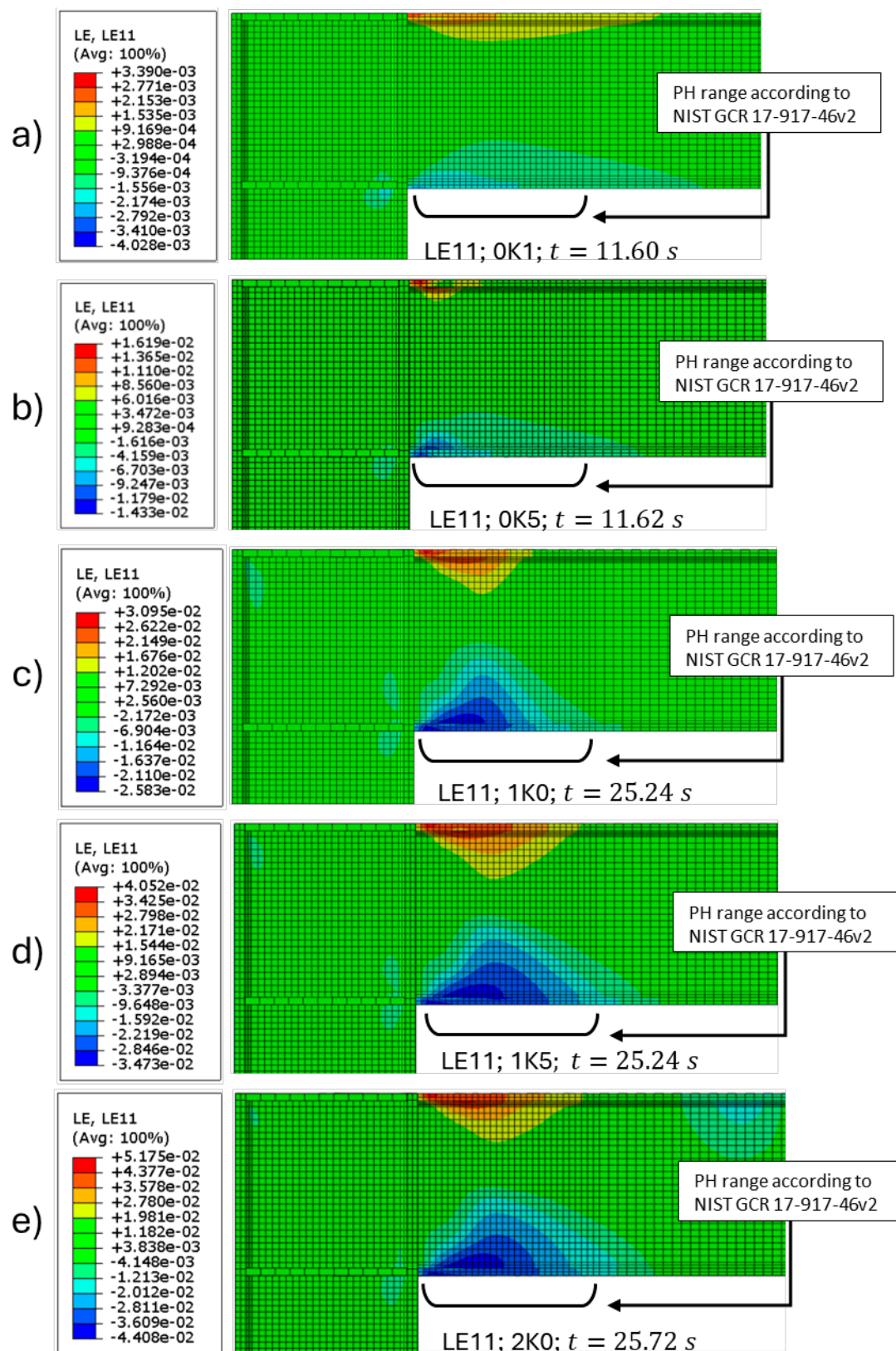


Figure 12. Logarithmic strain (LE11) maps of frame corner obtained from simulation in Abaqus software during stress extrema for various intensities: (a) 0K1, (b) 0K5, (c) 1K0, (d) 1K5, (e) 2K0.

Due to the three-dimensional nature of a strain tensor, the equivalent plastic strains (PEEQ—scalar, monotonically cumulated measure of multiaxial plastic deformation based on von Mises yield criterion) may be a better indicator of plastic zone range in 3D continuum elements. Figure 13 shows PEEQ values measured for the elements corresponding to the integration sections of the fiber model and clearly shows that plastic deformation propagates along the length of the beam. For an intensity of 0K5, the noticeable equivalent plastic strains disappear at a distance of 234 mm from the cross-section axis (71 mm from the beam face). For an intensity of 1K0, the decline is noticeable at a distance of ~400 mm. For higher intensities, plastic deformation extends as far as 550–650 mm into the beam. It can also be observed that the equivalent strains at a distance of 250 mm from the column axis (at the point where the CPH was assumed according to NIST GCR 17-917-46v2 code) range from 25% to 43% (depending on intensity) of the peak equivalent strains recorded at the beam-to-column joint.

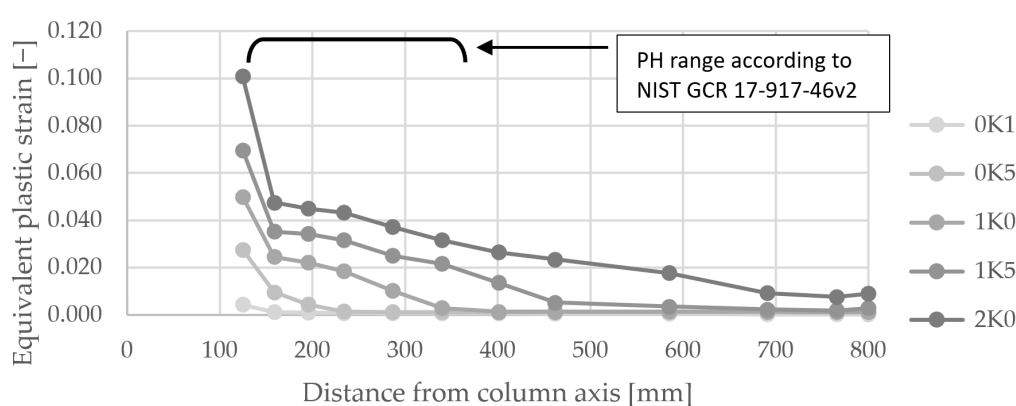


Figure 13. Plastic Equivalent Strain (PEEQ) obtained from time history analyses performed on solid models as permanent residual deformations in solid elements.

The location of the plastic hinge has a crucial effect on the structure's response to seismic excitation. The beam model (SAP2000) has rigidly defined plastic hinges at a distance of 250 mm from the column axis, in accordance with generally accepted standards for concentrated hinges. Simulation results for fiber and solid models show that the plasticity is concentrated on the beam face—specifically at the connection to the column (welds)—for lower intensities. At medium intensities, the zone propagates to a distance of one cross-section height ($1.0d_{beam}$). Higher intensities indicate that the plastic zone ceases at a distance of $1.5d_{beam} \sim 2.0d_{beam}$. RBS beams concentrate plastic deformation within the area of reduced cross-section. However, for nRBS beams, the propagation of the plastic zone must be considered when designing the structure.

4. Discussion

The results obtained from the static and dynamic analyses allow for the verification of the impact of numerical models on SHM problems in structures with nBRS beams.

An analysis of the distribution of deformations along the beam has shown that the level of numerical model refinement is crucial for accurately representing the geometry and range of the plastic zone. Simple models (such as those in SAP2000) reduce the plastic hinge to a single point, which makes it impossible to capture the actual propagation of plastic deformations. In contrast, a detailed solid model quickly identifies small plastic deformations that affect the stiffness of a joint even before large displacements occur. A useful compromise is the fiber model, which combines the advantages of the beam model in terms of design simplicity with those of the solid model by showing the propagation of

plastic deformation along the member. It should be noted that for irregular and complex joints, it is impossible to accurately capture local deformations without using a dense mesh of solid or shell elements (see the local deformation of the flanges shown in Figure 14).

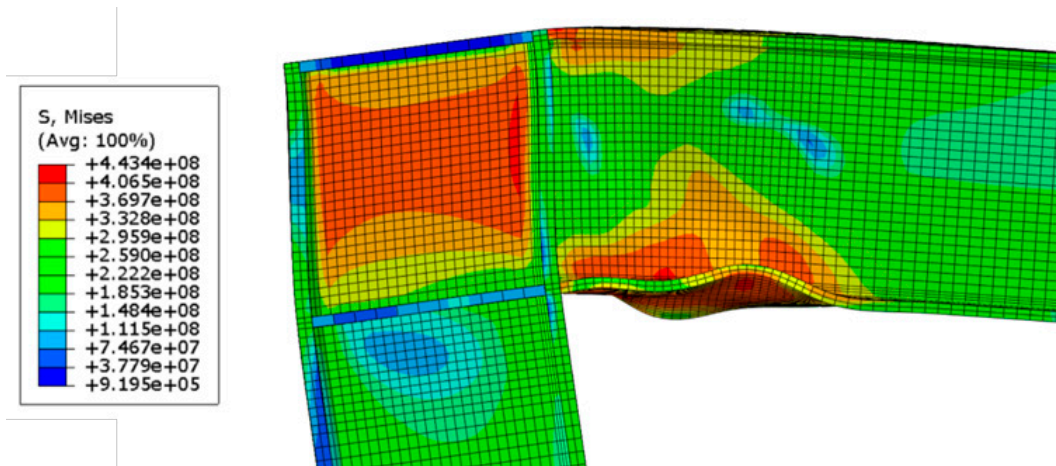


Figure 14. Map of stress during pushover analysis performed on a solid model (Abaqus) with visible deformations moments before collapse.

A key criterion for assessing the usability of the models was the duration of the non-linear time history analysis. Software dedicated to structural design demonstrated the highest computational efficiency: for SAP2000, the analysis took approx. 2 min, and in SeismoStruct approximately 1 min, whilst a detailed model in Abaqus required as much as 95 min of computation, indicating a significant increase in the cost of analysis for the most accurate representation of the structure. Computation times for beam models are very short due to the small number of elements. Increasing the discretization density of the system or the occurrence of large deformations can cause a sharp increase in computation time. Simulations were performed on a professional workstation with a 12-core CPU (AMD Ryzen 9 3900X), 32 GB of system RAM and Nvidia Quadro GV100 32 GB VRAM GPU.

According to the analyses carried out, beam models appear to be sufficient for obtaining a global response of the structure. However, for the purposes of designing SHM systems, it is necessary to use at least distributed plasticity models, which indicate the area of the plastic zone while saving computing power. On the other hand, for the analysis of complex connections and non-standard cross-sections, it is necessary to use a simulation environment such as Abaqus, and the many hours of computation required are justified and necessary to obtain detailed stress and strain maps.

In practical engineering situations, it is also important to consider the cost of carrying out such analyses, not only in terms of the time required for the calculations, but also in terms of software proficiency. SAP2000, which models beam structures, is a widely used and intuitive general-purpose software for the design of engineering structures, and training materials are commonly available. SeismoStruct, on the other hand, which is used for modeling fiber elements, is gaining popularity as a versatile tool for rapid seismic analyses. However, Abaqus, using its full, nonlinear capacity with solid finite elements, presents the greatest challenge, as the design of a structure usually requires numerous attempts on simpler models before proceeding. It is important to take into account the relatively cost-effective scope of additional analyses when designing structures, depending on the level of design risk. General comparison of the SAP2000, SeismoStruct and Abaqus is provided in Table 8.

Table 8. General comparison of the FEA software used in this paper.

FEA Software	SAP2000	SeismoStruct	Abaqus
Modeling skills necessary	Low	Low to moderate	Very High
Example computational time of a nonlinear time-history analysis	2 min	1 min	95 min
nDoF	33	45	214,017
Prediction accuracy	Low—plastic strain distribution unavailable	Moderate—no local phenomena can be analyzed	High

The findings from the above studies point to certain implications for the design and operation of SHM systems, regarding sensor placement and the interpretation of recorded data, particularly modern rotation-rate sensors useful in plastic hinge detection. The use of incorrect parameters for the yield zone in simplified numerical models creates a risk of installing point sensors (e.g., strain gauges) outside the stress concentration zone and consequently of receiving underestimated damage signals.

To avoid installing sensors in areas of minor deformation within frames containing nRBS elements, the optimal location and length of the sensors' measurement area should be determined on the basis of preliminary analyses using at least a distributed plasticity model (e.g., fiber). For non-standard structures with irregular shapes or asymmetrical connections, it is recommended to use analyses based on solid models, which show the structure's detailed response to a given load.

The research methodology also justifies the use of a two-level structural monitoring approach, where measurements of IDR using, for example, accelerometers are useful for a rapid, overall assessment of the exceedance of limit states during an event, while local measurements from strain gauges in the plastic deformation zones of nRBS components are analyzed in detail to locate and assess the condition of damaged elements and connections.

5. Conclusions

The above study has resulted in the following conclusions:

1. Simple beam models (CPH) correctly estimate the propagation of the Peak IDR extremum as intensity increases, but they underestimate the Residual IDR, which may lead to an incorrect post-event assessment of a structure's condition. Furthermore, concentrated plasticity is characterized by a sudden point jump in plastic deformation when the yield point is exceeded, distorting the actual propagation of plasticity along the beam.
2. In contrast to RBS connections, in nRBS joints the plastic zone migrates along the length of the beam from the column face over a distance ranging from $1.0d_{beam}$ for lower intensities to $2.0d_{beam}$ for higher ones. Therefore, it is recommended to adopt a wider range when designing SHM systems for structures using CPH models in the design of structures exposed to seismic risk. To accurately capture this phenomenon and determine stress concentrations, it is necessary to use distributed plasticity (fiber) or solid models.
3. Designing sensor systems based on the CPH definition carries a high risk of sensors (e.g., strain gauges) being incorrectly installed outside the zone of significant deformation. Therefore, the calibration of sensor locations and SHM alarm thresholds should be based on results from at least fiber-class models or detailed analyses using solid-body models. For engineers limited to CPH based solutions, a conservative

sensor placement according to theoretical equations of l_{PH} and industry standard guidelines is recommended.

4. While CPH models are sufficient for structural design purposes, the precise definition of critical parameters for SHM systems requires reference to higher-order models. Due to the high computational cost of analyses in solid models, it is justified to use fiber models for recognized types of joints. In the case of complex systems, it appears necessary to use the most accurate solid models.

This study has certain limitations, which also indicate the scope for future publications. Given the numerical constraints and the time required to run simulations of a large solid model, it seems important to also carry out further analyses for substantially larger steel structures with much longer natural periods (e.g., tall buildings) and perhaps shorter duration seismic records. Similarly, the node geometry could be extended to include various types of connections in order to simulate the effect of node stiffness on the global response of the frame. This study is limited to comparison of modeling refinement using different software approaches. Therefore, phenomena like residual welding stresses, which can decrease load intensity when plastic strains appear, were not analyzed. Responses for tall, multi-story structures could also be simulated to confirm the recommendations for this category of buildings. It is also advisable to carry out an experiment to further validate the results of the numerical simulations.

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