

Research paper

Passive and semi-active nonlinear negative stiffness control of an offshore jacket platform under earthquake-induced hydrodynamic interaction

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ABSTRACT

Earthquake-induced hydrodynamic interaction can substantially alter the seismic response and control performance of offshore platforms. This study investigates nonlinear negative stiffness control of a jacket platform considering motion-induced hydrodynamic feedback. A five-degree-of-freedom (5DOF) formulation is validated against a published benchmark and adapted to a lightweight-topside configuration. Motion-dependent hydrodynamic forces are evaluated using the Morison equation. A fixed-parameter passive nonlinear negative stiffness device (P-NNSD) and a displacement-triggered semi-active NNSD (DTSA-NNSD) are assessed. For a representative record, hydrodynamic interaction increases peak Level-5 relative displacement and absolute acceleration by approximately 50% and 25%, respectively, and Level-4-to-5 inter-story displacement by 40%. Across five ground motions, the P-NNSD reduces mean peak inter-story displacement, absolute acceleration, and base shear by 61.1%, 11.8%, and 10.5%, respectively; the corresponding DTSA-NNSD reductions are 73.5%, 15.0%, and 18.0%. Relative to the P-NNSD, the DTSA-NNSD further reduces peak inter-story displacement and absolute acceleration by 31.9% and 3.6% for this record. The NNSD contains no explicit energy-dissipation component, and the assessment is limited to a simplified lightweight-topside configuration, five ground motions, prescribed device location and control parameters, and idealized DTSA switching without experimental validation. Within these limits, response-dependent regulation outperforms fixed-parameter control.

1. Introduction

Offshore jacket platforms are widely used in offshore oil and gas production and marine energy infrastructure and are exposed to complex environmental and seismic actions. Unlike conventional onshore structures, their dynamic behaviour is influenced not only by structural properties but also by interactions with the surrounding water and foundation system. Ghazi et al. (2022) investigated a jack-up platform subjected to multiple environmental hazards and showed that different loading conditions can substantially alter structural demands. Coupled analyses of offshore wind energy systems have likewise highlighted the importance of interactions among structural, hydrodynamic, and foundation subsystems (Yang et al., 2024; Wang et al., 2026). Numerical and experimental studies on jacket-supported structures have further shown that hydrodynamic effects, foundation conditions, soil behaviour, and seismic characteristics can significantly influence structural response (Zhao et al., 2021; Natarajan and Madabhushi, 2022; Soleimani et al., 2023; Huang et al., 2024). These findings indicate that the seismic

performance of offshore structures should be assessed within an integrated structure-environment interaction framework rather than using dry-structure assumptions alone.

Among these interactions, hydrodynamic effects are particularly important for submerged offshore structures. Fluid-structure interaction studies have shown that structural motion can alter the surrounding fluid pressure and flow field, while the resulting forces in turn affect the structural response (Badshah et al., 2020). For jacket platforms, Liu et al. (2024) experimentally and numerically investigated a coupled soil-water-structure system and found that hydrodynamic pressure altered the frequency characteristics of structural acceleration. It is therefore important to distinguish externally imposed wave loading from hydrodynamic forces induced by earthquake-driven structural motion. During an earthquake, the absolute velocity and acceleration of submerged members disturb the surrounding water, generating motion-dependent forces that feed back into the structure. Hosseini Lavassani et al. (2023) further showed that fluid-structure interaction and vibration-induced hydrodynamic effects can influence the

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vibration-control response of jacket platforms. Thus, earthquake excitation, structural motion, and hydrodynamic feedback form an inherently coupled dynamic process.

Various vibration-control strategies have been developed to mitigate the dynamic responses of offshore structures. [Chen et al. \(2021\)](#) compared single and multiple tuned mass dampers for jacket-type offshore wind turbines and showed that tuned mass systems can effectively suppress structural vibration, while distributed tuning can improve robustness. Brace-viscous damper systems have also been shown to be strongly affected by device configuration and supporting stiffness ([Vaezi et al., 2021](#)). Similarly, multiple tuned mass damper systems have demonstrated effective response mitigation under combined seismic, wind, and wave actions ([Wang et al., 2020](#)). More recently, [Wang et al. \(2023a\)](#) developed an amplifying damping transfer system for wind-turbine structures, which transfers upper-tower motion to the bottom through a mechanical amplification mechanism, thereby alleviating damper-installation constraints and enhancing damping efficiency. Although these passive systems can provide effective vibration reduction, their performance generally depends on the compatibility between predefined control parameters, structural dynamic characteristics, and excitation conditions.

This limitation has motivated the development of variable-property, semi-active, and nonlinear control strategies. [Leng et al. \(2021\)](#) introduced a magnetorheological-elastomer-based isolation system for jacket platforms and subsequently employed neuro-fuzzy control to achieve response-dependent stiffness adjustment under earthquake and irregular-wave excitations ([Leng et al., 2022](#)). [Zheng et al. \(2023\)](#) applied nonlinear energy sinks to mitigate the vibration of deep-water jacket platforms subjected to combined wave and earthquake loading, whereas [Liu et al. \(2025\)](#) evaluated the effectiveness of multiple tuned mass dampers under wave and earthquake excitations. [Mohasel et al. \(2026\)](#) further validated an innovative damping system for the seismic response control of offshore jacket platforms through real-time hybrid simulation. Collectively, these studies indicate a shift from conventional fixed-parameter devices toward control systems capable of adapting their mechanical properties to variations in structural response and environmental conditions.

Negative stiffness provides an alternative vibration-control mechanism to conventional approaches based primarily on supplemental damping or tuned auxiliary masses. [Li et al. \(2020\)](#) reviewed the fundamental principles of negative stiffness devices and demonstrated that appropriately designed negative stiffness can modify the effective dynamic stiffness of a structural system. Experimental studies have further established the feasibility of smooth and bistable negative stiffness mechanisms for structural vibration mitigation ([Cain et al., 2020](#); [Fan et al., 2022](#)). [Li et al. \(2022\)](#) developed a quasi-active negative stiffness damper that combines negative stiffness with controllable damping to improve seismic adaptability. However, increasing the magnitude of negative stiffness does not necessarily improve the control performance. [Li et al. \(2023\)](#) demonstrated that negative stiffness nonlinearity can significantly alter the frequency-response characteristics and may induce undesirable nonlinear behaviour. Further parametric studies of negative stiffness inerter systems have shown that their control effectiveness and system stability are sensitive to stiffness-related parameters ([Li et al., 2023](#); [Tai et al., 2025](#)). Negative stiffness modifies the effective restoring characteristics of a structure and, when combined with supplemental damping, can amplify damper deformation and energy dissipation ([Wang et al., 2023b, 2025](#)). The KDamper oscillator ([Kapasakalis et al., 2020](#)) and its extended variants ([Kapasakalis et al., 2021](#)) build upon the conventional tuned mass damper concept by incorporating a negative-stiffness element and, in some configurations, inerter components. Their applicability to the vibration control of offshore wind-turbine towers has recently been investigated ([Kapasakalis et al., 2024](#)). In contrast, the P-NNSD considered herein is modelled solely as a displacement-dependent stiffness element without an explicit damping term. Its parameters

must therefore be selected to balance response mitigation and stability over the anticipated deformation range.

To account for the geometric nonlinearity inherent in practical negative stiffness mechanisms, [Zhang et al. \(2025\)](#) developed a nonlinear negative stiffness device (NNSD) and evaluated its mechanical behaviour and vibration-control performance through real-time hybrid simulation. Their results showed that the compression state governs both the magnitude and effective operating range of the negative stiffness. Therefore, a passive NNSD with a fixed compression ratio (P-NNSD) can only be optimized within a limited response range. [Zhang et al. \(2026\)](#) subsequently proposed a displacement-triggered semi-active NNSD (DTSA-NNSD), whose operating state is adjusted according to structural displacement. This strategy enables response-dependent regulation of the device characteristics, providing moderate negative stiffness action at small response levels and stronger regulation as structural demand increases. It therefore offers a potential means of balancing vibration mitigation, response adaptability, and system stability.

Despite recent advances, the coupling between nonlinear negative-stiffness control and earthquake-induced hydrodynamic feedback in jacket platforms remains insufficiently understood. Previous offshore studies have mainly investigated vibration-control systems employing tuned masses, supplemental damping, or inerters, whereas NNSD research has focused primarily on conventional or isolated structures. In a submerged jacket platform, NNSD-induced changes in structural motion also alter the hydrodynamic forces, creating a coupled control problem. The main contributions are as follows:

- (1) A verified 5DOF jacket model is coupled with motion-dependent Morison forces to capture the feedback between the controlled structural response and the surrounding water.
- (2) The P-NNSD and DTSA-NNSD are evaluated within the same coupled framework. The NNSD acts directly as a displacement-dependent restoring element without a tuned auxiliary mass, inerter, or explicit damping component, while the DTSA-NNSD adapts by switching between predefined negative-stiffness states according to structural displacement.
- (3) The influence of hydrodynamic feedback and the additional benefit of response-dependent regulation over fixed-parameter control are quantified across five ground motions.

The remainder of this paper is organized as follows. Section 2 verifies the five-degree-of-freedom (5DOF) structural formulation against a published reference jacket model and subsequently defines the modified lightweight-topside benchmark adopted for the coupled analyses. Section 3 formulates the earthquake-induced hydrodynamic interaction using the Morison equation. Section 4 presents the NNSD model and the P-NNSD and DTSA-NNSD control strategies. Section 5 describes the numerical implementation and coupled solution procedure. Section 6 evaluates the hydrodynamic effects and compares the two control strategies. Section 7 summarizes the main conclusions.

2. Reference-model verification and modified 5DOF jacket-platform model

A 5DOF shear-type lumped-mass formulation is adopted to represent the global horizontal dynamics of the offshore jacket platform, as illustrated in [Fig. 1](#). Each degree of freedom represents the equivalent structural mass at a different elevation, with only horizontal translational motion considered. The published jacket platform model of [Enferadi et al. \(2019\)](#) is first reconstructed using its original mass and stiffness parameters to verify the numerical implementation of the 5DOF formulation. A separately defined lightweight topside configuration is then introduced and used for the subsequent earthquake-induced hydrodynamic interaction and NNSD control analyses.

The structural displacements relative to the ground are adopted as

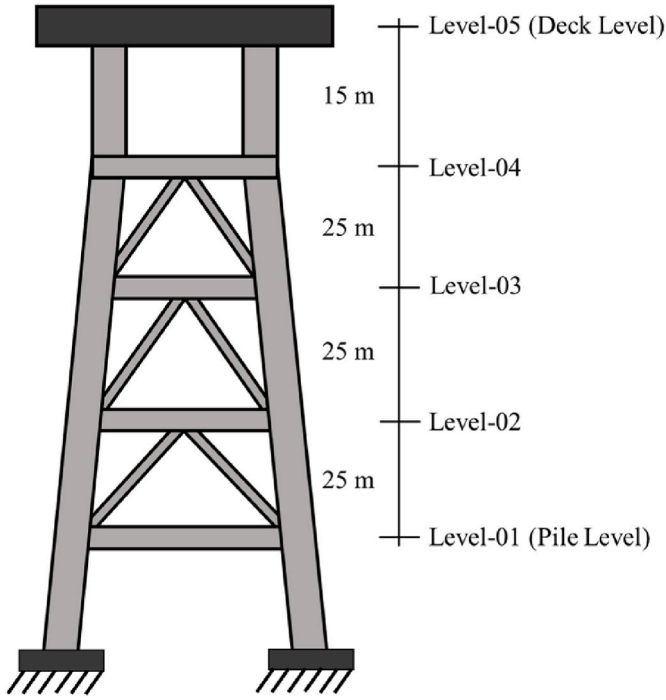


Fig. 1. 5DOF idealization of the offshore jacket platform.

the primary unknowns. In the absence of hydrodynamic and control forces, the equation of motion of the 5DOF platform subjected to horizontal ground motion is expressed as:

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = -\mathbf{M}\mathbf{r}\ddot{x}_g \quad (1)$$

where $\mathbf{M}$, $\mathbf{C}$, and $\mathbf{K}$ denote the mass, damping, and stiffness matrices of the structure, respectively; $\mathbf{x}$, $\dot{\mathbf{x}}$ and $\ddot{\mathbf{x}}$ are the displacement, velocity, and acceleration vectors relative to the ground, respectively; $\ddot{x}_g(t)$ denotes the horizontal ground acceleration; and $\mathbf{r} = [1, 1, 1, 1, 1]^T$ is the seismic influence vector.

The mass matrix of the structure is given by:

$$\begin{bmatrix} m_1 & 0 & 0 & 0 & 0 \\ 0 & m_2 & 0 & 0 & 0 \\ 0 & 0 & m_3 & 0 & 0 \\ 0 & 0 & 0 & m_4 & 0 \\ 0 & 0 & 0 & 0 & m_5 \end{bmatrix} \quad (2)$$

For the 5DOF shear-type model, the stiffness matrix is expressed as:

$$\begin{bmatrix} k_1 + k_2 & -k_2 & 0 & 0 & 0 \\ -k_2 & k_2 + k_3 & -k_3 & 0 & 0 \\ 0 & -k_3 & k_3 + k_4 & -k_4 & 0 \\ 0 & 0 & -k_4 & k_4 + k_5 & -k_5 \\ 0 & 0 & 0 & -k_5 & k_5 \end{bmatrix} \quad (3)$$

The structural damping is represented using the Rayleigh damping model:

$$\mathbf{C} = \alpha_R \mathbf{M} + \beta_R \mathbf{K} \quad (4)$$

where α_R and β_R are the mass- and stiffness-proportional Rayleigh damping coefficients, respectively. The Rayleigh damping matrix $\mathbf{C}$ is established for the uncontrolled structural model and retained unchanged in the P-NNSD and DTSA-NNSD analyses. Therefore, no additional or recalibrated structural damping is introduced when the NNSD is installed.

The mass and inter-story stiffness parameters used for validation of the baseline model are listed in Table 1. These parameters are adopted from the equivalent 5DOF model of the reference offshore jacket

Table 1

Structural parameters of the reference 5DOF jacket model used for numerical verification (Enferadi et al., 2019).

Parameter	Level 1	Level 2	Level 3	Level 4	Level 5
Mass (ton)	220	200	195	130	4850
Inter-story stiffness (N/m)	90×10^6	350×10^6	210×10^6	115×10^6	42×10^6

platform.

Based on the above mass and stiffness parameters, an eigenvalue analysis was conducted to obtain the first five mode shapes and the corresponding natural circular frequencies of the baseline 5DOF model, as shown in Fig. 2.

To verify the accuracy of the reconstructed 5DOF model, the calculated natural circular frequencies were compared with the reference results, as summarized in Table 2.

Good agreement is obtained between the reconstructed model and the published reference results for all five modes. The maximum relative error is 0.38%, while the discrepancies for the remaining modes are below 0.05%. This comparison verifies the numerical implementation of the 5DOF lumped-mass formulation using exactly the mass and stiffness parameters reported in the reference model (Enferadi et al., 2019).

The published reference model and the modified model used in the subsequent analyses serve different purposes. The original configuration, with a Level-5 lumped mass of 4850 t, is reconstructed to verify the numerical implementation of the 5DOF formulation against the published modal results. The subsequent analyses adopt a prescribed Level-5 mass of 200 t to define an idealized lightweight-topside configuration. This value is not an estimate of the residual mass obtained by removing auxiliary equipment from the original platform, and the modified model is not intended to be dynamically equivalent to that platform. The lower-level masses and inter-story stiffnesses are retained to provide a common numerical basis for examining hydrodynamic feedback and comparing the two control strategies within the prescribed configuration. This constitutes a modelling assumption rather than a verified engineering design for a particular lightweight platform.

The reference to small offshore installations with masses of approximately 200 t (Leporini et al., 2019) provides context for the selected mass scale only; it does not establish the dynamic equivalence or engineering representativeness of the present configuration. The purpose of this model is to evaluate how fixed and displacement-triggered negative-stiffness regulation affect the response of a specified coupled structure-water system. Accordingly, the agreement with the published reference frequencies verifies the reference-model implementation, whereas the modal properties and control responses of the modified configuration are evaluated separately.

The modal properties of the modified analysis model were therefore recalculated independently. As summarized in Table 2, its natural circular frequencies are 7.6435, 16.0118, 31.6730, 46.5432, and 66.3333 rad/s, corresponding to periods of 0.8220, 0.3924, 0.1984, 0.1350, and 0.0947 s, respectively. The first two modes account for approximately 98.77% of the total effective modal mass, and the Level 4-Level 5 interface exhibits the largest relative deformation among the four candidate inter-level locations in both dominant modes. This interface was therefore selected as the single-device location in the present model.

3. Earthquake-induced hydrodynamic interaction model

Earthquake-induced hydrodynamic feedback is incorporated into the 5DOF structural model through motion-dependent forces acting on the submerged jacket members. The platform has a total height of 90 m and is located in a water depth of 75 m. Its foundation is assumed to be fixed to a rigid seabed, and soil-structure interaction is neglected in the

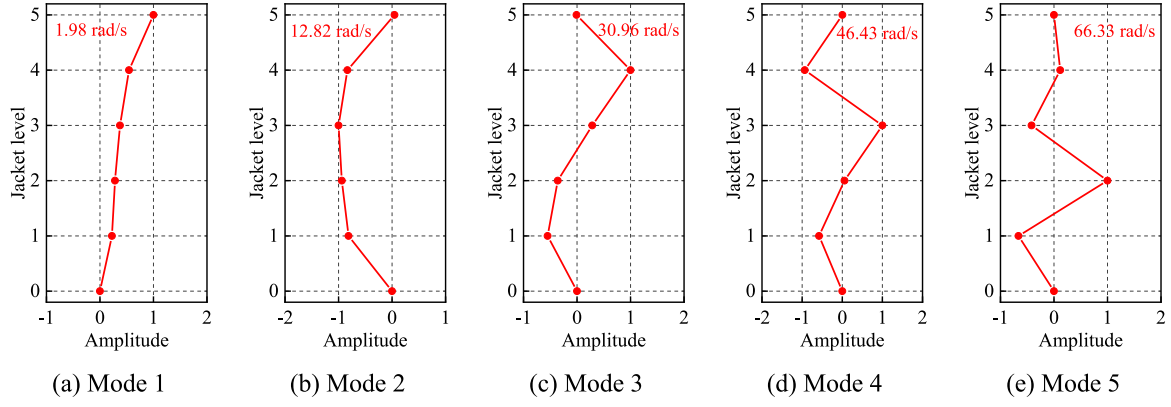


Fig. 2. Mode shapes and natural circular frequencies of the reconstructed reference 5DOF jacket model.

Table 2

Modal characteristics of the reconstructed reference model and the modified lightweight topside analysis model.

Mode	Reconstructed reference model (rad/s)	Reference (Enferadi et al., 2019) (rad/s)	Error (%)	Modified analysis model (rad/s)	Modified model period (s)
1	1.9876	1.98	0.38	7.644	0.822
2	12.823	12.83	0.05	16.012	0.392
3	30.958	30.96	0.006	31.673	0.198
4	46.435	46.43	0.01	46.543	0.135
5	66.333	66.34	0.01	66.333	0.095

present study. During earthquake excitation, the motion of the submerged jacket members disturbs the surrounding water, generating hydrodynamic forces that vary with the instantaneous structural response and, in turn, feed back into the structural dynamic system.

3.1. Morison-based hydrodynamic formulation

For submerged slender cylindrical members, the horizontal hydrodynamic force per unit length is calculated using the Morison equation based on the relative motion between the fluid and the structure (Nasseri et al., 2014):

$$f_{w,c}(z,t) = \rho C_M \pi \frac{D^2}{4} \ddot{u}_w - \rho C_A \pi \frac{D^2}{4} \ddot{x}_a + \frac{1}{2} \rho D C_D |\dot{u}_w - \dot{x}_a| (\dot{u}_w - \dot{x}_a) \quad (5)$$

where $f_{w,c}(z,t)$ denotes the horizontal hydrodynamic force per unit length acting on the cylindrical member at elevation (z); ρ is the seawater density; D is the outer diameter of the cylindrical member; C_M , C_A , and C_D are the Morison inertia, added-mass, and drag coefficients, respectively, with $C_A = C_M - 1$; $\dot{u}_w$ and $\ddot{u}_w$ denote the horizontal velocity and acceleration of the undisturbed water, respectively; and $\dot{x}_a$ and $\ddot{x}_a$ are the absolute velocity and absolute acceleration of the structural member, respectively.

The present study considers hydrodynamic forces induced by earthquake-driven structural motion in otherwise still water. Therefore,

$$\begin{cases} \dot{u}_w = 0 \\ \ddot{u}_w = 0 \end{cases} \quad (6)$$

Accordingly, Eq. (5) can be simplified as

$$f_{w,c}(z,t) = -\rho C_A \pi \frac{D^2}{4} \ddot{x}_a - \frac{1}{2} \rho D C_D |\dot{x}_a(z,t)| \dot{x}_a(z,t) \quad (7)$$

The two terms in Eq. (7) represent the inertia force associated with the acceleration of the structural member and the drag force arising from its motion relative to the surrounding still water, respectively.

Since the structural equations of motion are formulated in terms of displacements relative to the ground, the absolute structural displacement is expressed as

$$\mathbf{x}_a(t) = \mathbf{x}(t) + \mathbf{r}\mathbf{x}_g(t) \quad (8)$$

The corresponding absolute velocity and absolute acceleration are given by

$$\dot{\mathbf{x}}_a(t) = \dot{\mathbf{x}}(t) + \mathbf{r}\dot{\mathbf{x}}_g(t) \quad (9)$$

$$\ddot{\mathbf{x}}_a(t) = \ddot{\mathbf{x}}(t) + \mathbf{r}\ddot{\mathbf{x}}_g(t) \quad (10)$$

Thus, although the structural equations of motion are formulated in relative coordinates, the absolute velocity and acceleration defined in Eqs. (9) and (10) are used in the Morison formulation to characterize the actual motion of the submerged members relative to the surrounding still water.

The distributed hydrodynamic forces are evaluated along the submerged length of all underwater jacket members and then transformed into equivalent forces at the adjacent structural degrees of freedom according to the member connectivity and shape functions. The resulting equivalent hydrodynamic force vector of the 5DOF model is expressed as

$$\mathbf{F}_{w,c}(t) = [F_{w,c,1}(t) \ F_{w,c,2}(t) \ F_{w,c,3}(t) \ F_{w,c,4}(t) \ F_{w,c,5}(t)]^T \quad (11)$$

where $F_{w,c,i}(t)$ denotes the equivalent hydrodynamic force acting on the i th degree of freedom.

Because the water surface is located at an elevation of 75 m, whereas Level 5 is located at an elevation of 90 m, only the portions of the jacket members below the water surface are included in the hydrodynamic force calculation.

3.2. Coupled structural-hydrodynamic equation

With both horizontal earthquake excitation and motion-induced hydrodynamic forces taken into account, the dynamic equilibrium equation of the 5DOF platform is expressed as

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = -\mathbf{M}\mathbf{r}\ddot{\mathbf{x}}_g + \mathbf{F}_{w,c} \quad (12)$$

where $\mathbf{F}_{w,c}$ is the equivalent hydrodynamic force vector evaluated from the instantaneous structural motion.

Unlike a one-way analysis in which hydrodynamic forces are prescribed in advance as external loads, the hydrodynamic forces considered here depend directly on the instantaneous structural motion. The earthquake excitation first induces the structural response, from which the absolute velocities and accelerations of the submerged members are obtained using Eqs. (9) and (10). The hydrodynamic forces are then updated through the Morison formulation and fed back into Eq. (12), thereby modifying the subsequent structural response. This procedure

establishes a two-way coupling between the structural motion and the earthquake-induced hydrodynamic forces.

Fig. 3 illustrates the 5DOF jacket-platform model incorporating earthquake-induced hydrodynamic feedback. The platform base is fixed to the seabed and subjected to horizontal ground motion. The submerged members generate motion-dependent hydrodynamic forces, which are continuously updated according to the structural response and fed back into the structural dynamic system in the form of equivalent nodal forces.

4. Nonlinear negative stiffness control model

To mitigate the dynamic response of the upper part of the platform under coupled earthquake and hydrodynamic effects, a NNSD is installed between Levels 4 and 5, as shown in Fig. 4. The NNSD exploits geometric nonlinearity to generate tunable negative stiffness characteristics and modifies the local restoring-force relationship, thereby regulating the dynamic response of the upper part of the platform.

4.1. Fifth-order NNSD restoring-force model

The mechanical behaviour of the NNSD was examined through quasi-static force-displacement tests conducted at several compression ratios λ_p , using the dual-orthogonal actuator setup shown in Fig. 4. The resulting hysteretic responses were represented using polynomial functions, with third- and fifth-order formulations considered for comparison. The fifth-order formulation showed better agreement with the experimental results, particularly in reproducing the nonlinear curvature and asymmetric evolution of the restoring force. As shown in Fig. 5, decreasing the compression ratio from $\lambda_p = 0.8$ to $\lambda_p = 0.7$ produces a stronger negative stiffness effect within the relevant deformation range, as reflected by the increased restoring-force magnitude and the corresponding change in tangent stiffness. Therefore, $\lambda_p = 0.8$ is regarded as the moderate negative stiffness state, whereas $\lambda_p = 0.7$ represents the stronger negative stiffness state adopted when the structural deformation becomes large.

The physical NNSD consists of a pre-compressed helical spring combined with a geometric constraint mechanism. Structural relative displacement changes the geometry of the spring-force transmission path, so that the horizontal component of the spring force can oppose the conventional positive restoring-stiffness contribution over part of the deformation range. The compression ratio λ_p controls the spring precompression and therefore the magnitude and width of the negative-

tangent-stiffness region. In the dual-actuator tests of Fig. 4, one actuator imposes the device deformation while the orthogonal actuator adjusts the spring compression state; this arrangement reproduces the mechanical action used by the semi-active strategy.

The complete fifth-order restoring-force relationship contains two mechanically distinct regions. The branch with $k_{\text{NNSD}} < 0$ is defined as the negative stiffness region, whereas the subsequent branch with $k_{\text{NNSD}} > 0$ corresponds to the positive stiffness region. The boundary between these regions is the transition deformation x_{tr} , at which $k_{\text{NNSD}} = 0$. Thus, Fig. 5 presents the complete nonlinear constitutive relationship, whereas the structural response may activate only a limited portion of one branch.

The nonlinear restoring force of the NNSD is represented by a fifth-order polynomial (Zhang et al., 2025):

$$F_{\text{NNSD}} = a_1(\lambda_p)x_N + a_3(\lambda_p)x_N^3 + a_5(\lambda_p)x_N^5 \quad (13)$$

where x_N denotes the relative deformation between the two ends of the NNSD; λ_p is the compression ratio, which serves as the primary parameter governing the stiffness characteristics of the device; and a_1 , a_3 , and a_5 are the coefficients of the fifth-order model, determined by the device geometry, spring stiffness, and compression state.

The tangent stiffness of NNSD is

$$k_{\text{NNSD}} = \frac{\partial F_{\text{NNSD}}}{\partial x_N} = a_1(\lambda_p) + 3a_3(\lambda_p)x_N^2 + 5a_5(\lambda_p)x_N^4 \quad (14)$$

The negative stiffness region corresponds to $k_{\text{NNSD}} < 0$, and a larger negative stiffness magnitude refers to a larger absolute value $|k_{\text{NNSD}}|$ within this region.

The transition between the negative and positive stiffness regions occurs when the tangent stiffness becomes zero. Therefore, the transition deformation x_{tr} is determined from the smallest positive root of $a_1(\lambda) + 3a_3(\lambda)x_{tr}^2 + 5a_5(\lambda)x_{tr}^4 = 0$. Based on the fitted fifth-order models, the transition deformations are approximately 0.110 m for $\lambda_p = 0.8$ and 0.135 m for $\lambda_p = 0.7$. Accordingly, the negative stiffness regions are approximately $|x_N| < 0.110$ m and $|x_N| < 0.135$ m, respectively, while the subsequent branches correspond to positive tangent stiffness.

Because the NNSD is installed between Levels 4 and 5, its deformation is defined as

$$x_N = x_5 - x_4 \quad (15)$$

The corresponding relative velocity is

$$\dot{x}_N = \dot{x}_5 - \dot{x}_4 \quad (16)$$

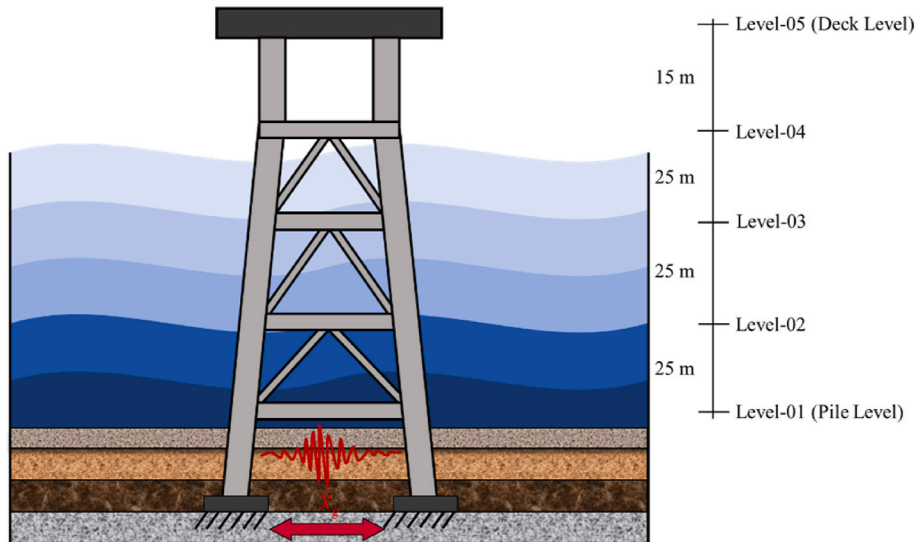


Fig. 3. Five-degree-of-freedom jacket platform model considering earthquake-induced hydrodynamic interaction.

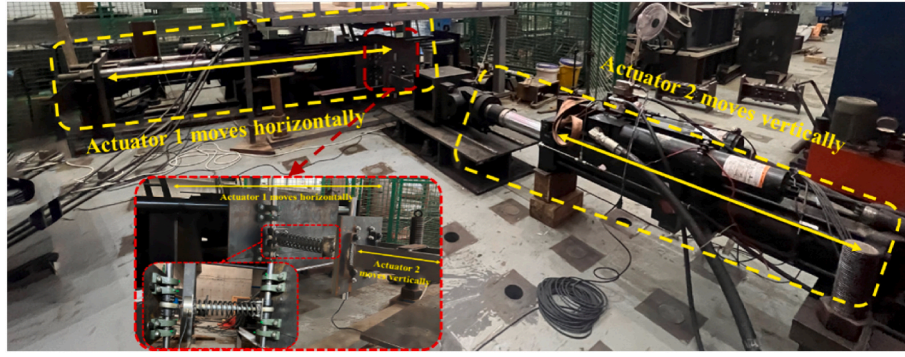


Fig. 4. Experimental setup of the NNSD using two orthogonal actuators.

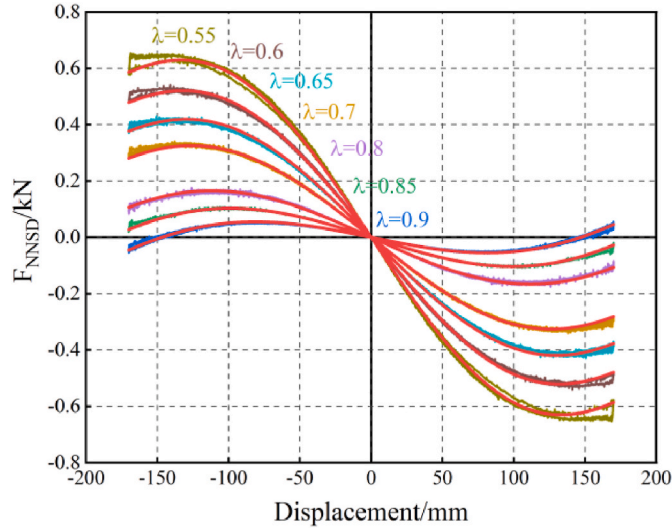


Fig. 5. Force-displacement hysteresis curves of the NNSD and the fifth-order Taylor series model.

By varying the compression ratio λ_p , the magnitude of the negative stiffness, the effective negative stiffness range, and the subsequent restoring stiffness of the NNSD can be adjusted. Consequently, the restoring-force characteristics of the device can be tailored to different

levels of structural response.

Since the present study performs equivalent dynamic analyses of a full-scale offshore platform, the stiffness and other mechanical parameters of the NNSD are treated as equivalent design parameters matched to the dynamic characteristics of the structure, rather than being directly assigned the absolute stiffness values of a laboratory-scale device.

4.2. NNSD-controlled offshore jacket platform

Based on the dominant-mode relative-deformation assessment described in Section 2, the NNSD is installed between Levels 4 and 5, as shown in Fig. 6. Because the device connects two adjacent structural degrees of freedom, its restoring force acts on Levels 4 and 5 with equal magnitude and opposite directions. The corresponding force-location vector is defined as

$$\mathbf{b}_N = [0 \ 0 \ 0 \ 1 \ -1]^T \quad (17)$$

Therefore, the NNSD force contribution to the structural system is $\mathbf{b}_N F_{NNSD}$, corresponding to F_{NNSD} at Level 4 and $-F_{NNSD}$ at Level 5 under the adopted sign convention. This formulation satisfies force equilibrium and is consistent with the relative device deformation $x_N = x_5 - x_4$.

Accordingly, when earthquake excitation, hydrodynamic feedback, and NNSD control are simultaneously considered, the equation of motion can be expressed as

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = -\mathbf{M}\mathbf{r}\ddot{x}_g + \mathbf{F}_{w,c} + \mathbf{b}_N F_{NNSD} \quad (18)$$

where F_{NNSD} denotes the restoring force generated by the NNSD and

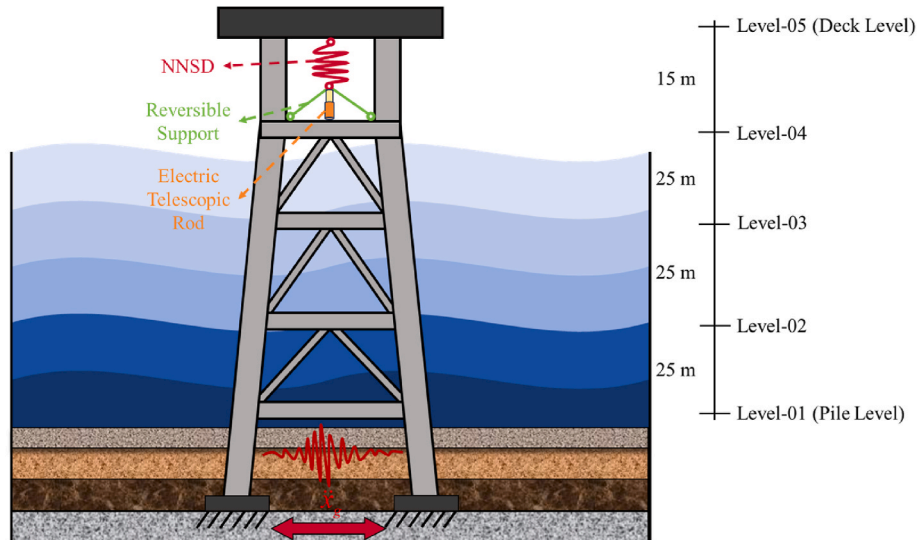


Fig. 6. Jacket platform equipped with an NNSD between Levels 4 and 5.

calculated using Eq. (13).

4.3. Passive NNSD control

A passive NNSD with fixed parameters (P-NNSD) is first adopted as the baseline control scheme. For the P-NNSD, the compression ratio remains constant throughout the earthquake response, i.e.,

$$\lambda(t) = \lambda_p \quad (19)$$

The corresponding restoring force is given by

$$F_p(t) = a_1(\lambda_p)x_N(t) + a_3(\lambda_p)x_N^3(t) + a_5(\lambda_p)x_N^5(t) \quad (20)$$

In this study, $\lambda_p = 0.8$ is selected as a representative baseline for passive control rather than being regarded as a globally optimal parameter. This selection is supported by the experimentally characterized restoring-force behavior of the NNSD and previous parametric investigations (Zhang et al., 2025, 2026). The $\lambda_p = 0.9$ state produces a relatively small negative stiffness force and a narrow effective negative stiffness range, resulting in limited control capacity during large-response stages. Although $\lambda_p = 0.7$ provides stronger negative stiffness and can further reduce displacement, its additional acceleration-reduction benefit is comparatively limited, while the stronger negative stiffness also reduces the available stability margin. Therefore, $\lambda_p = 0.8$ is selected as a compromise among displacement control, acceleration mitigation, and stability and is used as both the passive reference state and the initial state of the DTSA-NNSD.

In the present numerical model, F_p depends only on the device deformation and contains no velocity-dependent or hysteretic term. The P-NNSD is therefore treated as a stiffness-modifying element rather than an energy-dissipation device. Energy dissipation is provided by the existing structural damping and the velocity-dependent Morison drag term, whose parameters are kept unchanged between the uncontrolled and controlled cases.

The P-NNSD passively generates a nonlinear restoring force through the relative deformation between Levels 4 and 5. Because the compression ratio remains fixed, its force-displacement relationship does not change during the earthquake. Consequently, the control performance depends strongly on the compatibility between the prescribed negative stiffness characteristics and the actual range of structural response. A stronger negative stiffness effect may provide greater displacement reduction, but it does not necessarily produce a proportional improvement in acceleration response and may simultaneously reduce the local stability margin. Conversely, an excessively weak negative stiffness state may be insufficient for controlling large structural responses.

Therefore, a fixed-parameter P-NNSD can only be optimally matched to a limited response range. This limitation motivates the development of a response-dependent variable negative stiffness control strategy.

4.4. Displacement-triggered semi-active NNSD control

To improve the adaptability of the NNSD to different levels of structural response, a displacement-triggered semi-active NNSD (DTSA-NNSD) control strategy is further introduced (Zhang et al., 2026). The relative displacement between Levels 4 and 5 is adopted as the triggering variable for regulating the compression state of the NNSD. In the present two-state control scheme, the device initially operates at $\lambda_p = 0.8$, corresponding to a moderate negative stiffness state. When the absolute inter-story displacement reaches the prescribed threshold, the compression ratio is changed to $\lambda_p = 0.7$. The lower compression-ratio state produces a larger negative stiffness magnitude and restoring-force demand within the relevant deformation range, thereby providing stronger regulation during large-response stages.

The displacement-triggering variable is defined as

$$x_{tr}(t) = |x_5(t) - x_4(t)| \quad (21)$$

Based on the displacement amplitude, the NNSD is assigned to different operating states:

$$\lambda(t) = \begin{cases} 0.8, & x_{tr}(t) < x_{th} \\ 0.7, & x_{tr}(t) \geq x_{th} \end{cases} \quad (22)$$

where x_{th} denotes the prescribed displacement-triggering threshold. In the present analysis, $x_{th} = 0.015$ m is selected based on a previously published threshold-sensitivity analysis (Zhang et al., 2026), in which the control performance under different triggering thresholds was systematically evaluated. A relatively small threshold activates the stronger $\lambda_p = 0.7$ state earlier and maintains it for a longer duration, whereas a relatively large threshold delays activation and may reduce the displacement-control benefit during major response peaks. The value of 0.015 m provided a suitable compromise between activation timing, displacement reduction, and acceleration response in that study. It is therefore used herein as a literature-informed reference value rather than as a universally optimal threshold for all offshore jacket platforms. In the present two-state strategy, $\lambda_p = 0.8$ is retained during the low-response stage, while $\lambda_p = 0.7$ is activated only when the inter-story displacement reaches the prescribed threshold.

The compression ratios and displacement threshold adopted here are prescribed settings for comparing the P-NNSD and DTSA-NNSD within the modified 200 t configuration; they are not proposed as universal or optimized design values. Although the compression ratio is dimensionless, its effect on the coupled response depends on the device stiffness and geometry, the structural properties, and the excitation. Likewise, the dimensional switching threshold is specific to the displacement demand of the adopted model. Their effectiveness for the original 4850 t configuration has not been established in this study. Application to a particular platform therefore requires reassessment of the device stiffness, compression ratios, switching threshold, force and stroke capacities, and stability requirements using the properties and loading conditions of the target structure.

Physically, the change in compression ratio is realized by adjusting the axial precompression of the helical spring in the NNSD. As illustrated by the dual-actuator configuration in Fig. 4, one actuator imposes the horizontal deformation corresponding to the structural relative displacement, while the orthogonal actuator regulates the axial compression of the spring. When a switching command is generated by the displacement-triggering rule, the compression-adjustment actuator changes the spring from the $\lambda_p = 0.8$ configuration to the $\lambda_p = 0.7$ configuration. This adjustment changes the geometric configuration and axial force of the spring and consequently modifies the nonlinear restoring-force and tangent-stiffness characteristics of the NNSD. Thus, the DTSA-NNSD regulates the structural response by changing its mechanical properties rather than by continuously applying an externally prescribed control force.

At any instant, the restoring force of the DTSA-NNSD is expressed as

$$F_D(t) = a_1[\lambda_p]x_N(t) + a_3[\lambda_p]x_N^3(t) + a_5[\lambda_p]x_N^5(t) \quad (23)$$

and the corresponding equation of motion of the controlled structure is

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = -\mathbf{M}\mathbf{r}\ddot{x}_g + \mathbf{F}_{w,c} + \mathbf{b}_N F_D \quad (24)$$

Unlike the fixed-parameter P-NNSD, the DTSA-NNSD adjusts its operating state according to the structural relative displacement, allowing different stages of the earthquake response to be associated with different nonlinear restoring-force characteristics. During small-response stages, the device operates in a moderate negative stiffness state that is better matched to the current structural demand, thereby avoiding unnecessarily strong negative stiffness effects. As the absolute relative displacement between Levels 4 and 5 reaches the prescribed threshold, the device switches from the moderate negative stiffness state at $\lambda_p = 0.8$ to the stronger negative stiffness state at $\lambda_p = 0.7$. This switching is physically realized by adjusting the axial precompression of the helical spring, thereby changing the geometric configuration and the

corresponding nonlinear restoring-force characteristics of the NNSD.

Therefore, the DTSA-NNSD does not achieve vibration mitigation by continuously applying an external active control force. Instead, it modifies the intrinsic nonlinear restoring-force characteristics of the device through a finite number of parameter-state switches, thereby matching the negative stiffness effect to the instantaneous structural response level. This strategy preserves the passive mechanical mechanism of the NNSD while improving its adaptability to different response stages during an earthquake and provides a variable-parameter control framework for balancing vibration-control performance and system stability.

5. Numerical implementation

To evaluate the effects of earthquake-induced hydrodynamic forces on the dynamic response of the offshore jacket platform and the control performance of the NNSD, the coupled structure-water system described above is solved using time-domain analysis. All subsequent coupled-response and control analyses are conducted using the modified model defined in Section 2, whose modal properties are summarized in Table 2.

The structural equations of motion are implemented in the MATLAB/Simulink environment, with the 5DOF offshore jacket platform represented in state-space form. The nonlinear hydrodynamic forces induced by earthquake-driven structural motion are evaluated at each time step using the Morison model and fed back into the structural dynamic system. In this way, a time-domain coupled analysis framework is established in which the structural response and hydrodynamic forces are updated interactively.

5.1. Selection and scaling of ground motions

A target response spectrum specified by the Chinese seismic design code is adopted as the basis for selecting the earthquake inputs. An initial set of 50 natural ground-motion records with different spectral characteristics is selected from the PEER Ground Motion Database. Because the coupled analyses are conducted using the modified lightweight-topside model, the spectral compatibility of the candidate records is assessed with respect to the modal characteristics of this configuration. Ground-motion compatibility was assessed with reference to the fundamental period of the modified model, $T_1 = 0.822$ s, as indicated in Fig. 7.

For an original ground-acceleration time history, the scaled ground motion is expressed as

$$a_g(t) = S_f a_g^0(t) \quad (25)$$

where S_f is the amplitude scaling factor. When scaling to a prescribed peak ground acceleration (PGA),

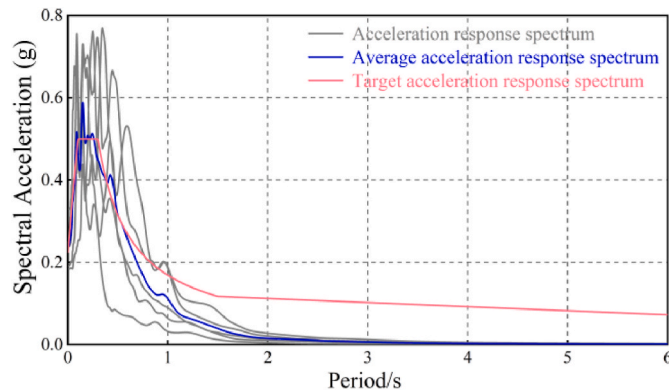


Fig. 7. Acceleration response spectra of the selected ground motions and the target spectrum.

$$S_f = \frac{PGA_{target}}{PGA_{original}} \quad (26)$$

The scaling procedure changes only the amplitude of the acceleration time history while preserving the frequency content and phase characteristics of the original record. The basic information of the ground motions is shown in Table 3. Fig. 7 compares the 5%-damped acceleration response spectra of the five scaled ground motions with the target response spectrum.

Amplitude scaling does not remove the velocity characteristics of an individual record. After baseline correction, scaling the acceleration by S_a scales the compatible ground-velocity history by the same factor, so the Peak Ground Velocity (PGV)/PGA ratio is invariant under this operation. Using the values in Table 3, PGV/PGA is 0.327, 0.222, 0.237, 0.255, and 0.294 s for GM1-GM5, respectively, demonstrating non-negligible record-to-record differences in velocity content. More importantly, the Morison drag term is not calculated from PGA or PGV as a single scalar measure; it is updated at each time step from the instantaneous absolute velocity of each submerged member. Consequently, the different velocity histories of the selected records are retained in the hydrodynamic-feedback calculation even when their amplitudes are scaled.

The selected ground motions exhibit different spectral characteristics over the principal response-period range of the modified lightweight-topside model, thereby reducing the dependence of the control-performance assessment on any single earthquake record or its incidental spectral compatibility with the structure. These ground motions are subsequently used to provide a consistent basis for evaluating earthquake-induced hydrodynamic effects and the performance of the different NNSD control strategies.

5.2. Hydrodynamic parameters and equivalent nodal forces

The seawater density is taken as $\rho = 1030 \text{ kg m}^{-3}$. The hydrodynamic forces induced by earthquake-driven structural motion are calculated using the Morison model. For a structure moving in otherwise still water, the inertia component associated with structural acceleration is described using the added-mass coefficient C_A , with $C_A = C_M - 1$. Following the hydrodynamic parameters adopted for the reference offshore jacket model, $C_M = 2.0$, $C_A = 1.0$ and $C_D = 0.8$ are used in the present analysis.

It should be emphasized that the hydrodynamic forces considered in this study are not externally prescribed wave loads acting independently on the structure. Instead, they arise from the interaction between the earthquake-induced motion of the platform and the surrounding water. Accordingly, the absolute velocity and absolute acceleration defined in Eqs. (9) and (10) are used in the Morison formulation and updated at each time step according to the instantaneous structural response, thereby establishing a feedback interaction between the hydrodynamic forces and structural motion.

Because the original offshore jacket is a spatially continuous structure, whereas the present dynamic model is represented by five lumped degrees of freedom, the distributed hydrodynamic forces acting on the submerged members are transformed into equivalent forces at the adjacent structural degrees of freedom. For a submerged member located between degrees of freedom i and $i+1$, the structural motion

Table 3
Basic information of ground motions.

Station	Magnitude	PGA ($\text{cm} \cdot \text{s}^{-2}$)	PGV ($\text{cm} \cdot \text{s}^{-1}$)
KAU073 (GM1)	7.6	28.025	9.156
KAU030 (GM2)	7.6	33.815	7.511
TAP049 (GM3)	7.62	72.14	17.129
TAP005 (GM4)	7.6	83.027	21.14
ILA001(GM5)	7.2	24.946	7.342

along the member height is interpolated from the motions at its two end nodes. The resulting distributed hydrodynamic force, denoted by $f_{w,c}(z, t)$, is then obtained using the Morison model. The corresponding equivalent nodal hydrodynamic forces are expressed as

$$F_{w,i}^{(e)} = \int_{L_e} N_i(z) f_{w,c}(z, t) dz \quad (27)$$

$$F_{w,i+1}^{(e)} = \int_{L_e} N_{i+1}(z) f_{w,c}(z, t) dz \quad (28)$$

where N_i and N_{i+1} are the linear interpolation functions associated with the two adjacent degrees of freedom. For a member segment extending from z_i to z_{i+1} , with $L_i = z_{i+1} - z_i$ and $z_i \leq z \leq z_{i+1}$, they are explicitly defined as $N_i(z) = (z_{i+1} - z)/L_i$ and $N_{i+1}(z) = (z - z_i)/L_i$. These functions satisfy $N_i(z_i) = 1$, $N_i(z_{i+1}) = 0$, $N_{i+1}(z_i) = 0$, and $N_{i+1}(z_{i+1}) = 1$, and are used consistently for both motion interpolation and equivalent nodal-force transformation.

By assembling the contributions from all submerged members, the equivalent hydrodynamic force vector of the 5DOF model is obtained as

$$\mathbf{F}_{w,c} = [F_{w,c,1} \ F_{w,c,2} \ F_{w,c,3} \ F_{w,c,4} \ F_{w,c,5}]^T \quad (29)$$

This equivalent-force formulation reduces the structural degrees of freedom while retaining the fundamental vertical distribution of the hydrodynamic forces, allowing the hydrodynamic feedback to be incorporated into the structural dynamic system according to the motion of each degree of freedom.

5.3. Numerical solution scheme

The coupled earthquake-hydrodynamic equations are solved in the time domain using MATLAB/Simulink. The 5DOF structural system is implemented in state-space form, with the state vector defined as

$$\mathbf{z} = [\mathbf{x}^T \ \dot{\mathbf{x}}^T]^T \quad (30)$$

At each time step, the relative responses of the structural degrees of freedom are obtained from the current state variables. The corresponding absolute velocities and absolute accelerations are then determined by combining the structural responses with the prescribed ground motion. The Morison hydrodynamic forces are subsequently updated, transformed into equivalent forces at the structural degrees of freedom, and fed back into the structural model. In this manner, the structural response and hydrodynamic forces are updated sequentially within the time domain, forming a closed-loop coupled solution procedure.

For the NNSD-controlled system, the relative deformation between Levels 4 and 5 is further calculated as $x_N = x_5 - x_4$, and the corresponding control force is determined according to the selected NNSD restoring-force model. For both the P-NNSD and DTSA-NNSD cases, the calculated device restoring force is assembled into the structural force vector through $\mathbf{b}_N = [0 \ 0 \ 0 \ 1 \ -1]^T$, so that equal and opposite forces are simultaneously applied to Levels 4 and 5, respectively. A fixed integration time step of $\Delta t = 1/1024$ s is adopted so that the structural response, nonlinear hydrodynamic forces, and NNSD restoring force are

updated synchronously on a unified time scale.

6. Results and discussion

Unless otherwise stated, all results in this section correspond to the modified lightweight-topside benchmark model defined in Section 2. To avoid presenting repetitive time-history plots, Figs. 8–14 show one ground-motion record as an illustrative case for explaining the earthquake-hydrodynamic interaction and NNSD-control mechanisms. However, all five selected ground motions were analyzed under the same conditions, and Table 4 reports the mean peak-response reduction ratios obtained from the five records. The quantitative comparison is therefore based on the five-record mean results rather than on the illustrative record alone.

6.1. Effect of earthquake-induced hydrodynamic interaction

To illustrate the influence of earthquake-induced hydrodynamic interaction on the dynamic response of the offshore jacket platform, the uncontrolled response under the selected illustrative record is first compared under two conditions: without hydrodynamic effects and with earthquake-hydrodynamic coupling.

Level 5 corresponds to the top of the platform, and its response reflects both the overall motion of the upper structure and the dynamic coordination between the adjacent upper degrees of freedom. Therefore, the following discussion focuses on the relative displacement of Level 5, the inter-story displacement between Levels 4 and 5, and the absolute acceleration of Level 5.

The relative displacement time histories of Level 5 under the two conditions are shown in Fig. 8.

When earthquake-induced hydrodynamic interaction is considered, the overall temporal characteristics of the Level 5 relative displacement remain similar to those obtained without hydrodynamic effects, whereas the response amplitude increases noticeably. For the illustrative record considered herein, the peak relative displacement increases by approximately 50%.

This result also indicates that earthquake-induced hydrodynamic forces should not be treated simply as prescribed external loads independent of the structural response. The absolute motion of the structure governs the generation of hydrodynamic forces, while the resulting forces feed back into the structure and modify its subsequent response, thereby forming a two-way dynamic interaction. For the present model, neglecting this feedback may therefore lead to a substantial underestimation of the displacement demand at the platform top.

Fig. 9 further compares the inter-story displacement between Levels 4 and 5 and the absolute acceleration of Level 5.

As shown in Fig. 9(a), hydrodynamic interaction increases the peak Level 4–Level 5 inter-story displacement by approximately 40%. This indicates that earthquake-induced hydrodynamic forces affect not only the overall displacement of the platform top but also the relative motion between adjacent structural degrees of freedom.

The corresponding peak absolute acceleration increases by approximately 25%, as shown in Fig. 9(b). Compared with the relative and

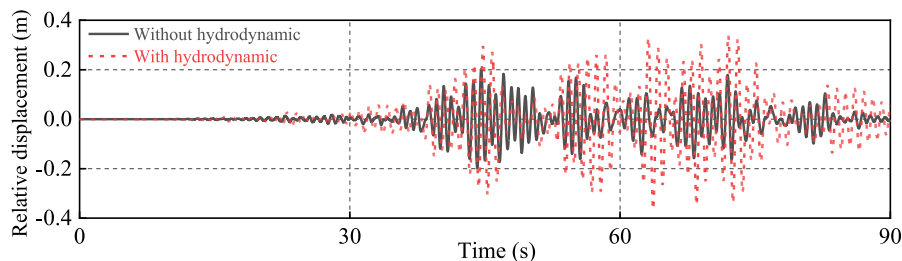


Fig. 8. Relative displacement responses of Level 5 with and without hydrodynamic interaction.

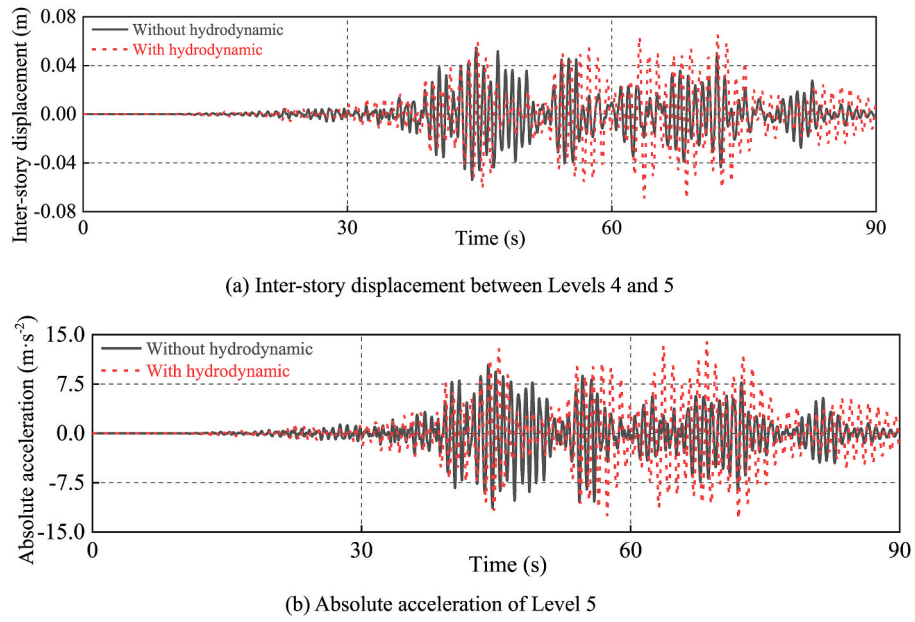


Fig. 9. Inter-story displacement between Levels 4 and 5 and absolute acceleration of Level 5 with and without hydrodynamic interaction.

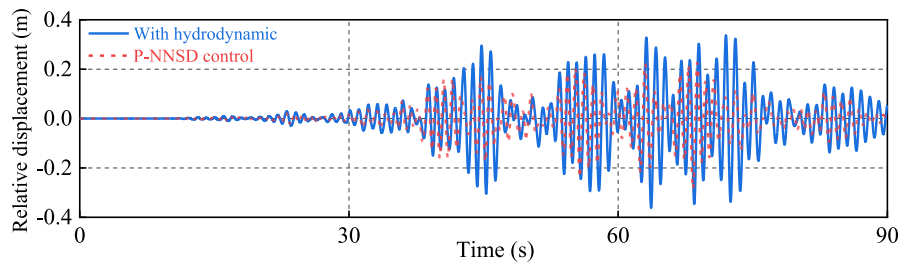


Fig. 10. Relative displacement responses of Level 5 for the uncontrolled and P-NNSD controlled cases.

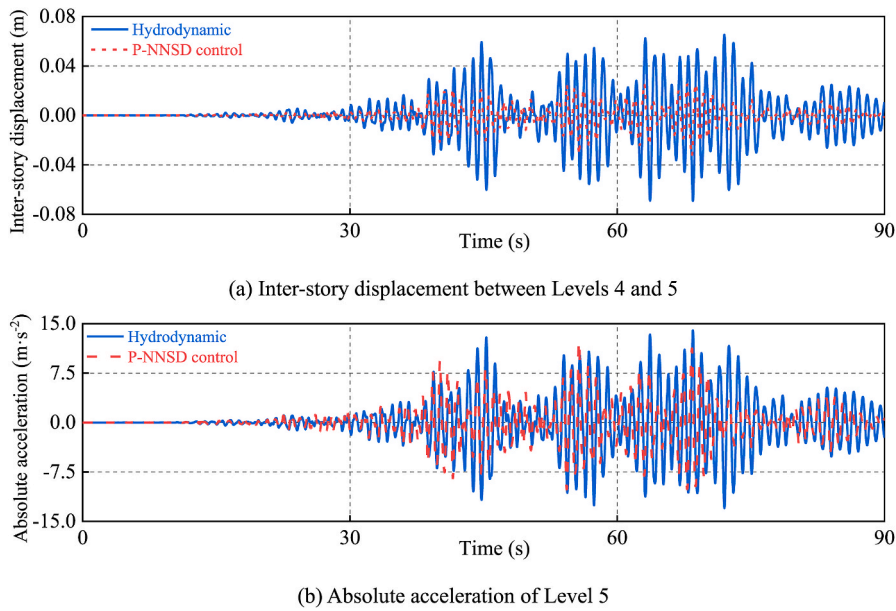


Fig. 11. Inter-story displacement and absolute acceleration responses of Level 5 for the uncontrolled and P-NNSD controlled cases.

inter-story displacements, the increase in absolute acceleration is less pronounced, indicating that hydrodynamic feedback affects different

response quantities to different extents. For the present model, displacement-related responses exhibit greater sensitivity to

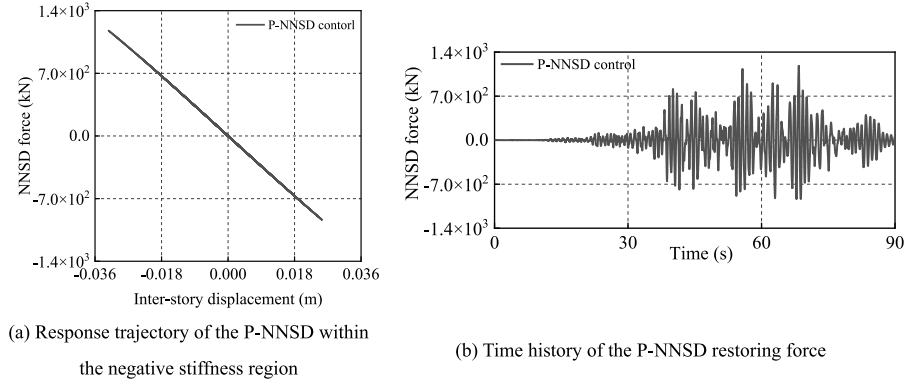


Fig. 12. Mechanical responses of the P-NNSD under earthquake-hydrodynamic interaction.

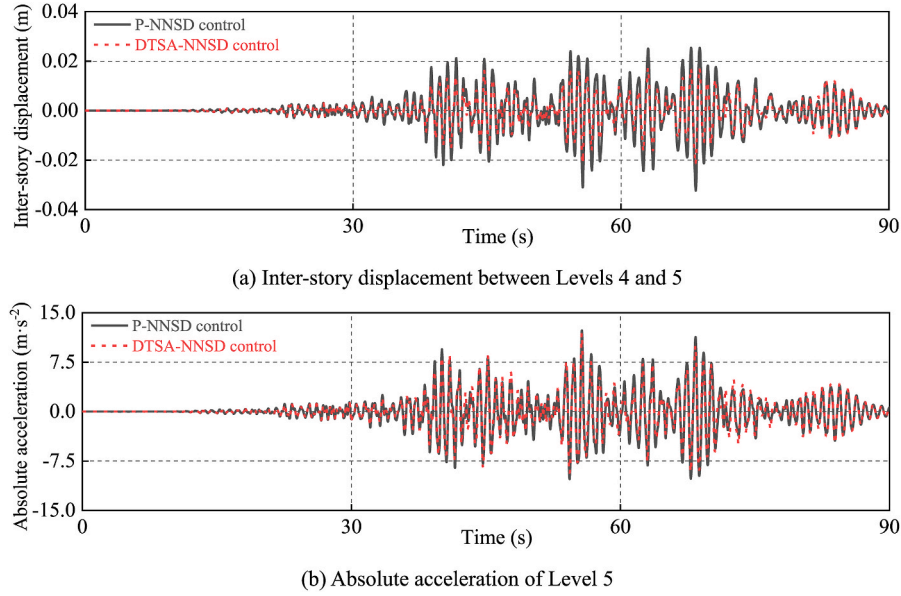


Fig. 13. Inter-story displacement and absolute acceleration responses of Level 5 under P-NNSD and DTSA-NNSD control.

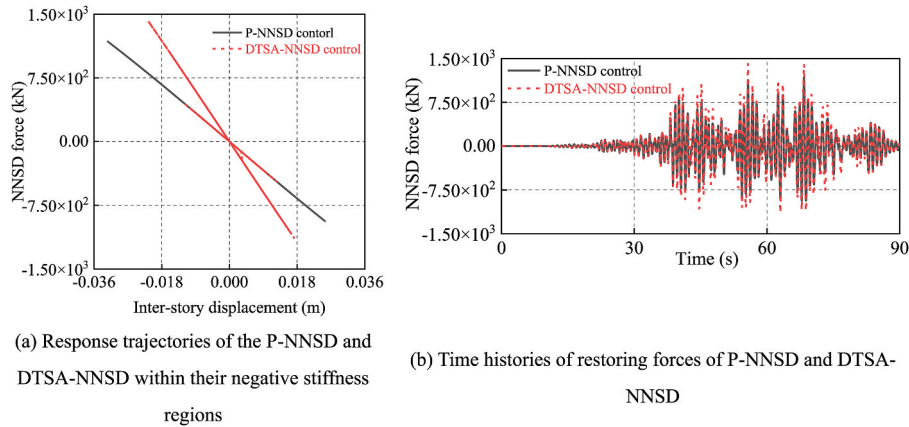


Fig. 14. Comparison of the mechanical responses of the P-NNSD and DTSA-NNSD under earthquake-hydrodynamic interaction.

hydrodynamic interaction.

The inclusion of earthquake-induced hydrodynamic interaction increases the relative displacement, inter-story displacement, and absolute acceleration responses of Level 5. Accordingly, all subsequent evaluations of NNSD control performance are conducted using the coupled

model with hydrodynamic feedback, thereby avoiding potential bias associated with evaluations based on a dry structural model.

Table 4
Mean peak-response reduction ratios for the five ground motions.

Case	Inter-story displacement	Absolute acceleration	Base shear
P-NNSD	61.1%	11.8%	10.5%
DTSA-NNSD	73.5%	15.0%	18.0%

6.2. Seismic performance of P-NNSD

Following the assessment of earthquake-induced hydrodynamic effects, the control performance of the fixed-parameter P-NNSD is evaluated. The P-NNSD is installed between Levels 4 and 5, and its parameters remain unchanged throughout the earthquake. The device regulates the dynamic interaction between the two adjacent degrees of freedom through its nonlinear negative stiffness restoring force.

The relative displacement responses of Level 5 before and after the installation of the P-NNSD are shown in Fig. 10.

For the illustrative record, Fig. 10 shows that the peak relative displacement of Level 5 is reduced by approximately 25% after installation of the P-NNSD. Under the representative ground motion, the P-NNSD reduces the peak Level 5 relative displacement by approximately 25%. Because the device is installed locally at the Level 4 - Level 5 interface, its effect differs from the uniform period elongation commonly associated with base isolation. The local stiffness modification changes the coupled response and force transmission of the multi-degree-of-freedom structure-water system. Nevertheless, in the absence of a dedicated modal-decomposition analysis, the observed reduction should be regarded as a model- and record-specific numerical result rather than an inherent displacement-reduction property of negative stiffness.

Because the P-NNSD directly connects Levels 4 and 5, the overall displacement of the platform top alone is insufficient to fully characterize its local control effect. Therefore, the inter-story displacement between the connected degrees of freedom and the absolute acceleration of Level 5 are further compared in Fig. 11.

Figs. 10 and 11 show that, under the representative ground motion, the P-NNSD reduces the peak Level 5 relative displacement, Level 4 - Level 5 inter-story displacement, and Level 5 absolute acceleration by approximately 25%, 70%, and 25%, respectively. These time-history results illustrate the response characteristics under one record, whereas the overall control performance is assessed using the five-record mean values reported in Table 4.

The force-displacement response and restoring-force time history of the P-NNSD under earthquake-hydrodynamic interaction are shown in Fig. 12.

Fig. 12(a) shows the actual operating trajectory of the P-NNSD. Its maximum deformation is approximately 0.036 m, well below the transition deformation of 0.110 m for $\lambda_p = 0.8$. The device therefore operates within a locally quasi-linear portion of the globally nonlinear negative stiffness branch, and the subsequent positive stiffness branch is not activated. Because the numerical NNSD contains no velocity-dependent or hysteretic term, it provides no supplemental energy dissipation; dissipation remains associated with the inherent structural damping and Morison drag.

Fig. 12(b) further shows the time history of the P-NNSD restoring force. The restoring force varies cyclically with the structural vibration and evolves consistently with the device deformation. Larger restoring forces occur mainly during periods of stronger structural response, whereas the device force decreases as the structural vibration subsides. This behavior confirms that the P-NNSD force is passively generated by the structural relative motion rather than being externally prescribed independently of the response.

Accordingly, neither supplemental device damping nor activation of the subsequent positive stiffness branch contributes to the response reduction observed in this case. The P-NNSD acts only by modifying the local restoring stiffness, while energy dissipation remains associated with the unchanged structural damping and Morison drag terms. The

observed response reduction is therefore associated with the modified coupled response of the structure-water system rather than with device-level dissipation.

6.3. Comparative performance of DTSA-NNSD

The mechanical characteristics of the fixed-parameter P-NNSD remain unchanged throughout the earthquake, and its control performance therefore depends on the compatibility between the prescribed device parameters and the actual structural response range. To improve the adaptability of the NNSD to varying response levels, the displacement-triggered semi-active NNSD (DTSA-NNSD) is further investigated.

The DTSA-NNSD employs the two-state control law defined in Section 4.4, with $\lambda_p = 0.8$ as the initial state, $\lambda_p = 0.7$ as the stronger negative stiffness state, and $x_{th} = 0.015$ m as the triggering threshold. The comparison is based on prescribed, literature-informed control parameters and should not be interpreted as a global parameter optimization.

The inter-story displacement between Levels 4 and 5 and the absolute acceleration of Level 5 under P-NNSD and DTSA-NNSD control are compared in Fig. 13.

As shown in Fig. 13, both NNSD strategies effectively suppress the structural response under coupled earthquake-hydrodynamic conditions, while the DTSA-NNSD provides further improvement over the fixed-parameter P-NNSD. The improvement is particularly evident in the inter-story displacement between Levels 4 and 5, indicating that displacement-triggered adjustment of the compression state enhances the control of local relative deformation. Compared with the P-NNSD-controlled case, the DTSA-NNSD further reduces the peak inter-story displacement and absolute acceleration by approximately 31.9% and 3.6%, respectively.

To evaluate the overall control performance without relying on a particular ground motion, the peak-response reduction ratios obtained from the five selected records are averaged with equal weighting. Table 4 reports these five-record mean values for the P-NNSD and DTSA-NNSD.

The P-NNSD achieves mean reductions of 61.1%, 11.8%, and 10.5% in peak inter-story displacement, absolute acceleration, and base shear, respectively. The corresponding reductions achieved by the DTSA-NNSD are 73.5%, 15.0%, and 18.0%, indicating better overall performance for the structural model and ground-motion set considered.

To further clarify the mechanical basis of the different control performances, the force-displacement relationships and restoring-force time histories of the P-NNSD and DTSA-NNSD are compared in Fig. 14.

Fig. 14(a) shows that the DTSA-NNSD initially follows the $\lambda_p = 0.8$ restoring-force branch and switches to the stronger $\lambda_p = 0.7$ branch when the prescribed displacement threshold is reached. The device response remains within the negative stiffness region of each branch, with the maximum deformation well below the corresponding stiffness-transition values. Consequently, both operating segments appear locally quasi-linear, although the complete fifth-order restoring-force relationships are globally nonlinear. The additional response reduction achieved by the DTSA-NNSD is therefore associated with response-dependent switching between two negative-tangent-stiffness states rather than with activation of the subsequent positive stiffness region.

The two-state strategy enables the magnitude of the negative tangent stiffness to be adjusted according to the structural response level. The moderate $\lambda_p = 0.8$ state is retained at lower response levels, whereas the stronger $\lambda_p = 0.7$ state is activated only after the prescribed displacement threshold is exceeded. This arrangement is intended to provide stronger control during large-response stages while avoiding the continuous application of the stronger negative stiffness state over the entire response history. Although this strategy is designed to balance control intensity and stiffness-related stability, a quantitative assessment of the stability margin is beyond the scope of the present study.

The restoring-force histories in Fig. 14(b) further show that the improved control performance of the DTSA-NNSD is not associated with an increase in the peak device force. Instead, its effectiveness depends on adjusting the restoring-force characteristics to the evolving structural demand through displacement-triggered switching. This result indicates that the timing and duration of the stronger negative stiffness state are important to the control performance, rather than the peak restoring force alone.

For the present platform model, the peak equivalent NNSD force is approximately 1.5×10^3 kN, substantially exceeding the capacity of the laboratory prototype shown in Fig. 4. For a potential full-scale implementation, multiple NNSD modules could be arranged in parallel between common end connections. Under the idealized assumption of compatible deformation among the modules, their restoring forces and tangent stiffnesses would be additive. Such a modular arrangement therefore provides a potential means of accommodating the required force demand; however, the present analysis does not establish its full-scale feasibility. Further investigation is required to assess individual-module capacity, force-displacement compatibility, load distribution among modules, connection design, fatigue resistance, corrosion protection, and the power and stroke demands associated with compression-state adjustment.

Overall, the results presented in Figs. 13 and 14 and Table 4 indicate that, for the structural model and ground motions considered, the DTSA-NNSD provides better overall response mitigation than the fixed-parameter P-NNSD. The most pronounced improvement is observed in the Level 4 - Level 5 inter-story displacement, with additional reductions in the Level 5 absolute acceleration and base shear. These results suggest that displacement-triggered regulation of the compression state can improve the response adaptability of the NNSD by matching its negative stiffness characteristics more closely to the evolving structural demand. Nevertheless, the reported performance is specific to the prescribed control parameters, structural configuration, and excitation set considered in this study.

7. Conclusions

This study investigated nonlinear negative-stiffness control of an offshore jacket platform under earthquake-induced hydrodynamic interaction. A coupled 5DOF model was established, and the performance of the fixed-parameter P-NNSD and displacement-triggered DTSA-NNSD was compared. The main conclusions are summarized as follows:

- (1) The reconstructed 5DOF model reproduces the reference natural frequencies with a maximum discrepancy of 0.38%, confirming the numerical implementation. The subsequent coupled analyses employ a separately defined lightweight-topside model with $m_5 = 200$ t and $T_1 = 0.822$ s; hence, the quantitative results apply specifically to this modified configuration.
- (2) For the illustrative ground motion, hydrodynamic feedback increases the peak Level-5 relative displacement, the peak inter-story displacement between Levels 4 and 5, and the peak Level-5 absolute acceleration by approximately 50%, 40%, and 25%, respectively. The greater sensitivity of the displacement responses indicates that neglecting hydrodynamic interaction may underestimate seismic demand and bias the evaluation of vibration control performance.
- (3) Across the five ground motions, the P-NNSD achieves mean reductions of 61.1%, 11.8%, and 10.5% in peak inter-story displacement, absolute acceleration, and base shear, respectively. Because the device contains no explicit damping term and remains within its negative stiffness region, these reductions represent model- and record-dependent responses of the coupled structure-water system rather than device-level energy dissipation.

- (4) The DTSA-NNSD achieves mean reductions of 73.5%, 15.0%, and 18.0% in peak inter-story displacement, absolute acceleration, and base shear, respectively, across the five ground motions. Under the representative record examined in detail, it further reduces the peak inter-story displacement and absolute acceleration by approximately 31.9% and 3.6%, respectively, relative to the P-NNSD.

The findings should be interpreted within the scope of the present study. Because the NNSD contains no explicit energy-dissipation component, its effectiveness depends on the structural and excitation characteristics. The assessment is also limited to a simplified 5DOF lightweight-topside model, five ground motions, prescribed device location and control parameters, and idealized switching without experimental validation. Soil-structure interaction and concurrent wave-earthquake loading are not considered. Therefore, the reported response reductions and control parameter settings apply specifically to the modified lightweight-topside configuration and the ground motions examined. Their applicability to the original 4850 t platform has not been established and requires structure-specific parameter assessment and verification of stability and device capacity.

Future work will extend the assessment to different platform configurations and a broader range of ground motions, incorporate soil-structure interaction and combined wave-earthquake loading, and optimize the device location and control parameters under stability and practical implementation constraints. Experimental studies will also be conducted to evaluate the feasibility and robustness of the proposed strategy.

CRediT authorship contribution statement

Yafei Zhang: Writing – review & editing, Writing – original draft, Methodology, Funding acquisition, Data curation. **Li Sun:** Writing – review & editing, Writing – original draft, Visualization, Validation, Project administration, Methodology.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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