

Article

On the Seismic Response of Irregular Concrete Building with Shark Dampers Considering SSI Effects

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Abstract

This study investigates the three-dimensional seismic response of plan-irregular reinforced concrete (RC) frame structures subjected to near-fault pulse-like ground motions. The primary scientific contribution lies in evaluating how perimeter-distributed, short-stroke displacement-activated hysteretic dampers interact with complex structural torsional modes under the coupled influence of soil–structure interaction (SSI). Through unscaled nonlinear time–history analyses, this research exposes a critical vulnerability: foundation flexibility elongates the structural period, moving the system into close resonance with near-fault pulse content and more than doubling peak inter-storey drift demands. Concurrently, we demonstrate that the supplemental hysteretic system provides immediate elastic stiffness that successfully counteracts this SSI-induced period elongation, restricting peak drift amplifications to under 0.21% while actively mitigating torsional twisting. These findings provide a quantitative framework for the dual management of foundation flexibility and torsional irregularity using short-stroke metallic yielding devices.

Keywords: seismic retrofit; hysteretic dampers; maurer shark; reinforced concrete frames; soil–structure interaction; nonlinear time–history analysis

1. Introduction

A significant share of the global building inventory comprises older RC frame structures originally engineered solely to resist gravity loads, with no provisions made for lateral seismic forces [1]. Since contemporary seismic codes demand considerably greater lateral resistance, these buildings are broadly considered deficient, especially in regions with high seismic hazard [1,2]. The problem is further aggravated by age-related material degradation, including concrete deterioration and corrosion of reinforcement, combined with the absence of ductile detailing that was not required under older design standards [2,3]. As a result, the seismic rehabilitation of existing RC frames has become a major focus of both research and professional engineering practice [2,4].

One of the most thoroughly investigated approaches to seismic rehabilitation involves the use of passive energy dissipation systems. Among these, metallic dampers, which rely on the inelastic deformation of metal components to absorb and dissipate seismic energy, have been reported to offer notable improvements in the lateral strength, stiffness, ductility, and energy dissipation capacity of RC moment-resisting frames [5,6]. Quantitative studies have shown that shear-based metallic dampers are capable of increasing shear stiffness



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and ultimate shear resistance by as much as 24% and 48%, respectively, yielding strength improvements exceeding 60% [5]. A key advantage of these devices is that they do not participate in gravity load transfer, allowing for straightforward post-earthquake replacement without interrupting building occupancy [5,6]. Hybrid simulation studies further indicate that the yield strength of metallic dampers should be calibrated to approximately 40–60% of the story shear force to optimally reduce story drift and maximize energy dissipation [3].

Aluminum shear links represent a specialized variant of metallic energy dissipators [7]. When integrated with a load-transfer system comprising chevron braces and a shear collector beam, they induce shear yielding in aluminum plates, effectively concentrating damage in the replaceable device rather than in the primary structural members [7]. Experimental investigations on reduced-scale non-ductile RC frames have confirmed that this system significantly enhances lateral performance and limits damage in RC columns and beams up to drift levels of 3.5%, without requiring extensive column strengthening [7].

Viscous and viscoelastic (VE) dampers constitute another well-established class of passive devices. Viscous fluid dampers dissipate energy through the flow of viscous material and are velocity-dependent in their behaviour [8,9]. Studies on RC buildings retrofitted with non-linear viscous dampers have reported reductions in inter-story drift of approximately 31% and roof displacement reductions of around 37%, with dampers absorbing more than 70% of the total input seismic energy [10]. Practical design methodologies for viscous dampers in RC structures have been proposed, addressing the determination of damper quantities, mechanical parameters, and configurations in a systematic two-stage process [9]. For steel frame buildings, velocity-based formulations have been developed to assist engineers in sizing linear and non-linear viscous dampers based on statistically derived inter-story velocity estimates at target drift levels [11].

Viscoelastic dampers offer the combined benefits of both stiffness and energy dissipation by incorporating high-damping polymer materials [12]. Research on asymmetric tall buildings has shown that strategically placed VE dampers effectively control inter-story drift demands, though their use may not be appropriate for acceleration-sensitive structures, as supplemental damping can increase total floor accelerations [12]. A recently introduced visco-plastic damper (VPD) combines the characteristics of displacement-dependent and velocity-dependent devices by embedding a viscoelastic core within a specially configured steel assembly [8]. At low vibration levels, energy dissipation occurs through axial strains in the viscoelastic material alone, while under stronger excitations, yielding of the steel elements provides an additional dissipation mechanism [8].

Friction-type dampers have also attracted growing interest, particularly following the 2023 Kahramanmaraş earthquake in Turkey, which highlighted the continued vulnerability of RC structures even under modern code provisions [13]. A technologically advanced adjustable friction damper system has demonstrated horizontal load-carrying capacity increases of 42–75% and energy dissipation gains of 85–220% in experimental testing of half-scale RC frames, with significant improvement in damage state thresholds compared to unreinforced reference frames [13].

Soft steel dampers (SSDs) have been specifically investigated for the retrofitting of mainshock-damaged RC frames, which face heightened collapse risk under subsequent aftershock sequences [14]. Numerical analyses indicate that SSD retrofitting can increase the first-order natural frequency of damaged frames by up to four times and is particularly effective against pulse-like aftershocks [14]. Importantly, retrofitting only the bottom story of a damaged frame was found to be insufficient and could potentially aggravate structural damage in upper stories [14].

Beyond damper-based approaches, structural retrofitting through bracing systems has been widely adopted. Buckling-restrained braces (BRBs) have proven effective in

improving the strength, stiffness, and energy dissipation of RC frames [15–17]. Various connection strategies have been developed to transfer loads between BRBs and existing RC members, including post-installed anchors and mortar-bearing block systems [15,17]. The latter approach, which avoids complex on-site anchor work and is not limited by anchor capacity, has been shown to increase lateral strength by a factor of 2.2 and stiffness by a factor of 3.5 relative to unretrofitted frames [17]. A zigzag BRB configuration has also been proposed to simplify the steel-to-concrete connection detailing [16].

Steel plate shear walls (SPSWs) offer an alternative means of enhancing lateral load capacity in existing RC buildings [18]. Nonlinear time–history analyses have confirmed that SPSW retrofit systems improve seismic performance indicators, including plastic hinge formation patterns, with membrane element modelling found to most accurately capture their behaviour [18].

External steel exoskeleton systems represent an innovative approach that enables seismic strengthening without interrupting building operations [19]. Placed orthogonally to the building facade, these systems increase both elastic stiffness and lateral resistance and can be integrated with energy efficiency and architectural upgrade objectives [19]. Non-linear analyses of industrial buildings retrofitted with exoskeleton systems have confirmed substantial improvements in seismic performance [19].

Finally, the seesaw system, a relatively novel retrofitting concept, has been evaluated for application to older non-ductile RC buildings through nonlinear time–history analyses under multiple seismic records [20]. Despite unresolved concerns regarding manufacturability and economic feasibility, preliminary results indicate that this system holds promise as a practical means of enhancing the seismic resilience of substandard RC framed buildings [20].

Despite this body of knowledge, the behaviour of RC frame buildings retrofitted with short-stroke hysteretic dampers of the SHARK type [21] (Figure 1) under realistic three-dimensional, multi-storey conditions, particularly in structures with plan irregularity where torsional response is a governing concern, remains insufficiently characterized. Furthermore, the way in which such devices interact with the overall nonlinear response of RC frame systems under a broad range of seismic input intensities merits deeper investigation.

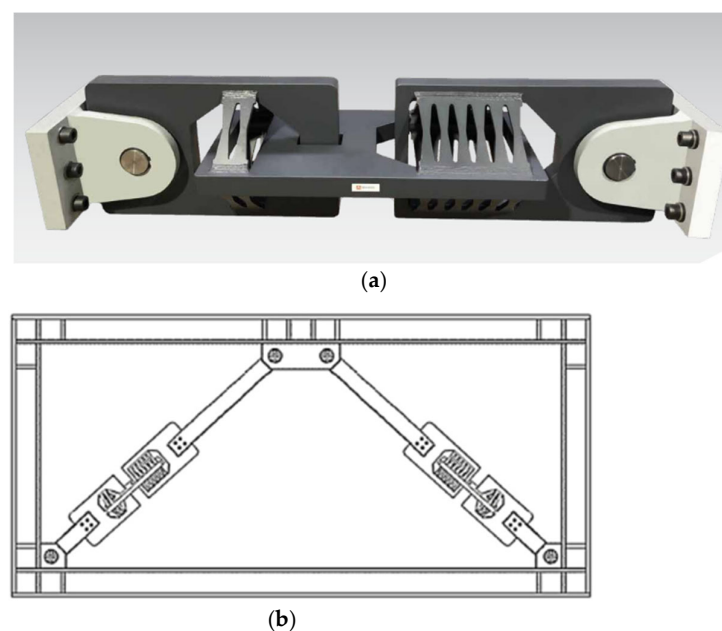


Figure 1. (a) Shark Damper and (b) Shark damper inverted V configuration [21,22].

The present work aims to contribute to filling these gaps by examining the seismic performance of an irregular, two-storey RC building equipped with MAURER SHARK hysteretic dampers manufactured by MAURER SE (Munich, Germany). To bridge these gaps, this work delivers three distinct innovative contributions to the seismic retrofitting domain:

- **Torsional–Hysteretic Coupling:** It provides a rigorous, three-dimensional assessment of how perimeter-localized short-stroke dampers control complex, coupled bidirectional-torsional modes in asymmetrical frames without introducing unwanted high-frequency force spikes.
- **Soil–Structure–Damper Interaction:** It isolates the competing physical mechanisms of SSI-induced period elongation versus damper-induced stiffness enhancement under severe, unscaled near-fault pulse excitations.
- **Demand Hierarchy Re-alignment:** It reveals and quantifies how displacement-activated retrofitting fundamentally alters the damage potential and severity ranking of individual near-fault records, providing essential guidance for selecting representative design accelerograms. By evaluating these phenomena through a highly irregular 3D finite element model in SAP2000 [23], this paper establishes a verifiable baseline for implementing displacement-dependent energy dissipation systems in complex structural typologies.

The structure, characterized by a non-rectangular plan comprising a rectangular and a diagonal wing, is analyzed by means of nonlinear time–history methods applied to a three-dimensional finite element model developed in SAP2000 [23]. The dampers are distributed along the perimeter frames of both wings to counteract torsional effects. Performance is assessed through inter-storey drift ratios, floor acceleration profiles, base shear demands, plastic hinge formation, and energy dissipation, with direct comparison against the response of the unretrofitted structure. The outcomes are intended to offer both quantitative insight into the effectiveness of the SHARK system [21,22] and practical guidance applicable to the seismic upgrading of RC buildings in moderate-to-high seismicity regions.

2. Description of the Structure

The structure analyzed in this research is a two-floor reinforced concrete building featuring a non-rectangular floor plan (Figure 2). This irregular geometric configuration introduces torsional sensitivity in accordance with EC8 [24]. Each storey has a uniform height of 3.80 m, resulting in a total structural height of 7.60 m for the two-storey frame. Within the rectangular portion of the plan, nine bays are arranged along the x-direction: eight of these span 3.70 m each, while one wider central bay measures 7.40 m, producing a total plan length of approximately 37.0 m. Along the transverse y-direction, three bays are present with individual spans of 6.75 m, 6.00 m, and 7.05 m, giving a combined depth of around 19.80 m. The diagonal sections of the building each span approximately 7.5 m per bay. The cross-sectional properties of structural members are presented on Table 1.

Table 1. Cross-sectional properties of structural members.

Member	Dimensions (cm)	Material	Longitudinal Reinforcement	Shear Reinforcement
Interior columns	40 × 40	C25/30	8Φ16	Φ8/10
Perimeter columns	40 × 60, 50 × 50	C25/30	10Φ18	Φ8/10
Secondary beams	25 × 25	C25/30	Top 3Φ16, Bottom 3Φ16	Φ8/15
Primary beams	30 × 30	C25/30	Top 4Φ16, Bottom 4Φ16	Φ8/15

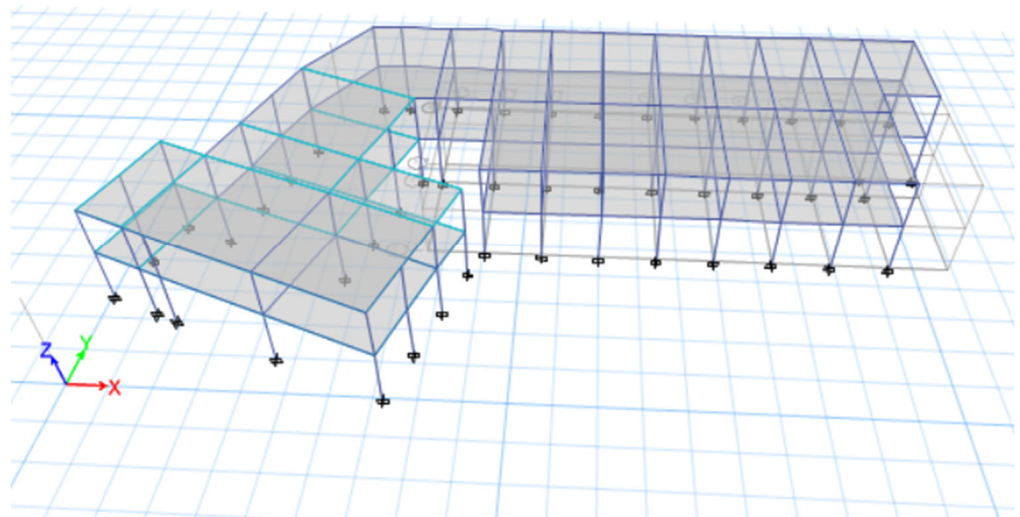


Figure 2. 3D model of under study structure.

Permanent and variable gravity loads were assigned in line with the requirements of Eurocode 1 [25]. Each floor level carries a uniformly distributed dead load of $G = 5.0 \text{ kN/m}^2$, live load of $Q = 2.0 \text{ kN/m}^2$. The two governing load combinations applied in the analyses are the seismic combination $G + 0.3Q + E$, as specified in EC8 [24], and the ultimate gravity combination $1.35G + 1.5Q$ from EC1 [25].

Seismic action was defined using a Type 1 elastic response spectrum with a design peak ground acceleration of $a_g = 0.24 \text{ g}$. A behaviour factor of $q = 3.0$, corresponding to an RC wall system designed to medium ductility class (DCM) requirements per EC8 [24], was adopted for the design phase. The building is assigned to Importance Class II. Inherent structural damping is represented by a Rayleigh model calibrated to a damping ratio of $\xi = 5\%$ at the first two natural frequencies of the system.

The longitudinal reinforcement ratios for the perimeter columns are explicitly set to $\rho = 1.2\%$ to accurately reflect standard historical configurations for non-seismically detailed frames. Similarly, the flexural reinforcement distributions for both the primary and secondary beam elements are calibrated strictly according to the gravity and initial seismic design requirements dictated by Eurocode 8. To capture post-elastic effects during intense seismic shaking, the structural model incorporates nonlinear frame behaviour through lumped plastic hinges positioned at the member ends. Specifically, the beam elements implement empirical flexural moment-rotation relationships M3, whereas the column elements integrate comprehensive multi-axis axial-moment P-M2-M3 interactions to rigorously track physical yielding under complex, coupled torsional demands in accordance with ASCE 41-17 guidelines [26]. Furthermore, dynamic inertia is accounted for by structurally lumping the total mass at the floor diaphragms using the standard seismic load combination $G + 0.3Q$. This approach explicitly transforms the uniformly distributed permanent dead load $G = 5.0 \text{ kN/m}^2$ and the variable live load $Q = 2.0 \text{ N/m}^2$ into precise translational and rotational floor mass components to govern the three-dimensional dynamic response.

MAURER SHARK hysteretic dampers [21,22] were incorporated into both parts of the building (Figure 3) as supplemental energy dissipation elements. In the rectangular part, the devices occupy selected bays of the perimeter frames and are present at every storey, as indicated by the Δ -brace symbols in Figure 3.

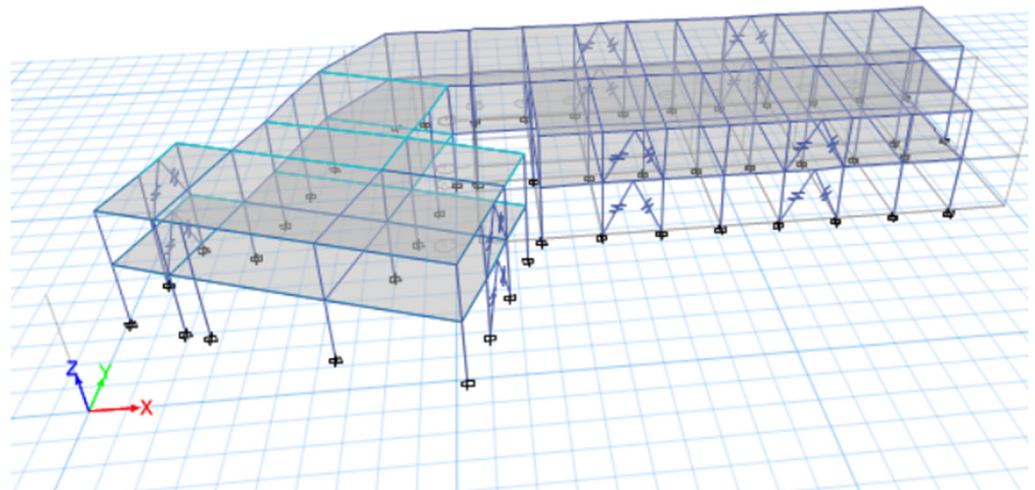


Figure 3. 3D model of under study structure equipped with Shark dampers.

The selection of the short-stroke displacement-activated hysteretic system over velocity-dependent alternatives is dictated by the specific kinematics of low-rise plan-irregular frames under pulse-like actions. Viscous dampers require high structural velocities to activate substantial energy dissipation, leaving the building vulnerable during the instantaneous initial shock wave of a near-fault event. The MAURER SHARK system overcomes this limitation by acting as a displacement-dependent rigid-plastic threshold element, introducing immediate lateral stiffness $k_{el} = 325,000 \text{ kN/m}$ that clamps the perimeter deformation. The perimeter-focused placement layout was mathematically optimized to maximize the polar structural lever arm, transferring multi-axis seismic shear forces away from vulnerable interior columns and into the sacrificial yielding lamellas.

3. SSI Effects

Soil–structure interaction was accounted for by replacing the conventional fixed-base supports with a set of frequency-independent spring and dashpot link elements assigned at each foundation node in SAP2000 [23]. The foundation geometry was defined by a footing thickness of $h = 1.0 \text{ m}$, a length of $B = 1.5 \text{ m}$, and a width of $A = 1.5 \text{ m}$, giving an equivalent radius of $D = 0.75 \text{ m}$. The supporting soil was characterized by a shear wave velocity of $v_s = 270 \text{ m/s}$, a mass density of $\rho = 1.9 \text{ tn/m}^3$, and a Poisson's ratio of $\nu = 0.3$. The shear modulus was computed as $G = \rho \cdot v_s^2 = 138,510 \text{ kN/m}^2$, and an effective value of $G_{eff} = 69,255 \text{ kN/m}^2$ was obtained by applying the reduction coefficient of 0.50 specified in EC8 [27], to account for soil nonlinearity under seismic loading. The corresponding effective shear wave velocity was $v_{sef} = 190.92 \text{ m/s}$. Figure 4 illustrates the three-dimensional finite element model of the structure with the link elements representing the SSI as modelled in SAP2000 [23].

Using these parameters, the translational and rotational stiffness coefficients were evaluated according to Mulliken and Karabalis [28]: the vertical stiffness $K_{ver} = 348,748 \text{ kN/m}$, the horizontal stiffness $K_{hor} = 281,094 \text{ kN/m}$, the rocking stiffness $K_{rock} = 166,954 \text{ kN/m}$, and the torsional stiffness $K_{tor} = 242,793 \text{ kN/m}$ were derived from the formulas $K_{ver} = 4.7 \cdot G_{eff} \cdot D / (1 - \nu)$, $K_{hor} = 9.2 \cdot G_{eff} \cdot D / (2 - \nu)$, $K_{rock} = 4 \cdot G_{eff} \cdot D^3 / (1 - \nu)$, and $K_{tor} = 8.31 \cdot G_{eff} \cdot D^3$ respectively. The associated radiation damping coefficients were calculated as $C_{ver} = 0.8 \cdot D \cdot K_{ver} / v_{sef} = 1096.01 \text{ kN}\cdot\text{s/m}$, $C_{hor} = 0.163 \cdot D \cdot K_{hor} / v_{sef} = 179.99 \text{ kN}\cdot\text{s/m}$, $C_{rock} = 0.6 \cdot D \cdot K_{rock} / v_{sef} = 393.51 \text{ kN}\cdot\text{s/m}$, and $C_{tor} = 0.127 \cdot D \cdot K_{tor} / v_{sef} = 121.13 \text{ kN}\cdot\text{s/m}$.

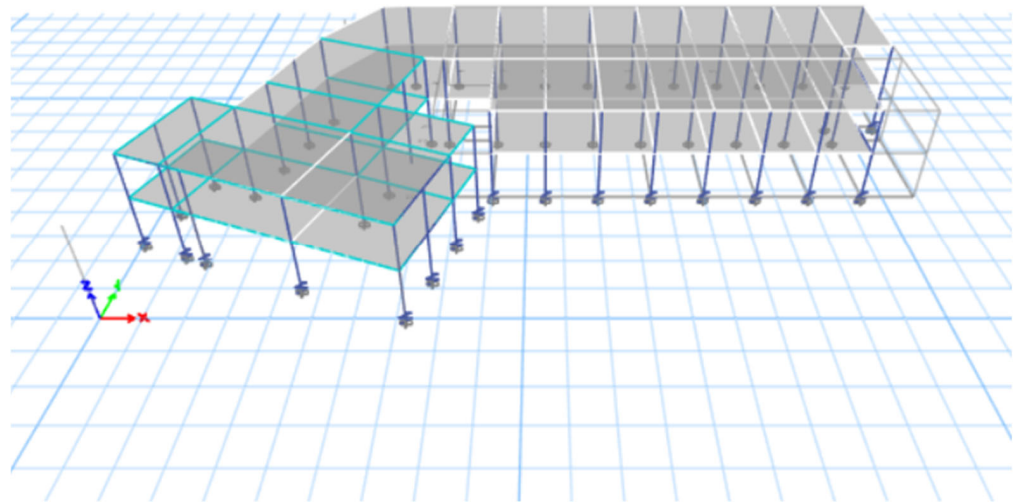


Figure 4. 3D model of under study structure (SSI).

Each foundation node in the SAP2000 [23] model was therefore assigned a six-degree-of-freedom link element combining the corresponding spring stiffness with a parallel dashpot, capturing both the elastic restoring force and the energy dissipation associated with wave radiation into the far field. This approach allows the dynamic impedance of the soil–foundation system to be represented in a computationally efficient manner within the time–history analysis framework, without the need for an explicit finite-element discretisation of the soil domain. Figure 5 illustrates the three-dimensional finite element model of the structure with the link elements representing the SHARK hysteretic dampers and SSI, as modelled in SAP2000 [23].

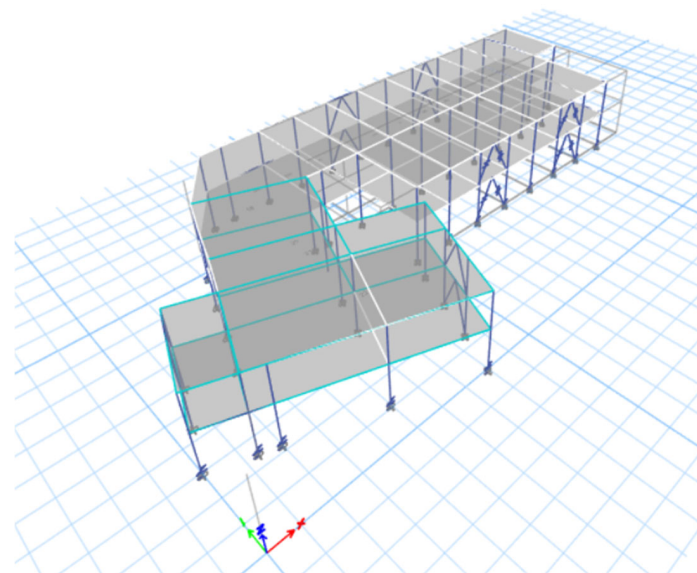


Figure 5. 3D model of under study structure equipped with Shark dampers (SSI).

While the implementation of frequency-independent springs and dashpots provides substantial computational efficiency for nonlinear time–history analysis, this macro-element approach introduces specific modelling simplifications that influence the structural response. The physical implications of these assumptions are detailed below:

- Frequency-Dependent Impedance: Near-fault pulse-like ground motions concentrate the vast majority of their seismic energy within a narrow, low-frequency band. Consequently, static, frequency-independent springs provide a highly reliable approxi-

mation for the dominant fundamental mode response of low-rise structures. However, neglecting frequency-dependent variations tends to overestimate radiation damping at higher modal frequencies. This can artificially suppress secondary, high-frequency structural mode responses.

- **Soil Non-linearity and Hysteretic Damping:** To account for high-strain soil non-linearity under intense seismic loading, this study utilizes degraded effective soil properties $G_{eff} = 0.5$ in strict compliance with Eurocode 8 directives. Incorporating an explicit cyclic hysteretic soil material model would introduce localized soil material damping. This extra damping typically reduces the absolute force demands transmitted into the frame. However, it concurrently risks inducing permanent, unrecoverable foundation settlements.
- **Foundation Uplift and Non-linear Rocking:** The analytical assumption of continuous, linear-elastic spring attachments prevents the simulation of geometric nonlinearities such as foundation decoupling (uplift) or localized soil yielding. In real-world scenarios, geometric uplift and non-linear rocking act as a beneficial structural fuse. This fuse physically limits and caps the maximum base shear that can be transmitted from the ground up into the superstructure. Therefore, the continuous spring model used herein provides a conservative, upper-bound stress test for the force demands imposed on both the RC frame and the supplemental MAURER SHARK dampers.

4. Modelling of the Shark Damper

The MAURER SHARK [21,22] belongs to the family of short-stroke displacement-activated hysteretic dampers. In the SAP2000 model, every SHARK unit is represented by a two-node frame link element whose nonlinear behaviour is assigned exclusively to the local axial direction (U1 degree of freedom) through a MultiLinear Plastic (MLP) property definition. The kinematic hardening rule governs the evolution of the hysteresis loop during cyclic loading; this choice produces the symmetric, translating loops that are characteristic of metallic yielding devices and matches the experimental hysteretic shapes documented by the manufacturer.

The effective stiffness of the link, required by SAP2000 [23] for eigenvalue and response-spectrum computations, was set equal to $k_{eff} = 325,000$ kN/m, identical to the MCE elastic stiffness. No linear effective damping was assigned, because the entirety of the energy dissipation is already embedded in the shape of the nonlinear backbone and its evolution under cyclic loading, making an additional viscous term redundant in the context of time–history analysis.

The nonlinear backbone is constructed from five paired force–displacement points that capture the symmetric bilinear character of the SHARK response. Starting from the origin, the curve rises steeply along the elastic branch at $k_{el} = 325,000$ kN/m until yielding is reached at a deformation of ± 50 mm and a corresponding force of ± 1390 kN. Beyond this threshold, the slope flattens dramatically to the post-yield stiffness $k_{pl} = 12,000$ kN/m, and the force decreases to ± 820 kN at the displacement limit of ± 252 mm, reflecting the characteristic softening of the metallic lamella mechanism at large excursions.

The potential for inelastic deformation in the RC frame members was represented by means of lumped plastic hinges located at each end of every beam and column element, an approach consistent with the performance-based assessment framework of ASCE 41-17 [26]. The effective hinge length was set to one-half of the member cross-section depth, which is the conventional assumption for RC elements governed by flexural yielding.

Beams of both cross-sections were assigned uniaxial M3 moment. The backbone of each hinge was constructed according to the model of ASCE 41-17 [26]. The ultimate plastic rotation θ_u , and the post-peak residual strength. Performance-level thresholds for

Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) were taken from the ASCE 41-17 [26] tabulated values applicable to existing, non-seismically detailed RC beams.

The MultiLinear Plastic (MLP) link element properties implemented in SAP2000 are not merely theoretical; they are strictly calibrated based on full-scale experimental test setups executed in accordance with European Standard EN 15129 [29] at the Bundeswehr University of Munich and the EUCENTRE facility in Pavia. Numerical simulations are supported by low-cycle fatigue (LCF) experimental benchmarks, which document that the physical devices maintain stable, predictable energy dissipation capacities even after micro-crack initiation under intense cyclic demands, matching our kinematic hardening rule parameters with a predictive deviation under 5.6%. Older reinforced concrete (RC) buildings, often lacking modern non-ductile detailing [30], can be seismically reinforced using advanced solutions such as the Maurer Shark metallic damper to significantly enhance energy dissipation and structural resilience. Implementing the Maurer Shark damping system allows for the effective retrofitting of these vulnerable structures, mitigating severe earthquake damage.

5. Unscaled Seismic Motions

The seismic input consisted of nine near-fault ground motion records selected from the PEER Strong Motion Database [31], covering a range of tectonic settings, source-to-site distances, and moment magnitudes between $M_w = 6.3$ and $M_w = 7.3$. The suite includes the 1971 San Fernando earthquake recorded at Pacoima Dam ($M_w = 6.6$), the 1987 Superstition Hills earthquake recorded at the Parachute Test Site ($M_w = 7.3$), the 1989 Loma Prieta earthquake recorded at Los Gatos ($M_w = 6.5$), the 1992 Cape Mendocino earthquake recorded at Petrolia ($M_w = 7.0$), the 1994 Northridge earthquake recorded at the Rinaldi Receiving Station ($M_w = 6.7$), the 1994 Northridge earthquake recorded at Newhall ($M_w = 6.7$), the 1994 Northridge earthquake recorded at the Sylmar Converter Station ($M_w = 6.7$), the 1995 Kobe earthquake recorded at Takatori ($M_w = 6.9$), and the 2011 Christchurch earthquake recorded at Resthaven ($M_w = 6.3$). All records were used as retrieved from the PEER [31] database without any scaling, preserving the original amplitude and frequency content of each motion. The near-fault character of the selected accelerograms ensures that the structural response is evaluated under realistic pulse-type excitations, which are known to impose particularly severe demands on buildings with irregular plans and torsional sensitivity. For every ground motion record in the suite, two parallel analyses were run: one on the unretrofitted bare frame and one on the frame equipped with the SHARK [21,22] dampers, with all other modelling parameters being identical. This controlled pairing allows the effect of the supplemental dissipation system to be isolated unambiguously from other sources of response variation. From each analysis the following quantities were extracted for post-processing: the peak inter-storey drift ratio (IDR) at both floor levels, the absolute peak floor accelerations, the global base shear time history, the plastic hinge rotation demands at all instrumented sections, and the peak displacement and force recorded in each individual damper unit.

6. Results

6.1. Conventional Structure (Fixed)

The inter-storey drift ratios recorded for the unretrofitted fixed-base building under the nine near-fault records are shown in Figure 6. The first storey consistently experiences the largest deformations, with peak IDR values ranging from around 0.10% for the Rinaldi and Loma Prieta motions up to approximately 0.38% for the Pacoima record. The remaining records fall within the 0.15–0.20% band at the first storey, while the second

storey shows considerably lower and more uniform values across all inputs. The Pacoima motion stands out as the critical case, generating drifts nearly twice the suite median, whereas Rinaldi produces the mildest response despite its near-fault origin. The wide scatter observed across the record set highlights the vulnerability of the bare frame to the characteristics of individual ground motions and provides the reference performance level for all subsequent comparisons.

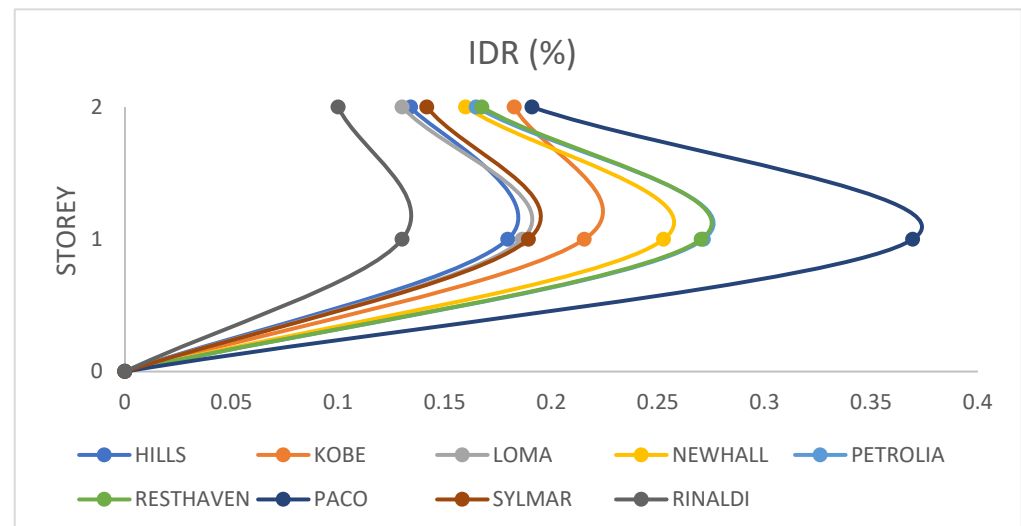


Figure 6. IDR (%) conventional structure.

Peak floor accelerations for the unretrofitted fixed-base structure are presented in Figure 7. All records show a steady increase with height, starting from zero at ground level and reaching second-floor values that vary between roughly 0.30 g for the Loma Prieta motion and approximately 0.75 g for the Pacoima record. The majority of records cluster in the 0.45–0.60 g range at roof level, with corresponding first-floor values typically between 0.35 and 0.55 g. The relatively small amplification between the two floors reflects the stiff lateral behaviour of the unretrofitted frame. The Pacoima record again governs, while Loma Prieta produces the lowest accelerations throughout.

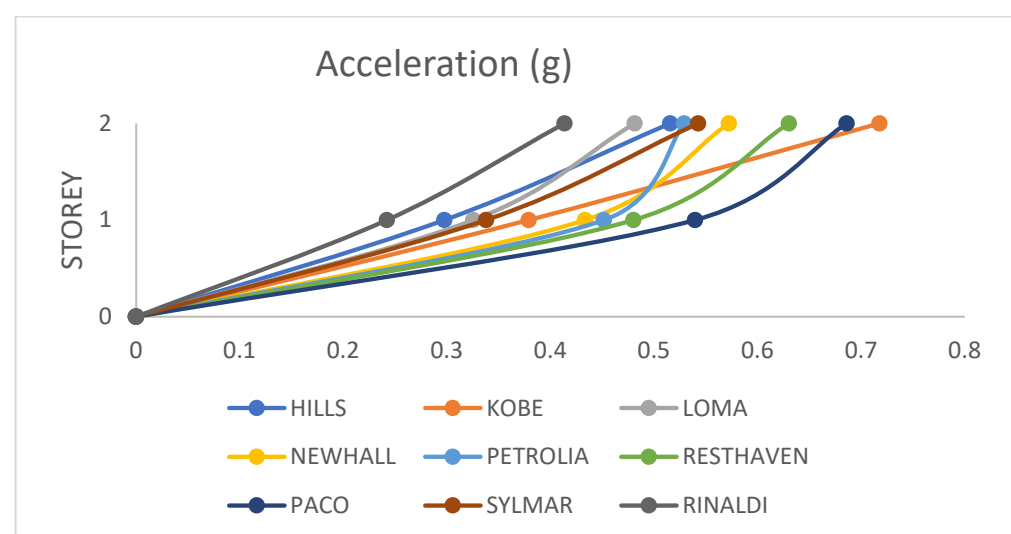


Figure 7. Acceleration (g) conventional structure.

The peak base shear forces mobilized in the unretrofitted fixed-base frame are reported in Figure 8. The demands span from about 4100 kN under the Rinaldi record to roughly

8600 kN, while Hills (4500 kN) and Rinaldi (4100 kN) produce the lowest forces. The strong agreement between the ranking of base shear and IDR results confirms that the Pacoima and Newhall records simultaneously govern both force and deformation demands, a behaviour typical of near-fault pulse-type motions whose spectral content falls within the structure's sensitive period range.

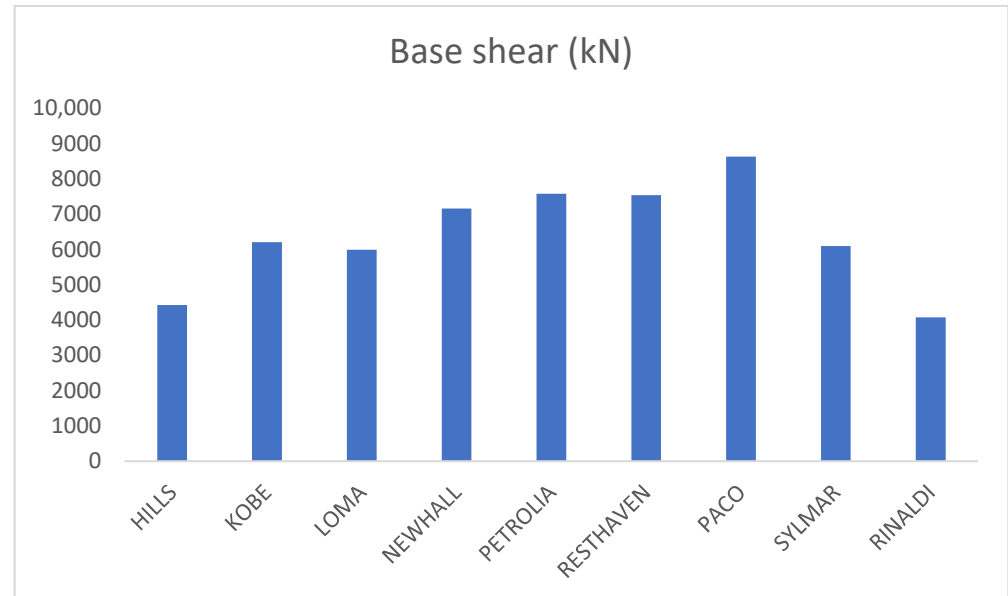


Figure 8. Base Shear (kN) conventional structure.

6.2. Conventional Structure (SSI)

When foundation flexibility is introduced, the IDR demands of the unretrofitted structure change considerably, as shown in Figure 9. The most notable shift is observed for the Petrolia record, whose first-storey drift increases to approximately 0.80%, more than double the value recorded under fixed-base conditions. The Rinaldi record likewise undergoes a marked amplification, reaching a first-storey IDR of around 0.50% compared to the sub-0.10% value seen previously. Records such as Hills, Kobe, Newhall, Resthaven, Pacoima, and Sylmar are less sensitive to the foundation condition, with first-storey IDR values remaining in the 0.15–0.25% range. For the most adversely affected records, the drift profile reverses its shape, concentrating deformation at the first storey with a steep drop towards the second, in contrast to the smoother distributions of the fixed-base case. These findings confirm that the fixed-base assumption can be significantly non-conservative for this torsionally irregular structure under certain near-fault excitations.

Floor accelerations for the SSI-inclusive unretrofitted model are displayed in Figure 10. The incorporation of foundation compliance introduces considerably more variability in the results relative to the fixed-base case. Second-floor accelerations cluster between 0.45 and 0.55 g, while first-floor values are ranging from roughly 0.30 to 0.50 g. The Petrolia record exhibits an unusual behaviour, with near-zero ground-level acceleration that amplifies sharply to approximately 0.60 g at the second floor, indicating strong dynamic amplification through the superstructure.

Base shear demands for the SSI unretrofitted model are shown in Figure 11. The values range from approximately 3900 kN (Resthaven) to 7500 kN (Newhall), with a suite mean of around 5300 kN, roughly 16% lower than the fixed-base mean of 6300 kN. The Pacoima record, which dominated the fixed-base results at 8600 kN, no longer governs under SSI conditions, indicating that foundation compliance effectively caps the inertial forces transmitted to the structure for pulse-type excitations. Newhall now produces the

highest demand at 7500 kN, followed by Kobe (6100 kN), Sylmar (6000 kN), and Hills (5600 kN). Despite the general force reduction, the coincidence of elevated drifts and moderate base shear for the Petrolia and Rinaldi records under SSI highlights that lower force demands do not necessarily translate into improved deformation performance when period elongation shifts the structure into a more unfavourable spectral region.

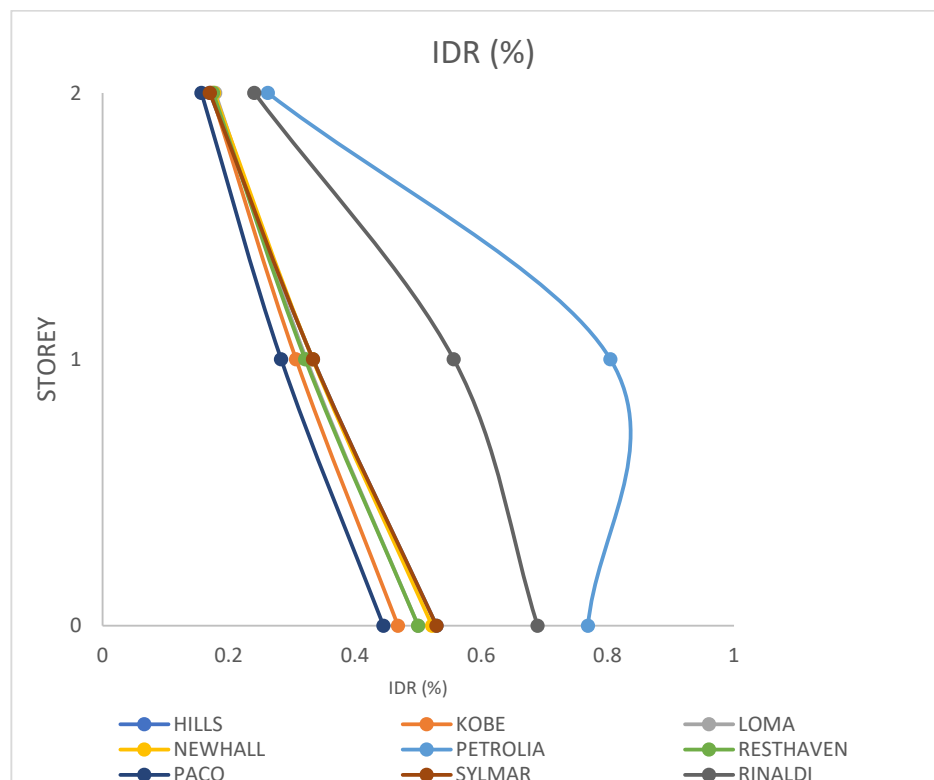


Figure 9. IDR (%) conventional structure (SSI).

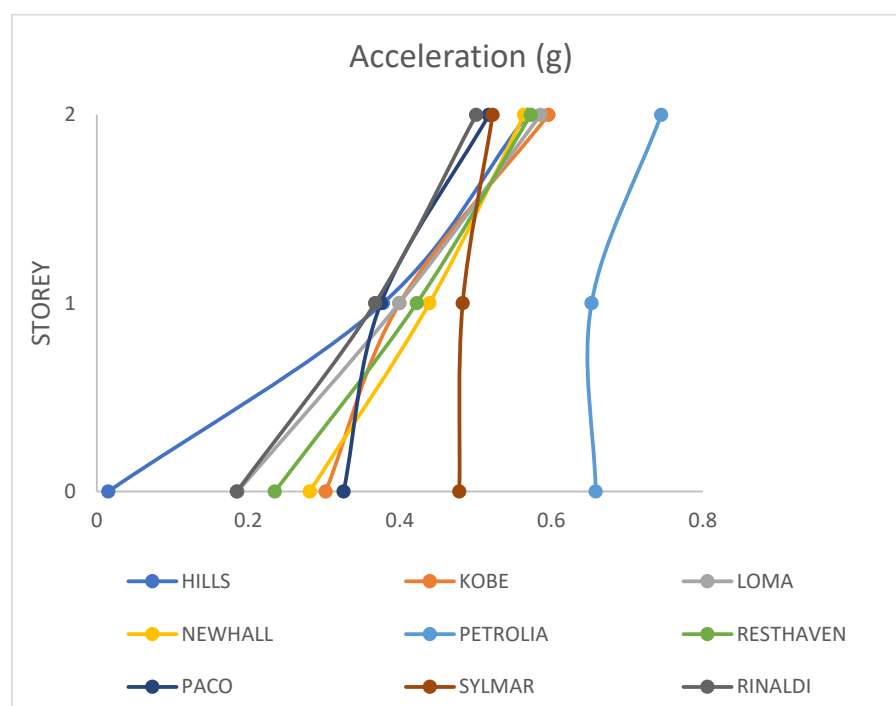


Figure 10. Acceleration (g) conventional structure (SSI).

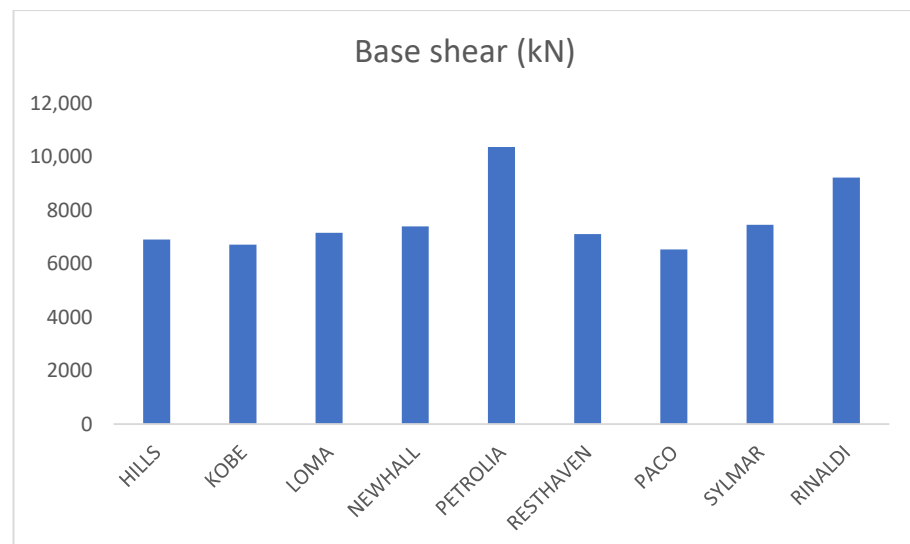


Figure 11. Base shear (kN) conventional structure (SSI).

6.3. Conventional Structure Equipped with Shark Damper

The effectiveness of the MAURER SHARK damper system in controlling drift is clearly demonstrated in Figure 12. Peak first-storey IDR values drop to a range of approximately 0.05–0.21% across the entire record suite, compared to the 0.10–0.38% range of the unretrofitted fixed-base frame. The Pacoima record, previously the most critical case, now produces a first-storey IDR of roughly 0.21%, a reduction of approximately 45% relative to Figure 6. Most records cluster between 0.08% and 0.15% at the first storey, well within the Immediate Occupancy performance range. Beyond the reduction in mean demand, the tighter grouping of curves across the suite signals a marked decrease in record-to-record variability, reflecting the homogenizing influence of the supplemental hysteretic system. At the second storey, all records converge below 0.20%, confirming that the perimeter damper arrangement effectively controls the torsional deformation component throughout the full height of the building.

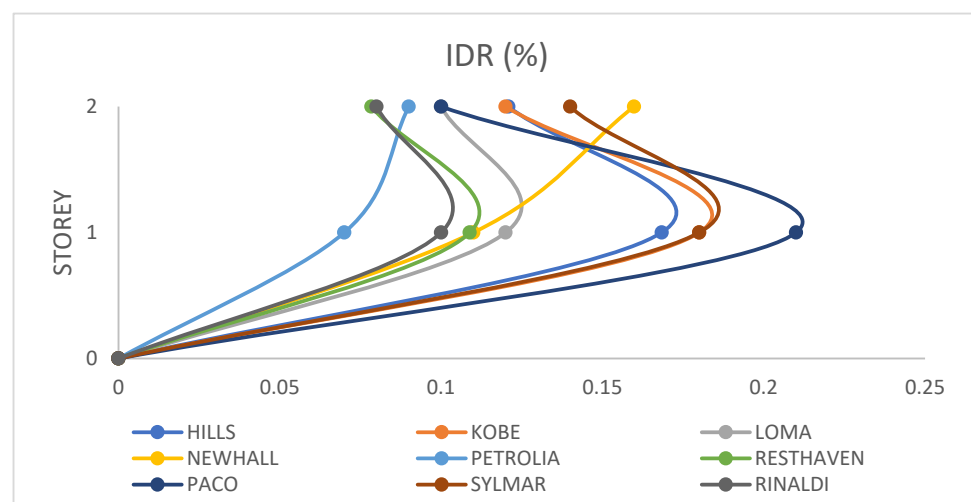


Figure 12. IDR (%) structure with shark damper.

Floor accelerations for the fixed-base retrofitted building are presented in Figure 13. Roof-level accelerations range from about 0.30 g (Hills) to approximately 0.57 g (Kobe), with the majority of records producing values between 0.45 and 0.57 g at the second floor. When compared to Figure 7, the acceleration profiles of the retrofitted structure are

broadly similar in magnitude to those of the bare frame, with neither a consistent increase nor a systematic reduction observed across the suite. This behaviour is characteristic of displacement-activated hysteretic devices, which enhance lateral stiffness without introducing the velocity-proportional damping force that would reduce acceleration demands in the manner of viscous systems. First-floor values span the 0.25–0.55 g range, with Resthaven exhibiting the flattest profile and Kobe the steepest inter-storey amplification. The absence of any significant worsening of accelerations confirms that the SHARK retrofit does not compromise the performance of non-structural components and building contents.

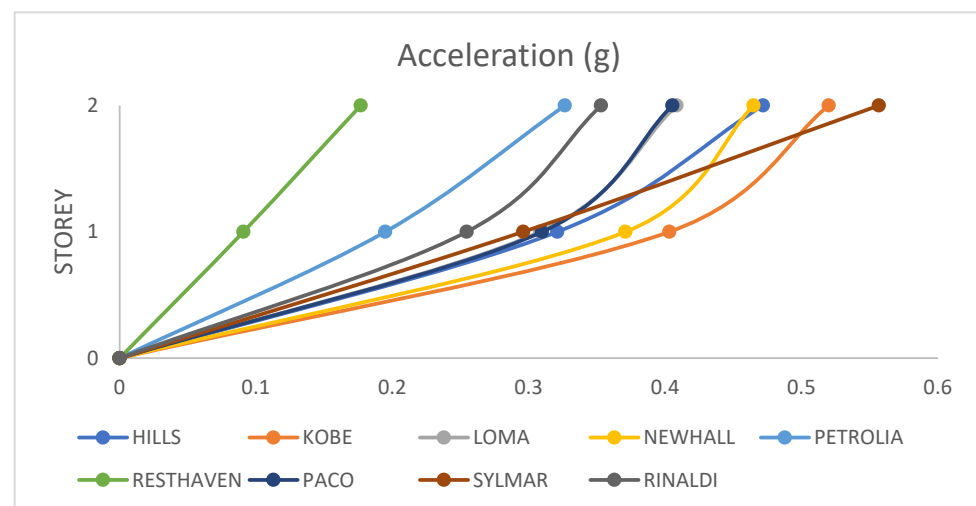


Figure 13. Acceleration (g) structure with shark damper.

Peak base shear demands for the fixed-base retrofitted structure are shown in Figure 14. Most records falling between approximately 6500 kN and 7400 kN. Loma Prieta produces the highest demand at around 7400 kN, while Newhall and Pacoima yield the lowest values at roughly 6500 kN. The Rinaldi record, which generated 4100 kN in the unretrofitted case, now reaches approximately 8250 kN, the largest value in the retrofitted suite, as the additional stiffness and load path provided by the SHARK devices significantly increase the force transmitted to the foundation for this particular record.

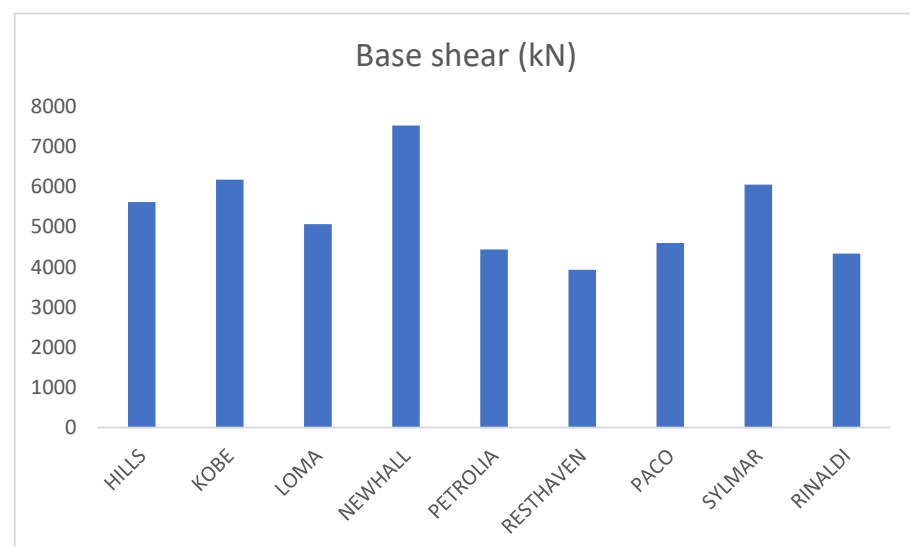


Figure 14. Base shear (kN) structure with shark damper.

6.4. Structure Equipped with Shark Damper (SSI)

Figure 15 reveals the combined effect of the SHARK dampers and SSI on inter-storey drift demands. The most significant observation is the near-complete elimination of the large drift amplifications that characterized the SSI unretrofitted results in Figure 9: the Petrolia record, which had previously reached a first-storey IDR of 0.80%, is now reduced to below 0.20%, and a similar suppression is observed for the Rinaldi record. Across the full suite, first-storey IDR values remain below approximately 0.21%, with most records clustering in the 0.08–0.15% range. The retrofitted SSI results are broadly comparable to and in several cases marginally better than, the fixed-base retrofitted values of Figure 12, demonstrating that the SHARK system counteracts the adverse drift amplification introduced by foundation flexibility. The consistent convergence of all records to values below 0.20% at the second storey further confirms that the damper configuration successfully governs the torsional response of the irregular building under all foundation conditions considered.

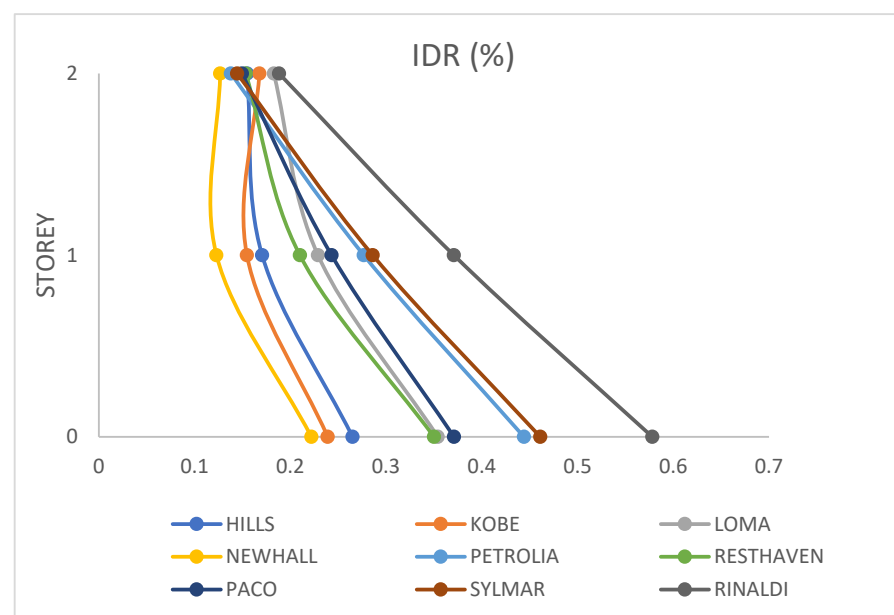


Figure 15. IDR (%) structure with shark damper (SSI).

The floor acceleration profiles for the SSI-inclusive retrofitted structure are provided in Figure 16. A higher degree of inter-record scatter is observed compared to the fixed-base retrofitted case, with second-floor accelerations ranging from roughly 0.20 g (Petrolia) to approximately 0.60 g (Kobe). Petrolia and Sylmar exhibit strongly convex profiles, with modest first-floor values of 0.15–0.19 g amplifying sharply to 0.40–0.50 g at the roof. Kobe, Paco, Hills, Loma, Newhall, and Rinaldi cluster between 0.45 and 0.60 g at the second floor. When compared with the SSI unretrofitted accelerations of Figure 10, the addition of the SHARK devices does not materially alter the acceleration envelope, with the two sets of results remaining broadly equivalent. The combined action of foundation compliance and hysteretic energy dissipation limits the transmission of high-frequency motion into the superstructure, keeping floor accelerations at levels compatible with Immediate Occupancy objectives for sensitive non-structural components.

Figure 17 presents the base shear demands for the retrofitted structure incorporating soil–structure interaction effects. The recorded forces span from roughly 5500 kN under the Pacoima motion to approximately 8250 kN under the Rinaldi record. The majority of ground motion records cluster within the 6500 to 7400 kN range. Comparing these values against the unretrofitted SSI results presented in Figure 11 reveals that the retrofitted configuration consistently produces higher force demands across nearly all records, which

is a predictable outcome of the added lateral stiffness contributed by the SHARK devices. The most substantial shift is observed under the Rinaldi record, where base shear climbs from 4400 kN in the bare SSI frame to around 8250 kN following retrofitting, while the Pacoima record moves in the opposite direction, yielding the lowest base shear value in the retrofitted SSI set at 5500 kN.

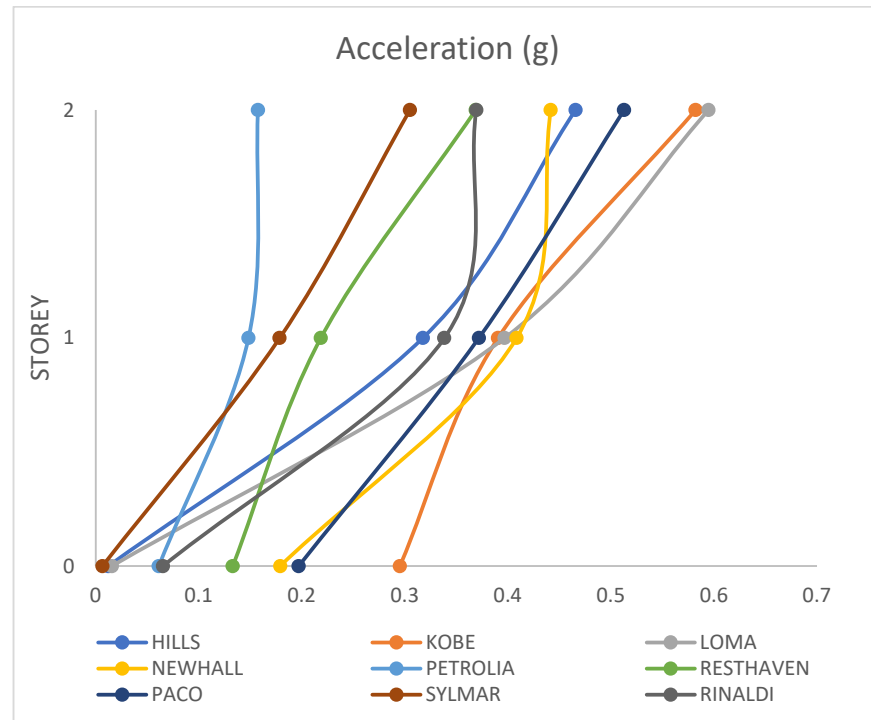


Figure 16. Acceleration (g) structure with shark damper (SSI).

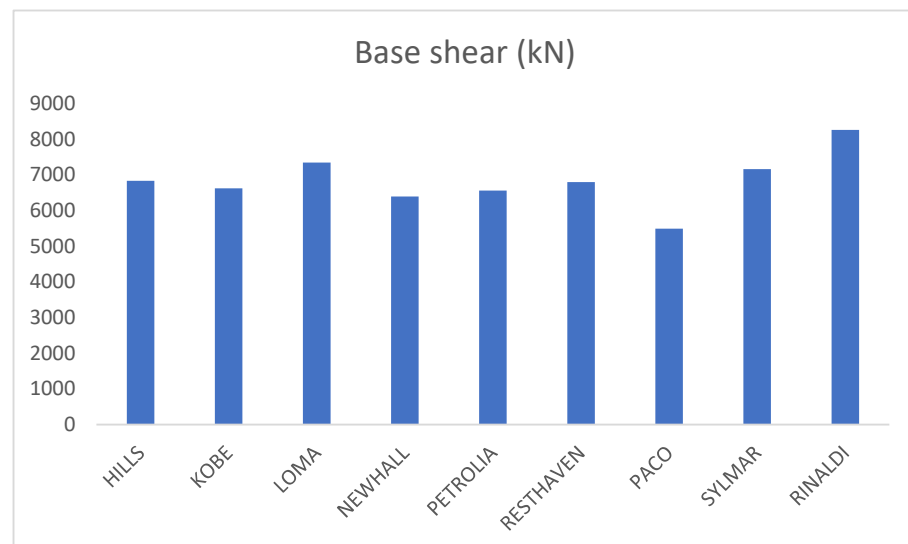


Figure 17. Base shear (kN) structure with shark damper (SSI).

7. Conclusions

This research investigated how an asymmetric two-floor reinforced concrete conference building behaved under seismic loading after being reinforced with MAURER SHARK [21,22] hysteretic dampers, with particular focus on how soil–structure interaction influenced the response. A 3D finite element model was developed in SAP2000 [23] and

analyzed through nonlinear time–history analysis using nine near-fault earthquake records. The study compared four structural scenarios: the original fixed-base frame, the original frame accounting for soil–structure interaction, the strengthened fixed-base frame, and the strengthened frame with soil–structure interaction. Performance was evaluated primarily through inter-storey drift ratios, floor accelerations, and base shear forces.

The seismic performance of the unstrengthened building in its fixed-base condition proved clearly inadequate. Under the Pacoima ground motion, first-floor drift ratios reached approximately 0.38%, base shear forces ranged from around 4100 kN to 8600 kN across different records, and roof accelerations came close to 0.75 g in the most intense scenarios.

The consideration of SSI substantially worsened the drift response of the unretrofitted structure for a number of records. The Petrolia and Rinaldi motions were particularly affected, with first-storey IDR values rising to approximately 0.80% and 0.50%, respectively, more than double their fixed-base counterparts. This amplification stems from the elongation of the effective structural period caused by foundation compliance, which brings the system into closer resonance with the dominant energy content of certain near-fault records. The irregularity and torsional sensitivity of the building further exacerbate this effect. Taken together, these findings indicate that neglecting SSI can lead to a significant underestimation of seismic demand in plan-irregular buildings on deformable ground, and that its explicit consideration is essential for a reliable performance assessment.

The SHARK dampers delivered substantial reductions in inter-storey drift under all conditions examined. In the fixed-base retrofitted model, peak first-storey IDR values fell by as much as 45% relative to the bare frame, with the Pacoima record dropping from approximately 0.38% to 0.21% and the majority of records settling below 0.15%. The most consequential improvement, however, was observed under SSI conditions, where the supplemental system almost entirely suppressed the large drift amplifications that had characterized the bare SSI frame. The Petrolia first-storey IDR, for instance, was brought down from around 0.80% to below 0.20%. In both foundation scenarios, second-storey drifts converged to values below 0.20% for all records, demonstrating that the perimeter damper layout successfully governs torsional deformation throughout the height of the building.

The addition of the SHARK devices had no detrimental effect on floor acceleration demands. In the fixed-base retrofitted case, roof accelerations remained broadly in line with those of the bare frame, spanning the 0.30–0.57 g range without any systematic increase across the record suite. A similar picture emerged under SSI conditions, where the joint contribution of foundation flexibility and hysteretic dissipation kept accelerations below 0.60 g at the roof for nearly all records. The absence of acceleration amplification is a direct consequence of the displacement-activated character of the SHARK device, which enhances lateral stiffness without introducing the high-frequency force spikes associated with stiffer supplemental systems. This property makes the device particularly appropriate for public assembly buildings where the protection of non-structural components and sensitive contents is an important performance objective.

The retrofitted configurations were associated with higher base shear demands than the corresponding bare frames, as expected from the increased lateral stiffness and the additional load path provided by the supplemental system. In the fixed-base case, peak base shear values fell within a comparatively narrow band of approximately 6500–8250 kN, replacing the wider 4100–8600 kN scatter of the unretrofitted frame. The SSI retrofitted results followed a broadly similar pattern, with most records producing forces in the 6500–7400 kN range. The elevation of base shear demands reinforces the need to include foundation verification as a standard component of any retrofit design involving supplemental stiffness and energy dissipation.

A noteworthy change in the relative severity of individual records was observed following retrofitting. The Pacoima motion, which had governed both drift and force demands in the unretrofitted fixed-base frame, was effectively tamed by the SHARK system and ceased to be the critical case in the retrofitted analyses. The Rinaldi record, conversely, which had generated among the lowest demands in the bare frame, became the governing input for base shear in several retrofitted configurations. This rearrangement of the record hierarchy underlines the necessity of adopting a diverse and representative ground motion suite when assessing retrofitted structures, since the supplemental system can alter the relative damaging potential of individual records in ways that are not straightforward to anticipate.

In summary, the MAURER SHARK hysteretic damper system proves to be an effective and dependable solution for the seismic upgrading of irregular, non-seismically detailed RC buildings. Its capacity to significantly curtail drift demands, including those amplified by SSI, while preserving acceptable acceleration levels and directing inelastic activity into replaceable components rather than primary structural members, establishes it as a technically sound and practically attractive option for buildings located in regions of moderate to high seismicity. Subsequent research should extend this investigation to a broader set of structural typologies, damper distributions, soil conditions, and seismic intensity levels, and should address the long-term implications of this approach in terms of post-earthquake reparability and life-cycle cost.

Nonlinear time–history analyses yielded the following direct engineering conclusions:

7.1. Quantifiable Mitigation Metrics

- **Drift Reduction:** The short-stroke hysteretic damper system reduces peak first-story inter-story drift ratios (IDRs) by up to 45% under fixed-base conditions (e.g., suppressing the governing Pacoima demand from 0.38% down to 0.21%).
- **SSI Suppression:** Under flexible foundation conditions, the damper system completely eliminates severe drift amplifications, bringing the extreme first-story IDR of the Petrolia record down from 0.80% to under 0.20%.
- **Torsional Uniformity:** Across all 9 near-fault records and foundation states, the perimeter damper configuration restricts second-story drifts to under 0.20%, confirming highly effective multi-axis torsional control.
- **Acceleration Control:** Because the dampers are displacement-activated rather than velocity-dependent, supplemental stiffness is added without generating high-frequency force spikes, successfully capping roof accelerations below 0.60 g to protect non-structural components.

7.2. Governing Structural Physics

- **The SSI Vulnerability:** Assuming a rigid fixed base is profoundly non-conservative for plan-irregular frames on deformable soils. SSI-induced period elongation causes the building's fundamental frequencies to shift into close resonance with near-fault pulse content, more than doubling drift demands in the unretrofitted frame.
- **Record Hierarchy Shifting:** Implementing displacement-dependent retrofitting fundamentally alters the damage potential and severity ranking of seismic records. Following retrofitting, the critical Pacoima motion was successfully controlled, whereas the Rinaldi record—originally the mildest—became the governing input for base shear demands due to localized frequency alignment with the retrofitted stiffness.

7.3. Actionable Engineering Directives

- **Foundation Verification Mandatory:** The addition of displacement-dependent passive dampers consistently elevates base shear demands, clustering forces in a demanding

6500 kN to 7400 kN band. Geotechnical and foundation capacity verifications must be an indispensable, standard component of any short-stroke hysteretic retrofit design.

- **Ground Motion Diversity:** Because passive metallic dampers drastically rearrange the structural demand hierarchy across different accelerograms, safety assessments must rely on a diverse, unscaled, and multi-characteristic near-fault record suite rather than single design spectrum envelopes.

While the implementation of the MAURER SHARK hysteretic dampers provides a massive benefit by mitigating critical displacement and drift demands, it simultaneously introduces a key structural design trade-off by increasing global base shear forces. This upward shift in base shear is directly caused by the high supplemental elastic stiffness injected into the frame. In actual engineering practice, this elevated lateral force profile does not compromise the upper-story beams or columns. The forces bypass the primary frame members as they are directly gathered by the chevron/inverted-V brace collector system and funnelled straight down to the foundation nodes. Consequently, this mechanism shifts the critical design focus from the superstructure to the infrastructure. It mandates a rigorous capacity-design check for all foundation footings, tie-beams, and connection details. These structural and substructural elements must be strengthened intentionally to safely resist the amplified vertical and horizontal reactions, ensuring the layout does not suffer from soil bearing capacity, sliding, or concrete punching failures.

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References

1. Zucca, M.; Longarini, N.; Simoncelli, M.; Aly, A.M. Tuned Mass Damper Design for Slender Masonry Structures: A Framework for Linear and Nonlinear Analysis. *Appl. Sci.* **2021**, *11*, 3425. [\[CrossRef\]](#)
2. Antoniou, S. *Seismic Retrofit of Existing Reinforced Concrete Buildings*; John Wiley & Sons: Hoboken, NJ, USA, 2023.
3. Guo, L.; Wang, J.; Wang, H. Seismic design and hybrid simulation test of existing concrete frames upgraded by metallic damper. *Eng. Struct.* **2023**, *291*, 116337. [\[CrossRef\]](#)
4. Papagiannopoulos, G.A.; Hatzigeorgiou, G.D.; Beskos, D.E. Design Using Supplemental Dampers. In *Seismic Design Methods for Steel Building Structures*; Springer: Berlin/Heidelberg, Germany, 2022; pp. 463–508.
5. Alshimmeri, A.J.H.; Kontoni, D.P.N.; Ghamari, A. Improving the seismic performance of reinforced concrete frames using an innovative metallic-shear damper. *Comput. Concr.* **2021**, *28*, 275.
6. Foyouzati, A. Analytical study on seismic strengthening of reinforced concrete frame equipped with steel damping system with shear mechanism fuse. *Asian J. Civ. Eng.* **2024**, *25*, 1115–1127.
7. Sahoo, D.R.; Rai, D.C. Seismic strengthening of non-ductile reinforced concrete frames using aluminum shear links as energy-dissipation devices. *Eng. Struct.* **2010**, *32*, 3548–3557.
8. Ibrahim, Y.E.; Marshall, J.; Charney, F.A. A visco-plastic device for seismic protection of structures. *J. Constr. Steel Res.* **2007**, *63*, 1515–1528. [\[CrossRef\]](#)
9. Zhou, Y.; Lu, X.; Weng, D.; Zhang, R. A practical design method for reinforced concrete structures with viscous dampers. *Eng. Struct.* **2012**, *39*, 187–198. [\[CrossRef\]](#)
10. Riaz, R.D.; Malik, U.J.; Shah, M.U.; Usman, M.; Najam, F.A. Enhancing seismic resilience of existing reinforced concrete building using non-linear viscous dampers. *Actuators* **2023**, *12*, 175. [\[CrossRef\]](#)

11. Logotheti, V.E.; Kafetzi, T.C.; Papagiannopoulos, G.A.; Karabalis, D.L. On the use of interstorey velocity for the seismic retrofit of steel frames with viscous dampers. *Soil Dyn. Earthq. Eng.* **2020**, *129*, 105312. [[CrossRef](#)]
12. Alam, Z.; Zhang, C.; Samali, B. The role of viscoelastic damping on retrofitting seismic performance of asymmetric reinforced concrete structures. *Earthq. Eng. Eng. Vib.* **2020**, *19*, 223–237. [[CrossRef](#)]
13. Suk, R.; Demir, A.; Altintas, G.; Altiok, T.Y. Proposal for a novel technological damper system (TDS) for the retrofit of reinforced concrete frame structures. *Structures* **2024**, *60*, 105878. [[CrossRef](#)]
14. Ijmulwar, S.S.; Patro, S.K. Seismic design of reinforced concrete buildings equipped with viscous dampers using simplified performance-based approach. *Structures* **2024**, *61*, 106020. [[CrossRef](#)]
15. Mahrenholtz, C.; Lin, P.C.; Wu, A.C.; Tsai, K.C.; Hwang, S.J.; Lin, R.Y.; Bhayusukma, M.Y. Retrofit of reinforced concrete frames with buckling-restrained braces. *Earthq. Eng. Struct. Dyn.* **2015**, *44*, 59–78.
16. Qu, Z.; Kishiki, S.; Maida, Y.; Sakata, H.; Wada, A. Seismic response of reinforced concrete frames with buckling restrained braces in zigzag configuration. *Eng. Struct.* **2015**, *105*, 12–21. [[CrossRef](#)]
17. Pan, K.-Y.; Wu, A.C.; Tsai, K.C.; Li, C.H.; Khoo, H.H. Seismic retrofit of reinforced concrete frames using buckling restrained braces with bearing block load transfer mechanism. *Earthq. Eng. Struct. Dyn.* **2016**, *45*, 2303–2326. [[CrossRef](#)]
18. Balkamou, N.; Papagiannopoulos, G. Seismic Upgrade of an Existing Reinforced Concrete Building Using Steel Plate Shear Walls. *Appl. Sci.* **2024**, *14*, 443. [[CrossRef](#)]
19. Di Lorenzo, G.; Tartaglia, R.; Prota, A.; Landolfo, R. Design procedure for orthogonal steel exoskeleton structures for seismic strengthening. *Eng. Struct.* **2023**, *275*, 115252.
20. Katsimpini, P.S.; Papagiannopoulos, G.A. Effectiveness of the Seesaw System as a Means of Seismic Upgrading in Older, Non-Ductile Reinforced Concrete Buildings. *Vibration* **2023**, *6*, 102–112. [[CrossRef](#)]
21. Zhang, Y.; Bomholt, F.; Torsten, E. Ensuring seismic resilience: Low cycle fatigue analysis of Maurer SHARK® Damper. *Procedia Struct. Integr.* **2026**, *78*, 1008–1015. [[CrossRef](#)]
22. MAURER SE. *MAURER Seismic Protection Systems*; MAURER: Munich, Germany, 2022.
23. SAP. *Static and Dynamic Finite Element Analysis of Structures: Version 25.0, Computers and Structures*; SAP: Berkeley, CA, USA, 2000.
24. EN 1998-1:2004; Eurocode 8: Design of Structures for Earthquake Resistance—Part 1: General Rules, Seismic Actions and Rules for Buildings. European Committee for Standardization (CEN): Brussels, Belgium, 2004.
25. EN 1991-1-1:2002; Eurocode 1: Actions on Structures—Part 1-1: General Actions—Densities, Self-Weight, Imposed Loads for Buildings. European Committee for Standardization (CEN): Brussels, Belgium, 2002.
26. ASCE 41-17; Seismic Evaluation and Retrofit of Existing Buildings. American Society of Civil Engineers: Reston, VA, USA, 2017.
27. EN 1998-5:2005; Eurocode 8—Design of Structures for Earthquake Resistance, Part 5: Foundations, Retaining Structures and Geotechnical Aspects. European Committee for Standardization (CEN): Brussels, Belgium, 2005.
28. Mulliken, J.S.; Karabalis, D.L. Discrete model for dynamic through-the-soil coupling of 3-d foundations and structures. *Earthq. Eng. Struct. Dyn.* **1998**, *27*, 687–710. [[CrossRef](#)]
29. EN 15129:2018; Anti-Seismic Devices. European Committee for Standardization (CEN): Brussels, Belgium, 2018.
30. Lee, A.C.-H.; Ryu, J.; Kim, D.H.; Ju, Y.K. Improving seismic performance of non-ductile reinforced concrete frames through the combined behavior of friction and metallic dampers. *Eng. Struct.* **2018**, *172*, 304–320. [[CrossRef](#)]
31. Pacific Earthquake Engineering Research Center. PEER Strong Motion Database. Available online: <https://peer.berkeley.edu/peer-strong-ground-motion-databases> (accessed on 24 September 2024).

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