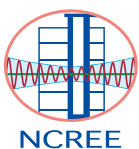


# 2025 NCREE Research

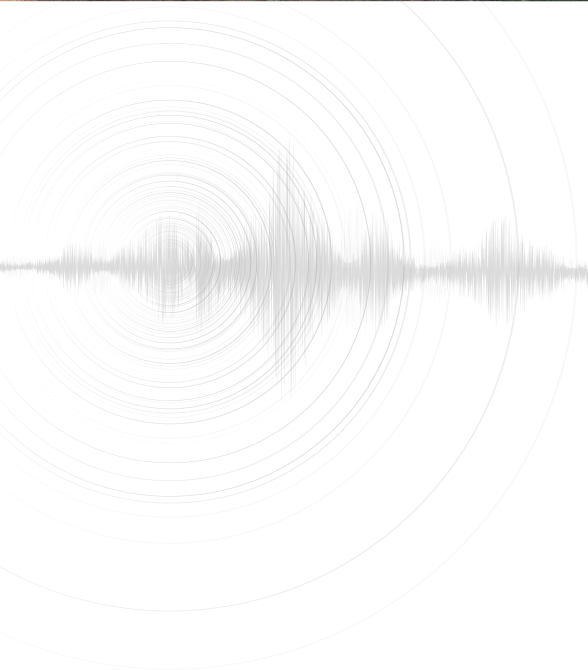
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# Development of Construction Diagrams of the In-Situ Cast-in-Place Retrofitting Method for Ultra-High-Performance Concrete Used in Brick Walls

Yuan-Tao Weng<sup>1</sup> Chung-Chan Hung<sup>2</sup> You-Ting Wei<sup>3</sup>

翁元滔<sup>1</sup>、洪崇展<sup>2</sup>、魏鈞廷<sup>3</sup>

## Abstract

In recent years, events such as the 2016 Kaohsiung Meinong earthquake and the 2018 and 2024 Hualien earthquakes have caused damage to many buildings, including serious collapses. However, most of these buildings are privately owned, making rehabilitation projects difficult to implement. In addition to challenges in raising funds, the lengthy construction period affects the use of space, which significantly reduces user willingness to proceed with reinforcement. The brick walls of the RC frame can effectively improve the lateral strength and stiffness of the entire frame, preventing collapse under strong earthquakes. This method uses a simple ultra-high-performance concrete (UHPC) reinforcement technique to enhance the earthquake resistance and collapse resistance of RC walls and other wall structures. The cladding construction methods can be divided into grouting, hand trowel, and shotcrete. The first three methods are considered wet construction, while the latter is semi-wet construction. Among these methods, the hand trowel involves using a trowel to directly apply and shape UHPC material on the reinforcement surface, whereas the shotcrete method involves delivering the mixed UHPC slurry to pumping equipment and spraying the slurry evenly onto the construction surface through the spraying system.

**Keywords:** ultra-high-performance concrete, seismic retrofitting, construction diagrams

## Introduction

Ultra-high-performance concrete (UHPC) is produced by incorporating fine aggregates such as quartz sand and silica fume to increase the density of the slurry and reduce weak surfaces associated with coarse aggregates. Fibers are added to address the limitations of traditional concrete, particularly its poor tensile and crack resistance. UHPC exhibits excellent crack suppression capabilities. It is widely used in earthquake-resistant structures, where its enhanced toughness and strength significantly improve seismic performance, reduce the need for confining and shear reinforcement, increase workability, and simplify the construction process. Research and development results indicate that UHPC can achieve compressive strengths of 120–140 MPa, and tensile strengths exceeding 7 MPa, both of which are more than four times greater than those of traditional concrete. UHPC can reduce the cross-section of structural components,

increase construction efficiency, and expand usable space. Owing to its dense material structure, UHPC demonstrates porosity, water absorption, chloride ion permeability, and other durability indicators that are ten to a hundred times lower than those of traditional concrete, substantially reducing structural aging and extending the service life of buildings.

The brick walls of the RC frame can effectively enhance the lateral strength and stiffness of the entire structure, thereby preventing collapse during strong earthquakes. This study primarily employs a simple UHPC reinforcement method to improve the earthquake resistance and collapse resistance of brick walls and other wall structures. The covering construction methods used can be categorized as either hand trowel or shotcrete. In the hand trowel method, a trowel is used to directly apply and shape UHPC material on the reinforcement surface. In the shotcrete method, the mixed UHPC slurry is delivered to pumping equipment and sprayed evenly onto the

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construction surface using a spraying system. The following sections introduce the construction methods associated with shotcrete.

The applicable conditions for this method are as follows:

1. Reinforcement goal: To improve the shear strength within the wall structure and the deformation toughness within and outside the surface, thereby effectively enhancing the earthquake resistance and collapse resistance of the wall structures.
2. Reinforcement direction: The lateral strength of the wall.
3. Material properties: To increase the bond strength between the new and old interfaces, appropriate bolt components must be used in accordance with the wall thickness and actual conditions. For brick walls with poor surface conditions (e.g., many cracks, etc.), the bolt installation position or method should be adjusted.

The main features of this method are as follows:

1. The construction methods of hand trowel and shotcrete do not require the assembly of formwork, which can effectively reduce the number of temporary works.
2. Hand troweling and shotcrete reinforcement methods provide good construction efficiency.
3. This method can save construction time.
4. Compared with conventional concrete encapsulation methods, the use of UHPC can effectively reduce the reinforcement thickness.

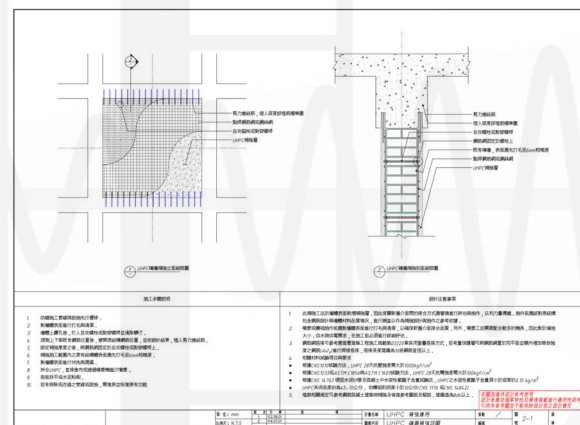
## Key Points of Retrofitting Design

1. Cladding is a process of adding a reinforcement layer to the wall surface. Therefore, the joints between the new and old surfaces should be carefully evaluated and constructed to facilitate force transfer. Before construction, the original structure, including the steel bar design and wall material quality, should be investigated as a reference for reinforcement design and construction.
2. The wall surface should be roughened and cleaned before coating to ensure the quality of the interface between the new and old surfaces. In addition, among the different coating methods, the spraying method requires more machinery and equipment, so the site size, water supply, and power supply requirements must be evaluated in detail before construction.
3. The overlap method of steel mesh can be used in accordance with Chapter 03220 of the Construction Engineering Specifications of the Construction and Planning Agency. If the protective layer is taken into consideration, the

steel mesh can be placed on the same plane, and additional steel bars of equivalent strength can be added for welding overlap. The overlap length is recommended to be at least 10 times the diameter of the steel bar.

4. The relevant material test items and requirements are as follows:
  - i. According to the CNS1010 test method, the 28-day compressive strength of UHPC must be greater than 1000 kgf/cm<sup>2</sup>.
  - ii. According to CNS1233 or ASTM C1856 and ASTM C1609 test methods, the 28-day flexural strength of UHPC must be greater than 120 kgf/cm<sup>2</sup>.
  - iii. According to CNS 14703 test method for water-soluble chloride ion content in hardened cement mortar and concrete, the water-soluble chloride ion content of UHPC must be less than or equal to 0.15 kg/m<sup>3</sup>.

## Development of Construction Diagrams



**Fig. 1.** Reference diagram of UHPC application in the brick wall cast-in-place reinforcement method.

The relevant description shall include the following parts:

1. The designer should draw detailed cross-sectional drawings of the relative relationship between beams and walls based on actual site conditions.
2. The thickness of UHPC should be determined after considering the design requirements and construction conditions and fully communicating with the owner. When steel mesh is used for reinforcement, the recommended reinforcement thickness is not less than 4 cm. If no steel mesh is used, the minimum reinforcement thickness should not be less than 2 cm.
3. The size and configuration of bolts and steel wire mesh can be determined according to the strength design results, but an appropriate



- protective layer thickness is required. Shear connecting bars should be reserved adjacent to the beam. It is recommended that shear connecting bars be #5 or below bamboo steel bars, and the strength of the steel bars must be less than 4900 kgf/cm<sup>2</sup>.
4. The UHPC reinforcement layer should transmit lateral forces through the connection between the shear reinforcement bars and the upper and lower beams. If the UHPC reinforcement layer cannot be connected to the upper and lower beams, it can be connected to the adjacent columns by embedding reinforcement bars. However, the force transmission path will pass through the plastic hinge areas above and below the columns. Therefore, the columns need to be further reinforced to avoid premature damage to the plastic hinges in the columns.
  5. The designer must calculate the number of shear tie bars. Before planting the bars, the designer can refer to the construction or design drawings of the existing beams to avoid conflicts between the bars. In addition, the designer can further use a bar scanning instrument to confirm the actual position of the beam bars before planting the bars.
  6. Waterproof design should be considered for areas around the reinforcement surface and at the interface between the new and old structures where there is contact with rainwater.
  7. Construction steps
  8. The construction steps for the UHPC brick wall reinforcement field cast-in-place cladding method are as follows:
  9. Prevent the relocation of construction pipelines and facilities.
  10. The surface of the original structure within the reinforcement construction area should first be roughened to a roughness of 6 mm.
  11. Crack treatment: Use a steel brush to remove debris from the cracks and use a vacuum cleaner or industrial air blower to remove dust and particles. For more serious cracks, use a stronger repair material. For deeper or wider cracks, use mortar, epoxy resin, or UHPC for repair. When troweling the surface, use a spatula to smooth the surface and ensure that the repaired area is flush with the surrounding area.
  12. After drilling holes in the wall, insert screw anchors or through-hole screws and clean out any dirt.
  13. After detecting the existing steel bar positions of the upper and lower beams, avoid the original structural steel bar positions and install shear connecting bars according to the design results.
  14. After setting the covering thickness, fix the steel mesh to the screw anchors or through-thread screws.
  15. Mix UHPC and apply the coating.
  16. Smooth the surface or apply cement plaster.
  17. UHPC maintenance: The material manufacturer's instructions on maintenance requirements must be followed, and the material must be kept moist. For larger repairs, keeping the repair area moist will help UHPC strength development. It can be covered with a wet cloth or plastic film to maintain humidity. Material storage and management measures must be followed during construction.
  18. If any pipelines or facilities have been removed or diverted, they must be restored to their original condition and function.

### Key Points of Retrofitting Design Review

1. Related material test items and requirements:
  - i. According to the CNS1010 test method, the 28-day compressive strength of UHPC must be greater than 1000 kgf/cm<sup>2</sup>.
  - ii. According to CNS1233 or ASTM C1856 and ASTM C1609 test methods, the 28-day flexural strength of UHPC must be greater than 120 kgf/cm<sup>2</sup>.
  - iii. According to CNS 14703 test method for water-soluble chloride ion content in hardened cement mortar and concrete, the water-soluble chloride ion content of UHPC must be less than or equal to 0.15 kg/m<sup>3</sup>.
  - iv. Anchor reinforcement is easily affected by the strength of existing concrete. Designers should consider the variability of materials and recommend further core sampling of existing concrete strength.
  - v. The steel mesh material manufacturer should be required to provide material-related reports. If the owner requires, the welded steel wire mesh can be inspected in accordance with Chapter 03220 of the Public Works Committee.
2. The size of the steel mesh or steel bar should be selected according to the analysis results and should have an appropriate protective layer.
3. The joint opening position should be designed according to the designed screw anchor hole position, but the number of bolts or screws should be sufficient to transmit the shear force of the UHPC reinforcement layer.

### Conclusions

UHPC is a recently developed cement-based composite material, both domestically and internationally. Due to its ultra-high strength and dense

microstructure, UHPC addresses the issue of reinforcement projects reducing usable space because of excessive volume, while also offering effective waterproofing. Regarding workability, the slump flow can be adjusted to suit construction conditions, thereby overcoming challenges where reinforcement cannot proceed due to unfavorable conditions.

To facilitate the application of this reinforcement method in practical projects, this study has developed a reference diagram and instruction document. These materials include the method's characteristics, applicable timing, reinforcement design process, precautions, construction steps, and design review items, enabling users to efficiently conduct reinforcement design and improve the quality of seismic reinforcement projects.

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# Toward Risk-Targeted Seismic Design in Taiwan

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## Abstract

In recent years, risk-targeted ground motion (RTGM) has gained increasing attention in civil engineering and building code development. RTGM defines the design ground motion that satisfies a specified structural collapse risk target by incorporating both seismic hazard and structural fragility, thereby achieving consistent reliability across regions. Compared with the traditional uniform-hazard ground motion (UHGM), RTGM provides a risk-consistent framework aligned with performance-based seismic design. This study presents a preliminary evaluation of RTGM in Taiwan using updated seismic hazard models. The inputs are based on the Taiwan SSHAC Level 3 PSHA Project (2015–2018), including refined three-dimensional areal source models and Taiwan-specific ground motion prediction equations (GMPEs) for crustal and subduction earthquakes. Fault source models are further updated using the 2020 technical report by the Geological Survey and Mining Management Agency (GSMMA). A range of fragility dispersion ( $\beta$ ) values is considered to assess its influence on RTGM. Results from 200 reference sites indicate that the risk coefficient ( $C_r$ ) averages approximately 1.0, with variability within  $\pm 15\%$ .  $C_r$  is primarily controlled by the hazard slope, which averages about 3.6, ranging from 2.3 to 5.6. These findings provide a basis for advancing seismic design in Taiwan toward a risk-consistent framework.

**Keywords:** risk-targeted ground motion (RTGM), performance-based seismic design, structural fragility, probabilistic seismic hazard analysis (PSHA), reliability analysis

## Introduction

The current Taiwan Building Code (TBC) for seismic design (NLMA, 2024) specifies nominal design ground motions in terms of site-specific spectral acceleration coefficients ( $S_{DS}$ ,  $S_{D1}$ ,  $S_{MS}$ , and  $S_{M1}$ ), which are determined based on uniform-hazard ground motions (UHGMs) derived from probabilistic seismic hazard analysis (PSHA). The PSHA input models rely primarily on early-generation ground motion prediction equations (GMPEs) and simplified background seismic source zonation, most of which were developed two decades ago. Fault sources are based on the 2012 edition of the active fault database published by the Central Geological Survey (CGS; reorganized as the Geological Survey and Mining Management Agency in 2023) (Lin et al., 2012).

The TBC defines three seismic hazard levels for UHGMs: the service-level earthquake (SLE), design

basis earthquake (DBE), and maximum considered earthquake (MCE), corresponding to return periods of 30, 475, and 2,500 years, respectively. The associated performance objectives are specified qualitatively. Under the SLE, structures are expected to remain essentially elastic, with no structural damage (Immediate Occupancy, IO). Under the DBE, inelastic demands should not exceed the allowable ductility capacity, and only repairable damage is permitted (Life Safety, LS). Under the MCE, structures may reach their nominal ductility capacity but should not collapse (Collapse Prevention, CP).

In recent years, risk-targeted ground motion (RTGM) has gained increasing attention in civil engineering and building code development. RTGM defines the level of ground motion that satisfies a specified structural collapse risk target by explicitly incorporating both seismic hazard and structural fragility, thereby achieving consistent reliability across

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regions and more directly supporting performance-based design objectives. The ASCE/SEI 7-10 standard in the United States was the first to adopt RTGM, defining the risk-targeted maximum considered earthquake ( $MCE_R$ ) corresponding to a 1% probability of collapse in 50 years. Other countries, including Canada, New Zealand, and members of the Eurocode, are also moving toward adopting RTGM in seismic design practice.

This study aims to perform a preliminary evaluation of RTGM characteristics in Taiwan by applying the RTGM framework with updated seismic hazard input models. The PSHA inputs are primarily based on the Taiwan SSHAC Level 3 PSHA Project (2015–2018), including refined three-dimensional areal source models and region-specific GMPEs for crustal and subduction earthquakes. In addition, the fault source models were updated using the 2020 technical report published by the CGS. A range of structural fragility dispersion ( $\beta$ ) values is considered to evaluate their influence on RTGM. The results provide a basis for advancing seismic design provisions in Taiwan toward a risk-consistent framework.

## PSHA Input Models

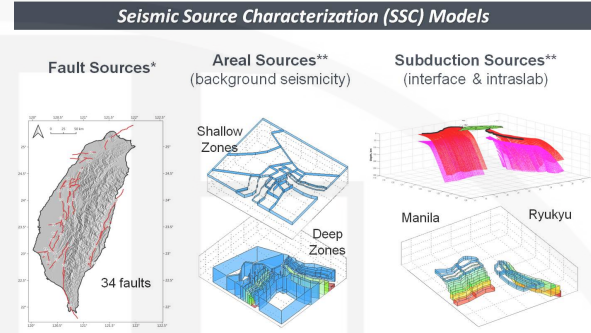
The PSHA in this study adopts seismic source characterization (SSC) and ground motion characterization (GMC) models, as summarized in Table 1. The areal source model, subduction source model, and Taiwan-specific GMPEs are primarily based on the “Reevaluation of Probabilistic Seismic Hazard of Nuclear Facilities in Taiwan Using SSHAC Level 3 Methodology” project conducted by Taiwan Power Company (2019). This project followed a highly structured SSHAC Level 3 process in accordance with nuclear regulatory guidelines (NUREG-2213, 2018), providing a rigorous framework with established quality assurance and technical oversight.

**Table 1** PSHA input models used in this study.

|                  |                                                                                                                                                                                                                                             |
|------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| SSC model        | <ul style="list-style-type: none"> <li>• <i>Areal sources</i>: Zone B</li> <li>• <i>Fault sources</i>: CGS 2012 active fault model and Chushiang Fault</li> <li>• <i>Subduction sources</i>: interface and intraslab earthquakes</li> </ul> |
| GMC model (GMPE) | <ul style="list-style-type: none"> <li>• <i>Crust type</i>: ASK14-adj, BSSA14-adj, CB14-adj, CY14-adj, Chao19, and Phung19</li> <li>• <i>Subduction type</i>: AGA16-adj, LL08-adj, Chao19, and Phung19</li> </ul>                           |

This study reconstructs a logic-tree framework for 34 active faults in Taiwan based on a research report commissioned by the CGS and conducted by Sinotech Engineering Consultants, Inc. (Yen et al., 2020). The integrated three-dimensional models of active faults, areal sources, and subduction sources are illustrated in Fig. 1.

PSHA calculations were performed for 200 reference sites distributed across Taiwan, assuming firm rock conditions ( $V_{S30} = 520$  m/s) and a ground motion variability parameter ( $\varepsilon$ ) of 2.0. Hazard computations were conducted using the validated software Haz45.3 (Hale et al., 2018).



**Fig. 1** 3D SSC models used in this study.

## Description of RTGM

Seismic risk ( $P_F$ ) can be calculated by combining the hazard curve  $H(a)$  with the fragility curve  $P_{F|a}(a)$ , as expressed in the following theoretical formula (McGuire, 2004; Kennedy and Short, 1994):

$$P_F = \int_0^{\infty} H(a) \left( \frac{dP_{F|a}(a)}{da} \right) da \quad (1)$$

where  $a$  represents the intensity measure (e.g., spectral acceleration).

Structural fragility curves are represented by a lognormal distribution characterized by two parameters: the median  $C_{50}$  and the logarithmic standard deviation  $\beta$ . The median  $C_{50}$  corresponds to the ground motion intensity at which the probability of collapse is 50%, while  $\beta$  represents the dispersion in structural seismic capacity. ASCE/SEI 7-22 defines structural fragility under the  $MCE_R$  level as having a 10% probability of collapse; therefore,  $P_{F|a}(a)$  can be expressed as follows:

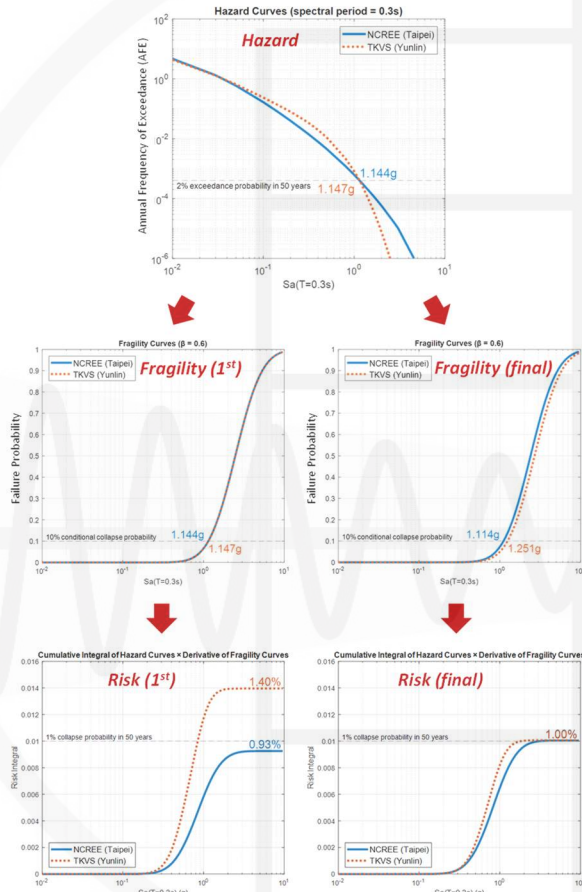
$$P_{F|a}(a) = \Phi \left[ \frac{\ln a - (\ln RTGM + 1.28)}{\beta} \right] \quad (2)$$

where  $\Phi[\cdot]$  denotes the standard normal cumulative distribution function. The RTGM is defined as the ground motion intensity that satisfies a target risk level. Based on ASCE/SEI 7-22, a  $\beta$  value of 0.6 is typically recommended for structural fragility representation.

Fig. 2 presents the RTGM and the corresponding probability of collapse for the Taipei and Yunlin sites under the same MCE condition. The spectral acceleration at a period of 0.3 s for the 2,500-year return period is approximately 1.15g for both sites, which is initially taken as the RTGM. The resulting probabilities of collapse are 0.93% for the Taipei site and 1.40% for the Yunlin site, both deviating from the 1% target risk level.



This indicates that the Yunlin site exhibits a steeper hazard curve and a greater contribution from moderate ground motions, resulting in an elevated collapse risk ( $>1\%$ ). Therefore, the RTGM must be increased to 1.25g to achieve the target 1% risk level. Conversely, the Taipei site is characterized by a flatter hazard curve, with ground motion hazard dominated by low-probability, infrequent earthquake events, leading to a lower collapse risk ( $<1\%$ ). As a result, the RTGM can be reduced to 1.11g while still satisfying the target 1% risk level.



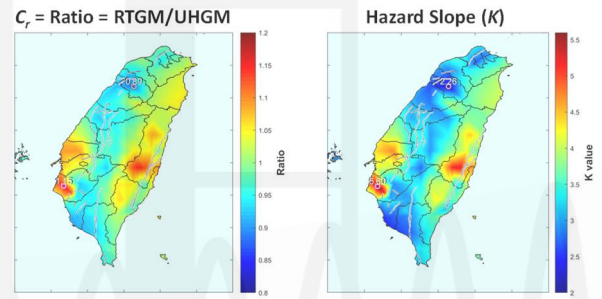
**Fig. 2** Example of RTGM calculation for the Taipei and Yunlin sites.

## Results and Discussion of Taiwan RTGM

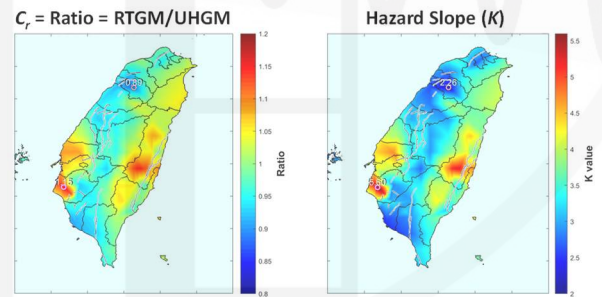
Based on the updated PSHA input models, the structural fragility assumption of a 10% probability of collapse under RTGM conditions with  $\beta = 0.6$ , and a target risk of 1% probability of collapse in 50 years, the spatial distributions of the risk coefficient ( $C_r$ ) and hazard slope ( $K$ ) for spectral accelerations at 0.3 s and 1.0 s (hereafter  $SA_{0.3s}$  and  $SA_{1.0s}$ ) are shown in Figures 3 and 4, respectively (Liu and Huang, 2025). The risk coefficient  $C_r$  is defined as the ratio of RTGM to the 2,500-year return period UHGM, while the hazard slope  $K$  is evaluated as the logarithmic slope between the 475- and 2,500-year return periods.

Comparison of Fig. 3 and Fig. 4 indicates that  $SA_{0.3s}$  and  $SA_{1.0s}$  exhibit broadly similar spatial patterns in both  $C_r$  and  $K$  values, with higher values concentrated in eastern and southwestern Taiwan, particularly in Hualien, Yunlin, and Tainan. A positive correlation between  $C_r$  and  $K$  is observed, indicating that the hazard slope is a primary controlling factor for RTGM. This also reflects that RTGM is determined by site-specific seismicity characteristics to satisfy a target collapse risk.

Statistically,  $C_r$  values in Taiwan have an average of approximately 1.0, ranging from 0.88 to 1.15, with an overall variability of about  $\pm 15\%$ . The  $K$  values average approximately 3.6, ranging from 2.3 to 5.6, which is broadly comparable to those observed in other high-seismicity regions such as the western United States (approximately 3.0–4.0).

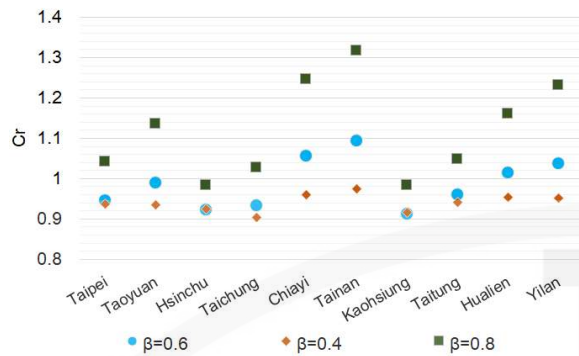


**Fig. 3**  $C_r$  and  $K$  values for 0.3 s spectral acceleration ( $SA_{0.3s}$ ) in Taiwan.

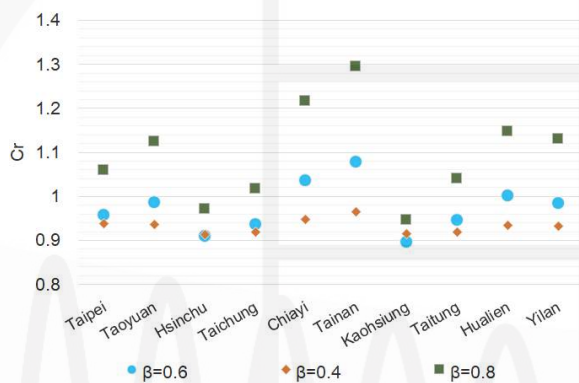


**Fig. 4**  $C_r$  and  $K$  values for 1.0 s spectral acceleration ( $SA_{1.0s}$ ) in Taiwan.

Fig. 5 and Fig. 6 present  $C_r$  values for  $SA_{0.3s}$  and  $SA_{1.0s}$  under three  $\beta$  levels (0.4, 0.6, and 0.8), evaluated in 10 reference cities (Liu and Huang, 2025). Overall, a smaller  $\beta$  value indicates that collapse risk is primarily concentrated in rare, high-intensity earthquakes. Under the target risk level of a 1% probability of collapse in 50 years, the resulting RTGM may be lower than the conservative UHGM. In contrast, a larger  $\beta$  value reflects greater uncertainty in structural capacity, leading to a broader range of ground motions contributing to collapse risk, including moderate events. Consequently, a higher RTGM is required to maintain the specified risk target.



**Fig. 5**  $C_r$  values for different  $\beta$  levels in 10 cities under 0.3 s spectral acceleration ( $SA_{0.3s}$ ) conditions.



**Fig. 6**  $C_r$  values for different  $\beta$  levels in 10 cities under 1.0 s spectral acceleration ( $SA_{1.0s}$ ) conditions.

## Conclusions and Future Work

Using RTGM as the basis for seismic design enables buildings in different regions to achieve consistent performance objectives and reliability levels by explicitly accounting for both seismic hazard (reflecting site-specific seismicity) and structural fragility (representing building seismic capacity). In contrast to conventional approaches that apply a uniform hazard level (e.g., a 2,500-year return period) across all regions, RTGM avoids spatial inconsistency in collapse risk among different locations. Therefore, the RTGM framework not only improves structural safety and resilience, but also provides a quantitative basis for governmental decision-making in resource allocation and disaster risk management, thereby strengthening national-level disaster prevention and mitigation strategies.

The updated PSHA results indicate higher seismic hazard levels in certain regions than those implied by the current UHGM-based design maps, with notable differences particularly in near-fault areas. Future work will focus on further refining SSC models, including key parameters such as fault slip rates, time-dependent behavior, and rupture segmentation models, to improve the reliability and applicability of both PSHA and RTGM results.

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# Suggestions for Implementing the Width-limit Effect Source Scaling Models in M-based Hazard Analysis

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黃雋彥<sup>1</sup>、林哲民<sup>2</sup>

## Abstract

Huang et al. (2024; hereafter referred to as the Hea24 model) proposed global-scale earthquake source scaling models applicable to Taiwan that differ from previous approaches. By incorporating the rupture width-limit effect and constructing the model primarily based on fault geometry, it effectively addresses one of the major limitations in seismic hazard assessment associated with conventional source scaling models by looking at the difficulty of reliably constraining fault mechanism prior to earthquake occurrence. Instead, the Hea24 model establishes a global framework based on the relationship between fault dip angle and seismogenic thickness, while also accounting for regional variations, such as those in Taiwan, in terms of magnitude-dependent scaling of fault aspect ratio (length-to-width) and rupture area.

Because the Hea24 model focuses on regional-scale characterization, Taiwan is treated as a region with a low aspect ratio, reflecting its gentle fault dip angles and deeper seismogenic zones. This regionalized approach offers a clear advantage over models based solely on global scaling, making it more suitable for earthquake assessments and seismic hazard applications in Taiwan. However, the current framework does not explicitly account for variations in seismogenic depth among individual faults, even though some faults are significantly deeper or shallower than the regional average. Therefore, this study proposes a methodology to extend the width-limit-effect-based source scaling model of Hea24 from the regional scale to fault-specific applications. The regional free-surface effect is omitted due to the difficulty of determining surface rupture status before an earthquake. Additionally, the model replaces the regional normalized depth terms with fault-specific smoothed depth functions within the remaining rupturable fault zone. This approach aims to improve the reliability and applicability of fault rupture scaling (including rupture width and length) for use in fault-specific seismic hazard analysis, source inversion, and ground motion simulation.

**Keywords:** source scaling model, width-limit effect, seismic hazard

## Introduction

Huang et al. (2024; hereafter referred to as the Hea24 model) proposed an earthquake source scaling model applicable both globally and to Taiwan. This model incorporates the rupture width-limit effect and is primarily based on fault geometry. Unlike previous scaling relationships, which classify faults by magnitude and focal mechanism, the Hea24 model addresses the difficulty in seismic hazard assessment of determining the focal mechanism before an earthquake. Instead, it uses the relationship between fault dip angle

and seismogenic thickness to establish a global framework, while also accounting for regional differences in terms of fault aspect ratio (length-to-width) and how rupture area scales with magnitude.

Because fault dip angle ( $\nu$ ) and seismogenic thickness can be determined through geological, geophysical, and seismological methods, this approach is well-suited for constructing detailed, large-scale assessments in a target region. The Hea24 model emphasizes regional-scale characterization, treating Taiwan as a region with a low aspect ratio,

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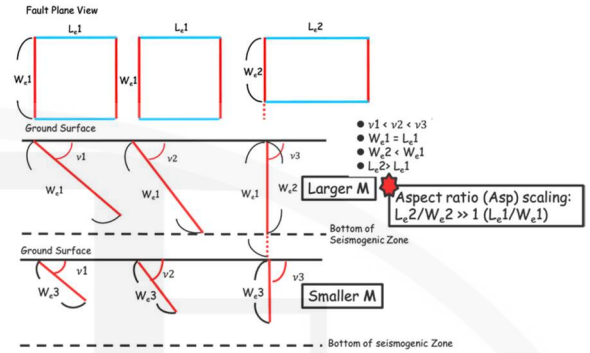
which reflects its gentler fault dip angles and deeper seismogenic zones. However, this framework is mainly applicable at the regional scale and does not explicitly address differences in seismogenic depth for individual faults compared to the regional background. Therefore, this study introduces a methodology to adapt the width-limit-effect-based source scaling model from the regional scale to individual faults. This approach aims to enhance the reliability and applicability of fault rupture scaling in fault-specific seismic hazard analysis, source inversion, and ground motion simulation.

In the Hea24 model, the rupture width-limit effect describes how faults with steeper dip angles and shallower seismogenic depths are more likely to be restricted in rupture width, leading to changes in aspect ratio at smaller magnitudes, and vice versa (see Fig. 1). This variation occurs because rupture initiation typically starts as an approximately circular patch that expands simultaneously along both the strike (length) and dip (width) directions on the fault plane. However, once the rupture reaches the base of the seismogenic zone, further downward propagation becomes increasingly difficult due to higher geothermal gradients, lithostatic pressure, confining pressure, and stress. At this depth, the mechanical behavior transitions from brittle to ductile, inhibiting further rupture growth in the width direction. As a result, rupture energy is released along the strike direction, increasing the aspect ratio and changing the area scaling.

Fig. 2 illustrates the fault aspect ratio and rupture area scaling relationships used in the Hea24 model. These relationships incorporate earthquake magnitude, fault dip angle  $\nu$ , and a regional term,  $RT(M)$ , which represents the magnitude-dependent threshold at which large earthquakes reach the bottom of the seismogenic zone. The model also accounts for ruptures that initiate at greater depths, where more ductile conditions further limit downward propagation and favor along-strike rupture growth, thereby modifying the aspect ratio and area scaling. In addition, the free-surface effect is included: ruptures propagating to the ground surface expend more energy due to weaker near-surface materials, resulting in reduced rupture extent along strike and thus altering the aspect ratio and area relationships. These depth-dependent scaling effects are distinguished using a normalized rupture bottom depth parameter ( $NZ_{bor}$ ), which captures differences associated with the free-surface effect.

Finally, the Hea24 model introduces a dip-dependent break magnitude, which represents  $M_{BP}(\nu)$ , the magnitude at which rupture reaches the base of the seismogenic zone. This parameter is defined at the regional scale. Building on this concept, the present study proposes methods to convert regional-scale evaluations into fault-specific assessments of allowable rupture width. These approaches are intended to

support broader use of the model in fault-based analyses.



**Fig. 1.** The example figure of the influence of the width-limit effect in the source scaling models used in the Hea24 model.

#### > Aspect ratio scaling relation

$$\checkmark \log_{10}(Asp_e) = S4_{ASP}(M, \nu, NZ_{bor}, \text{region})$$

- Width-limit effect:  $M, \nu, \text{region}$  ( $RT(M)$  for large- $M$ )
- Depth-Dependent and Free-surface effects:  $NZ_{bor}$

#### > Area scaling relation

$$\checkmark \log_{10}(A_e) = S3_A(M, \nu, NZ_{bor})$$

- Width-limit effect:  $M, \nu$
- Depth-Dependent and Free-surface effects:  $NZ_{bor}$

**Fig. 2.** Example schematic diagram illustrating the fault aspect ratio (length-to-width ratio) and area as used in Hea24. The functional forms shown represent those applied to each seismic and physical effect considered, depicted in stages 1 to 4.

## Recommended Procedure for Applying Width-Limited Source Scaling Relationships to Fault-Specific Cases

First, in seismic hazard assessment, ground motion simulation, or source inversion, the free-surface effect is difficult to determine in advance; therefore, it is omitted in the proposed procedure. In addition, for a specific fault with the earthquake that occurred, the location of the rupture bottom should be related to variations in rupture width with consideration of magnitude scaling. However, because this information is also difficult to constrain prior to an earthquake, this study recommends assuming that the rupture bottom is located within the remaining down-dip extent, defined as the difference between the maximum rupturable fault width and the rupture width estimated from the

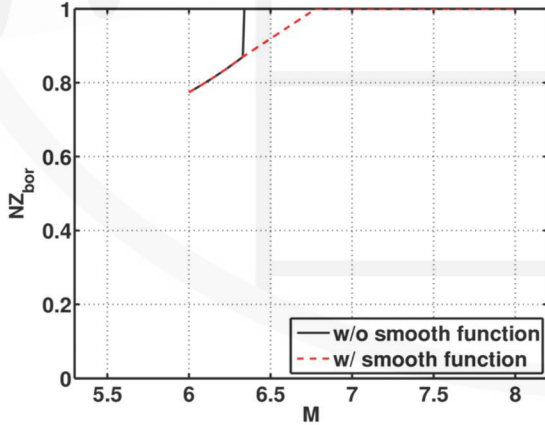


width-scaling relationship provided by Hea24. Furthermore, the rupture bottom is assumed to lie at the midpoint of this remaining rupture width. Based on this assumption, the normalized bottom rupture depth  $NZ_{bor}$  is estimated as a function of the rupture width ratio (RWR) using the following formulation (illustrated in Fig. 3):

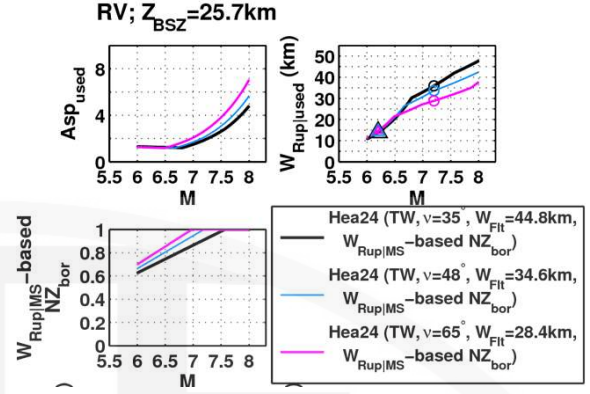
$$NZ_{bor} = \begin{cases} RWR_{MS}(M, \nu, NZ_{bor_{MS}}) + \\ (1 - RWR_{MS}(M, \nu, NZ_{bor_{MS}}))/2; & M \leq M_{BP} \\ 1; & M > M_{BP} \end{cases} \quad (1)$$

Because this formulation leads to saturation at a certain magnitude, beyond which the normalized bottom rupture depth approaches unity, a smoothing function is introduced to ensure continuity in practical applications (as shown by the red dashed line in Fig. 3).

Finally, Fig. 4 presents example applications of the translated source scaling relationships under different conditions. For instance, when the seismogenic depth ( $Z_{BSZ}$ ) is held constant and the fault dip angle varies, the normalized bottom rupture depth  $NZ_{bor}$  changes accordingly. In these scenarios, the magnitude-dependent scaling relationship for fault width can be evaluated using the proposed approach.



**Fig. 3.** Schematic illustration of the parameters proposed in this study for translating the regionally applicable Hea24 model to fault-specific applications with the smooth function.



**Fig. 4.** Schematic representation of the translated source scaling models for fault-specific conditions based on the regionally applicable Hea24 model. Examples are shown for varying fault dip angles under a fixed seismogenic depth ( $Z_{BSZ}$ ) representative of Taiwan. The upper-left panel shows the aspect ratio, the upper-right panel shows rupture width, and the lower-left panel presents the magnitude-dependent relationship of the normalized bottom rupture depth ( $NZ_{bor}$ ) as a function of the rupture width ratio (RWR). Different colors denote different fault dip conditions.

## Conclusions

This study adapts the Hea24 source scaling model, originally developed for regional use, into scaling relationships suitable for individual faults. The approach relies on prior characterization of fault geometry using geological, geophysical, or seismic data specific to the fault. Once these parameters are determined, the methodology can be applied to individual fault scenarios, offering significant practical value for engineering applications.

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NCREE

# Characterizing Effective Supplemental Damping Properties to Estimate the Seismic Response of Non-Proportionally Damped Two-Way Asymmetric Buildings

Jui-Liang Lin<sup>1</sup>

林瑞良<sup>1</sup>

## Abstract

Building codes stipulate that the design response spectrum may be reduced when the damping ratio of the building exceeds 5%. To reduce the 5%-damped design response spectrum, it is necessary to quantify the effective damping of non-proportionally damped buildings. It is recognized that both the resultant damping and the distribution characteristics of viscous dampers across the floors affect the seismic response of asymmetric-plan buildings. Therefore, this study proposes a method to characterize the effective damping properties of non-proportionally damped two-way asymmetric-plan buildings. The proposed method involves constructing a non-proportionally damped three-degree-of-freedom (3DOF) oscillator in modal space, which reflects the characteristics of the dominant mode in each of the three directions of the two-way asymmetric-plan building. By mapping the seismic response of the 3DOF oscillator into physical space, the seismic response of a 20-story example building was satisfactorily estimated.

**Keywords:** supplemental damping, viscous damper, asymmetric building, seismic response, modal analysis

## Introduction

Damping reduction factors are used to adjust the 5%-damped design response spectra specified by building codes when supplemental damping is incorporated into buildings (ASCE/SEI 41-13, 2014). Before applying damping reduction factors to modify the 5%-damped design response spectra, it is essential to determine the level of damping achievable with added dampers. Occhiuzzi (2009) proposed a methodology based on the state-space representation of dynamical systems for computing modal damping ratios of structures. Charney and McNamara (2008) compared the following three methods: modal strain energy using undamped mode shapes, free vibration log decrement, and complex eigenvalue-eigenvector analysis—to determine equivalent viscous damping ratios in structures with added viscous dampers. Lee et al. (2004) introduced a method for evaluating the equivalent damping ratio of a structure equipped with linear or nonlinear supplemental damping devices. The equivalent damping ratio was assessed using the

Lyapunov function and its derivative. They noted that the proposed approach offers little advantage in predicting the equivalent damping of a linear system compared to conventional methods. However, the procedure provides a convenient tool for estimating the damping effect of nonlinear damping devices.

ASCE/SEI 41-13 (2014) stipulates the computation of effective damping, denoted as  $\beta_{eff}$ , resulting from linear viscous dampers as follows:

$$\beta_{eff} = \beta_0 + \beta_{sd} = \beta_0 + \frac{T \sum_j C_j \cos^2 \theta_j \phi_{rj}^2}{4\pi \sum_i \left(\frac{w_i}{g}\right) \phi_i^2} \quad (1)$$

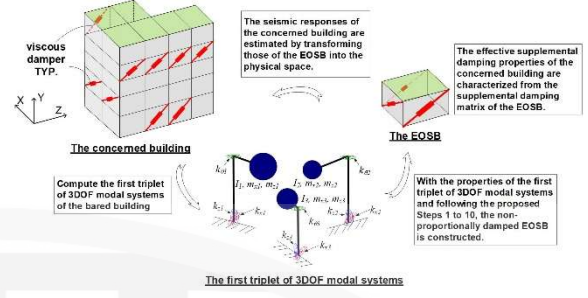
where  $\beta_0$  is the inherent damping of the building and  $\beta_{sd}$  is the supplemental damping,  $T$  is the fundamental period of the building,  $\theta_j$  is the angle of inclination of device  $j$  to the horizontal,  $\phi_{rj}$  is the first mode relative displacement between the ends of device  $j$  in the

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horizontal direction,  $w_i$  is the reactive weight of floor level  $i$ , and  $\phi_i$  is the first mode displacement at floor level  $i$ . Given the effective damping  $\beta_{eff}$  (Eq. 1), modal response spectrum analysis or modal response history analysis can be performed. Due to non-proportional or non-classical damping, seismic responses between vibration modes are typically coupled. As a result, real-valued modal response spectrum analyses are only approximate methods for analyzing non-proportionally damped buildings. Additionally, supplemental damping in buildings generally leads to different effective damping ratios for various vibration modes. Therefore, applying a single supplemental damping ratio to reduce spectral values across all spectral periods in modal response spectrum analyses is questionable.

It is common practice to install viscous dampers symmetrically in a symmetrical building. In this case, the only supplemental damping property requiring quantification is the supplemental damping  $\beta_{sd}$ . When viscous dampers are placed in an asymmetric-plan building, it is necessary to consider not only the resultant damping but also the distribution characteristics of the added viscous dampers across the floor plans. Goel (2000) noted that, in addition to the effective supplemental damping ratio (denoted as  $\xi_{sd}$ ), the effective supplemental damping eccentricity (denoted as  $e_{sd}$ ) and the effective supplemental damping radius of gyration (denoted as  $\rho_{sd}$ ) are properties that collectively characterize the resultant effect of supplemental damping on the seismic response of a one-way asymmetric-plan building.

Lin (2017) developed the effective one-story building (EOSB) model for proportionally damped two-way asymmetric-plan buildings. The EOSB, a three-degree-of-freedom (3DOF) oscillator in modal space, represents the 3DOF modal properties associated with the first triplet of vibration modes of the two-way asymmetric-plan building. These modes include the fundamental mode in each of the three directions: two translational and one rotational. This study aims to develop a non-proportionally damped EOSB for an  $N$ -story two-way asymmetric-plan building with linear viscous dampers, particularly when the  $3N \times 3N$  supplemental damping matrix of the  $N$ -story building is unavailable. Once the supplemental damping matrix of the EOSB is constructed, the effective supplemental damping properties of the  $N$ -story building can be characterized. The non-proportionally damped EOSB, a 3DOF modal oscillator reflecting the dominant mode in each of the three directions, is then used to estimate the seismic response of the  $N$ -story building (Fig. 1).



**Fig. 1.** Schematic diagram of the proposed EOSB for non-proportionally damped two-way asymmetric-plan buildings.

## Methodology

The EOSB of a proportionally damped  $N$ -story two-way asymmetric-plan building is provided by Lin (2017). This type of building has rigid floor diaphragms, with the centers of mass and rigidity for all stories located along two vertical lines. The damping matrix of a non-proportionally damped EOSB comprises two components: inherent damping (denoted as  $\mathbf{C}_0^*$ ) and supplemental damping (denoted as  $\mathbf{C}_{sd}^*$ ). Inherent damping is assumed to Rayleigh damping, with the damping ratios of the EOSB's first two vibration modes matching those of the first two vibration modes of the first triplet of vibration modes of the building without dampers. The supplemental damping  $\mathbf{C}_{sd}^*$  is defined as:

$$\mathbf{C}_{sd}^* = \begin{bmatrix} C_{xx} & 0 & -C_{xx}e_{sdz} \\ 0 & C_{zz} & C_{zz}e_{sdzx} \\ -C_{xx}e_{sdz} & C_{zz}e_{sdzx} & C_{\theta\theta} + C_{xx}e_{sdz}^2 + C_{zz}e_{sdzx}^2 \end{bmatrix} \quad (2)$$

Five parameters in Eq. 2, including three supplemental damping coefficients ( $C_{xx}$ ,  $C_{zz}$ , and  $C_{\theta\theta}$ ) and two supplemental damping eccentricities ( $e_{sdz}$  and  $e_{sdzx}$ ), need to be determined. Ideally, the determination of these five parameters should be independent of input ground motions, since actual ground motions from future earthquakes are inherently unpredictable. It is recognized that the displacement response of a single-degree-of-freedom (SDOF) oscillator subjected to a unit impulse is the unit impulse-response function. The complete response history of an SDOF oscillator subjected to a ground motion is a composition of unit impulse-response functions. The unit impulse-response function is the inverse Fourier transform of the frequency response function, which is a general dynamic property of an oscillator and is independent of the characteristics of input excitations. Therefore, by utilizing a unit impulse to excite the EOSB, the procedure for determining the values of the five parameters ( $C_{xx}$ ,  $C_{zz}$ ,  $C_{\theta\theta}$ ,  $e_{sdz}$ , and  $e_{sdzx}$ ) is as follows:



Step 1: The peak  $x$ -directional roof displacement (denoted as  $u_{xr,peak}$ ) of the bare building (i.e., the building without dampers) subjected to an  $x$ -directional unit impulse (i.e.,  $x$ -directional ground acceleration with a unit acceleration at a single time point and zero accelerations at other time points) is computed. Similarly, the peak  $x$ -directional displacement (denoted as  $u_{x,peak}^*$ ) of the bare EOSB (i.e., the EOSB without supplemental damping matrix  $\mathbf{C}_{sd}^*$ ) subjected to an  $x$ -directional unit impulse is computed. The  $x$ -directional mapping factor is then obtained as  $\eta_x = \frac{u_{xr,peak}}{u_{x,peak}^*}$ .

Step 2: This is similar to Step 1, except that the displacement is calculated for an input  $z$ -directional unit impulse excitation applied to both the bare building and the bare EOSB. Accordingly, the  $z$ -directional mapping factor is obtained as  $\eta_z = \frac{u_{zr,peak}}{u_{z,peak}^*}$ .

Step 3: The  $x$ - and  $z$ -directional roof displacement histories (denoted as  $u_{xr}(t)$  and  $u_{zr}(t)$ ) of the non-proportionally damped building (i.e., the building with dampers) subjected to  $x$ - and  $z$ -directional unit impulses are computed, respectively. Similarly, the roof rotational history (denoted as  $u_{\theta r}(t)$ ) of the non-proportionally damped building subjected to a  $y$ -rotational unit impulse is also computed.

Step 4: From the decaying amplitudes of the displacement histories  $u_{xr}(t)$ ,  $u_{zr}(t)$ , and  $u_{\theta r}(t)$ , the initial conjectured values of  $C_{xx}$ ,  $C_{zz}$ , and  $C_{\theta\theta}$  (denoted as  $C_{xx}^{(0)}$ ,  $C_{zz}^{(0)}$ , and  $C_{\theta\theta}^{(0)}$ ) are computed as follows:

$$\begin{aligned} C_{xx}^{(0)} &= 2m_x \omega_x \xi_x = 2m_x \omega_x \frac{\ln\left(\frac{u_{xr,1}}{u_{xr,2}}\right)}{2\pi} \\ &= \frac{\omega_x}{\pi} \ln\left(\frac{u_{xr,1}}{u_{xr,2}}\right) \\ C_{zz}^{(0)} &= \frac{\omega_z}{\pi} \ln\left(\frac{u_{zr,1}}{u_{zr,2}}\right), \quad C_{\theta\theta}^{(0)} = \frac{\omega_\theta}{\pi} \ln\left(\frac{u_{\theta r,1}}{u_{\theta r,2}}\right) \end{aligned} \quad (3)$$

in which  $u_{xr,1}$  and  $u_{xr,2}$  represent the first two maximum amplitudes of  $u_{xr}(t)$ ,  $u_{zr,1}$  and  $u_{zr,2}$  are the first two maximum amplitudes of  $u_{zr}(t)$ , and  $u_{\theta r,1}$  and  $u_{\theta r,2}$  denote the first two maximum amplitudes of  $u_{\theta r}(t)$ . Additionally,  $\omega_x$ ,  $\omega_z$ , and  $\omega_\theta$  are the angular frequencies of the  $x$ -translational,  $z$ -translational, and  $y$ -rotational dominant modes of the EOSB. It should be noted that the  $x$ -translational,  $z$ -translational, and  $y$ -rotational dominant modes do not necessarily correspond to the lowest, intermediate, and highest vibration

modes of the first triplet of vibration modes of the concerned building.

Step 5: The iteration counter is set to  $j = 0$ , and the initial conjectured  $\mathbf{C}_{sd}^*$  is set as follows:  $C_{xx} = C_{xx}^{(0)}$ ,  $C_{zz} = C_{zz}^{(0)}$ ,  $C_{\theta\theta} = C_{\theta\theta}^{(0)}$ , and  $e_{sdx} = e_{sdx}^{(0)} = k_{23}/k_{22}$ ,  $e_{sdz} = e_{sdz}^{(0)} = -k_{13}/k_{11}$ , where  $k_{23}$ ,  $k_{22}$ ,  $k_{13}$ , and  $k_{11}$  are the elements of the stiffness matrix of the EOSB (Lin, 2017). Because  $k_{23}/k_{22}$  and  $-k_{13}/k_{11}$  represent the structural eccentricities, the initial conjectured supplemental damping eccentricities ( $e_{sdx}^{(0)}$  and  $e_{sdz}^{(0)}$ ) are presumed to be the same as their structural eccentricity counterparts.

Step 6: The three directional roof displacement histories (denoted as  $u_{xr}^b(t)$ ,  $u_{zr}^b(t)$ , and  $u_{\theta r}^b(t)$ ) of the non-proportionally damped building subjected to a bi-directional unit impulse (i.e., by simultaneously applying unit impulses in  $x$ - and  $z$ -directions) are computed. The superscript  $b$  denotes the responses resulting from a bi-directional unit impulse.

Step 7: The iteration counter is set to  $j = j + 1$ .

Step 8: By varying  $e_{sdx}$  and  $e_{sdz}$  from  $-3e_{sdx}^{(j-1)}$  to  $3e_{sdx}^{(j-1)}$  and from  $-3e_{sdz}^{(j-1)}$  to  $3e_{sdz}^{(j-1)}$ , the three directional displacement histories (denoted as  $u_{x1}^{*b}(t)$ ,  $u_{z1}^{*b}(t)$ , and  $u_{\theta 1}^{*b}(t)$ ) of the non-proportionally damped EOSB subjected to a bi-directional impulse are computed. The bi-directional impulse comprises  $x$ -directional and  $z$ -directional unit impulses multiplied by  $\eta_x$  and  $\eta_z$ , respectively. The suggested ranges for varying  $e_{sdx}$  and  $e_{sdz}$  (i.e., from  $-3e_{sdx}^{(j-1)}$  to  $3e_{sdx}^{(j-1)}$  and from  $-3e_{sdz}^{(j-1)}$  to  $3e_{sdz}^{(j-1)}$ ) and their increments can be adjusted depending on a compromise between the required precision and the affordable computation time. Of all the considered combinations of  $e_{sdx}$  and  $e_{sdz}$ , the values of  $e_{sdx}$  and  $e_{sdz}$  are updated as the pair (denoted as  $e_{sdx}^{(j)}$  and  $e_{sdz}^{(j)}$ ) for which the corresponding displacement discrepancy index (denoted as  $I_{D1}$ ) is a minimum, where  $I_{D1}$  is computed as:

$$\begin{aligned} I_{D1}^{(j)} &= \frac{1}{3NDP} \sum_i^{NDP} \left[ \left( u_{x1}^{*b}(t_i) - u_{xr}^b(t_i) \right)^2 \right. \\ &\quad + \left( u_{z1}^{*b}(t_i) - u_{zr}^b(t_i) \right)^2 \\ &\quad \left. + \left( au_{\theta 1}^{*b}(t_i) - au_{\theta r}^b(t_i) \right)^2 \right] \end{aligned} \quad (4)$$

$NDP$  represents the number of data points in the response history, and  $a$  denotes the width of the

building plan. Consequently,  $\mathbf{C}_{sd}^*$  has been updated by replacing  $e_{sdx}^{(j-1)}$  and  $e_{sdz}^{(j-1)}$  with  $e_{sdx}^{(j)}$  and  $e_{sdz}^{(j)}$ , respectively.

Step 9: With the updated  $\mathbf{C}_{sd}^*$  obtained from the previous step and varying  $C_{xx}$  from  $0.5C_{xx}^{(j-1)}$  to  $1.5C_{xx}^{(j-1)}$ ,  $C_{zz}$  from  $0.5C_{zz}^{(j-1)}$  to  $1.5C_{zz}^{(j-1)}$ , and  $C_{\theta\theta}$  from  $0.5C_{\theta\theta}^{(j-1)}$  to  $1.5C_{\theta\theta}^{(j-1)}$ , the three directional displacement histories (denoted as  $u_{x2}^b(t)$ ,  $u_{z2}^b(t)$ , and  $u_{\theta2}^b(t)$ ) of the non-proportionally damped EOSB subjected to a bi-directional impulse are computed. The bi-directional impulse consists of x-directional and z-directional unit impulses multiplied by  $\eta_x$  and  $\eta_z$ , respectively. The suggested ranges for varying  $C_{xx}$ ,  $C_{zz}$ , and  $C_{\theta\theta}$  (i.e., from  $0.5C_{xx}^{(j-1)}$  to  $1.5C_{xx}^{(j-1)}$ , from  $0.5C_{zz}^{(j-1)}$  to  $1.5C_{zz}^{(j-1)}$ , and from  $0.5C_{\theta\theta}^{(j-1)}$  to  $1.5C_{\theta\theta}^{(j-1)}$ ) and their increments can be adjusted depending on a compromise between the required precision and the affordable computation time. Of all the considered combinations of  $C_{xx}$ ,  $C_{zz}$ , and  $C_{\theta\theta}$ , the values of  $C_{xx}$ ,  $C_{zz}$ , and  $C_{\theta\theta}$  are updated as the trio (denoted as  $C_{xx}^{(j)}$ ,  $C_{zz}^{(j)}$ , and  $C_{\theta\theta}^{(j)}$ ) for which the corresponding displacement discrepancy index (denoted as  $I_{D2}$ ) is a minimum, where  $I_{D2}$  is computed as:

$$I_{D2}^{(j)} = \frac{1}{3NDP} \sum_i^{NDP} \left[ \left( u_{x2}^b(t_i) - u_{xr}^b(t_i) \right)^2 + \left( u_{z2}^b(t_i) - u_{zr}^b(t_i) \right)^2 + \left( au_{\theta2}^b(t_i) - au_{\theta r}^b(t_i) \right)^2 \right] \quad (5)$$

As a result,  $\mathbf{C}_{sd}^*$  has been updated by replacing  $C_{xx}^{(j-1)}$ ,  $C_{zz}^{(j-1)}$ , and  $C_{\theta\theta}^{(j-1)}$  with  $C_{xx}^{(j)}$ ,  $C_{zz}^{(j)}$ , and  $C_{\theta\theta}^{(j)}$ , respectively.

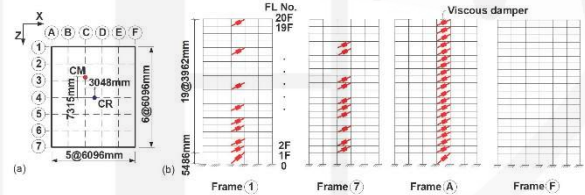
Step 10: If the discrepancy between the values of  $I_{D2}^{(j)}$  and  $I_{D2}^{(j-1)}$  is not acceptable, the iteration continues with Step 7. Otherwise, the eventual  $\mathbf{C}_{sd}^*$  is obtained and the iteration is stopped. In other words, the non-proportionally damped EOSB has been constructed.

## Numerical Validation

The example building is a 20-story steel moment-resisting frame building designated as ASY20. ASY20 is a variation of the proportionally damped symmetrical prototype building used in the steel research project to represent a typical office building

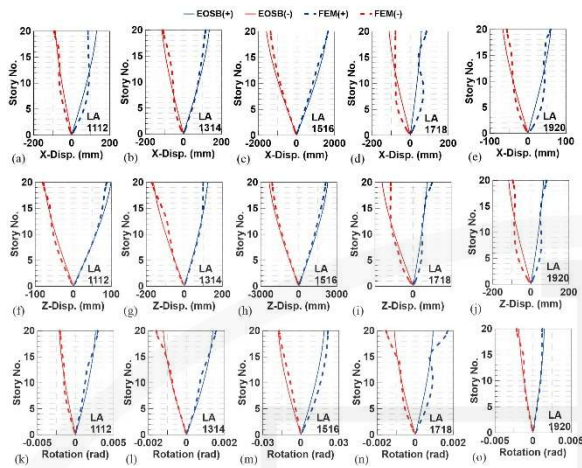
situated on stiff soil in Los Angeles (FEMA-355C, 2000). In addition to adding linear viscous dampers, the alteration included the intended movement of the center of mass (CM) of each floor away from the center of rigidity (CR) to achieve eccentricity ratios of 20% and 10% in the x- and z-directions (Fig. 2a). Each floor is simulated with a rigid diaphragm and a lumped mass located at the center of mass. The details of the building are shown in the related report (FEMA-355C, 2000). Rayleigh damping with 2% damping ratios for the first two vibration modes of the first triplet of vibration modes of ASY20 was adopted. The Newmark- $\beta$  numerical integration method provided by the structural analysis program PISA3D (Lin et al., 2009) was employed in the dynamic analyses of the numerical building model. The added linear viscous dampers, all with a damping coefficient of  $20 \text{ kN} \times \text{s/mm}$ , were arbitrarily distributed on the four sides of the perimeter moment-resisting frame (Fig. 2b).

The ensemble of 20 ground motion records (denoted as LA01–LA20), originally used in the steel research project, was also employed in this numerical validation. This ensemble, scaled to a seismic hazard level corresponding to a 475-year return period, consists of ten pairs of bi-directional ground motion records. The first and second components of each pair (e.g., (LA01, LA02), (LA03, LA04)) were applied in the x- and z-directions, respectively. For example, LA03 and LA04 were applied in the x- and z-directions, respectively. Additionally, the pair of ground motions LA03 and LA04 is denoted as LA0304.



**Fig. 2.** (a) Typical floor plan and (b) elevations of ASY20.

Fig. 3 illustrates the peak story displacements in three directions along the height of ASY20, subjected to the five pairs of ground motions, LA1112 to LA1920. As shown in Fig. 3, the effectiveness of the proposed approach is confirmed.



**Fig. 3.** The x-directional story displacements of ASY20 compared with those estimated from the EOSB under (a) LA1112, (b) LA1314, (c) LA1516, (d) LA1718, and (e) LA1920 ground motions. Panels (f) to (j) correspond to (a) to (e) for the z-directional story displacements. Panels (k) to (o) correspond to (a) to (e) for the y-rotational displacements.

## Conclusions

The present study proposes a non-proportionally damped 3DOF oscillator as a substitute for a non-proportionally damped two-way asymmetric-plan building in modal space. The 3DOF oscillator effectively characterizes the supplementary damping properties of the fundamental mode in each of the three directions. The displacement histories of a non-proportionally damped building (ASY20), subjected to ten pairs of bi-directional ground motions, were satisfactorily estimated using the proposed 3DOF oscillator.

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# Seismic Design of Varying Section Truss-Confined Buckling-Restrained Braces

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## Abstract

The key feature of the varying-section truss-confined buckling-restrained brace (TC-BRB) is an additional truss confining system attached outside the central steel casing. The truss system consists of several steel open-web truss frames, which provide the required restraining rigidity and reduce both the size of the central casing section and the amount of infilled mortar. The overall self-weight is reduced compared to a conventional buckling-restrained brace (BRB), particularly in long-span and high force-capacity BRB applications. Analytical results show that the chord primarily contributes to the flexural rigidity of the restraining system, while the post mainly contributes to the overall shear rigidity. The ultimate compressive strength of a TC-BRB is subsequently evaluated, incorporating the effects of chord residual stress, length differences between the restrainer and the entire brace, and initial in-plane flexural deformation, based on available experimental data.

**Keywords:** buckling-restrained brace, truss-confined restrainer, effective rigidity, buckling load

## Introduction

The key feature of the varying-section truss-confined buckling-restrained brace (TC-BRB) is an additional truss confining system attached outside the central steel casing. The truss system consists of several steel open-web truss frames, providing the required restraining rigidity and reducing the size of the central casing section and the amount of infilled mortar. The hysteretic behavior of TC-BRBs, including truss member response, buckling resistance, and cumulative deformation capacity, was investigated through cyclic loading tests on physical specimens with different yield capacities, truss frame shapes, and sizes. Results confirmed that the proposed TC-BRB can be designed and fabricated to achieve excellent seismic performance (Guo et al., 2017a, 2017b; Chen et al., 2021, 2022; Wu et al., 2023, 2025). In this study, the effective flexural and shear rigidities of the entire truss confining system, contributed by individual truss members, are first quantified. Based on classical mechanics theory, the elastic buckling resistance of the truss-confined restrainer is computed from a stability analysis that considers the effect of shear deformation. The ultimate compressive strength of a TC-BRB is

predicted by accounting for the effects of residual stress in the chord members, the effective length ratio between the restrainer and the entire BRB, and the initial in-plane flexural deformation.

## Key Experimental Results

Details of the six TC-BRB specimens, the test setup, loading procedure, hysteresis responses, and failure modes were thoroughly described in a previous experimental study (Chen et al., 2021, 2022; Wu et al., 2023, 2025). As shown in Fig. 1, specimens 2VT and 3VT were equipped with a varying-section truss confining system. The design dimensions of their steel cores and truss-confined restrainers are provided in Tables 1 and 2. Both specimens had a nominal tensile yield capacity  $P_{ym}$  of 853 kN and were fabricated from CNS SN490B steel plates with a nominal yield stress of 325 MPa. The expected maximum compressive strength  $P_{max}$  was 1730 kN for both specimens, considering material over-strength, strain hardening, and compression strength adjustment factors of 1.2, 1.3, and 1.3, respectively. The axial deformation–force relationships and failure modes of the two

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specimens are shown in Fig. 2 and Fig. 3, respectively. Their cyclic peak responses, including maximum tensile strength ( $T_{max}$ ) and compressive strength ( $C_{max}$ ),

core strain ( $\epsilon_c$ ), and cumulative inelastic deformation (CID) capacity, are summarized in Table 3.

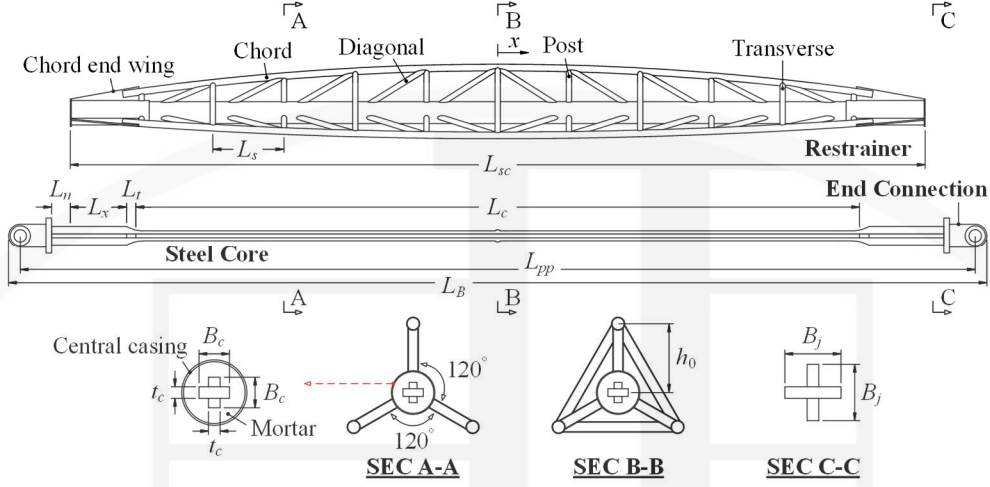


Fig. 1. Details of the varying-section TC-BRB.

## Buckling load

Under the same deflection, the chord and diagonal each provide axial rigidity to jointly resist the bending moment. The equivalent flexural rigidity of a single truss segment ( $EI_{seg}$ ) is calculated by summing the flexural rigidities of the chord ( $EI_{ch}$ ) and diagonal ( $EI_d$ ), which act in parallel:

$$EI_{seg} = EI_{ch} + EI_d \quad (1)$$

The equivalent shear rigidity of the single truss segment ( $GA_{seg}$ ) can be determined by considering the shear rigidities of the chord ( $GA_{ch}$ ), diagonal ( $GA_d$ ), and post ( $GA_{po}$ ), which operate in series:

$$GA_{seg} = \frac{1}{-1/GA_{ch} + 1/GA_d + 1/GA_{po}} \quad (2)$$

where  $E$  and  $G$  are the elastic modulus and shear modulus of steel, respectively. The effective flexural ( $EI_{ts}$ ) and shear ( $GA_{ts}$ ) rigidities of the truss confining system, which consists of three truss frames of identical size and equal inclined angles between each other, in the considered buckling direction, can be determined as follows:

$$EI_{ts} = \frac{3}{2}EI_{seg}, \quad GA_{ts} = \frac{3}{2}GA_{seg} \quad (3)$$

A TC-BRB restrainer consists of a truss confining system and a central casing. Accordingly, the effective rigidity of the complete restraining system is the sum of the individual rigidities. The equivalent flexural ( $\Sigma EI$ ) and shear ( $\Sigma GA$ ) rigidities of the restrainer at each truss segment with an interval  $L_s$  are expressed as follows:

$$\Sigma EI = EI_{ts} + EI_{sc} + E_m I_m \quad (4)$$

$$\Sigma GA = GA_{ts} + \lambda GA_{sc} \quad (5)$$

where  $A_{sc}$  and  $I_{sc}$  denote the cross-sectional area and moment of inertia of the central casing, respectively;  $\lambda = 0.05$  represents the reduction factor of the casing shear area.  $E_m$  and  $I_m$  denote the elastic modulus and sectional moment of inertia of the mortar filling in the central casing, respectively.

The global stability analysis was performed using the energy method, and the corresponding total potential energy was calculated from a buckling shape function  $\phi(x)$ . The elastic buckling load  $P_{cr}^R$  of a TC-BRB restrainer, considering the shear reduction effect, is given as follows [Chen *et al.*, 2021]:

$$P_{cr}^R = \frac{\int_0^{L_{sc}} \phi' \phi' dx}{\int_0^{L_{sc}} \frac{\phi \phi}{\Sigma EI} dx + \int_0^{L_{sc}} \frac{\phi' \phi'}{\Sigma GA} dx} \quad (6)$$

The shape function  $\phi(x) = \sin(\pi x/L_{sc})$  is used for the calculation. When the restrainer has homogeneous section properties, with constant effective flexural ( $EI_{eff}$ ) and shear ( $GA_{eff}$ ) rigidities along its longitudinal direction, the typical shear-affected buckling resistance can be evaluated as follows:

$$P_{cr,s}^R = \frac{P_e}{1 + P_e/GA_{eff}}, \quad P_e = \frac{\pi^2 EI_{eff}}{L_{sc}^2} \quad (7)$$

where  $L_{sc}$  is the restrainer length. The chord of the varying-section truss confining system is cold-rolled to the required curve through plastic bending. This mechanical process induces residual stress within the chord (Fig. 4), which reduces its compressive resistance. Consequently, the buckling resistance of the varying-section restraining system decreases to  $P_{cr}^{R,rs}$  because of residual stress, as follows:

$$P_{cr}^{R,rs} = n_{rs} P_{cr}^R \quad (8)$$

where  $n_{rs}$  is the residual stress reduction factor.

Analytical results (Wu *et al.*, 2025) indicate that an  $n_{rs}$  factor of 0.91 was obtained for specimen 2VT (Table 4). Assuming continuous deformation between these parts under overall flexure, the global buckling strength of the TC-BRB can be calculated using the effective length difference:

$$P_{cr}^B = n_l P_{cr}^{R,rs}, \quad n_l = \frac{L_{sc}^2}{L_{pp}^2} \quad (9)$$

where the pin-to-pin distance  $L_{pp}$  denotes the effective TC-BRB length; and  $n_l$  denotes the effective length

reduction factor. The decrease in buckling resistance resulting from the initial in-plane flexural deformation of the TC-BRB can be explained by the relationship between axial force and in-plane deformation, as recommended by the BRB stability concept:

$$P_{cr}^B = n_l P_{cr}^{R,rs}, \quad n_{ip} = \frac{y_{ip} - a_{ip}}{y_{ip}} \quad (10)$$

where  $n_{ip}$  is the in-plane deformation reduction factor. Based on the test data recorded (Wu *et al.*, 2023), the  $n_{ip}$  value for the buckled specimen 2VT is 0.78.

**Table 1.** Design dimensions of the steel cores.

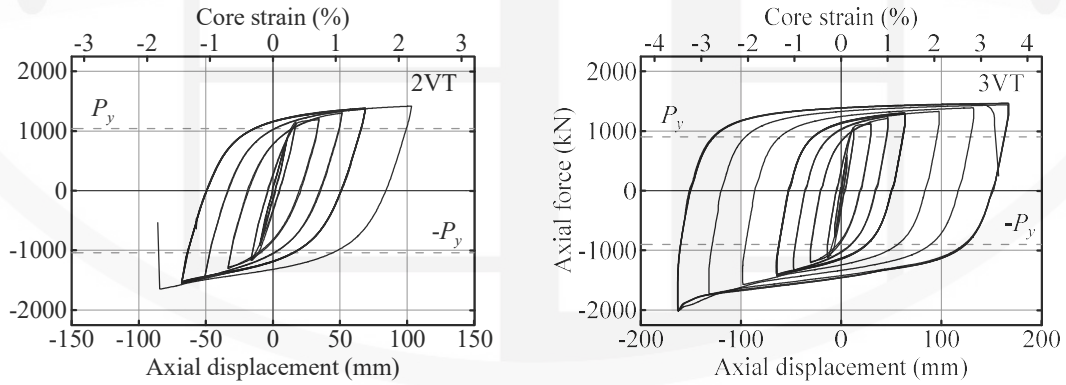
| Specimen | $t_c$<br>(mm) | $B_c$<br>(mm) | $B_j$<br>(mm) | $L_c$<br>(mm) | $L_t$<br>(mm) | $L_x$<br>(mm) | $L_n$<br>(mm) | $L_{pp}$<br>(mm) | $L_B$<br>(mm) |
|----------|---------------|---------------|---------------|---------------|---------------|---------------|---------------|------------------|---------------|
| 2VT      | 25            | 65            | 120           | 4638          | 60            | 360           | 120           | 6120             | 6270          |
| 3VT      |               |               |               |               |               |               |               |                  |               |

**Table 2.** Design dimensions of the restrainers.

| Specimen | Central casing<br>(mm) | Chord<br>(mm) | Post<br>(mm) | Diagonal & transverse<br>(mm) | $L_{sc}$<br>(mm) | $h_0$<br>(mm) |
|----------|------------------------|---------------|--------------|-------------------------------|------------------|---------------|
| 2VT      | 139.6×4                | 48.4×3.2      | 48.4×3.2     | 33.8×2.8                      | 5478             | 290           |
| 3VT      |                        | 60.5×3        | 48.4×3       | 33.8×2.8                      |                  |               |

**Table 3.** Key experimental results.

| Specimen | Failure mode | $T_{max}$<br>(kN) | $C_{max}$<br>(kN) | Max. $\varepsilon_c$ (%)<br>Positive Negative | Max. $P_T/P_v$ | Max. $P_C/P_T$ | CID |
|----------|--------------|-------------------|-------------------|-----------------------------------------------|----------------|----------------|-----|
| 2VT      | Buckling     | 1426              | 1647              | 2.12 1.69                                     | 1.37           | 1.14           | 208 |
| 3VT      | Fracture     | 1465              | 2015              | 3.61 3.52                                     | 1.63           | 1.40           | 605 |



**Fig. 2.** Experimental responses of the specimens.



**Fig. 3.** Failure of specimen 2VT.

The calculated ultimate compressive strengths for specimens 2VT and 3VT are shown in Table 4. Because specimen 3VT did not experience buckling failure, the in-plane and out-of-plane deformations associated with the buckling load were unavailable

for calculating the residual stress reduction and in-plane deformation factors. However, the experimental buckling load of specimen 2VT can be reasonably predicted from the ultimate compressive strength  $P_n$  using the three reduction factors.

Table 4. Buckling resistance assessments of the specimens.

| Specimen | $P_{cr}^R$ (kN) | $n_{rs}$ | $n_l$ | $n_{ip}$ | $P_n$ (kN) | $P_n / C_{max}$ |
|----------|-----------------|----------|-------|----------|------------|-----------------|
| 2VT      | 3159            | 0.91     | 0.8   | 0.78     | 1794       | 1.09            |
| 3VT      | 3556            | -        | -     | -        | -          | -               |

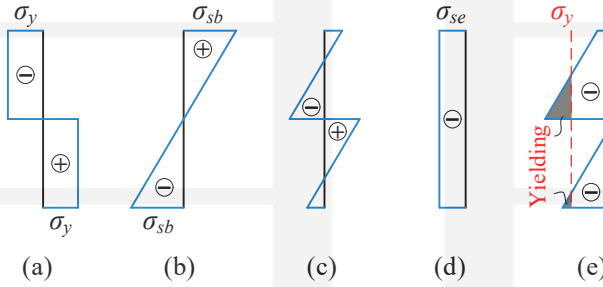


Fig. 4. Illustration of the effects on the curved chord of (a) plastic bending, (b) elastic spring-back, (c) residual stress, (d) secondary axial force, and (e) partially yielded sections.

## Conclusions

This study estimated the elastic buckling resistance of the restraining system in a varying-section TC-BRB, considering the shear reduction effect. The ultimate compressive strength of a complete TC-BRB was analyzed, incorporating the strength-reduction effects of residual stress in the curved chord, length effects, and initial in-plane flexural deformation. Experimental results confirm that the ultimate compressive strength of a TC-BRB can be predicted from the restrainer's buckling load when strength-reduction effects are considered. A strength reduction factor of 0.9 is recommended in accordance with LRFD design procedures. Based on the results of this research and engineering judgment, a resistance factor of 0.9 appears reasonable. However, this value should be further investigated and supported by additional experimental results and detailed reliability analysis.

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# Field Investigation on Seismic Performance of the Retrofitted RC Buildings of Tainan Municipal Nansi Junior High School

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## Abstract

At 0:17:27 a.m. on January 21, 2025, Taiwan time, an earthquake with a magnitude of ML 6.4 on the Richter scale (referred to as the 0121 Chiayi Dapu Earthquake) occurred in Nansi District, Tainan City, approximately 44.9 km northeast of Tainan City Government, near Dapu Township, Chiayi County. The epicenter was located at 23.22° north latitude and 120.55° east longitude, with a depth of 15.8 kilometers. The maximum seismic intensity was 6-, recorded in Dapu Township, Chiayi County, while Nansi District, Tainan City experienced intensity 5+. The earthquake caused structural damage to many buildings in Chiayi County and Tainan City near the epicenter. The National Center for Research on Earthquake Engineering dispatched researchers to the affected area to assess structural damage, focusing on the main distribution areas of red- and yellow-tagged buildings. Among the damaged structures inspected, Tainan Municipal Nansi Junior High School was selected for case study. Notably, the front building sustained severe damage, despite having been retrofitted in 2010 with added shear walls and enlarged columns and beams as part of a nationwide seismic retrofit project. Before reinforcement, the front building was constructed with a cantilever corridor design, which accounted for two-thirds of all school buildings. Therefore, this paper examines the seismic performance of the retrofitted reinforced concrete building.

**Keywords:** Field inspection, retrofitted buildings, damaged school buildings

## Introduction

On January 21, 2025, Taiwan time, an earthquake with a magnitude of ML 6.4 on the Richter scale, referred to as the 0121 Chiayi Dapu Earthquake (also known as Tainan Nansi) occurred in Nansi District, Tainan City (44.9 km northeast of Tainan City Government) near Dapu Township, Chiayi County [CWA, 2025], with a maximum seismic intensity of 6- in Dapu Township, Chiayi County, and a seismic intensity of 5+ in Nansi District, Tainan City. The earthquake caused damage to many buildings in Chiayi County and Tainan City near the epicenter, so the National Center for Research on Earthquake Engineering, National Institutes of Applied Research (hereinafter referred to as “NCREE”) dispatched personnel to the disaster area to investigate the

damage to building structures based on the main distribution areas of red- and yellow-tagged buildings.

The main buildings of Tainan Municipal Nansi Junior High School (NJHS) are the front building, the back building, and the activity center (Fig. 1). The front and back buildings were severely damaged in the 0121 Chiayi Dapu (Nansi, Tainan) earthquake. Notably, the front building sustained the most severe damage (Fig. 2 and Fig.3), despite being retrofitted in 2010 with additional shear walls and enlarged columns and beams as part of a nationwide seismic upgrade project [Hwang et al., 2022]. Before this reinforcement, the front building was a reinforced concrete structure with cantilevered corridors, which has a design used in about two-thirds of all school buildings.

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Fig. 1. Schematic diagram of the locations of the main buildings at Nansi Junior High School

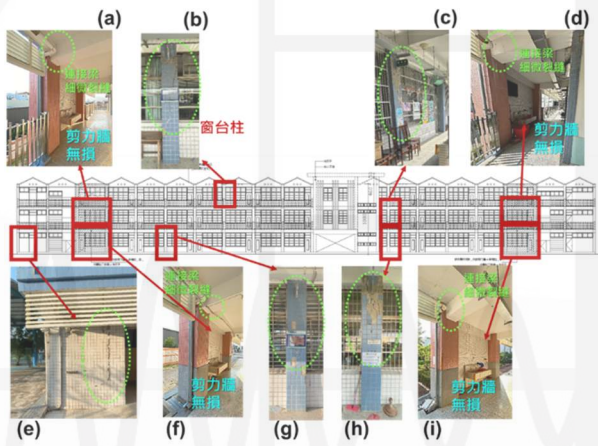


Fig. 2. Earthquake damage to the front corridor of Nansi Junior High School (front, corridor)

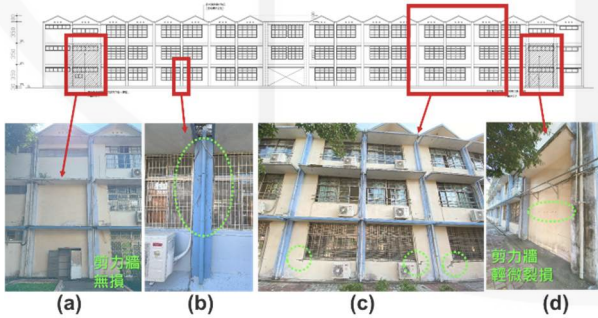


Fig. 3. The back façade of the NJHS front building shows minor damage

### Observed Strong Ground Motions

NJHS is approximately 8 kilometers from the epicenter of the 0121 Chiayi Dapu earthquake, and the nearest seismic station is Nansi (CHY079), located about 4 km away. Figure 4 presents the three-component surface acceleration record from the Nansi station, where the east-west, north-south, and vertical directions are denoted as EW, NS, and Z,

respectively. As shown in Fig. 4, a comparison of the pseudo-acceleration response spectra ( $PS_a$ ) derived from the observed ground motions at the station with the current and previous design spectra for the two horizontal components at Nansi station reveals that the  $PS_a$  values for both horizontal earthquakes at the Nansi Station significantly exceed the design basis earthquake (DBE) spectra in the short-period range, and surpass the maximum considered earthquake (MCE) spectra in several period ranges. Additionally, the peak ground acceleration recorded at the Nansi station exceeds the requirement for the 475-year return period. These results indicate that this seismic event imposes a significant seismic demand on the three-story (low-rise) school building at NJHS.

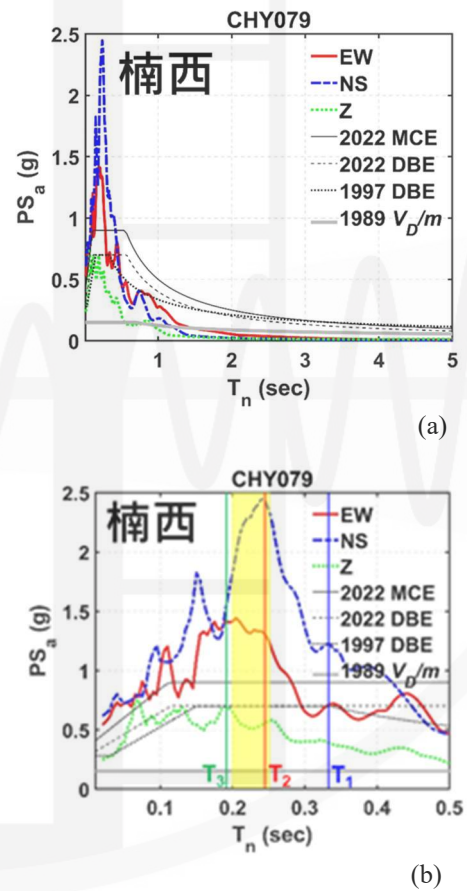


Fig. 4. Pseudo-acceleration response spectra of ground accelerations observed at Nansi Station: (a) natural period  $T_n$ : 0 - 5 s, (b)  $T_n$ : 0 - 0.5 s

### Structural Analysis

In this study, the PISA3D structural analysis software [Lin et al., 2009] developed by NCREE was used to create a numerical model of the front building (Fig. 5), and appropriate analytical methods and material models were applied to simulate the behavior of the school buildings. The results of the modal analysis are shown in Fig. 6. Based on the first three

natural periods (0.323 s, 0.246 s, and 0.199 s) and the response spectra from the Nansi station (Fig. 4), it is evident that the front building was subjected to significant seismic forces during the earthquake. Compared to the back façade (Fig. 3), the frame on the front corridor side experienced greater structural damage (Fig. 2). The newly constructed shear walls on the corridor side did not show damage, although minor cracks were observed at the connections between the girders and the new shear walls. Therefore, further investigation into the longitudinal transmission of seismic forces in this structure is necessary. Structural analyses with different assumptions for rigid and flexible diaphragms were performed (Fig. 7).

Field investigation and structural analysis revealed that the seismic damage to the front building of NJHS can be attributed to (1) high seismic force demands; (2) insufficient strength and ductility of certain structural members, such as windowsill columns; and (3) an idealized assumption of a rigid diaphragm. A similar failure mechanism was observed in the west wing of the back building. The field investigation of the retrofitted NJHS school building and the numerical simulation showed the potential failure mode of cantilevered corridor school buildings retrofitted using added shear walls. Although some structural members in both the front and back buildings at NJHS were significantly damaged, the two retrofitted school buildings effectively avoided collapse. This demonstrates the necessity and benefits of seismic evaluation and retrofitting projects for school buildings in Taiwan.

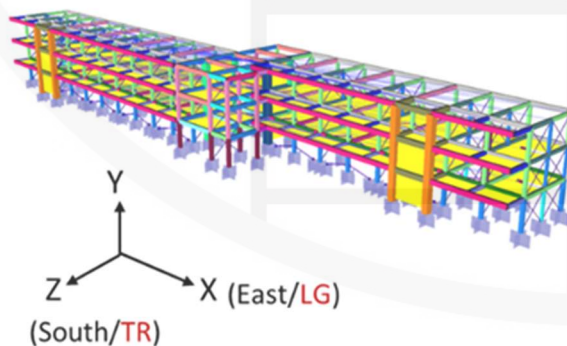


Fig. 5. PISA3D model

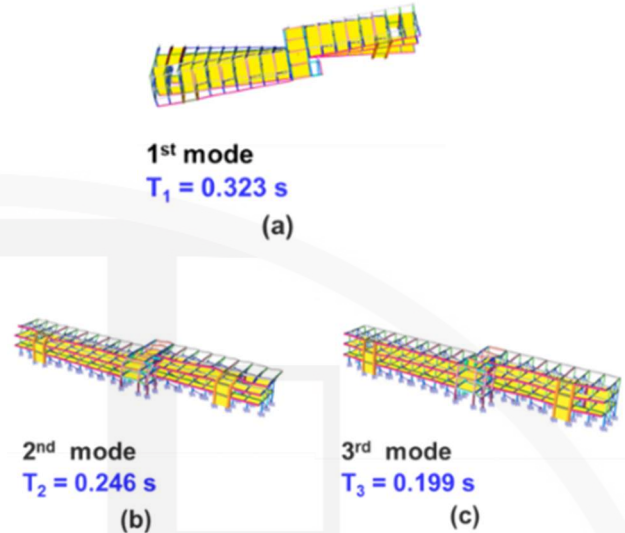


Fig. 6. Modal analysis using PISA3D

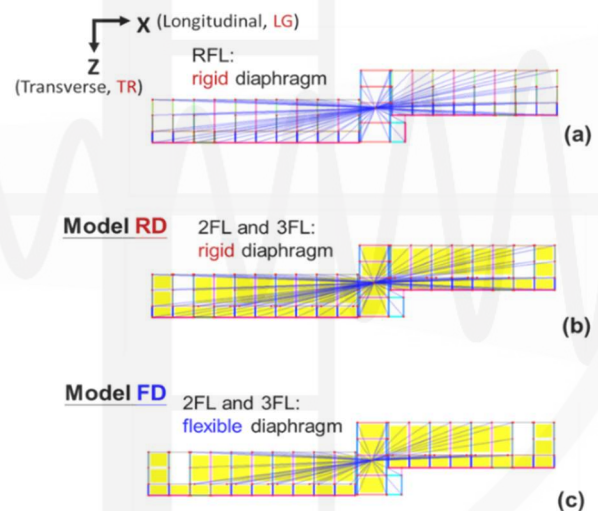


Fig. 7. Simulation of rigid and flexible diaphragms

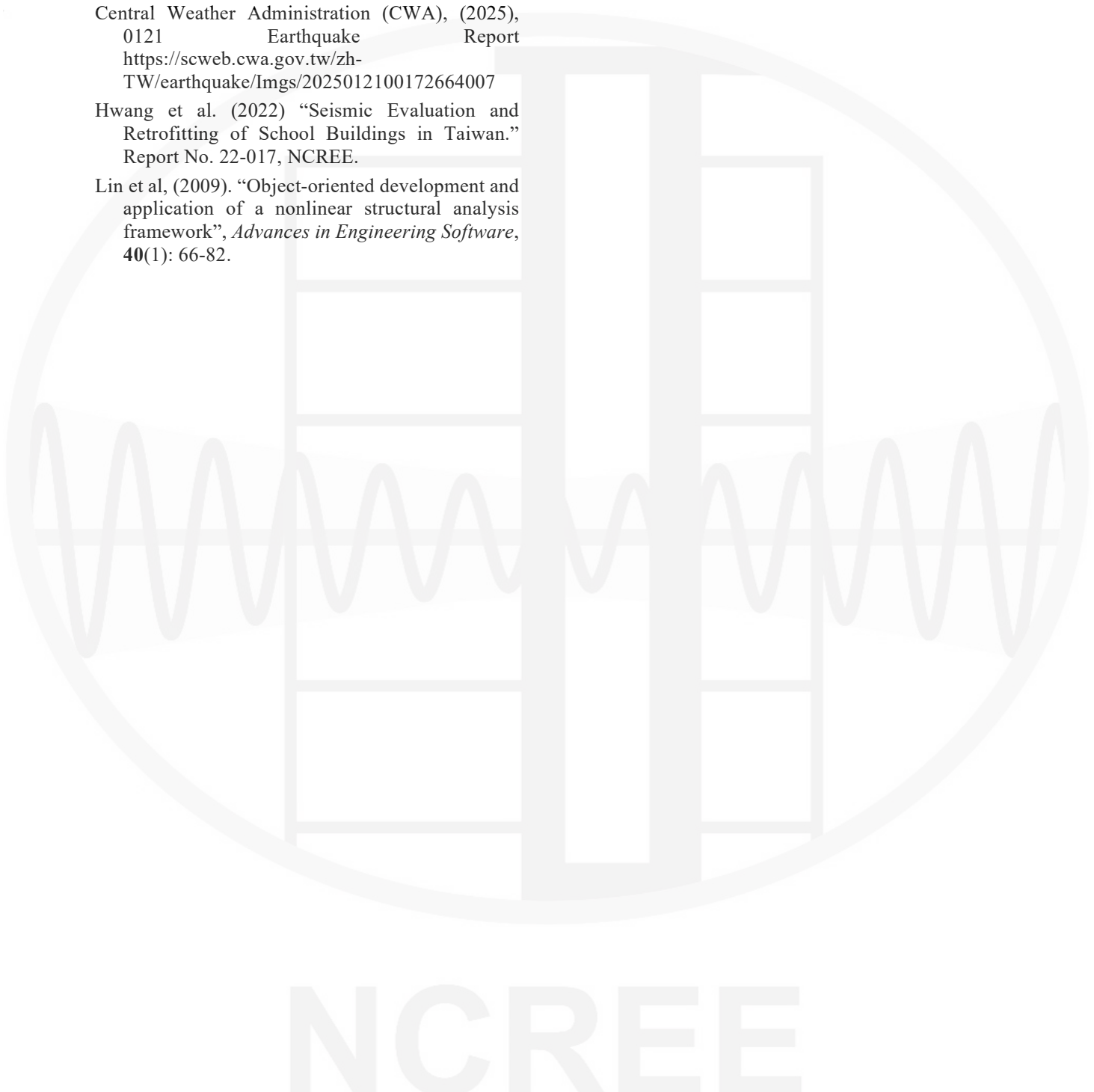
## Conclusions

The field investigation on the seismic performance of the retrofitted reinforced concrete buildings of NJHS demonstrates that when using the retrofit method of adding shear walls for seismic reinforcement, engineers must carefully confirm the rationality of the rigid floor assumption and examine the force transmission mechanism between the floor slab and the shear wall. This diligence is essential to prevent unexpected damage to the connecting beams and to avoid overestimating the horizontal seismic force that can be allocated by the new shear wall. Although the local components of the front and back buildings of NJHS were significantly damaged, these two reinforced school buildings avoided collapse,

thereby proving the necessity and positive impact of NCREE's assistance in the government's nationwide school building retrofitting project.

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# NCREE



# A Generalized Framework for Structural Analysis Model Conversion: Case Study on ETABS to PISA3D

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## Abstract

Cross-platform transfer of structural models is a major challenge in structural engineering due to the coexistence of multiple analysis software packages with differing modeling assumptions, data structures, and computational frameworks. As a result, transferring structural models between platforms is increasingly important in both engineering practice and research. This study presents the development of an automated structural model conversion system, the E2P Converter (ETABS to PISA3D), which integrates model conversion, analysis, and validation within a unified framework. A Data Hub for Structures (DHS) serves as the core data hub, reorganizing essential model information into a consistent, software-independent format. This method enables systematic mapping between different analysis platforms while preserving key structural characteristics. The system automatically converts ETABS models to PISA3D format, and conversion accuracy is evaluated by comparing modal periods, mode shapes, and pushover responses. The results show good agreement between the two platforms, indicating that the proposed methodology offers an effective solution for cross-platform structural model conversion and has strong potential for extension to other structural analysis software.

**Keywords:** structural model conversion, automated model conversion, Data Hub for Structures, ETABS, PISA3D

## Introduction

In structural engineering, various analysis software are used for different engineering tasks due to differences in modeling methods, analytical features, and computational performance. For instance, ETABS [1] is commonly used for building design and seismic analysis because of its comprehensive and user-friendly interface. In contrast, PISA3D [2], developed by the National Center for Research on Earthquake Engineering, National Institutes of Applied Research (hereinafter referred to as “NCREE”), demonstrates strong potential in nonlinear dynamic analysis and large-scale repeated computations, especially in defining nonlinear material behavior. Consequently, engineers and researchers often need to transfer structural models between platforms when changing tools to

meet specific analytical needs.

However, cross-platform structural model conversion remains challenging due to fundamental differences among software in data structures, unit systems, modeling formats, element definitions, and nonlinear behavior representations. Current methods still rely heavily on manual reconstruction and step-by-step verification, which are time-consuming and prone to human error, affecting the consistency and reliability of analysis results. To address these issues, this study proposes an automated cross-platform framework for converting structural models, called the E2P system. Its key contribution is the introduction of a Data Hub for Structures (DHS), which organizes essential structural information into a unified, software-independent format. By transforming models at the level of engineering parameters instead of direct file

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mapping, the proposed approach systematically resolves discrepancies between ETABS and PISA3D, and improves consistency, traceability, and verifiability in the conversion process.

## E2P System and Data Hub for Structures (DHS)

The E2P system developed in this study uses a modular architecture, as shown in Fig. 1. The workflow includes data extraction, unit conversion, DHS creation, data processing, PISA3D input file generation, analysis execution, and result comparison. The system automatically reads model data exported from ETABS and integrates key structural information, such as story definitions, nodal coordinates, element connectivity, material properties, section characteristics, mass distribution, and boundary conditions. After data consolidation and standardization, the corresponding PISA3D input files are generated, followed by automated analysis and validation.

A key feature of the proposed system is the DHS, which serves as the central data structure for model conversion. Rather than mapping ETABS table fields directly to PISA3D input formats, the DHS reorganizes the original model data into a unified, software-independent representation. This model includes geometric information, material properties, element definitions, mass distribution, boundary conditions, and nonlinear parameters, providing a consistent foundation for further model transformation and reconstruction.

With the adoption of the DHS, the conversion process advances from basic file translation to comprehensive mapping and reorganization of key engineering parameters. Discrepancies between ETABS and PISA3D, such as element behavior, material models, unit definitions, and modeling assumptions, are systematically resolved within a unified framework [3]. This approach enhances the consistency, traceability, and verifiability of conversion results while minimizing errors from manual model reconstruction.

Furthermore, using an intermediate data model improves the system's extensibility and maintainability. Additional analysis platforms, new element types, or advanced nonlinear material models can be integrated by creating compatible data interfaces and mapping rules, without altering the core conversion logic. From a design perspective, this framework enables model conversion between ETABS and PISA3D and offers a scalable foundation for future cross-platform structural analysis applications.

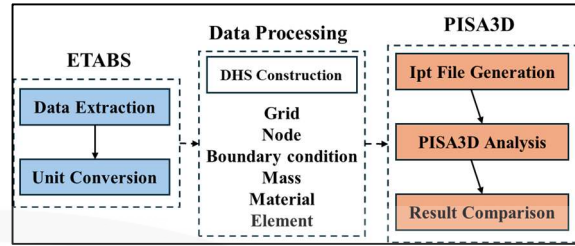


Fig. 1. Workflow of the proposed E2P system.

## Automated Conversion Workflow

ETABS offers a built-in “Display–Show Tables” function for exporting model data; however, manual selection and extraction of tables are inefficient and error-prone for large-scale models. To improve efficiency and reliability, the ETABS Application Programming Interface (API) is used as the primary data source. The system automatically launches ETABS, loads the specified model, and retrieves the required data tables through external program control, ensuring consistent and automated data extraction. To maintain reliability under varying conditions, a fallback mechanism allows pre-exported Excel tables to be used as an alternative input when API access is unavailable.

During the model transformation stage, the E2P system resolves key differences between ETABS and PISA3D, such as unit systems, coordinate definitions, degrees of freedom, boundary conditions, element formulations for beams, columns, and braces, rigid end offsets, plastic hinge representations, nonlinear material models, wall modeling strategies, rigid diaphragm assumptions, mass distribution, and loading conditions. Basic geometric transformations and unit conversions are managed through direct mapping, but representing nonlinear behavior and mechanical properties requires more complex parameter interpretation and is the main source of discrepancies in analysis results.

The conversion workflow involves several sequential steps: extracting ETABS model data, parsing and converting units, constructing the DHS, reorganizing data according to PISA3D modeling logic, generating PISA3D input files, and automatically executing structural analysis. Results from ETABS and PISA3D are then compared based on modal properties and structural responses to assess the consistency of the converted model.

Unlike conventional tools that focus only on file-based model translation, the proposed framework integrates a validation mechanism into the conversion process. By comparing modal periods, mode shapes, and other response characteristics, the system evaluates whether the converted model retains the essential engineering behavior of the original structure. This approach allows the E2P system to serve as both a model

converter and a verification bridge between structural analysis platforms.

## Preliminary Results

The current implementation of the E2P system supports the conversion of common structural components, including beams, columns, slabs, and braces. Several building models have been tested to evaluate the performance of the proposed framework. The results indicate that the generated PISA3D models can be successfully analyzed without numerical instability, and their modal characteristics exhibit trends consistent with those obtained from ETABS. These findings demonstrate the basic correctness and robustness of the proposed model conversion process.

A representative case study involving a two-story three-dimensional braced frame is presented, as shown in Fig. 2. The comparison of modal periods and mode shapes (Figs. 3 and 4) indicates that the discrepancies between ETABS and PISA3D results remain within acceptable ranges, suggesting that the essential dynamic characteristics of the structure are well preserved after conversion. In addition, the pushover analysis results shown in Fig. 5 reveal that the relationship between base shear and structural displacement exhibits a consistent overall trend between the two platforms. This observation further confirms that the converted model reasonably captures the nonlinear structural behavior.

From an engineering application perspective, the proposed system offers several practical advantages. First, it greatly reduces the time and effort needed for manual model reconstruction. Second, the automated workflow and DHS improve the consistency of model data across platforms. Third, the built-in comparison mechanism streamlines model validation. These features indicate that the E2P system is an effective tool for cross-platform analysis, model verification, and workflow automation in structural engineering.

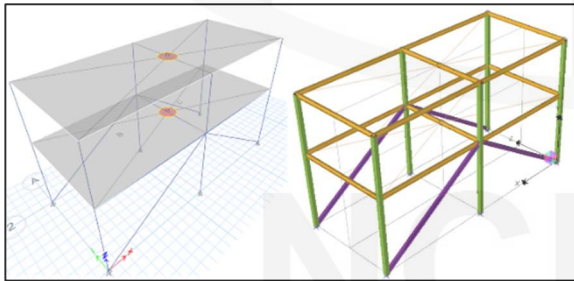


Fig. 2. Model conversion example.

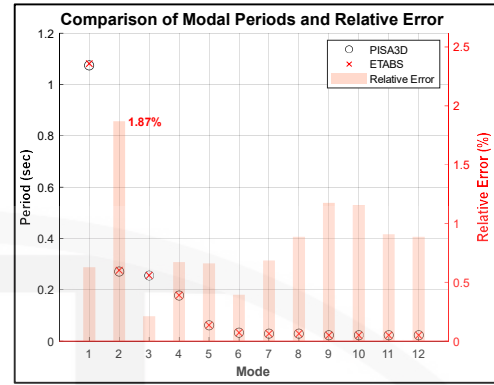


Fig. 3. Comparison of modal periods.

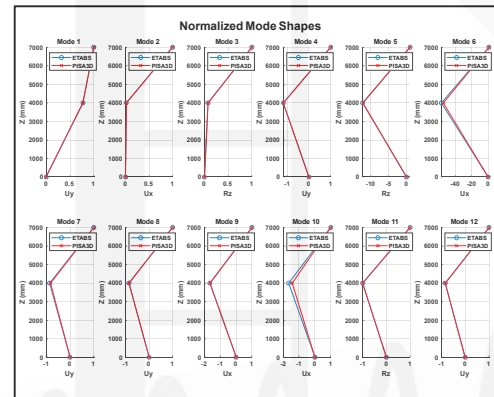


Fig. 4. Mode shape comparison.

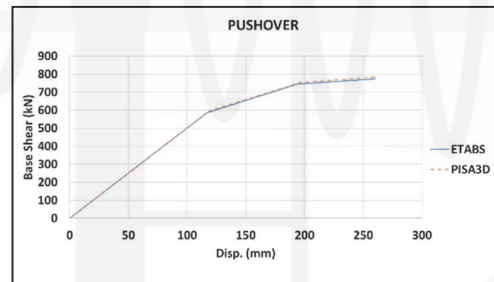


Fig. 5. Comparison of base shear-displacement curves from pushover analysis.

## Conclusions and Future Work

This study introduces an automated framework for converting structural models between ETABS and PISA3D, using a DHS as the central data structure. By reorganizing key structural information into a software-independent format, this approach enables consistent cross-platform model transformation.

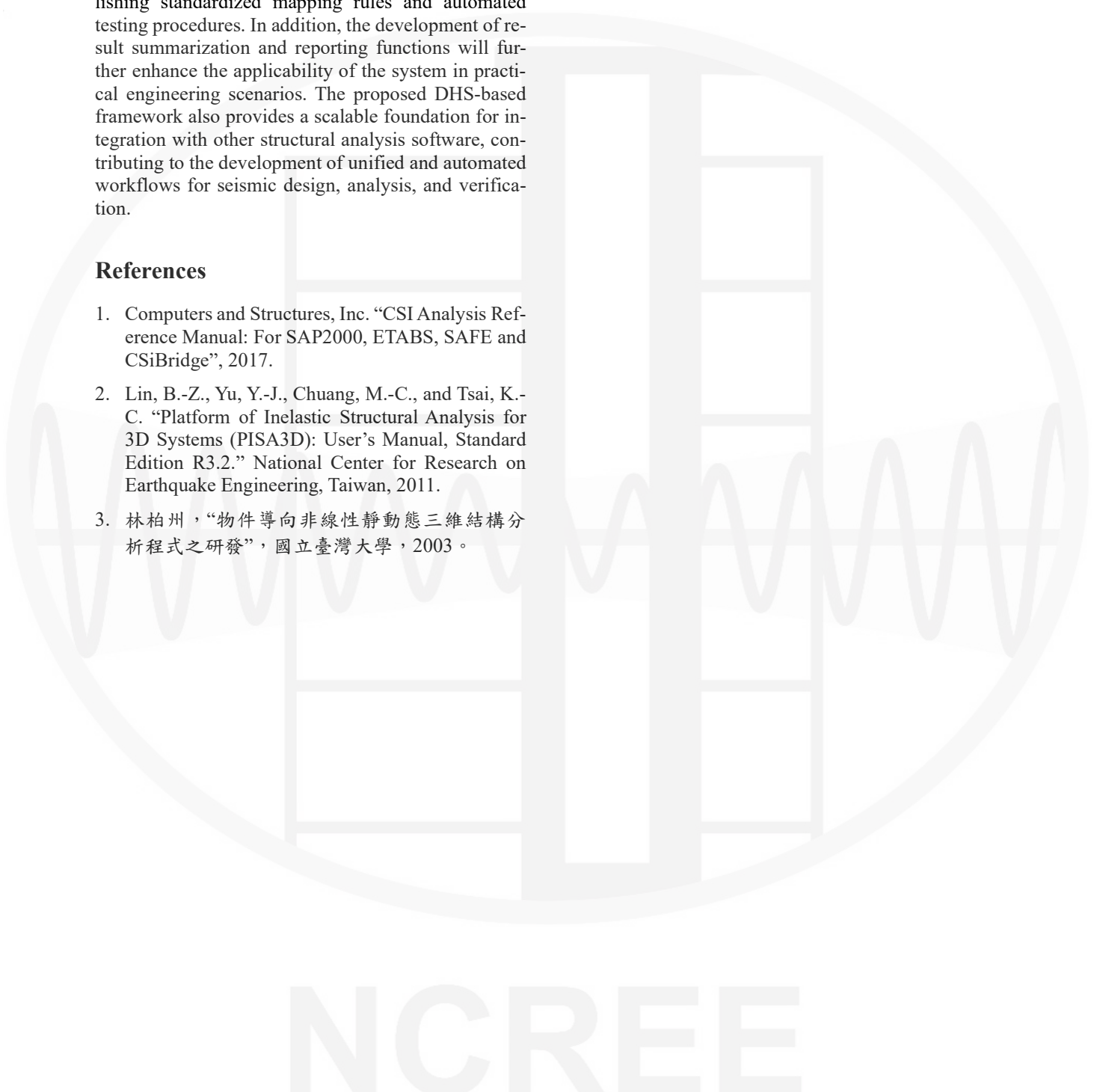
The results show that the proposed framework successfully converts structural models while preserving essential dynamic and nonlinear characteristics. By integrating a modular workflow, DHS-based representation, and automated validation, the conversion process shifts from manual and error-prone to systematic and verifiable. These features enhance efficiency, consistency, and reliability, enabling the system to

serve as both a conversion tool and a verification bridge between structural analysis platforms.

Future work will focus on extending the proposed framework to support a wider range of nonlinear behaviors and structural components, as well as establishing standardized mapping rules and automated testing procedures. In addition, the development of result summarization and reporting functions will further enhance the applicability of the system in practical engineering scenarios. The proposed DHS-based framework also provides a scalable foundation for integration with other structural analysis software, contributing to the development of unified and automated workflows for seismic design, analysis, and verification.

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# NCREE



# Nonlinear Hinge Models for H-shaped Steel Columns in Pushover Analysis

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## Abstract

In steel buildings, in addition to common hollow structural sections (HSS), H-shaped steel columns are also widely used in construction practice. However, when conducting nonlinear static analysis for new and existing buildings in Taiwan, engineers currently rely exclusively on nonlinear hinge parameters recommended by international codes such as ASCE 41-17 and AISC 342-22. This study compares experimental data for H-shaped steel columns with these codes and proposes a nonlinear hinge model that more accurately reflects the actual test results.

**Keywords:** steel frame, moment-resisting frame, H-shaped steel column, nonlinear hinge model, seismic evaluation, nonlinear pushover analysis.

## 1. Introduction

This study collected data from 25 H-shaped steel column specimens. By analyzing the envelope curves derived from experimental hysteresis loops, the nonlinear behavior of each specimen was characterized. The data were organized and used in statistical regression to develop a recommended nonlinear hinge model. This model is defined by three primary parameters:  $a$  (plastic rotation at peak strength),  $b$  (total plastic rotation capacity), and  $\alpha$  (strain-hardening ratio).

After comparing the regression-based model with the experimental envelope curves, the results indicate that the predicted maximum moments for most specimens accurately represent the experimental data, with only approximately 18% exhibiting overestimation. This indicates that the model is highly accurate and avoids significant overprediction. Regarding deformation capacity, the regressed parameters  $a$  and  $b$  closely correspond to the experimental results, confirming strong predictive capability. Additionally, the experimental envelope curves were compared with the

ASCE 41-17 (2017) and AISC 342-22 (2022) standards.

## 2. Methodology

This study collected data from 25 experimental H-shaped steel column specimens from international sources. The corresponding envelope curves were extracted from the resulting hysteresis loops. By combining these results with parameters such as cross-section, axial load ratio, and slenderness ratio, their relationships were analyzed to develop a statistically regressed nonlinear hinge model for H-shaped steel columns.

The study analyzed domestic and international experimental data to develop a nonlinear hinge model for H-shaped steel columns. This model was validated through full-scale tests conducted by the National Center for Research on Earthquake Engineering (NCREE) on a two-story special concentrically braced frame (SCBF) and a full-scale steel moment-resisting frame (MRF). These structural specimens incorpo-

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rated H-shaped steel columns and beams. The proposed nonlinear hinge model was implemented in a nonlinear pushover analysis to evaluate the model's rationality and practical applicability.

### 3. Development of Nonlinear Hinge Models for H-shaped Steel Columns

This study referenced the definitions of nonlinear hinge models from both ASCE 41-17 (2017) and Chung (2023). As shown in Fig. 1, the model provided by ASCE 41-17 does not clearly specify a method for defining residual strength, which differs from the behavior commonly observed in local experimental data. In Taiwanese steel component experiments, the failure point is typically defined as the moment when the component's strength decreases to 80% of its maximum strength. Therefore, this study introduces the proposed backbone curve shown in Fig. 2, which adopts a five-point definition (A–E) to more accurately reflect these experimental observations.

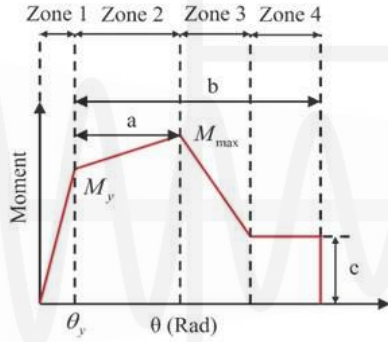


Fig. 1. ASCE 41-17 backbone curve.

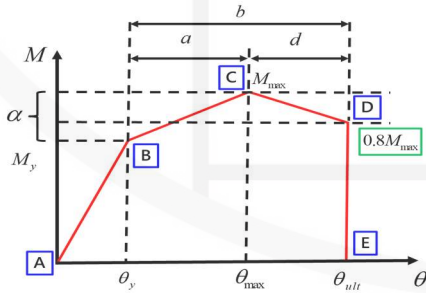


Fig. 2. Proposed backbone curve.

The study identified factors that potentially influence nonlinear hinge parameters and performed nondimensionalization on these variables. This process facilitates comparison between specimens and supports subsequent regression analysis. The representative parameters selected include  $b/t_f$  is the flange width-to-thickness ratio,  $h/t_w$  is the web depth-to-thickness ratio,  $P/P_{ye}$  is the axial load ratio, and  $L/r_y$  is the slenderness ratio with respect to the weak axis:

$$\alpha = 10^{0.036 \left( \frac{b_f}{2t_f} \right)^{0.275} \left( \frac{h}{t_w} \right)^{-0.350} \left( 1 - \frac{P}{P_{ye}} \right)^{-0.019} \left( \frac{L}{r_y} \right)^{0.300}} - 0.24$$

$$(1.848 \geq \alpha \geq 0)$$

$$a = 10^{3.840 \left( \frac{b_f}{2t_f} \right)^{-0.661} \left( \frac{h}{t_w} \right)^{-2.463} \left( 1 - \frac{P}{P_{ye}} \right)^{2.953} \left( \frac{L}{r_y} \right)^{-0.912}} - 0.002$$

$$(0.0224 \geq a \geq 0)$$

$$b = 10^{3.128 \left( \frac{b_f}{2t_f} \right)^{-0.582} \left( \frac{h}{t_w} \right)^{-1.999} \left( 1 - \frac{P}{P_{ye}} \right)^{3.617} \left( \frac{L}{r_y} \right)^{-0.621}} - 0.003$$

$$(0.051 \geq b \geq a)$$

The regression formulas in this study yield predictions that more closely correspond to experimental data for both maximum bending strength and deformation capacity, compared to current provisions as shown in .

In terms of peak moment prediction, the code-based models generally tend to underestimate performance, with approximately 82% of specimens having predicted values lower than the experimental results. In contrast, the model proposed in this study exhibited overestimation in only about 18% of cases, making it more reflective of actual behavior overall. Regarding deformation capacity, the  $a$  and  $b$  parameters obtained from the hysteresis envelope curves effectively represent the actual deformation limits of members under varying axial loads and cross-sectional conditions. Furthermore, the five-point curve definition (Points A–E) adopted in this research aligns with the experimental observations in Taiwan, which use an 80% reduction from maximum strength as the criterion for failure.

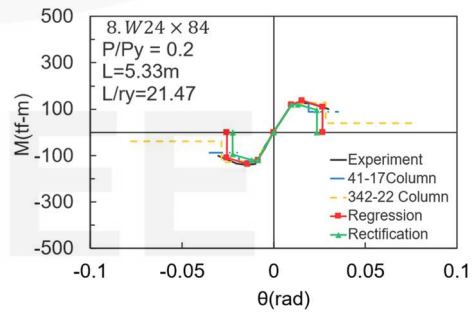
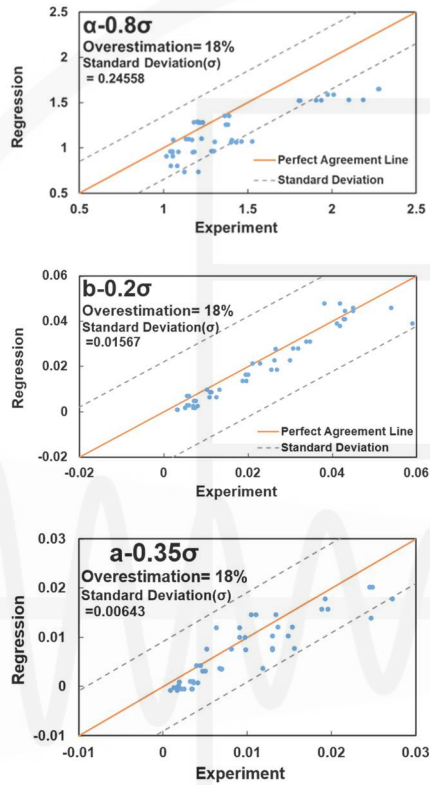


Fig. 3. Comparison of experimental backbone curves with the proposed nonlinear hinge curves for H-shaped steel columns.

To enhance model stability and prevent over-prediction, upper and lower bounds were established for the three primary parameters. To address the requirements for conservatism in practical design, a standard deviation correction factor was also introduced. As illustrated in the error plot in , this approach keeps the overestimation rate of the regression model below 20%, thereby strengthening its overall reliability.



**Fig. 4.** Error plot of the regression model following standard deviation correction.

#### 4. Model Analysis and Experimental Comparison

Verification was conducted using full-scale tests from the NCREE, specifically a full-scale MRF ( ) and a two-story SCBF ( Fig. 7). In the model, the columns and beams were designed as H-shaped steel sections. Nonlinear hinges for the beams and concentric braces were implemented using the models recommended in TEASPA V5.0 (2025). Nonlinear pushover analysis was performed using the structural analysis software ETABS. In this study, only the X-direction was analyzed to compare the analytical results with the experimental data.

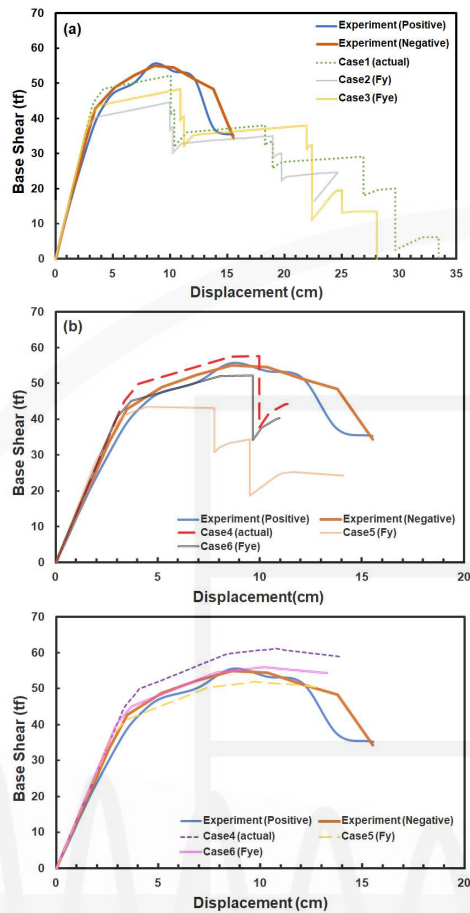


**Fig. 5.** Full-scale, single-bay MRF.

Analysis of the full-scale single-bay MRF reveals that when using the nonlinear hinge model recommended by ASCE 41-17, the resulting capacity curves consistently underestimate strength, regardless of whether nominal strength or actual material strength is used. In contrast, by adopting the recommended nonlinear hinge values for H-shaped steel columns proposed in this study, the analytical results at both the peak strength point and the maximum displacement point align most closely with the experimental envelope curve, demonstrating superior simulation accuracy, as shown in .

When the proposed nonlinear hinge model was applied using actual material strength, the peak strength was found to be slightly higher than the experimental envelope. To maintain design conservatism, it is recommended to use nominal yield strength for material properties to better represent actual performance. Additionally, incorporating the TEASPA V5.0 recommendations for enhanced beam members in the overall frame analysis resulted in a simulated capacity curve that more closely approximates the experimental values. The nonlinear hinge recommendations for H-shaped steel columns proposed in this study outperform the ASCE 41-17 model in simulating failure modes, peak strength, and deformation capacity, demonstrating higher applicability.





**Fig. 6.** Comparison of capacity curves: (a) ASCE 41-17 nonlinear hinges, (b) TEASPA V5.0 conventional beams and proposed column nonlinear hinges, and (c) TEASPA V5.0 enhanced beams and proposed column nonlinear hinges.



**Fig. 7.** Full-scale two-story SCBF.

## 5. Conclusions and Future Work

A regression analysis was performed on the nonlinear behavior of H-shaped steel columns, considering parameters such as section sizes (depth-to-thickness ratio: 84.5 to 23.32; width-to-thickness ratio: 9.77 to 7.84), slenderness ratios (10.03 to 45.21), and axial load ratios (0 to 0.6). The

experimental hysteresis loops were digitized into envelope curves to enable a unified parametric analysis.

The experimental results indicate that the magnitude of axial load significantly affects the moment-resisting capacity and deformation behavior of H-shaped steel columns. Under high axial loads, both moment strength and deformation capacity decrease—with the plastic rotation at peak strength (a) reaching approximately 0.012 to 0.018 rad and the total plastic rotation capacity (b) dropping below 0.03 rad. This phenomenon aligns with the axial-bending interaction effects considered in standard codes, demonstrating that high axial loads adversely impact column ductility and toughness. Conversely, tests conducted under low axial loads showed higher moment capacity and superior deformation capability (parameter *a* between 0.025 ~ 0.035 rad, and parameter *b* reaching up to 0.07 rad). Specifically, under identical sectional conditions, the maximum moment values for low axial load specimens were significantly higher (15% to 25%) than those under high axial loads, identifying the axial load ratio as a critical factor influencing nonlinear capacity.

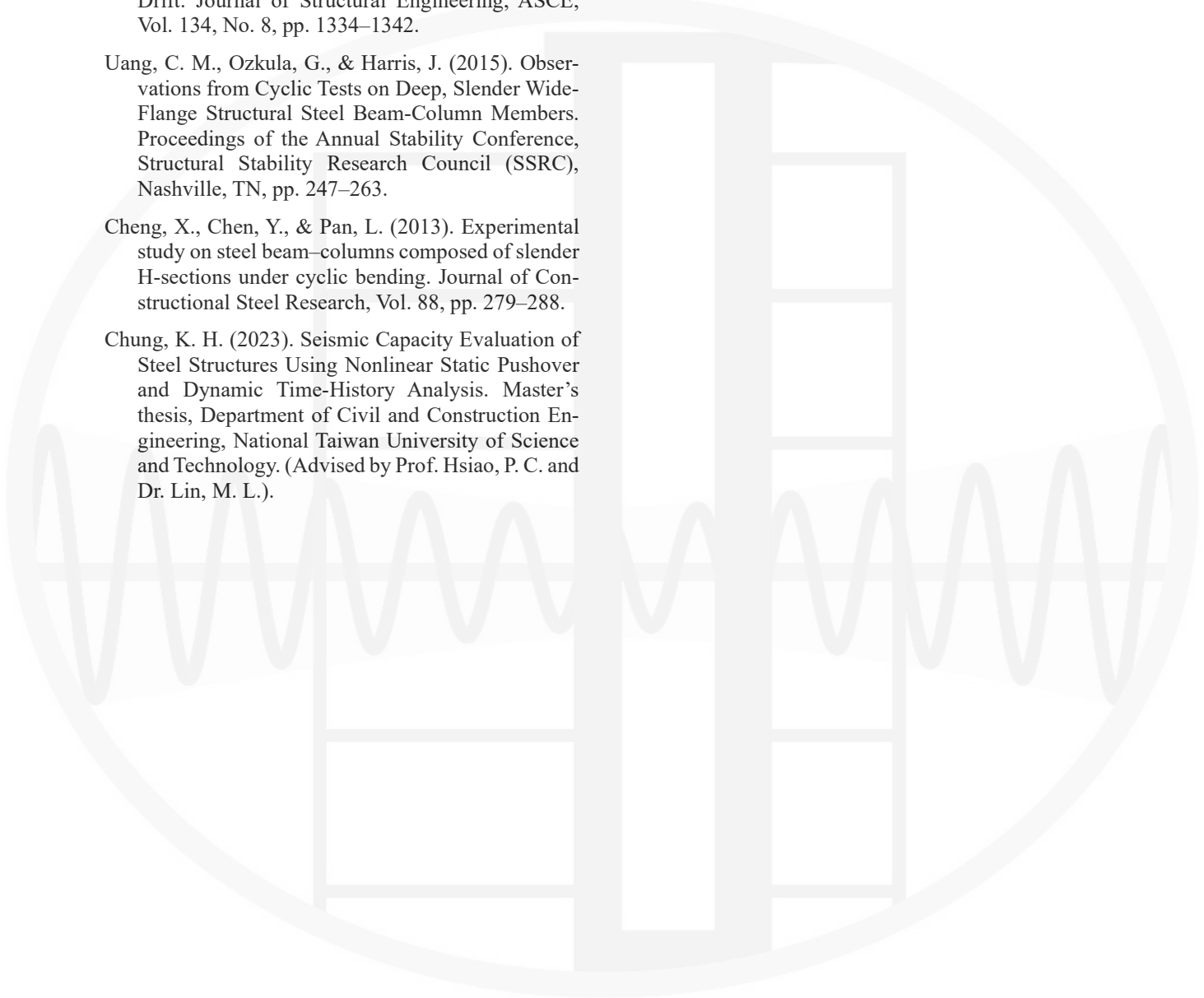
Regarding the nonlinear hinge parameters for the weak axis, there is currently insufficient experimental data to perform a statistical regression analysis. Therefore, the strong-axis regression model established in this study was temporarily used as the analytical basis for weak-axis components. Preliminary results indicate that the strong-axis model does not accurately predict the seismic behavior of weak-axis members. Given that weak-axis characteristics and failure modes likely differ from those of the strong axis, it is recommended that additional full-scale experiments targeting weak-axis specimens be conducted in the future. This would address the data gap and enable the development of a dedicated regression model, thereby improving the accuracy and practicality of weak-axis component evaluations.

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# NCREE



NCREE

# Technical Handbook and Software Development for Seismic Performance Assessment and Retrofit of Building Structures

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## Abstract

This major update of TEASPA V5 emphasizes the integration of a steel structure evaluation module and the implementation of the PMM biaxial axial-moment interaction nonlinear hinge model. In addition, the platform has been migrated from a web-based environment to a standalone desktop application.

To facilitate a systematic understanding of the underlying theoretical framework and the operation of the standalone version for professional engineers, two documents have been developed and published: Technical Handbook for Taiwan Earthquake Assessment and Strengthening of Structures by Pushover Analysis (NCREE-2025-003F) and Operating Manual for Taiwan Earthquake Assessment and Strengthening of Structures by Pushover Analysis (NCREE-2025-010F). Building upon the framework established in TEASPA V4, these documents further advance methodologies for seismic assessment and retrofit of structures.

**Keywords:** seismic performance assessment, building public safety inspection and reporting system, TEASPA

## Introduction

With the aging of buildings and the ongoing revision of design codes, the ability of existing building structures in Taiwan to withstand future consecutive earthquakes has become a critical issue. After the 2016 southern Taiwan earthquake, on-site disaster inspections by relevant professionals revealed that older buildings with soft stories frequently sustained severe damage. In addition to increasing public awareness of seismic safety in buildings, the development of evaluation and retrofit techniques for existing structures has become equally urgent.

TEASPA V5 integrates the previous web-based application into a standalone desktop application. The program interface is shown in Fig. 1, allowing

engineers to use the software without limitations due to poor internet connectivity. Building on the framework of TEASPA 4.3, the program retains six major modules: structural member cross-section design, model construction, member plastic hinge assignment, PGA calculation, model verification, and report generation. Consequently, users can perform building public safety inspection and reporting tasks more efficiently.

The following sections explain the newly added steel structure evaluation functionality and the PMM biaxial axial-moment interaction nonlinear hinge feature, with reference to the technical handbook and operating manual, and summarize the latest updates in TEASPA V5.

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Fig. 1. Program interface of TEASPA V5.

## PMM Biaxial Axial–Moment Interaction Nonlinear Hinge

In this version, the RC module incorporates the PMM biaxial axial–moment interaction nonlinear hinge functionality. Compared with the previous version (V4.3), which only provided the PM uniaxial axial–moment interaction nonlinear hinge, the PMM nonlinear hinge is applicable to high-rise or irregular structures, where significant torsional effects may occur during unidirectional nonlinear static pushover analysis. This enhancement enables the model to capture member behavior involving simultaneous biaxial deformation and substantial axial force variations induced by torsion, as shown in Fig. 2.

To evaluate the analytical accuracy of TEASPA V5, a collaborative study was conducted with the National Research Institute for Earth Science and Disaster Resilience (NIED), Japan. Experimental data obtained from an E-Defense shaking table test of a ten-story RC frame were used for validation. Fig. 3 shows a comparison between analytical and experimental results. The initial stiffness predicted by the analysis agrees well with the experimental results, while the maximum strength is slightly conservative compared to the test data. Thus, the RC module of TEASPA V5 reliably evaluates structural stiffness and strength, providing an effective and accurate tool for engineering applications.

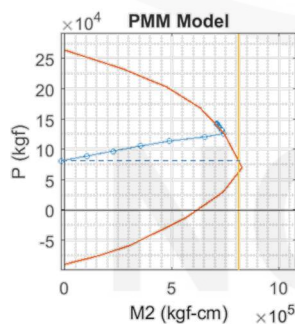


Fig. 2. Variation of column axial forces in mid- to high-rise buildings subjected to seismic excitation.

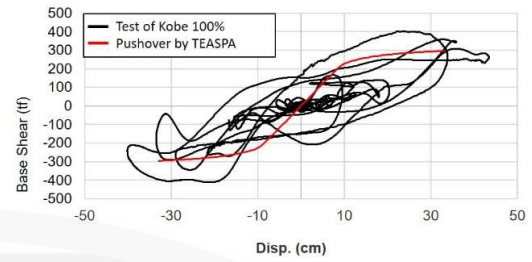


Fig. 3. Comparison of TEASPA V5 analysis and experimental results.

## Steel Structure Evaluation Module

In addition to the new functionalities in the RC module, TEASPA V5 introduces the Steel Structure Evaluation Module. Given that the seismic capacity of existing steel structures in Taiwan may be insufficient and that the number of newly constructed steel buildings has increased in recent years, there is a growing demand for seismic assessment and retrofitting of steel structures. Building on the principles used for detailed seismic capacity assessment of RC buildings, TEASPA V5 offers simulation methods for the behavior of critical steel structural members to evaluate the seismic performance of steel structures.

The Steel Structure Evaluation Module was validated using experimental data from an E-Defense shaking table test of a ten-story steel frame. As shown in Fig. 4, TEASPA V5 accurately predicts the initial stiffness of the structure, although its maximum strength estimate is slightly conservative compared to the test results. The interstory drift ratios at each floor also follow trends consistent with experimental observations, offering engineers a reliable tool for practical applications.

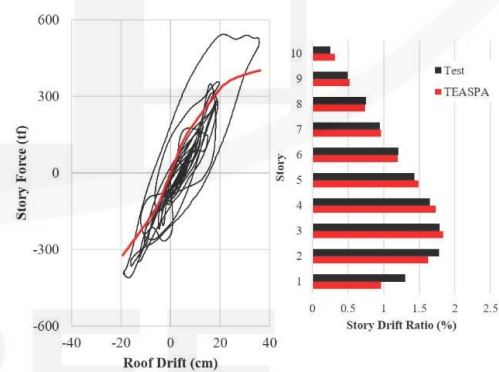


Fig. 4. Comparison of TEASPA V5 analysis and experimental results.

## Updates on the Displacement Reference and Nonlinear Hinge Assignment of RC Columns

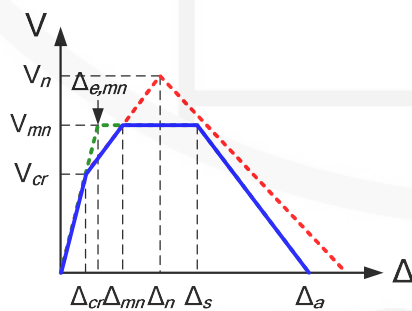


The development team has updated certain displacement references in TEASPA V5. Detailed descriptions can be found in Chapter 2 of the Technical Handbook. The main revisions concern the demand criteria for seismic capacity assessment of newly constructed RC buildings, where the displacement-controlled ( $A_{p2}$ ) reference has been changed from  $D_r^T = 3.0\%$  to  $3.2\%$ . For existing steel structures, the previous displacement-controlled ( $A_{p2}$ ) references for seismic assessment were  $D_r^T = 2.0\%$  for conventional beam-column connections and  $D_r^T = 4.0\%$  for improved connections; these have now been unified to  $D_r^T = 4.0\%$  regardless of connection type. Similarly, the performance criteria for existing steel structures, independent of connection type, have been updated for displacement-controlled ( $A_{p2}$ ) limits, as summarized in Table 1.

For the nonlinear hinge assignment of RC columns, the reduction factor has been eliminated. The initial elastic stiffness of the columns (as shown in Fig. 5) was previously calculated based on the initial axial force (DL + 0.5LL) and then adjusted according to  $E_c I_g$  the variation of axial force to determine the reduction factor. This approach has now been revised to use the full-section moment of inertia  $E_c I_g$  without any reduction. For further details, refer to Section 3.1 of the Technical Handbook.

**Table 1.** Updated Performance Criteria for Seismic Capacity Assessment of Existing Steel Structures.

| Building Use Classification | Displacement-Controlled ( $A_{p2}$ ) |
|-----------------------------|--------------------------------------|
| $I = 1.5$                   | $D_r^T = 2.0\%$                      |
| $I = 1.25$                  | $D_r^T = 3.0\%$                      |
| $I = 1.0$                   | $D_r^T = 4.0\%$                      |



**Fig. 5.** Bending-shear failure lateral load-displacement curve.

## TEASPA V5 Technical Handbook and Operating Manual

To facilitate users' understanding of the background theory and operation of TEASPA, our team has compiled two technical reports: Technical Handbook for Taiwan Earthquake Assessment and

Strengthening of Structures by Pushover Analysis (TEASPA V5) and Operating Manual for Taiwan Earthquake Assessment and Strengthening of Structures by Pushover Analysis (TEASPA V5). As shown in Fig. 6, these reports are available for download on the Technical Reports page of the NCREE website. The Technical Handbook presents the background theory of the program and the calculation methods for each section, while the Operating Manual details the operational procedures of the TEASPA program and the methods for constructing various structural members.

Both the Technical Handbook and the Operating Manual not only maintain the principles of TEASPA V4.3 but also introduce the PMM nonlinear hinge and behavioral simulation methods for steel structural members. Furthermore, the program has transitioned from an online web-based version to a standalone application.

In the past, many users encountering the TEASPA for the first time found that limited classroom time during workshops compressed the training content. When practicing independently at home without prior experience with analysis software, users often encountered difficulties. Furthermore, unlike students, most professionals cannot devote long hours to learning how to use TEASPA, which may result in reluctance to adopt the program.

To help users quickly familiarize themselves with TEASPA V5, the development team, as always, provides demonstration cases and explanations in the final chapter of the Technical Handbook. This edition includes six building demonstration models: a ten-story RC shear wall building, a four-story apartment with brick walls, an experimental verification of an RC shear wall frame, a fifteen-story steel frame, a four-story steel residential structure, and an experimental verification of a buckling-restrained brace steel frame. The Operating Manual, in Chapter 3, presents eight member input examples and parameter verifications, including rectangular columns (general columns and short columns), wing walls, RC T-beams, RC walls, RC open walls, four-sided and three-sided brick walls, and brick window spandrels. The final chapter also features demonstration examples of RC and steel structures, with step-by-step explanations and detailed instructions. Related parameter settings are available for reference on the TEASPA website.

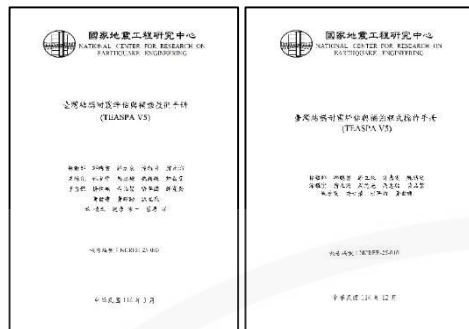


Fig. 6. TEASPA V5 Technical Handbook (left) and Operating Manual (right).

## RebarGen

In recent years, users have frequently reported that constructing rectangular columns or beam sections using the detailed rebar input method is both time-consuming and inconvenient. Occasionally, after carefully entering a series of coordinates, TEASPA returns an error indicating that the rebar coordinates in the section do not satisfy symmetry conditions. To address these challenges, the TEASPA team developed the RebarGen program, as shown in Fig. 7. This tool enables users to quickly calculate rebar coordinates for rectangular sections and generates part of the required input data for the “4.0.1 Sec.CR.Long” sheet in the INP.xlsx file.

When using RebarGen, users must input information such as section dimensions, cover thickness, stirrup diameter, and the “rebar position matrix.” Users can also define custom rebar sizes. After the calculation is completed, the program generates a schematic diagram of the rebar layout, as shown in Fig. 8, enabling users to verify the input rebar sizes and distribution. Additionally, it outputs a Rebarnote.xlsx file, from which users can directly copy the calculated rebar coordinates and paste them into the “4.0.1 Sec.CR.Long” worksheet in the TEASPA input file.

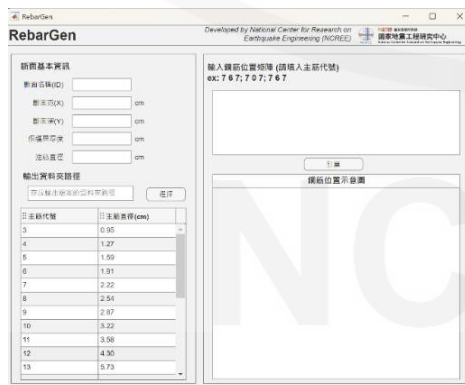


Fig. 7. Program interface of RebarGen.

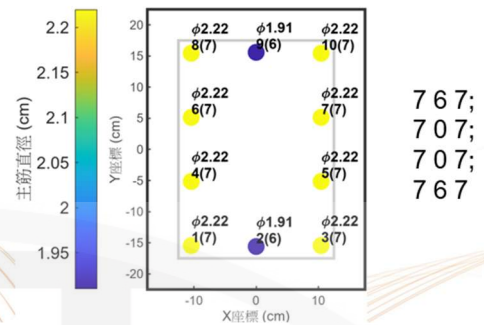


Fig. 8. Rebar layout preview.

## Conclusion

To ensure that the program remains aligned with the latest code developments and becomes increasingly user-friendly, the TEASPA team consistently incorporates feedback from practicing engineers. In addition to implementing suggestions from multiple sources in program updates, the protection of users' personal data remains a primary concern. The team aims to sustain ongoing communication and collaboration with the engineering community.

## Acknowledgements

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# Application of Nonlinear Dynamic Methods to the Simulation of RC Structural Specimens in Shaking Table Tests

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## Abstract

This study conducts nonlinear dynamic time history analysis using the Taiwan Earthquake Assessment for RC Structures by Dynamic Analysis (TEASDA 1.0). Reinforced concrete (RC) building specimens from shaking table tests are selected as validation targets. Numerical models are developed with the structural analysis software ETABS 2019. Nonlinear behavior is modeled using both the TEASDA auxiliary program and the ASCE/SEI 41 provisions. Recorded time history data from actual shaking table tests are used to investigate structural dynamic responses under seismic loading. Analytical results are then compared with experimental results from shaking table tests. The findings demonstrate that, with appropriate consideration of structural nonlinear behavior, nonlinear dynamic time history analysis can effectively capture structural responses under real earthquake excitations, thereby enhancing the accuracy and reliability of seismic design.

**Keywords:** Nonlinear dynamic time history analysis, shaking table test, reinforced concrete building structure, TEASDA, ASCE 41.

## Introduction

This study examines the seismic evaluation of reinforced concrete (RC) building structures, assessing the application and accuracy of nonlinear dynamic time history analysis in simulating structural responses under seismic excitation. As performance-based seismic design becomes increasingly important, traditional linear analysis methods are insufficient to fully represent the behavior of structures subjected to strong earthquakes. Consequently, analytical approaches that incorporate material nonlinearity and component plasticity are required to improve the reliability and accuracy of seismic assessments.

This study follows the Taiwan Earthquake Assessment for RC Structures by Dynamic Analysis (TEASDA 1.0) (Hsiao et al., 2021) and incorporates the ASCE/SEI 41-17 provisions (ASCE 2017) to define plastic hinges in beams and columns, thereby establishing a rational nonlinear numerical model. ETABS 2019 is used to construct the structural model,

and actual earthquake time history records are input to simulate structural dynamic responses under seismic loading.

To validate the feasibility of the analytical approach, multiple RC building specimens from shaking table tests were selected, encompassing various structural configurations and irregularities. Numerical simulation results were compared with experimental measurements to evaluate the accuracy and discrepancies of the analysis. The results indicate that, with appropriate consideration of structural nonlinear behavior and plastic hinge properties, nonlinear dynamic time history analysis effectively captures the response trends of structures under seismic loading, demonstrating good agreement with shaking table test results. This highlights the strong potential of the analytical method for seismic evaluation of RC structures and its applicability in engineering practice and future research.

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## Shaking Table Specimen Case Study

This study examines a 10-story RC building tested on a shaking table (Kajiwara et al., 2015; Sato et al., 2015; Tosauchi et al., 2017) as an analysis case. The structural model is developed using ETABS, and detailed building information is provided in Table 1. The story heights are as follows: 2.8 m for the first floor, 2.6 m for the second to fourth floors, 2.55 m for the fifth to seventh floors, and 2.5 m for the eighth to tenth floors. The total height of the specimen is 25.75 m. Shear walls are present from the first to seventh floors in the X-direction, while the Y-direction utilizes a moment-resisting frame. A three-dimensional view of the model is shown in Fig. 1.

Table 1. Building information.

|                |                           |
|----------------|---------------------------|
| Building type  | 10-story RC structure     |
| Structure type | Moment frame + Shear wall |
| Dimension      | 12 by 8 (m)               |
| Height         | 22.75 (m)                 |

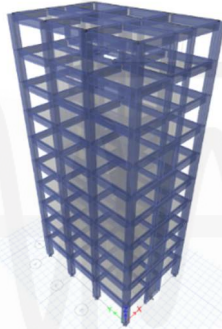


Fig. 1. Numerical model of the structure.

The loading calculation and distribution are based on specimen data reported by Kajiwara et al. (2015), Sato et al. (2015), and Tosauchi et al. (2017). The total designed weight of the specimen is 8,196 kN, as shown in Table 2. In addition to the self-weight (D.L.) of the main structural frame, the specimen weight includes additional loads (L.L.) from lifting and installation equipment. In the analytical model, the total weight at each floor (D.L. + L.L.) is applied as a uniformly distributed load and evenly assigned to each floor slab, as shown in Fig. 2.

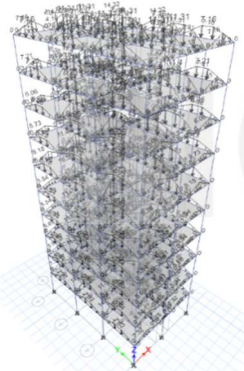


Fig. 2. Numerical model of the retrofitted structure.

After verifying the correctness of the model weights, the elastic periods of the analytical model are calibrated to match the initial periods of the shaking table specimen by adjusting member stiffness, as shown in Table 3. According to Kajiwara et al. (2015), Sato et al. (2015), and Tosauchi et al. (2017), the initial period is 0.85 s in the frame direction (Y-direction in this model) and 0.58 s in the shear wall direction (X-direction in this model). The modal analysis results for the first six modes, including modal periods and mass participation ratios, are presented in Table 4. It can be observed that, after stiffness adjustment, the initial modal periods in both directions correspond to those of the shaking table specimen.

Table 2. Designed weight of the specimen.

| Story | D.L. (kN) | L.L. (kN) | Sum  |
|-------|-----------|-----------|------|
| 10    | 725       | 0         | 725  |
| 9     | 740       | 57        | 797  |
| 8     | 694       | 28        | 722  |
| 7     | 716       | 28        | 744  |
| 6     | 949       | 28        | 977  |
| 5     | 318       | 188       | 806  |
| 4     | 780       | 28        | 808  |
| 3     | 798       | 28        | 826  |
| 2     | 817       | 28        | 845  |
| 1     | 889       | 57        | 946  |
| Total | 7726      | 470       | 8196 |

Table 3. Effective stiffness.

|                 | Flexural (I) | Shear (A) |
|-----------------|--------------|-----------|
| Column          | 0.35         | 1         |
| Beam            | 0.35         | 1         |
| Wall            | 1            | 1         |
| Beam above wall | 20           | 1         |

Table 4. Modal periods and mass participation ratios.

| Modal | Period (sec) | UX | UY | UZ | RZ |
|-------|--------------|----|----|----|----|
| 1     | 0.84         | 0  | 1  | 0  | 0  |
| 2     | 0.57         | 1  | 0  | 0  | 0  |
| 3     | 0.51         | 0  | 0  | 0  | 1  |
| 4     | 0.29         | 0  | 1  | 0  | 0  |
| 5     | 0.18         | 1  | 0  | 0  | 0  |
| 6     | 0.17         | 0  | 1  | 0  | 0  |

## Input Ground Motion Time Histories

To compare analytical results with shaking table test results, the same earthquake time histories used in the experiments are adopted as input motions. The input motion is the JMA-Kobe record, applied in three directions (X, Y, and Z) at five excitation levels (10%,



25%, 50%, 60%, and 100%).

In the analytical model, the X-direction corresponds to the frame direction in the experiment, while the Y-direction corresponds to the shear wall direction. To reduce computational time, only a segment of the original time history, from 8 s to 33 s (totaling 25 s), is used, as shown in Fig. 3.

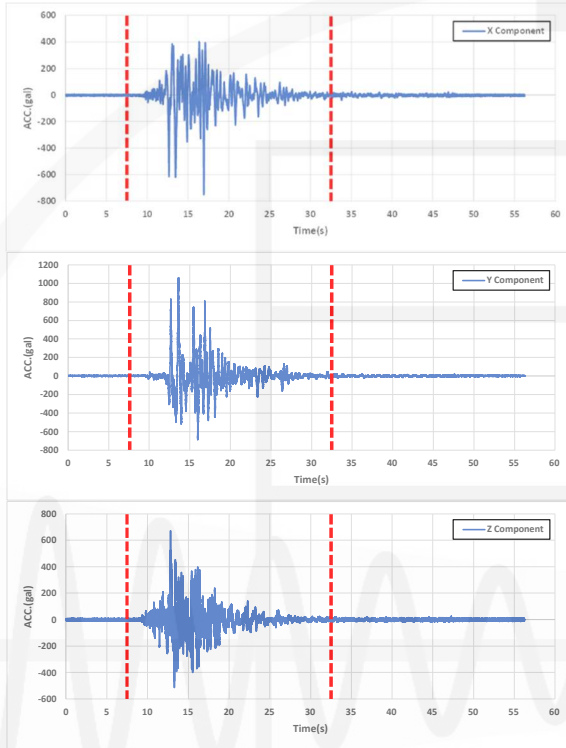


Fig. 3. Input ground motion time history.

## Comparison Between Experimental and Analytical Results

Nonlinear dynamic time history analyses are conducted in ETABS using two modeling approaches: the TEASDA auxiliary program (Hsiao et al., 2021) and plastic hinge definitions based on ASCE 41-17 (ASCE 2017). The structural responses under seismic loading are evaluated and compared.

To verify the accuracy of the numerical model, analytical results are compared with shaking table test results. The comparison includes acceleration and displacement time histories in both the elastic stage (10% excitation) and the plastic stage (100% excitation), as shown in Figs. 4–7.

Results indicate that, in the elastic stage, both the TEASDA-based and ASCE 41-based plastic hinge models produce similar acceleration and displacement responses in the frame direction, which are consistent with the experimental results (Figs. 4a and 5a). However, the acceleration and displacement responses in the shear wall direction are slightly smaller than the

experimental values (Figs. 4b and 5b). This is primarily due to both models sharing the same elastic period settings.

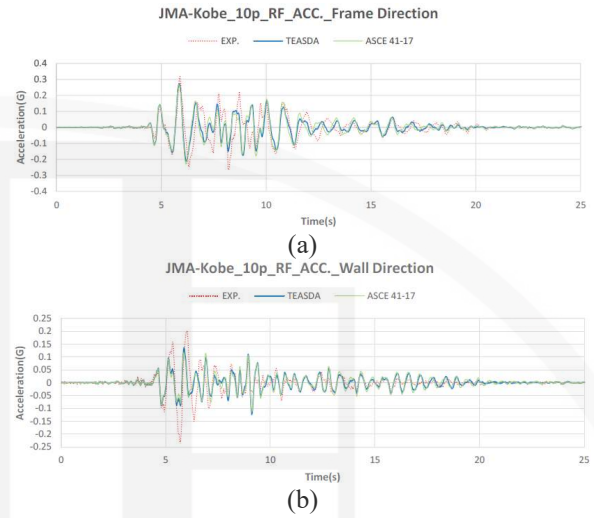


Fig. 4. Acceleration time histories during the elastic stage: (a) frame direction; (b) wall direction.

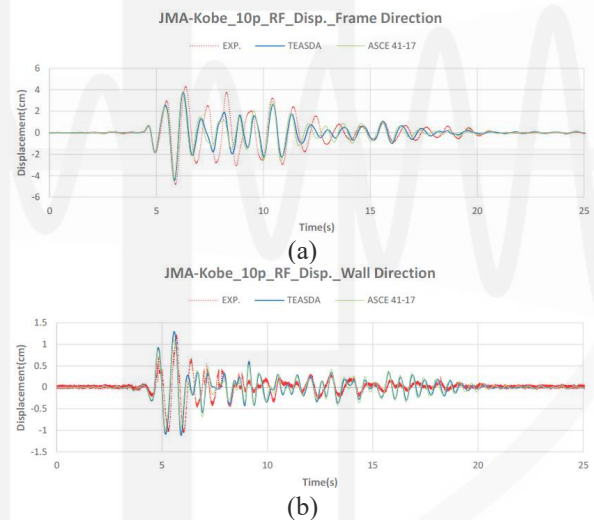
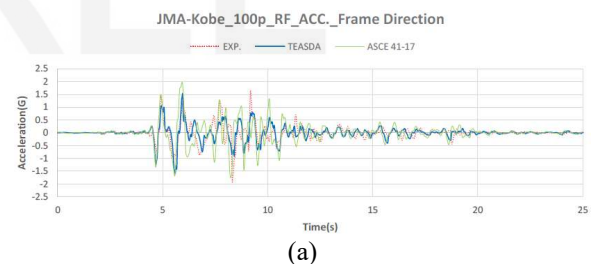


Fig. 5. Displacement time histories during the elastic stage: (a) frame direction; (b) wall direction.

In the plastic stage, the TEASDA-based model yields more conservative results than the experimental data (Figs. 6 and 7). The ASCE 41-based model aligns well with experimental responses in the frame direction (Figs. 6a and 7a), but overestimates responses in the shear wall direction (Figs. 6b and 7b).



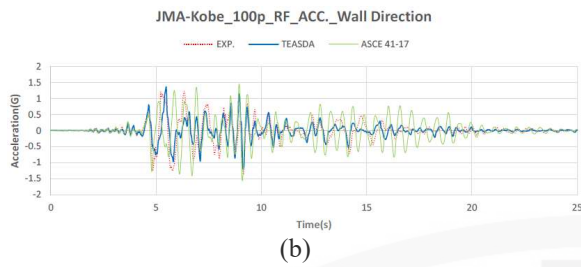


Fig. 6. Acceleration time histories during the plastic stage: (a) frame direction; (b) wall direction.

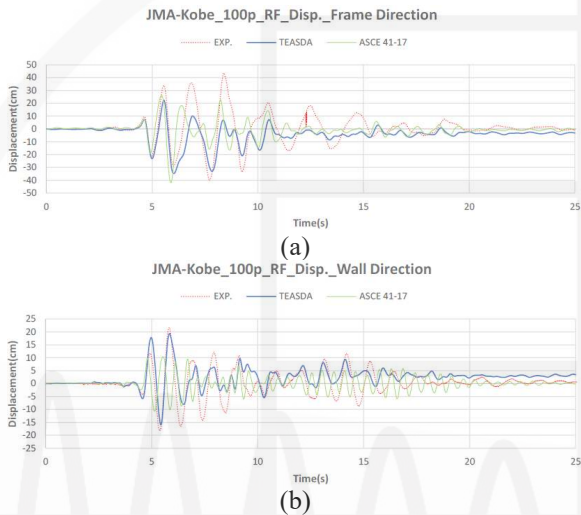


Fig. 7. Displacement time histories during the plastic stage: (a) frame direction; (b) wall direction.

## Conclusions

This study applies nonlinear dynamic time history analysis to simulate shaking table test structures and validates the analytical model by comparing it with experimental results. The results demonstrate that the developed building model effectively captures the actual structural behavior. Comparison of different plastic hinge modeling approaches shows that structural models developed using the TEASDA auxiliary program (Hsiao et al., 2021) can reasonably simulate the nonlinear behavior of RC components.

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# Performance of Existing RC Columns Strengthened with Recycled Aggregate Concrete Jacketing

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## Abstract

This study examines the seismic response of reinforced concrete (RC) columns retrofitted with recycled aggregate concrete (RAC) jacketing under sequential earthquake loading. The results demonstrate that RAC jacketing significantly enhances structural stiffness and effectively restrains lateral deformations, in contrast to the unretrofitted control specimen (UC), which exhibited rapid displacement escalation and severe hysteretic degradation due to cumulative damage. The strengthened specimen (SC-RAC) achieved yielding under identical loading conditions, delivering a 243% increase in lateral load capacity. During dynamic excitation, the UC reached a drift ratio of approximately 7% and experienced abrupt strength loss indicative of brittle failure resulting from insufficient confinement, whereas the SC-RAC maintained a stable response without sudden degradation. Furthermore, the retrofitted column sustained its load-carrying capacity over a wider displacement range, with only minor deterioration observed at the final stage. These findings highlight RAC jacketing as an effective and sustainable retrofitting strategy for improving the seismic resilience of RC columns.

**Keywords:** recycled aggregate concrete (RAC), retrofitting, RC columns, seismic performance, sustainable construction.

## Introduction

A large portion of the existing reinforced concrete (RC) building stock was designed according to earlier code provisions that did not adequately address seismic demands. Many of these older structures are classified as non-ductile because they were proportioned primarily for gravity loading and lack the confinement, reinforcement detailing, and capacity design requirements mandated by modern seismic codes [1,2]. Consequently, they often exhibit limited lateral stiffness and strength, which makes them susceptible to excessive interstory deformation, soft-story behavior, and other undesirable response mechanisms during earthquakes [3]. Their insufficient ductility and poor energy dissipation capacity further increase the risk of brittle damage, severe structural deterioration, or collapse under strong ground shaking [4].

Given these deficiencies, effective retrofit

strategies are essential not only for restoring structural capacity but also for improving the seismic reliability of vulnerable buildings. Among the available strengthening techniques, RC jacketing is one of the most established approaches for upgrading deficient columns and related structural members. This method enhances the original section by enlarging the cross-section and introducing additional longitudinal and transverse reinforcement, thereby improving strength, stiffness, and confinement [5,6]. Its effectiveness has been demonstrated in previous experimental studies, including the Taiwan–Thailand collaborative work reported by Weng et al. [7], in which jacketed columns with dowel anchors achieved approximately 10% greater lateral strength than comparable specimens without dowels, while both specimens exhibited similar ductility and flexural failure characteristics.

The rapid expansion of concrete construction has significantly increased the demand for natural aggregates, raising concerns about resource depletion.

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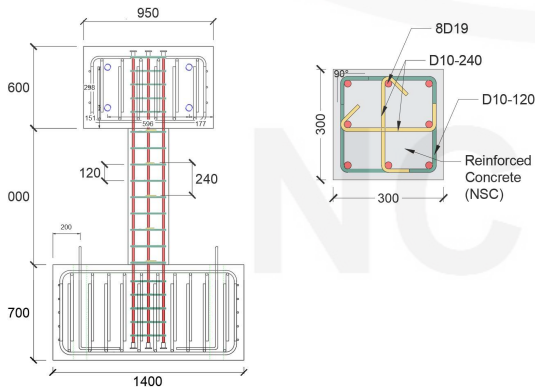
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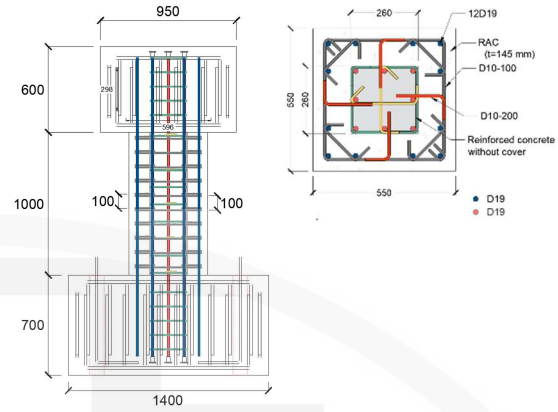
As a sustainable alternative, recycled aggregate concrete (RAC), derived from demolished concrete, has attracted attention for use in new concrete applications [8–10]. Despite its environmental benefits and promising mechanical properties, the application of RAC in seismic retrofitting remains insufficiently validated, particularly for non-ductile RC buildings in earthquake-prone regions. This issue is especially relevant in Taiwan and similar areas, where many older structures lack adequate detailing and confinement, as demonstrated by past earthquake damage. Therefore, this study evaluates the potential of RAC jacketing as an effective and sustainable solution for strengthening existing RC columns.

### Specimen Details

To assess the seismic effectiveness of RAC jacketing, two column specimens were fabricated and tested at NCEE. The unretrofitted control specimen (UC) was intentionally designed to replicate common detailing deficiencies found in pre-modern RC construction. It had a clear height of 1000 mm and a square cross-section of 300 mm × 300 mm, with a relatively thin concrete cover of 20 mm. The longitudinal reinforcement consisted of eight D19 bars arranged symmetrically, while confinement was provided by D10 closed stirrups spaced at 120 mm within the plastic hinge regions and 240 mm at mid-height, supplemented by D10 cross ties at the same spacing. The concrete mixture was designed to achieve a compressive strength of 20 MPa. Strengthening was accomplished by removing the existing concrete cover and installing a 145 mm-thick RAC jacket around the original column. The jacket was reinforced with twelve D19 longitudinal bars positioned at the corners and confined using D10 stirrups spaced at 100 mm along the column height. To ensure effective interaction between the existing core and the newly added layer, mechanical dowels were introduced in accordance with the anchorage provisions reported by Weng et al. [7]. The overall geometry and reinforcement configuration of the specimens are illustrated in Fig. 1.



(a)



(b)

Fig. 1. Geometric and reinforcement details: (a) UC specimen; (b) SC-RAC specimen.

### Test Setup and Loading Protocols

A dedicated experimental setup was developed to simulate combined axial and lateral demands representative of seismic loading. Lateral forces were applied using an actuator connected to the column head, while a constant axial load of 100 kN was maintained with a hydraulic jack. The foundation block was rigidly fixed to the strong floor to eliminate unintended movement during testing. This configuration allowed for realistic reproduction of gravity and earthquake-induced actions typically experienced by first-story columns in mid-rise RC buildings, as illustrated in Fig. 2.



Fig. 2. Experimental setup.

During the dynamic loading phase, six earthquake records were applied to evaluate the seismic response of the specimens under varying intensity levels. Each record was scaled to peak ground accelerations (PGA) of 100, 200, and 400 gal, representing a range of seismic demands. The selected motions included both far-field and near-fault characteristics and were imposed using a servo-controlled actuator operating in displacement mode. The loading sequence began with the El Centro 1940 S00E component (far-field), followed by the Kobe 1995 N–S record from the KJMA station and the Chi-Chi 1999 N–S record from TCU075, both representing near-fault conditions. The final set consisted of the Meinong 2016 CHY063 N–S record, applied at three



PGA levels (100, 200, and 400 gal).

After completing the ground motion phase, a displacement-controlled reversed cyclic loading protocol was applied to further investigate the inelastic behavior. Each drift level was repeated for two full loading cycles to capture the effects of cyclic degradation. The initiation of inelastic loading was determined by the maximum drift attained by each specimen during the preceding dynamic tests, ensuring a consistent transition into the nonlinear response range.

## General Results

Clear distinctions in hysteretic behavior were observed between the UC and the strengthened (SC-RAC) specimens under low-intensity excitations (PGA = 100 gal). For both the El Centro and Kobe records, the UC specimen developed peak lateral forces of 103.43 kN and 105.81 kN, respectively, with wider hysteresis loops and greater deformation demands, indicating an early inelastic response. In contrast, the SC-RAC specimen remained essentially elastic, mobilizing higher or comparable peak forces (239.26 kN and 226.67 kN) with steeper, narrower loops, reflecting increased stiffness and reduced displacement response.

As the loading sequence progressed, the disparity became more pronounced. Under the Chi-Chi excitation, the SC-RAC specimen sustained a peak lateral force of 192.97 kN, whereas the UC specimen dropped to 91.92 kN, indicating substantial degradation in load-carrying capacity. A similar trend was observed for the Meinong 100 gal record, where the SC-RAC reached 155.23 kN, compared to 101.89 kN for the UC specimen. The UC specimen exhibited increasing deformation without a corresponding strength gain, suggesting a transition into the inelastic regime and progressive stiffness deterioration due to cumulative damage.

Under higher-intensity motions (Meinong 200 and 400 gal), the SC-RAC specimen continued to demonstrate superior performance, maintaining a near-linear response with peak forces of 282.11 kN and 425.85 kN, respectively, which significantly exceeded those of the UC specimen (118.38 kN and 124.19 kN). Although the SC-RAC began to approach yielding at 400 gal, the UC specimen exhibited rapid displacement escalation and severely degraded hysteresis loops, indicating pronounced loss of strength and stiffness. As summarized in Fig. 3, the RAC-jacketed column achieved up to a 243% increase in peak lateral capacity under the most severe excitation, highlighting its enhanced seismic resistance.

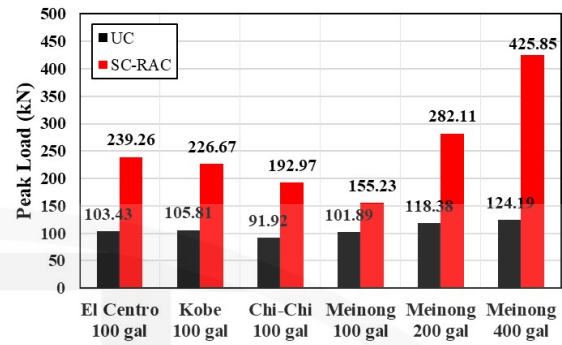


Fig. 3. Comparison of peak lateral loads between UC and SC-RAC specimens under six ground motions.

For the UC specimen, the previous Meinong 400 gal excitation had already driven the column to a drift level approaching 7% ( $\approx 91$  mm), making lower drift stages unnecessary for subsequent cyclic testing. Consequently, the loading protocol began directly at this drift level. However, after only one cycle, the lateral resistance decreased by more than 15% from its peak value, indicating imminent failure. This rapid strength deterioration was accompanied by highly pinched and unstable hysteresis loops (Fig. 4), reflecting severe damage accumulation and loss of stiffness.

The envelope response of the UC specimen further confirms its limited capacity and pronounced deformability. The lateral force increased gradually with displacement, reaching peak values of 124.86 kN at 62.47 mm in the positive direction and 120.25 kN at 70.59 mm in the negative direction. Beyond these points, the response exhibited a plateau and then declined, consistent with brittle behavior governed by insufficient confinement and poor energy dissipation.

In contrast, the SC-RAC specimen exhibited a markedly different response, characterized by significantly higher strength and reduced deformation demand. The envelope curve was steeper and more stable, reaching peak loads of 509.84 kN at 30.18 mm (positive) and 503.84 kN at 39.02 mm (negative), representing an increase of approximately 308% compared to the UC specimen. Additionally, the SC-RAC maintained its load-carrying capacity over a broader displacement range, with only minor degradation observed during the final stages of testing, indicating improved ductility and sustained structural integrity.

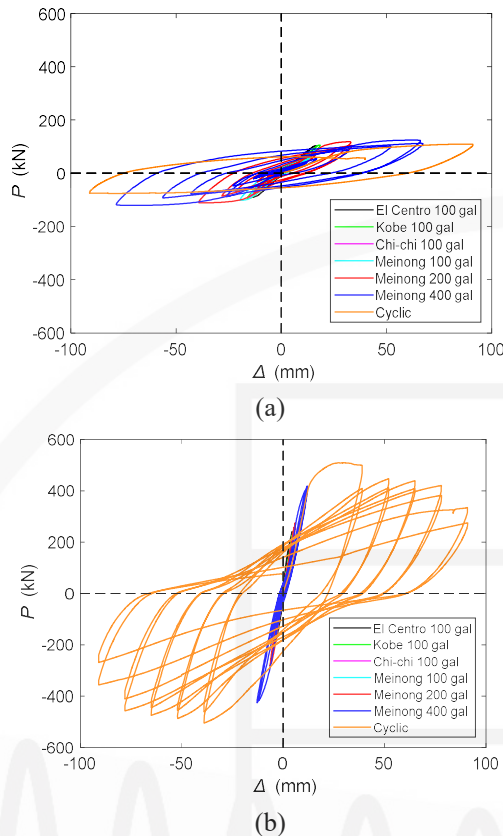


Fig. 4. Force–displacement combined hysteresis curves (static cyclic loading and dynamic ground motions): (a) UC specimen; (b) SC-RAC.

## Conclusions

This study confirms the viability of RAC jacketing as an effective seismic retrofitting technique for RC columns, demonstrating substantial improvements in both strength and overall structural response. Under dynamic loading, the SC-RAC specimen exhibited enhanced stiffness and consistently higher load resistance compared to the UC specimen. During the Meinong 400 gal excitation, the retrofitted specimen achieved an increase of up to 243% in peak lateral capacity, clearly indicating superior seismic performance. The combined hysteretic response further highlights this improvement, with the SC-RAC specimen reaching a peak lateral load of 509.84 kN, whereas the UC specimen failed at only 124.86 kN. In contrast, the UC specimen exhibited sharp post-peak strength degradation accompanied by pinched hysteresis loops, reflecting brittle behavior associated with inadequate confinement. These observations underscore the ability of RAC jacketing to enhance both load-carrying capacity and damage tolerance in non-ductile RC columns.

## Acknowledgments

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# International Collaborative Test Platform for Suspended Nonstructural Systems

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## Abstract

Recent earthquakes in Taiwan, Japan, and Korea have shown that suspended nonstructural systems, which include ceilings, suspended equipment, and mechanical, electrical, and plumbing (MEP) piping, are highly vulnerable to seismic damage and may seriously impair the post-earthquake functionality of essential facilities. Because suspended systems must be mounted on dedicated supporting frames during shake-table testing, large-scale experimental validation is costly, and it is difficult for a single facility to conduct all necessary tests. This project integrates the research resources and testing capabilities of the National Center for Research on Earthquake Engineering in Taiwan, the Hyogo Earthquake Engineering Research Center (E-Defense, NIED) in Japan, and the Seismic Research and Test Center (SESTEC) in Korea to establish a multinational joint testing and validation platform for suspended nonstructural systems. The study covers three main tasks: seismic strategy research, development of testing technology, and implementation of a multinational testing framework. Full-scale and large-area shaking-table tests were used to examine suspended busway systems, concealed ceilings, and vibration-isolated mechanical equipment. The project also developed an input-motion design approach in which the engineering objective is the response at the specimen anchorage point rather than at the shaking-table base. In the NCRE demonstration tests, using suspended vibration-isolated equipment as an example, the results showed that inadequate lateral restraint can lead to excessive displacement. These outcomes provide a technical foundation for future qualification methods, design guidance, and long-term multinational cooperation on suspended nonstructural systems.

**Keywords:** suspended nonstructural systems, overseas testing platforms, seismic strategies, seismic testing criteria

## Introduction

Recent earthquake experiences in Taiwan, Japan, and Korea indicate that suspended ceilings, equipment, and mechanical, electrical, and plumbing (MEP) piping systems inside buildings can be damaged even under moderate shaking, interrupting building functions and post-earthquake serviceability. Essential facilities such as hospitals, high-tech plants, and power facilities therefore require reliable seismic strategies and validation methods for critical suspended nonstructural systems. Because these systems exhibit

highly nonlinear behavior and their post-earthquake functionality must often be verified at the system level, full-scale dynamic testing on shaking tables remains the most direct and effective validation approach. However, suspended systems require supporting frames to be mounted on shaking tables, which increases testing costs and may exceed the capacity of a single domestic facility. In response, this project was organized around a multinational joint testing and validation platform that combines the shaking-table facilities, frame configurations, and research expertise of the National Center for Research on Earthquake

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Engineering, National Institutes of Applied Research (hereinafter referred to as “NCREE”) in Taiwan, the Hyogo Earthquake Engineering Research Center (E-Defense, NIED) in Japan, and the Seismic Research and Test Center (SESTEC) in Korea.

## Seismic Strategy Studies for Suspended Systems

This part of the study takes suspended concealed-ceiling systems as the research target. At present, concealed ceiling systems commonly used in Taiwan can be classified into four types: traditional, notched carrier, metal strip, and metal panel concealed ceilings. Except for the metal strip type, the framing systems of the other three types consist of main runners and carrying channels. In the metal strip ceiling system, the main runner and ceiling panel are combined into a single component, so system stability depends primarily on the carrying channels. For component connections, the hanger assemblies of traditional and metal panel concealed ceilings include fastening plates, threaded hanger rods, and hanger hooks, which suspend and secure the carrying channels. In contrast, the hanger assemblies of notched carrier and metal strip concealed ceilings use nuts and bolts to fasten the carrying channels, without hooks.

Because early concealed ceiling practices in Taiwan were largely adopted from Japan, the “traditional concealed ceiling” referenced in this study corresponds to the Japanese ceiling system. Through this international research collaboration, shaking table tests were conducted on the traditional concealed ceiling system in partnership with the NIED research team in Japan. The results from this study are expected to provide a foundation for the future design of ceilings and equipment layouts in large spaces, and to support the revision or development of relevant domestic design standards to ensure the safety and functional continuity of concealed ceiling systems during earthquakes.

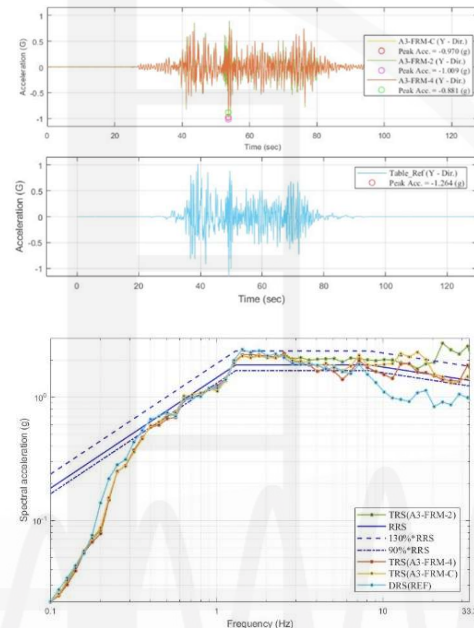


**Fig. 1.** Ten-story steel frame and 7th-floor busway specimen.

## Development of Testing Technology

This project reviewed the design concepts of

AC156, ASTM E3118, and major frame-based shake-table studies. The review showed that laboratories do not share a unified AC156-based testing approach and differ in shaking-table capacity, frame stiffness, specimen scale, and research objectives. Because suspended specimens are mounted on extended steel frames, the actual seismic demand at the specimen anchorage can differ from the shaking-table motion due to amplification or filtering by the frame (Fig. 2).



**Fig. 2.** Comparison of steel frame response with shaking table motion.

Consequently, in this study, the engineering control target for input-motion design is defined as the anchorage-point motion rather than only the motion of shaking table. Pre-tests were carried out to measure the response at specified anchorage locations on top of the frame, and the measured response spectra were compared to the required spectra. For the large international demonstration tests, the actual input correction primarily used a transfer-function-based approach: trial runs identified the frame transfer function, and the original input histories were iteratively modified in the frequency domain to better reproduce the target floor demand at the suspension anchorage.

Regarding suspended-system testing at the three experimental facilities in Taiwan, Japan, and Korea, NCREE has primarily used small supporting frames to conduct small-area shaking table tests, as the construction cost of large frames is relatively high. Consequently, large-area ceiling-system and equipment-system tests have been less common at NCREE. In contrast, the E-Defense shaking table operated by NIED has been used to install busway systems, HVAC systems, and fire-protection piping in a ten-story steel frame. It has also been used to install



concealed ceilings, indoor units, and total heat exchangers in a scaled gymnasium model to observe the interaction between roof structures and suspended equipment. SESTEC, meanwhile, has established a validation approach for ceiling-system testing using smaller steel frames suited to its facility capacity. Additionally, because it operates multiple shaking tables, SESTEC has strong potential for developing tests on suspended systems distributed over elongated areas. Although the testing concepts at the three institutions are similar, they differ in frame stiffness, shaking-table capacity, specimen size, and research objectives. Therefore, the key to platformization is not strict uniformity but rather the creation of a foundation for future collaboration through pre-testing, calibration, and a shared technical language.

Internationally, suspended systems are commonly tested using rigid supporting frames mounted on shaking tables; however, the relationships among frame area, stiffness, out-of-plane vibration, and the response at specimen anchorage points have not yet been fully clarified. Accordingly, this project developed input-motion adjustment methods based on the concept of “reproducing the target floor motion at the top of the supporting frame” to reduce the influence of the frame’s dynamic characteristics on the test results. To verify this concept, the study introduced six-axis sensors along with the MTS StexPro external adaptation process, aiming to back-calculate and update the shaking-table commands from measurements taken at the top of the frame so that the response at designated locations would more closely match the target motion. Preliminary results show that using external sensors for iterative input-motion adjustment is feasible. Although rotational measurements are still affected by noise, this approach already provides a sufficient technical basis for further development of testing procedures for suspended nonstructural systems.

### Procedure for Overseas Testing

Beyond technical development, the project aimed to establish a repeatable international process for future suspended-system testing. Implementation showed that such testing requires coordinated management of specimen modularization, three-dimensional layout, instrumentation planning, anchorage verification, sea transport, customs processing, import inspection, tax exemption, return shipping, frame fabrication, connection detailing, site scheduling, and pre-test site preparation. Thus, international suspended-system testing is not only an experimental challenge but also a project-management challenge that relies on schedule control and resource coordination. By conducting an international demonstration program in this study, the project gained practical knowledge to create a reusable process for future collaborative qualification testing

and standards development.

### Demonstration Test at NCREE

At the NCREE Tainan Laboratory, full-scale suspended-system tests were conducted on the long-stroke, high-velocity shaking table using a 4 m-high extended steel frame installed on the 8 m × 8 m table. As shown in Fig. 3, the frame allowed suspended specimens up to 10.98 m × 10.98 m, corresponding to a maximum test area of about 120 m<sup>2</sup>. The frame fundamental frequencies were approximately 15.5 Hz in the horizontal X and Y directions and 11.7 Hz in the vertical direction.



**Fig. 3.** Test area of the suspended nonstructural system.

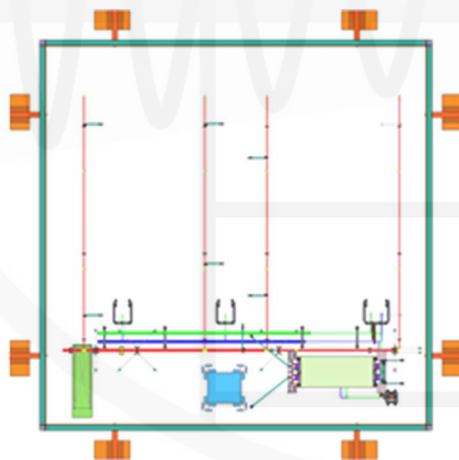
The demonstration program consisted of three test phases led by NCREE, SESTEC, and NIED, respectively. Before the formal multinational tests, NCREE designed and adjusted the input motions for the demonstration campaign. The motion set was divided into three categories: natural earthquake records, structural-response motions, and AC156-based spectrum-compatible motions. In the NCREE program, THH051R was a natural earthquake record, THHRF was a simulated floor-response motion generated from a simplified SAP2000 structural model, and TACRF\_CGT and TACRF\_KPH were used as standardized benchmark motions for comparison (Table 1). Of these, TACRF\_KPH was the primary benchmark motion because it incorporated spectral characteristics from the 1995 Kobe earthquake in Japan, the 2017 Pohang earthquake in Korea, and the 2024 Hualien earthquake in Taiwan. Due to shaking-table capacity limitations, the representative NCREE verification level was KPH95 (TACRF\_KPH at 95% intensity), with horizontal peak accelerations of about 1.0 g, which provided a rigorous basis for equipment qualification in this study.

As an example, the NCREE test matrix included a suspended pre-cooling air-handling unit with connected piping and nearby sprinkler components, allowing observation of spatial interactions between systems (Fig. 4). The three phases compared a fully reinforced configuration (Phase I), a partially reinforced configuration (Phase II), and a baseline configuration (Phase III). The baseline phase

replicated the Hualien damage scenario using only rods and spring isolators, while the reinforced phase maintained vertical vibration isolation and added rigid frames, bracing, and hinge-type seismic snubbers to provide horizontal restraint and multidirectional damping. Under the KPH95 benchmark, the difference between reinforced and baseline cases was pronounced. The baseline configuration exhibited excessive displacement, overturned during later testing stages, experienced rod bending and shearing near the anchorage, and repeatedly struck nearby sprinkler components. These findings indicate that the primary seismic vulnerability of suspended vibration-isolated equipment is due to inadequate displacement control, loss of lateral stability, and interaction with adjacent systems, rather than acceleration demand alone.

**Table 1.** Input motion set for NCREE tests.

| Category                  | Name      | Horizontal Intensity |            |
|---------------------------|-----------|----------------------|------------|
| Ground motion             | THH051R   | 0.547 g              | 5% to 100% |
| Structural response       | THHRF     | 0.939 g              | 5% to 90%  |
| Artificial Motion (AC156) | TACRF_CGT | 0.900 g              | 5% to 75%  |
|                           | TACRF_KPH | 1.264 g              | 5% to 95%  |



**Fig. 4.** Configuration of NCREE test (Phase I).

## Conclusions

This project has progressively established the technical foundation for a multinational testing and validation platform for suspended nonstructural systems. By combining seismic strategy studies, full-scale experiments, input-motion design and correction, and multinational execution planning, the project has formed a common basis for future seismic qualification and criteria development. The study also confirms that the central issue in suspended-system testing is the realistic reproduction of seismic demand

at specimen anchorage points because the supporting frame itself modifies the transmitted motion. Finally, the vibration-isolated equipment example demonstrates that inadequate lateral restraint leads to excessive displacement, instability, collision, and anchorage failure, while a strategy that preserves vertical isolation and adds horizontal restraint and damping can significantly improve seismic performance and system stability.

## Acknowledgements

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# Seismic Qualification Strategy for Large Equipment: A Case Study of a Wind Turbine Controller

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## Abstract

With advances in industrial technology and infrastructure, many systems and equipment are becoming larger in scale, more highly integrated, and more complex. When these large systems require seismic qualification, full-scale shaking table tests are often impractical due to limitations in facility size, capacity, or performance. Previous research has introduced a combined analysis and testing method for seismic qualification. This method divides large or complex equipment into a primary support structure, which is suitable for numerical simulation, and functional components, which require physical testing for verification. This approach addresses testing facility constraints while maintaining qualification accuracy and feasibility. Although current standards include combined qualification methods, many uncertainties remain regarding their practical implementation. In a previous shaking table test of a wind turbine nacelle, internal components such as the controller showed significant dynamic responses during seismic loading. The test data was also used to calibrate and validate the nacelle's finite element model (FEM). This paper uses the wind turbine controller as an example of combined seismic qualification for large equipment, applying simulated motions at the controller's anchorage point as input for shaking table tests on the controller.

**Keywords:** large equipment, shaking table test, system identification, finite element analysis

## Introduction

To evaluate the seismic performance of nonstructural components such as equipment, shaking table testing has long been considered one of the most intuitive and reliable methods. However, with advancements in industrial technology and infrastructure, many types of equipment and systems are becoming larger, more integrated, and more complex. For example, the capacity of individual wind turbines has increased from a few hundred kilowatts two decades ago to nearly 20 megawatts today, resulting in significant increases in the size and weight of the nacelle and associated equipment. Verifying the seismic performance of such large equipment is often impractical using full-scale shaking table tests due to limitations in facility size, payload capacity, or dynamic performance. Testing scaled models also introduces difficulties. Beyond validating scaling laws, it is challenging to determine whether the nonlinear behaviors observed in a scale model accurately represent those of

the full-scale prototype. Constructing very large testing facilities to accommodate such equipment is economically unfeasible. Therefore, the approach to seismic qualification for large equipment is expected to change. A combined qualification strategy that integrates numerical analysis with physical testing offers a way to overcome testing constraints while maintaining both accuracy and feasibility.

## Literature Review

Qualification techniques combining analysis and physical testing were initially developed in the 1970s to address the seismic qualification needs of equipment in nuclear facilities. Wood et al. (1979) focused on the main control boards of nuclear power plants, utilizing in-situ vibration testing to identify the modal shapes, frequencies, and damping ratios. This data was used to iteratively update the analytical model by adjusting the stiffness matrix until the modal parameters

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correlated sufficiently with the in-situ test results. By using the floor acceleration time history and the analytical model's transfer functions, the response spectrum at the location of maximum modal response was established.

Fischer et al. (1982a) used static deflection tests and forced resonant vibration tests on a typical single cabinet to validate their analytical model. The vibration tests induced resonance in assemblies of multiple cabinets using a motor actuator mounted on top of the cabinet. The literature recommended bottom springs to simulate the nonlinear behavior observed in the static deflection tests and suggested coupling tubes to connect cabinet models, with tube stiffness adjusted to match the resonance test results. This approach enabled the numerical model to be extended to modular multi-cabinet systems, and its seismic performance was validated by numerical analysis, demonstrating the model's conservatism. In another paper, Fischer et al. (1982b) described the procedure for verifying the seismic safety of multi-cabinet motor control centers using numerical models. After performing shaking table tests on the generic cabinet, they pointed out that the damping ratios at the top of the cabinet with and without overhead cables were 10% and 5%, respectively, while the fundamental frequencies remained similar. Following a calibration procedure similar to the previous study, the numerical model could be extrapolated from the generic cabinet to evaluate the seismic performance of new equipment. In addition to the customary comparison of the test response spectrum (TRS) and the required response spectrum (RRS) to ensure adequate strength, this paper also used the power spectral density ratio to ensure operability. From the generic cabinet tests, the transfer function between the cabinet base and the equipment mounting point was obtained. When the equipment is installed in a new seismic environment, this transfer function can be used to update the equipment's expected power spectral density (EPSD). If the new environment's EPSD is bounded by the original, and the new TRS is similarly enveloped, the equipment's functionality is assured. This procedure allows generic cabinet test data to be extrapolated to similar equipment and new seismic environments.

Hendricks et al. (1983) studied insulation systems, which typically use long, vertical cantilever bushings. The primary seismic concern is the relative movement between bushings, and inertial forces may cause excessive stress in ceramic or phenolic materials. Due to the complex geometry and materials, accurate analytical modeling is challenging and relies heavily on engineering assumptions. Although full-scale testing is the most reliable method, the size, weight, and cost of the equipment often make it impractical. Therefore, combining testing and analysis can take advantage of both approaches. The recommended steps included: (1) using portable exciters for low-level testing in the laboratory or in-situ to determine fundamental behavior

and seismic sensitivity; (2) defining structural failure modes that could impair functionality based on test results; (3) calibrating the analytical model using test data, and determining natural frequencies, mode shapes, operating stresses, and deflections that could cause failure; and (4) performing seismic analysis using the numerical model. For example, in-situ testing of circuit breakers and switchgear provided transfer functions. Due to material and system complexity, the equipment showed complex vibration modes. The frequencies and damping ratios identified from the transfer functions could be used to verify or adjust the numerical models.

After 2000, Sinha et al. (2000) used a 370-ton horizontal storage tank to demonstrate how to determine dynamic characteristics using low-level in-situ excitation. Although the test data may underestimate the damping ratio because of field support conditions, this was considered an acceptable conservative assumption.

Gatscher et al. (2010) applied a combined analysis and testing method to large non-nuclear equipment such as industrial UPS systems, cooling towers, and medical devices. Compared to the traditional focus on maintaining structural position, ensuring post-earthquake functionality has become an increasing priority. To address this, large equipment was divided into a mechanical subsystem requiring position retention and functional components requiring operability. The mechanical subsystem was verified through analysis, while the functional components were evaluated using response spectrum and random vibration testing. The paper noted that response spectrum testing, which follows methods similar to those in the IEEE 344 standard, may not excite higher mode responses, and the selection of test waveforms can vary between practitioners. To better capture the energy distribution and statistical properties of seismic input, acceleration spectral density (ASD) testing was also introduced. ASD testing is more conservative than response spectrum testing, so care is required to avoid over-testing.

Eljadei et al. (2017) illustrated a seismic qualification method combining testing, analysis, and similarity using pressure blowers and battery racks as case studies. By confirming that the target equipment and the numerical model had similar dominant frequencies via shaking table data and could withstand design earthquakes while maintaining functionality, the results could be extrapolated to similar equipment models.

Regarding testing standards, due to the limited availability and specialized nature of nuclear equipment, widely used seismic qualification standards, such as IEEE/IEC 60980-344 (2020) and ASME QME-1, allow seismic qualification through a combination of testing and analysis. For example, IEC/IEEE 60980-344 divides shaking table testing into Assembly and Component/Subcomponent levels:



- **Assembly-level testing:** Performed on an assembly with all available subcomponents to verify structural integrity and position stability. If absent subcomponents significantly affect the dynamic response, dummy weights are used to simulate them, and accelerometers are installed at the intended mounting points. Subcomponents should be mounted to withstand maximum deflection or stress cycles, ensuring that measured resonant frequencies reflect the full assembly. During the vibration test, transfer functions are derived from the control accelerometers to the subcomponent mounting points to obtain the in-cabinet RRS.
- **Component/Subcomponent-level testing:** Unqualified subcomponents are mounted to the shaking table or a rigid fixture and subjected to excitation. The resulting TRS shall envelop the derived in-cabinet RRS.

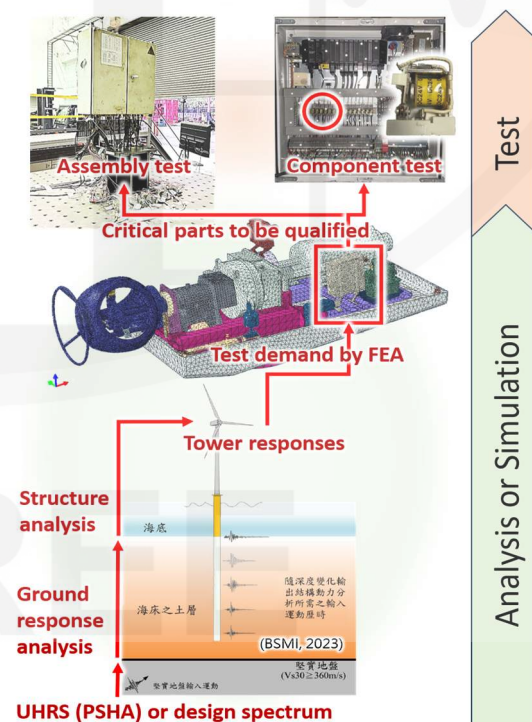
### Combined Seismic Qualification Strategy

Based on the literature review, the combined approach of analysis and testing mainly uses in-situ or laboratory test data to identify dynamic parameters, such as modal frequencies and shapes, to update or calibrate the numerical model. This differs from standard test procedures, which primarily aim to ensure that safety-critical components function during and after an earthquake. In this approach, modal testing calibrates the numerical model, which is then used to generate the input response at the mounting location of the critical component. This calculated input is applied during component testing to verify operability. The main objective is to qualify large equipment that cannot be fully tested or complex equipment whose functionality cannot be confirmed by analysis alone. This method focuses testing or analysis on critical features or seismic vulnerabilities, eliminating the need to test every design variation or individual component. The primary steps are screening components and conducting analysis or testing based on their characteristics (2020):

1. Screening for critical or fragile functional devices or mechanical components within the equipment;
2. Developing the numerical analysis model of the equipment and extracting dynamic parameters via modal testing to calibrate the model;
3. Establishing functionality acceptance criteria for the critical components;
4. Executing the qualification:
  - A. If a critical component's functionality is based on structural integrity or position retention, its seismic performance can be verified analytically using numerical models;

- B. If the critical component is an electrical or active mechanical device whose operability cannot be verified analytically, numerical analysis is used to derive the seismic response at its mounting location. A bounding RRS or ASD is established to conduct shaking table testing, ensuring functionality is maintained under assumed seismic conditions.

Taking the shaking table test of a wind turbine nacelle as an example, the test specimen, a Vestas V47-660 kW, is a product from over 20 years ago; however, its size already approaches the limits of the shaking table. Newer or larger models will require the combined method. This test utilized artificial seismic waveforms matching the design response spectrum given by the Taiwan building code at its original site to simulate tower responses, which were also used as input motions for the shaking table test. The testing results indicate that the controller exhibited significant dynamic responses, leading to its selection as the target for further seismic qualification research. Concurrently, a finite element model (FEM) of the complete nacelle was developed and calibrated using system parameters identified from white-noise testing, yielding reliable simulated responses. Subsequently, when conducting seismic qualification for the controller, FEM was used to simulate the vibrational response at the interface between the controller and the main nacelle supporting structure, or the interface between the controller and the subcomponent inside. This was used to determine the shaking table input motions for component testing. The workflow is illustrated in Fig. 1.



**Fig. 1.** Process for seismic qualification by combined analysis and testing (2023)

## Conclusions

To meet the seismic qualification requirements for large equipment, a combined strategy that integrates numerical simulation and physical testing offers a balanced approach in terms of cost, practicality, and reliability. Key factors for successful qualification include:

- Screening components suitable for simulation versus those requiring physical testing. However, there is no clear criterion. Generally, supporting structures whose goal is position retention and are easy to model are simulated, while complex components providing critical functions are tested.
- Ensuring correlation and conservatism between the numerical model and the actual equipment. The model shall be calibrated using system characteristic parameters obtained from local or global system identification testing. Literature indicates that low-level excitation tests may underestimate damping ratios, inherently adding conservatism to numerical results. Additional conservatism can be achieved by applying safety factors or enveloping the RRS/ASD.

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# Seismic Behavior of the Connection between a Shear Link Beam and Reinforced Concrete Members

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## Abstract

This study primarily investigates the seismic behavior of the connection between steel link beams and reinforced concrete (RC) members. The present goal is to replace the poorly constructible RC shear coupling beam with a steel shear-yielding link beam and to develop design methods for the shear deformation capacity and shear strength capacity of the link beam. This study fabricated four specimens using different connection methods, including simulated welding, precasting, and site fabrication, and conducted seismic tests. The test results show that the core segment of each specimen exhibits full and stable shear force-deformation hysteresis loops and effective energy dissipation behavior. Through bolt anchoring at the ends of the steel link beam, together with an appropriate extension length, no anchoring failure occurred at the ends. This research also proposes design suggestions and performance evaluation criteria for the connections between steel link beams and members. Based on the observations from this experiment and the results of the preliminary test, a design process for this type of steel link beam is provided, offering a practical reference for the seismic design of steel and RC hybrid structures, which will help promote the wider application of steel link beams in building structures.

**Keywords:** link beam, energy dissipation, extension length, hybrid

## Introduction

In traditional reinforced concrete (RC) frames, the coupling beams between shear walls are often short-span RC beams, and diagonal main bars are often used to meet high shear force requirements. While such beams can improve shear resistance, they require a large number of diagonal main bars during construction, which makes steel bar binding difficult. The area where the main bars are installed is extremely crowded and prone to quality defects such as honeycombing and voids. Furthermore, short-span RC beams are highly susceptible to concrete shear cracking after repeated earthquakes, and their shear energy dissipation capacity and ductility are difficult to maintain consistently, which can easily lead to brittle shear failure. Therefore, if they are designed in combination with coupling beams that have stable energy dissipation characteristics, the overall ductility and seismic performance of the frame can be improved. The link beam in this study replaces the RC coupling beam and the shear link of the eccentrically braced frame, generating a shear plastic hinge in the web, as shown in Fig. 1. The force and deformation mode of

the link beam, under the assumption that the connection segment remains elastic, are the same as those of the seismic stud column (SSC) without axial force; a comparison of the force models is shown in Fig. 2. If the required interstory drift is known, the shear deformation demand of the link beam can be obtained from the structural geometric deformation. Previous studies on SSCs have indicated that the shear deformation capacity ( $\gamma_c$ ) of the core segment with a shear plastic hinge is negatively correlated with the short side width–thickness ratio ( $\lambda_{ssc}$ ) of the core segment, with the ratio ( $\delta$ ) of the core segment length to the overall specimen length limited to 0.2–0.6.

The steel link beam in this study can be prefabricated in the factory, reducing on-site reinforcement and grouting work. It has good workability and quality-control potential, which is of substantial benefit to engineering applications. In this study, four full-scale shear-yielding link beam specimens were designed and then simulated to be connected to RC members using different connection methods. Cyclic loading tests were conducted using the multi-axial test system (MATS) in the large structural laboratory at the National Center for

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Research on Earthquake Engineering, National Institutes of Applied Research (hereinafter referred to as “NCREE”) to investigate the overall seismic behavior of the connection between the link beam and the RC member. The aim is to provide a basis for the detailed design of the connection between the shear-yielding link beam and the RC member.

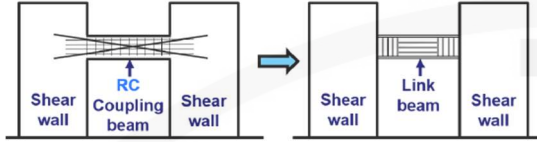


Fig. 1. Schematic diagram of an RC coupling beam replaced by a link beam

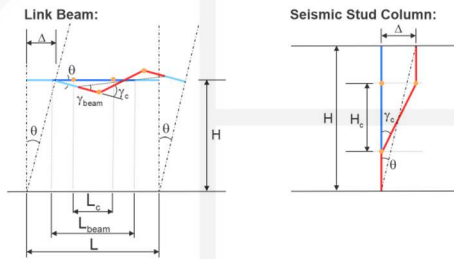


Fig. 2. Force model and structural geometric deformation of the SSC and link beam

## Research Methods

In this study, the seismic performance of the connection between the steel link beam and the RC member was mainly evaluated through four cyclic tests conducted on full-scale specimens. The specimens consisted of a steel link beam, RC bases (members), end plates, and shear tenons. Some specimens were constructed using steel fiber-reinforced concrete. The dimensions of the four specimens are shown in Fig. 3. All cross-sections were BH 800 × 300 × 6 × 25, and the material was A36. The core segment of the link beam was equipped with a longitudinal stiffener to form a shear sub-segment. After stiffening the core segment of each specimen, the width–thickness ratio of the short side of the core segment  $((b/t)_c)$  was 31.25. This value is the dimensionless width–thickness ratio ( $\lambda_{ssc}$ ) control parameter, and the ratio of the width–thickness ratio of the core segment to that of the joint segment  $(b/t)_c/(b/t)_j$  was 1.2. The standard width of the stiffener plate was 120 mm. The stiffener plate located in the end sub-segment was enlarged to 132 mm to meet the design requirements for larger bending moments in this area.

The four specimens are named CBW, CBS, CBM, and CBF, where the letters CB represent coupling beam. The last letter of each specimen's name represents the connection method between the steel coupling beam and the base: W represents welding, S represents site fabrication, M represents a matrix-modified structure using 70 MPa concrete, and F represents a fiber-modified structure using high-

toughness steel fiber materials. The steel link beams are divided into two categories based on the connection method: 2,350 mm and 2,500 mm. The ratio of the length of the core segment to the total length is 0.61 and 0.58, respectively. The 2,500 mm value is the design length of the CBS specimen, simulating the direct placement of the steel link beam between the RC members. The 2,350 mm value is the length of the precast specimen, which is slightly shorter because of the need to reserve installation space and account for the thickness of the welding plate. The main steps in the design of the connection between the steel link beam and the RC members are to determine the design bending moment at the connection, the strength of the anchoring reinforcement, and the design strength of the end plate and stiffener. Unlike traditional steel structure connections that use high-strength bolts, all specimens in this study used threaded steel bars instead of traditional anchor bolts as the main anchoring members for transmitting end bending moment and shear force. The design of the end plate thickness refers to the design procedures and related formulas suggested in AISC Design Guide 4: Extended End Plate Moment Connections, provided by the American Institute of Steel Construction.

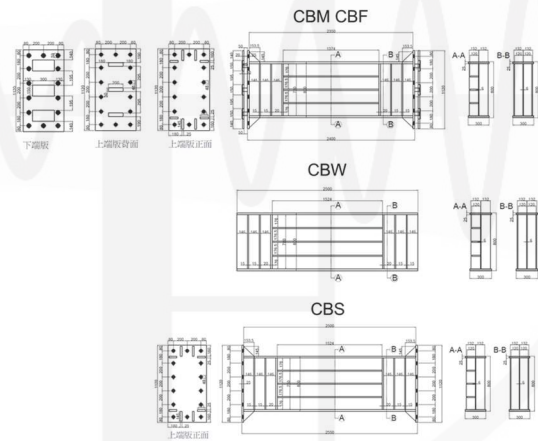


Fig. 3. Schematic diagrams of the link beam specimens.

All tests were conducted using MATS. The specimen height, including both the top and bottom fixtures, was 3,500 mm. The top and bottom ends of the specimens were fixed to the reaction frame and loading platform of MATS using an appropriate number of prestressed steel rods. On each of the south and north sides of the specimens, 1,200 kN vertical actuators were arranged. In addition, two hydraulic servo actuators were arranged horizontally to apply shear force to the specimens. One end of each actuator was connected to the loading platform, while the other end was fixed to the reaction wall. The actuators were controlled using the MATS control system, and all tests were conducted by applying interstory drift loading in accordance with the American Institute of Steel Construction (AISC) 341-22 sequence for beam-to-column moment connections at the bottom of the



fixed specimens. Before this test sequence, additional interstory drifts of 0.125% and 0.25% were applied. The overall configuration of the link beam test is depicted in Fig. 4. In these tests, displacement and strain measurements were taken at specific points using an optical image measurement system and strain gauges. The horizontal displacement difference between the top and bottom endpoints of the core segment was measured using a lateral displacement sensor. This displacement difference was divided by the core segment length to obtain the shear deformation angle ( $\gamma$ ) of the core segment. During the test, the timing and process of yielding in each part of the specimen were observed, and the angle and behavior of the tension field in the core segment web under shear force were measured and recorded.

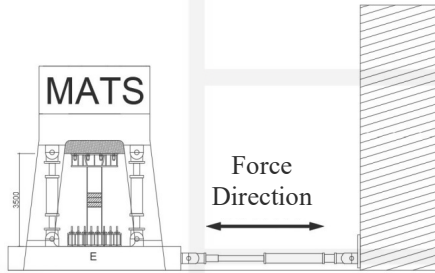


Fig. 4. Test configuration for the link beam specimens

## Test Results and Discussion

The failure modes of all specimens in this study involved concave folds and fractures due to repeated tension- and compression-induced buckling, with ruptures occurring at the web center and edge welds of the web panel. The failure conditions and the hysteresis loops of deformation and strength for the four specimens are shown in Figs. 5–8. The maximum deformation in the tests was determined as follows: (1) throughout the entire loading process, the test strength remained above 80% of the maximum shear strength of the link beam; (2) no significant shrinkage was observed after at least one complete cycle; and (3) the average of the maximum positive and negative deformation was used to determine the result. The test results indicate that the maximum overall drift angles of the link beam for specimens CBW, CBS, CBM, and CBF were 6.0%, 5.9%, 5.8%, and 5.8% radians, respectively, while the maximum shear deformation angles of the core segments of the four specimens were 8.2%, 8.4%, 9.1%, and 9.3% radians, respectively.

For the four specimens in this study, the shear strain hardening factors of the web of the core segment were calculated by dividing the maximum shear strength from the test by the actual plastic shear strength of the core segment, resulting in values of 1.45, 1.44, 1.42, and 1.43. All specimens in this study had longitudinal stiffeners installed in the web of the core segment. Based on the experimental results, the specimens with longitudinal stiffeners configured only in the web of the core segment were selected for comparison, as shown in Fig. 9. It is clear that the

larger the normalized width–thickness ratio, the smaller the strain hardening factor tends to be.

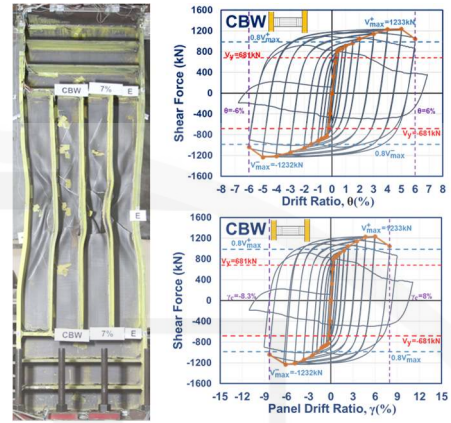


Fig. 5. Failure conditions and hysteresis loops of deformation and strength for specimen CBW

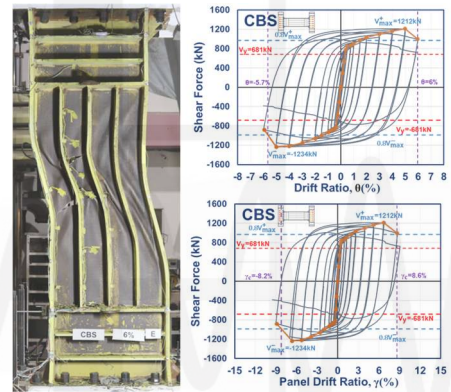


Fig. 6. Failure conditions and hysteresis loops of deformation and strength for specimen CBS

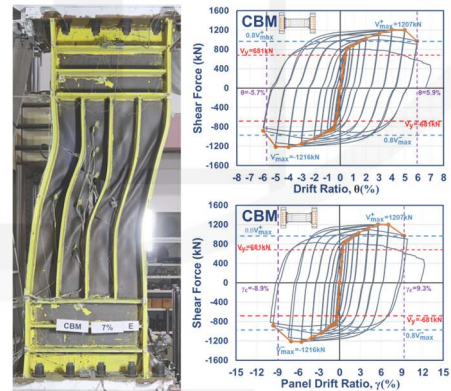
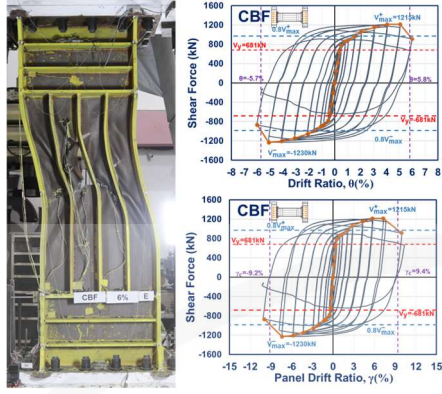
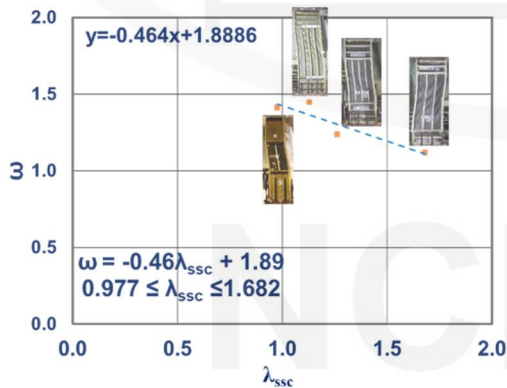


Fig. 7. Failure conditions and hysteresis loops of deformation and strength for specimen CBM

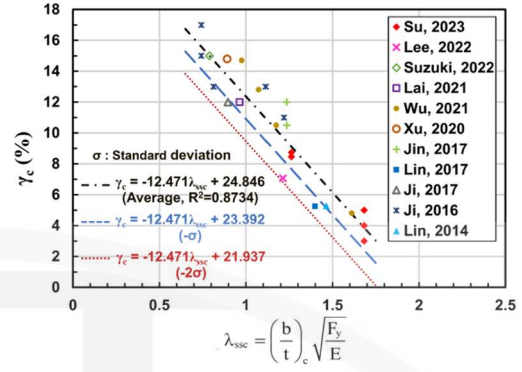


**Fig. 8.** Failure conditions and hysteresis loops of deformation and strength for specimen CBF

To make this study more widely applicable, we not only discuss the four shear-yielding link beam specimens but also incorporate test data from other SSC and steel link beam specimens reported in the literature. Using linear regression analysis, we found an approximately linear relationship between the shear deformation capacity ( $\gamma$ ) of the web of the core segment before the development of the tensile field and the dimensionless width–thickness ratio limit ( $\lambda_{ssc}$ ) in the short direction of the specimen web. This relationship is illustrated in Fig. 10 and simplified by the equation  $\gamma = -12.5\lambda_{ssc} + 23.0$ . This indicates that the smaller the short-side width–thickness ratio of the core segment web, the larger the shear deformation capacity. The shear deformation demand ( $\gamma_{dem}$ ) of the core segment was estimated based on the relationship between the interstory drift demand ( $\theta_{dem}$ ) and the ratio ( $\delta$ ) of the length of the core segment to the net length of the specimen (calculated as  $\gamma_{dem} = \theta_{dem}/\delta$ ). Based on the aforementioned relationship between shear deformation capacity ( $\gamma$ ) and the dimensionless short-side width–thickness ratio ( $\lambda_{ssc}$ ) of the core segment web, the core segment stiffener of the shear-yielding link beam can be accurately designed to provide effective shear deformation capacity.



**Fig. 9.**  $\omega$ - $\lambda_{ssc}$  relationship diagram of the specimens with only longitudinal stiffeners at the core segment



**Fig. 10.** Relationship between the maximum shear deformation and the width–thickness ratio of the short side of the link beam

## Conclusions and Suggestions

1. The cross-sections of the four steel link beam specimens in this study are identical, differing only in the connection and stiffening methods of the end link beam; the  $\lambda_{ssc}$  of the core segment and the joint segment are the same. The test results indicate that the entire link beam section enters shear yielding, but the final shear deformation is concentrated in the core segment with longitudinal stiffener plates.
2. For shear-yielding link beams without axial force, the core segment shear deformation capacity design formula  $\gamma_c = -12.5\lambda_{ssc} + 23$  proposed in this study can be used to determine the shear deformation capacity of the core segment. When a link beam designed using this formula reaches the expected core shear deformation, its shear strength can be maintained at 80% of the maximum shear strength.
3. The average ratio of the shear deformation capacity ( $\gamma_c$ ) of the core segment to the overall shear deformation capacity ( $\gamma_{beam}$ ) of the four specimens is 1.47. This indicates that the actual overall drift deformation capacity of the link beam is approximately equal to  $1.5\gamma_c$ , and the  $\delta$  value is between 0.58 and 0.61.

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# Methodology Developed for Improving the Performance of the On-Site Earthquake Early Warning System

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## Abstract

This study utilizes support vector machines (SVMs) to develop an optimized on-site earthquake early warning model. Using structural monitoring data from the National Center for Research on Earthquake Engineering building, an earthquake early warning model for strong floor vibration in a high-rise building, applicable to this structure, was developed as a test case. This early warning model adds a signal identifier after system triggering and employs an artificial intelligence prediction model with physical constraints. Its operational performance under low-alert and high-alert scenarios was tested. Analysis results confirm that the earthquake early warning model for strong floor vibration of a high-rise building developed using this method exhibits the best performance in terms of reporting success rate during earthquakes, false alarm rate during earthquakes, and false alarm rate during peacetime. The estimated additional computation time required is approximately 0.01 seconds. The findings of this study can be applied to optimize existing or newly constructed customized on-site earthquake early warning systems, which are expected to significantly improve operational performance and effectively expand their application areas and targets, thereby mitigating the overall social and economic impact of earthquakes in Taiwan.

**Keywords:** On-Site Earthquake Early Warning System, Support Vector Machine

## Introduction

The prediction models of the local earthquake early warning systems widely used in Taiwan can be mainly divided into methods using a single parameter with a physical basis, such as the peak amplitude of a 3-second P-wave (Wu et al., 2005), and methods using artificial intelligence, such as training a multi-parameter input model using support vector machines (SVMs) (Hsu et al., 2013). These methods were developed using historical earthquake motion data observed throughout Taiwan. Due to the inherent uncertainty of earthquake motion, model predictions cannot be completely accurate, which may lead to missed alarms, false alarms, and even false alarms during normal times. These situations occur more frequently at low alert levels, thus limiting the application areas and scenarios of local earthquake early warning systems. For the reasons mentioned above, this study aims to utilize SVMs to develop an optimized approach for on-site earthquake early warning systems. Using structural monitoring data from the National Center for Research on Earthquake Engineering building, an earthquake

early warning model for strong floor vibration of a high-rise building, applicable to this structure, was developed as a test case. This early warning model incorporates a signal identifier after triggering and employs an artificial intelligence (AI)-assisted alarm mode with physical constraints. Its operational performance was tested under low-alert and high-alert levels.

Analysis results confirm that the earthquake early warning model for strong floor vibration in high-rise buildings, developed using this method, exhibits the best performance in terms of earthquake-time reporting success rate, false alarm rate during earthquakes, and false alarm rate during peacetime. The estimated additional computation time required is less than 0.01 seconds. The findings of this study can be applied to optimize existing or planned new customized on-site earthquake early warning systems, significantly improving their operational performance and effectively expanding their application areas and targets, thereby mitigating the overall social and economic impact of earthquakes in Taiwan.

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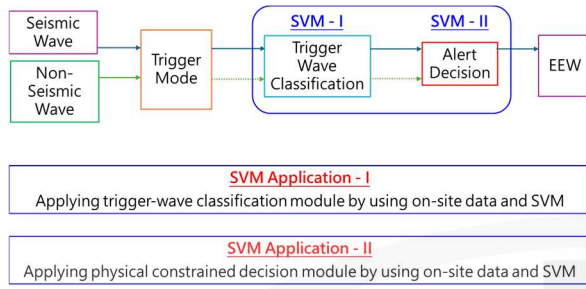


Fig. 1. A conceptual diagram illustrating the implementation process of the proposed method.

## Methodology

This section briefly explains feasible methods for improving the effectiveness of early warning models using SVMs, including reducing false alarms during peacetime and earthquakes. A conceptual diagram illustrating the implementation process and operation is shown in Fig. 1.

1. Adjusting the early warning model based on the seismic characteristics and warning targets of the target area: Due to significant regional differences in seismic characteristics and varying warning targets across target areas, customizing the input parameters of the early warning model based on the above-mentioned methods is the most fundamental way to improve its effectiveness. This relies on the complete collection of vibration data from the target area during peacetime and earthquakes, and the clear formulation of warning targets.
2. Importing triggered signal classification (TSC): Compared with regional early warning systems, local early warning systems cannot utilize multi-point detection to distinguish whether a trigger signal is a seismic wave or a localized environmental disturbance. Furthermore, in low-alert-level scenarios, increasing the trigger threshold does not reduce non-seismic-wave triggers. Therefore, importing a signal classifier into the early warning mode is necessary to reduce false alarms in low-alert-level scenarios.
3. Employing an AI-powered early warning module with physics-based constraints: With the increasing computing power of computers and the continuous accumulation of seismic motion observation data, using AI technology to improve the accuracy of early warning models in vibration signal classification and alarm decision-making is imperative. However, the potential non-reproducibility of seismic motions means that AI-developed early warning models may still be unsuitable for future earthquake scenarios. Therefore, using physics-based parameters as constraints when applying AI to signal classification and alarm decision-making can effectively reduce this risk. A conceptual illustration is shown in Fig. 2.

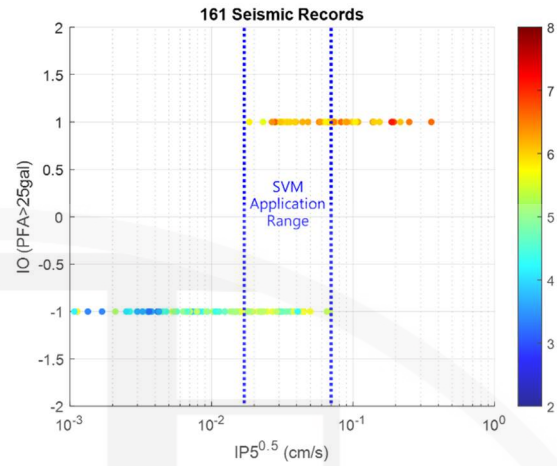


Fig. 2. A conceptual diagram illustrating the AI-prediction module with physics-based constraints.

## Verification Case Study

This study collected 7,416 trigger records from the structural monitoring system of the National Center for Research on Earthquake Engineering buildings during its operation from early February 2021 to the end of August 2025. Using the method, a high-rise strong earthquake early warning system applicable to this structure was developed, and its operational effectiveness was tested. The warning criteria were that the maximum peak acceleration response of each floor slab exceeded 2.5, 8.0, and 25 gal, corresponding to the threshold values of ground seismic intensity levels 2, 3, and 4, respectively.

To distinguish whether each trigger record was caused by seismic waves or other environmental disturbances, this study used the earthquake catalog published by the Central Weather Administration as a benchmark, comparing the trigger time of each trigger record with the earthquake occurrence time to determine whether the record was caused by seismic waves. Ultimately, 161 records were identified as triggered by earthquakes, while the rest were attributed to others e.g. environmental disturbances.

This study first selects the optimal single parameter for the trigger signal classification and alarm numerical model based on the records. Then, using this single parameter as a constraint, SVM technology is employed to train and optimize the signal classifier and alarm decision model required for the early warning model. The signal classifier is trained using all records, while the alarm decision model is trained using only earthquake-triggered records. After the models are trained, the operational effectiveness of the early warning model during earthquakes and in peacetime is tested using all trigger records.

To train the SVM to achieve optimal signal classification and alarm decision results, this study first se-



lects several parameter sets that can represent independent signal characteristics. Then, their effectiveness is tested using field observation data to select the best-performing signal feature set. The selected signal feature parameters can be divided into single parameters with units and normalized vector parameter sets without units, as listed below:

Single parameters with units:

1. Pa3: Peak Acc.
2. Pv3: Peak Vel.
3. Pd3: Peak Disp.
4. Tauc3: Effective Predominant Period
5. CAV3: Cumulative Absolute Vel.
6. ISV3: Integral of Square Vel.

7. IP3(T1): Instantaneous Power

Note: 3 means 3-second signal after triggering

Normalized vector parameter sets without units:

1. NFA: Normalized Fourier Amplitude spectrum (Sum = 1)
2. CAI: Cumulative Arias Intensity (Max. = 1)
3. NSV: Normalized Singular Value (Sum = 1) (from Singular Spectrum Analysis with  $L = 600$ )
4. HUP: Hilbert Unwrapped Phase
5. HIA: Hilbert Instantaneous Amplitude (Max. = 1)
6. NTH: Normalized Time History (Max. Abs. = 1)
7. EMD: Padding Intrinsic Mode Functions (IMFs) of NTH from Empirical Mode decomposition (EMD) one by one (from high to low frequency)
8. WPT: Padding reconstructed wavelet packet transform (WPT) of NTH (from high to low frequency, with  $N = 2^3$ )
9. NUM: Padding normalized unitary matrix (NUM) U from SSA (from high to low singular value with  $L = 100$ )
10. SLA: Short over Long term Average (SLA) time history calculated by Short-Term Average (STA) time histories over Long Term Average (LTA) time history

The single parameter is calculated using the signal from 3 seconds after triggering, while the unitless normalized vector parameter is calculated using the signal from 3 seconds before to 3 seconds after triggering. The optimal training parameters are expected to vary depending on the seismic characteristics of the

application area and the specific warning target.

Table 1 Performance comparison of different early warning models developed for two alert standards.

| On-Site<br>EEW<br>Module | Threshold<br>Values | PFA > 2.5 gal (Low)      |                          |                             | PFA > 25 gal (High)      |                          |                             |
|--------------------------|---------------------|--------------------------|--------------------------|-----------------------------|--------------------------|--------------------------|-----------------------------|
|                          |                     | Seismic<br>S-Sate<br>(%) | Seismic<br>F-Rate<br>(%) | Usual<br>F-Rate<br>(Counts) | Seismic<br>S-Sate<br>(%) | Seismic<br>F-Rate<br>(%) | Usual<br>F-Rate<br>(Counts) |
| Pa3                      | Low                 | 100.0%                   | 94.1%                    | 6737                        | 100.0%                   | 39.5%                    | 227                         |
|                          | Medium              | 69.4%                    | 0.0%                     | 392                         | 28.6%                    | 0.0%                     | 0                           |
|                          | High                | 86.1%                    | 5.9%                     | 1489                        | 61.9%                    | 9.2%                     | 15                          |
| IP3                      | Low                 | 100.0%                   | 82.4%                    | 6539                        | 100.0%                   | 26.1%                    | 3                           |
|                          | Medium              | 59.7%                    | 0.0%                     | 4                           | 40.5%                    | 0.0%                     | 0                           |
|                          | High                | 70.8%                    | 5.9%                     | 23                          | 73.8%                    | 9.2%                     | 0                           |
| IP3 +<br>TSC_SVM         | Low                 | 93.1%                    | 23.5%                    | 6                           | 100.0%                   | 26.1%                    | 1                           |
|                          | Medium              | 59.7%                    | 0.0%                     | 2                           | 40.5%                    | 0.0%                     | 0                           |
|                          | High                | 70.8%                    | 5.9%                     | 3                           | 73.8%                    | 9.2%                     | 0                           |
| IP3 SVM                  | —                   | 100.0%                   | 0.0%                     | 4626                        | 100.0%                   | 0.0%                     | 149                         |
| IP3_SVM +<br>TSC SVM     | —                   | 93.1%                    | 0.0%                     | 6                           | 100.0%                   | 0.0%                     | 1                           |

Table 1 presents a performance comparison of different local early warning models developed for two alert standards, including the success rate of alarm transmission during earthquakes, the false alarm rate during earthquakes, and the total number of false alarms during normal times. Five local early warning models were compared, including a simpler model and one obtained after the phased application of the optimization method proposed in this study. Their establishment methods and performance are described below:

1. Pa3: This model uses only the peak acceleration of the 3-second P-wave as the basis for alarm decision-making. The analysis results show that regardless of how the alarm threshold is set, the optimal success rate and false alarm rate cannot be obtained during earthquakes, and the total number of false alarms during normal times is quite high. These excessively frequent false alarms will limit practical applications. (This example is used only as a control group and is not a real-world application case.)
2. IP3: Only the instantaneous power peak of the 3-second P-wave (Chao et al., 2023) is used as the basis for alarm decision. Since it incorporates the structure's fundamental vibration period into the calculation of instantaneous power, it can achieve a better success rate and false alarm rate during earthquakes than the Pa3 mode. The total number of false alarms is low under normal conditions. However, under the condition of zero false alarms during earthquakes, its success rate can only reach 60%, so the excessively high rate of missed alarms also leads to limitations in practical applications.
3. IP3+TSC\_SVM: After applying the trigger signal identifier/classifier (TSC), IP3 is used for alarm decisions. The analysis results show that this method can significantly reduce false alarms during normal times, especially at low alert levels, where it can significantly improve the effectiveness of the early warning system. However, the early warning effect during earthquakes cannot be improved.

4. IP3\_SVM: In some situations, an AI model trained using an SVM is used for alarm decision-making. The analysis results show that it can significantly improve the early warning effect during earthquakes and can overcome the limitations of using a single parameter for early warning effectiveness. However, it cannot reduce the number of false alarms under normal conditions.
5. IP3\_SVM+TSC\_SVM: This method simultaneously applies to a trigger signal identifier/classifier and the AI model for alarm decision-making. Analysis results confirm that this approach comprehensively improves the performance of local early warning systems during both earthquake-prone and normal operation, demonstrating significant effectiveness in both low-alert and high-alert scenarios.

Compared with alarm decision-making methods using single parameters, using a trigger signal identifier and an AI model developed with SVMs requires additional computation. This study tested the computation time on a typical desktop personal computer. The trigger signal identifier required approximately 0.006 seconds, and the AI-powered alarm decision required approximately 0.004 seconds. Therefore, using the optimized method in this study, the system alarm time needs to be increased by approximately 0.01 seconds. This time difference should still meet the requirements of most application scenarios.

## Conclusions

This study utilizes SVMs to develop an optimized on-site earthquake early warning model. The optimized model adds trigger signal identification and uses an AI model with physical constraints for alarm decision-making.

To confirm the feasibility and effectiveness of the optimization technology, this study used structural monitoring data from the National Center for Research on Earthquake Engineering building to develop an on-site earthquake early warning model for strong floor vibration of a high-rise building applicable to this structure as a test case. The model's operational performance under low-alert and high-alert scenarios was tested, including its earthquake-time reporting success rate, earthquake-time false alarm rate, and the number of false alarms during normal times. The analysis results confirm that the earthquake early warning model for strong floor vibration of a high-rise building developed using this method exhibits the best performance in terms of earthquake-time reporting success rate, earthquake-time false alarm rate, and normal-time false alarm rate. The estimated additional computation time is approximately 0.01 seconds, which still meets the needs of most application scenarios.

The findings of this research can be applied to optimize existing or newly developed customized on-site

earthquake early warning systems, which are expected to significantly improve operational efficiency and effectively expand their applicable fields and targets, thereby mitigating the overall social and economic impact of earthquakes in Taiwan. However, the optimization techniques developed in this study still rely on observational data. Therefore, when applied to newly constructed early warning systems, only the triggering records of surrounding strong-motion stations can be used for preliminary development, and the impact on effectiveness still needs further evaluation. In addition, this technology is mainly based on SVMs. Considering that earthquake ground motion may be characterized by non-reproducibility, it is recommended that the effectiveness of the optimized early warning system be tracked over a long period of time, and that the need for further adjustments be continuously evaluated as observational data accumulates.

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# Strength Matching Between Confinement Reinforcement and Concrete in RC Columns Under High Axial Compression

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## Abstract

According to the seismic design provisions of ACI 318-19, columns subjected to high axial compression exceeding  $0.3 f'_c A_g$  require a strict demand on transverse confinement reinforcement depend on the axial compression, and all longitudinal reinforcement must be confined by the transverse reinforcement. The maximum allowable strength of transverse reinforcement for confinement is 690 MPa. In addition, columns must achieve a target deformation capacity corresponding to a 3% radian drift ratio. The current seismic design requirements for columns in the Taiwan Concrete Structure Design Code are based on those of ACI 318-19. According to these codes, the required amount of confinement reinforcement for columns under high axial compression decreases as the ratio of reinforcement yield strength to concrete compressive strength,  $n_c (= f_{yt}/f'_c)$ , increases. There are no additional design restrictions. However, this design requirement is not reasonable and does not ensure that the column is able to provide the target deformation capacity. In this paper, a total of seven specimens were tested to investigate the influence of  $n_c$  on columns under high axial compressive load. For all specimens, the design  $n_c$  values were ranged from 6.0 to 24.6 based on the material limitations of the Taiwan Code, and the actual  $n_{ca}$  (actual concrete strength,  $f'_{ca}$ , replaced design strength,  $f'_c$ ) values were between 4.7 and 19.2. Experimental results demonstrate that the deformation capacity of the columns decreases as the  $n_{ca}$  value increases. It is also found that when the  $n_{ca}$  value exceeds 13, the specimens failed to achieve the target 3% radian drift ratio. Based on these limited research results, this paper proposes that the  $n_c$  value should not be greater than 12 to obtain a conservative column deformation capacity. Furthermore, specimen C1 exhibited a good deformation capacity, exceeding the 3% radian drift ratio, even under 1.39 times the prescribed axial load. This result indicates that the confinement reinforcement requirement of the column with a lower  $n_c$  value (8.9) is overly conservative.

**Keywords:** RC column with high axial compression, transverse confinement reinforcement, strength ratio of transverse confinement reinforcement to concrete strength,  $n_c$ , seismic deformation capacity

## 1. Introduction

According to the ACI 318-11 building code and earlier versions, the required amount of seismic transverse confinement reinforcement for reinforced concrete (RC) columns must comply with two equations:

$$R_{t,a} = 0.3 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}} \quad (1)$$

$$R_{t,b} = 0.09 \frac{f'_c}{f_{yt}} \quad (2)$$

where,  $A_g$  is the gross cross-sectional area of the column;  $A_{ch}$  is the cross-sectional area of the member measured to the outside edges of the transverse reinforcement;  $f'_c$  is the design

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compressive strength of concrete; and  $f_{yt}$  is the design yield strength of the transverse reinforcement. Elwood et al. (2009a) examined the relationship between transverse confinement reinforcement and deformation capacity for columns by analyzing a database of 145 column specimens subjected to various axial compressive loads. The research results demonstrated that the deformation capacity of RC columns is significantly influenced by axial load levels. The column deformation capacity gradually diminishes as its axial compressive load increases. Consequently, Elwood et al. (2009b) proposed a design equation for seismic columns, suggesting that the required amount of confinement reinforcement should increase with the axial compressive load to provide a target deformation capacity of a 3% radian drift ratio. ACI 318-14 (ACI 2014), referencing Elwood et al. (2009b) research results and current Canadian codes, introduced a new requirement for the amount of transverse confinement reinforcement  $R_{t,c}$ , shown as Eq. (3), for columns with a high design axial force,  $P_u$ , greater than  $0.3f'_c A_g$ .

$$R_{t,c} = 0.2k_f k_n \frac{P_u}{f_{yt} A_{ch}} \quad (3)$$

where,  $k_f$  is the concrete strength factor, which must be greater than 1.0 when the concrete strength exceeds 70 MPa;  $k_n$  denotes the confinement effectiveness factor, which is equal to  $n_l/(n_l - 2)$ ; and  $n_l$  is the number of longitudinal bars laterally supported by the corners of hoops or seismic hooks. Additionally, every longitudinal bar must be laterally supported by transverse reinforcement. This differs from the requirement in ACI 318-11 and earlier versions, which only required every other longitudinal bar to be laterally supported. For the upper limit of the design strength of transverse confinement reinforcement for seismic design, ACI 318-11 sets the limit at 420 MPa, while ACI 318-19 substantially increases this limit to 690 MPa to reduce congestion of confinement steel. In Taiwan, the Building Design Code for Concrete Structures was updated in 2023 (NLMA, 2023), and the column seismic design section was revised based on the corresponding section of ACI 318-19.

Since the design axial compression,  $P_u$ , can be substituted by  $\eta f'_c A_g$ , Eq. (3) can be rewritten as Eq. (3a) below:

$$R_{t,c} = 0.2k_f k_n \eta \frac{A_g}{A_{ch}} \frac{f'_c}{f_{yt}} \quad (3a)$$

where  $\eta$  is the axial compressive load ratio and is equal to the ratio of  $P_u$  to  $f'_c A_g$ . Eq. (3) clearly shows that the required amount  $R_{t,c}$  of transverse confinement reinforcement for the column is proportional to  $f'_c/f_{yt}$ . Therefore, the most effective strategy for reducing  $R_{t,c}$  is decreasing the concrete strength and increasing the transverse reinforcement strength simultaneously.

However, strength matching is a crucial factor for RC members to fully utilize the material advantages of concrete and reinforcement and to achieve optimal performance. If the strength difference between concrete and reinforcement is large, the RC member may not perform as predicted. This is because low-strength concrete may fail prematurely, preventing the high design strength of the transverse confinement reinforcement from being fully developed. Therefore, reducing the required amount of transverse confinement reinforcement by using a high strength ratio of transverse confinement reinforcement to concrete,  $n_c = f_{yt}/f'_c$ , as shown in Eqs. (1) to (3), contradicts fundamental RC member design principles. Consequently, this paper asserts that the seismic design requirements for columns in ACI 318-19 may be insufficient, potentially failing to ensure the 3% radian drift ratio capacity mandated by the code. The primary objective of this study is to investigate the influence of this factor  $n_c$  on the seismic deformation performance of RC columns under high axial compressive loads. The 3% radian drift ratio is used as the criterion for evaluating experimental deformation capacity.

## 2. Test Program

A total of seven column specimens with a cross-section of  $650 \times 650$  mm and a height-to-width aspect ratio of 3.85 were tested in this study. SD550W rebar, the upper grade allowed for longitudinal reinforcement in the column seismic design provisions of the current ACI 318 Code, was utilized for all longitudinal reinforcements of all specimens, with reinforcement ratios of 2.86% (12-#11) and 3.81% (16-#11). The main design parameter in this study was the strength ratio of transverse confinement reinforcement to concrete,  $n_c$ . According to the seismic strength limits of the Taiwan Code, the lower and upper bounds for concrete strength are 28 and 70 MPa, respectively, and the lower strength used in engineering practice and the upper bound for transverse confinement reinforcement are 420 and 550 MPa, respectively. Hence, this study adopted four strengths of 28, 35, 42, and 70 MPa for concrete, and two strength grades of 420 and 550 MPa for the transverse confinement reinforcement. The design  $n_c$  values for the specimens ranged from the minimum value of 6.0 to the maximum value of 24.6 and included 10, 12, 16.4, and 19.7. The design amount, size, and spacing of the transverse confinement reinforcement were determined based on the axial load conditions, Eq. (3) or Eq. (3a), and a spacing of not more than 100 mm. Before the design, two parameters were defined: the axial load ratio ( $\eta = P_u/f'_c A_g$ ) and the demand-to-capacity ratio (DCR) of transverse confinement reinforcement,  $DCR = R_{t,c}/R_{t,d}$ . In the design process, the  $\eta$  was selected between 0.4 and 0.5, and the DCR approached 1.0. The original design results of all parameters for the specimens are shown in Table 1.



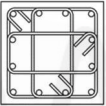
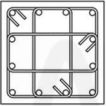
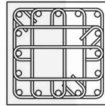
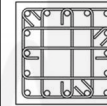
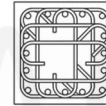
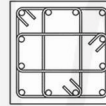
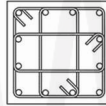
Table 1 also presents the actual parameters corresponding to the actual concrete compressive strength  $f'_c$  measured from concrete cylinders cured under the same conditions as the specimens and tested on the day of testing. The actual strength ratio  $n_c = f_{yt}/f'_c$ , test axial load ratio  $\eta_{test} = P_{test}/(f'_c A_g)$ , and the actual  $DCR_a = R_{t,ca} / R_{t,d}$  are shown in Table 1 as well, where  $R_{t,ca}$  is calculated based on Eq. (3) or Eq. (3a) with  $f'_{ca}$  and  $P_{test}$  or  $\eta_{test}$ . The test axial loads,  $P_{test}$ , for each specimen were constantly applied throughout the entire testing process. For the test axial load ratio  $\eta_{test}$  for all specimens, there were two levels. One was for the Specimens C1 and C2, which adopted the original design ratio of 0.5, so their  $DCR_a$  value was 1.39. This means that the amount of transverse confinement reinforcement provided,  $R_{t,d}$ , was insufficient, about 28% less than the amount required from  $R_{t,ca}$  of Eq. (3), as obtained using the test compressive load of  $P_{test} = 0.5 f'_{ca} A_g$ . For the other five specimens, the test

compressive loads were determined by the provided amounts of transverse confinement reinforcement provided,  $R_{t,d}$ , which were equal to the required amounts from  $R_{t,ca}$  in Eq. (3); that is, their  $DCR_a$  values were 1.0. As a result, the  $\eta_{test}$  values ranged from 0.27 to 0.5.

### 3. Test Results and Discussion

Table 2 summarizes the maximum strength and ultimate deformation of each specimen. The maximum strength  $V_{max}$  is the average of the two maximum strengths in the positive and negative directions,  $V_{max}^+$  and  $V_{max}^-$ , respectively. The ultimate deformation is defined by  $DR_{80}$ , which is the average of  $DR_{80}^+$  and  $DR_{80}^-$ , the drift ratios corresponding to the points where the strength drops to 80% of the positive and negative maximum strengths of  $V_{max}^+$  and  $V_{max}^-$  on the experimental positive and negative envelope curves, respectively.

**Table 1.** Design and actual parameters for all specimens

| Specimen                                                                      | C1                                                                                 | C2                                                                                 | C3                                                                                 | C4                                                                                 | C5                                                                                   | C2A                                                                                  | C2B                                                                                  |
|-------------------------------------------------------------------------------|------------------------------------------------------------------------------------|------------------------------------------------------------------------------------|------------------------------------------------------------------------------------|------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------|
| Section Type<br>650 × 650 mm                                                  |  |  |  |  |  |  |  |
| $\rho_s$ (%)                                                                  | 2.86 (12-#11)                                                                      |                                                                                    | 3.81 (16-#11)                                                                      |                                                                                    |                                                                                      | 2.86 (12-#11)                                                                        |                                                                                      |
| $f'_c$ (MPa)                                                                  | 35                                                                                 |                                                                                    | 42                                                                                 |                                                                                    |                                                                                      | 35                                                                                   | 28                                                                                   |
| $f'_{ca}$ (MPa)                                                               | 47.33                                                                              | 49.74                                                                              | 54.65                                                                              | 56.59                                                                              | 89.64                                                                                | 35.96                                                                                | 37.55                                                                                |
| $f_y$ (MPa)                                                                   | 550 (SD550W)                                                                       |                                                                                    |                                                                                    |                                                                                    |                                                                                      |                                                                                      |                                                                                      |
| $f_{yt}$ (MPa)                                                                | 420                                                                                | 690                                                                                | 420                                                                                | 690                                                                                | 420                                                                                  | 690                                                                                  |                                                                                      |
| $n_c = f_{yt}/f'_c$                                                           | 12.0                                                                               | 19.7                                                                               | 10.0                                                                               | 16.4                                                                               | 6.0                                                                                  | 19.7                                                                                 | 24.6                                                                                 |
| $n_{ca} = f_{yt}/f'_{ca}$                                                     | 8.9                                                                                | 13.9                                                                               | 7.7                                                                                | 12.7                                                                               | 4.7                                                                                  | 19.2                                                                                 | 18.4                                                                                 |
| Trans. Size,<br>@spacing (%)                                                  | #5<br>@110                                                                         | #4<br>@110                                                                         | #5<br>@120                                                                         | #4<br>@120                                                                         | #6<br>@130                                                                           | #4<br>@110                                                                           | #4<br>@130                                                                           |
| $h (=P_u/A_g f'_c)$                                                           | 0.5                                                                                |                                                                                    | 0.5                                                                                |                                                                                    |                                                                                      | 0.5                                                                                  | 0.5                                                                                  |
| $h_{test} (=P_{test}/A_g f'_{ca})$                                            | 0.5                                                                                | 0.5                                                                                | 0.38                                                                               | 0.38                                                                               | 0.27                                                                                 | 0.5                                                                                  | 0.4                                                                                  |
| $R_{t,d} = \frac{A_{sh}}{s b_c}$ (%)                                          | 1.27                                                                               | 0.81                                                                               | 1.45                                                                               | 0.93                                                                               | 1.93                                                                                 | 0.81                                                                                 | 0.68                                                                                 |
| $R_{t,a} = 0.3 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (%) | 0.75                                                                               | 0.46                                                                               | 0.90                                                                               | 0.55                                                                               | 1.50                                                                                 | 0.46                                                                                 | 0.37                                                                                 |
| $R_{t,b} = 0.09 \frac{f'_c}{f_{yt}}$ (%)                                      | 0.75                                                                               | 0.46                                                                               | 0.90                                                                               | 0.55                                                                               | 1.50                                                                                 | 0.46                                                                                 | 0.37                                                                                 |
| $R_{t,c} = 0.2 k_f k_n \eta \frac{A_g f'_c}{A_{ch} f_{yt}}$ (%)               | 1.30                                                                               | 0.79                                                                               | 1.49                                                                               | 0.91                                                                               | 1.98                                                                                 | 0.79                                                                                 | 0.63                                                                                 |
| $R_{t,ca} = 0.2 k_f k_n \eta_a \frac{A_g f'_{ca}}{A_{ch} f_{yt}}$ (%)         | 1.76                                                                               | 1.13                                                                               | 1.45                                                                               | 0.93                                                                               | 1.93                                                                                 | 0.81                                                                                 | 0.68                                                                                 |
| $DCR = R_{t,c} / R_{t,d}$                                                     | 1.03                                                                               | 0.98                                                                               | 1.02                                                                               | 0.98                                                                               | 1.02                                                                                 | 0.98                                                                                 | 0.93                                                                                 |
| $DCR_a = R_{t,ca} / R_{t,d}$                                                  | 1.39                                                                               | 1.39                                                                               | 1.00                                                                               | 1.00                                                                               | 1.00                                                                                 | 1.00                                                                                 | 1.00                                                                                 |

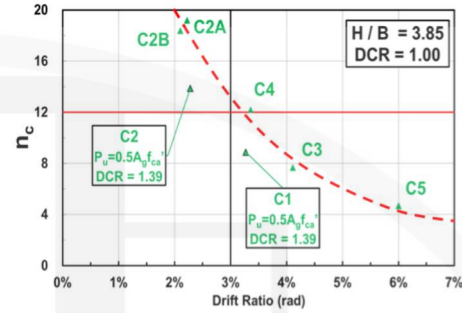
Observation of the ultimate deformation capacity  $DR_{80}$  in Table 2 indicates that a higher  $n_{ca}$  leads to a lower deformation capacity, and vice versa, the smaller the  $n_{ca}$ , the larger the deformation capacity. Specifically, for specimens C1, C3, C4, and C5, the  $n_{ca}$  values are less than 13, and the  $DR_{80}$  values all exceed 3% radian. In contrast, for specimens C2, C2A, and C2B with  $n_{ca}$  values greater than 13, the  $DR_{80}$  values fail to reach 3% radian. Additionally, the relationship between the actual  $n_c$  and  $DR_{80}$  for the seven specimens is plotted in Fig. 1. The distribution clearly shows a high correlation between the two parameters, where the interstory drift ratio capacity ( $DR_{80}$ ) decreases as the  $n_c$  value increases. The dashed line in Fig. 1 is the trend line for the five specimens with  $DCR_a$  equal to 1.0. This trend indicates that for columns where the amount of transverse confinement reinforcement is governed by axial compressive load in Eq. (3), the column cannot achieve a 3% radian drift ratio capacity once the  $n_c$  value exceeds approximately 13. Consequently, when columns are designed according to ACI 318-19 seismic design regulations and the transverse confinement reinforcement is determined based on Eq. (3), the required 3% radian drift ratio capacity cannot be guaranteed. It is therefore necessary for the seismic design of columns under high axial compressive load to consider the strength-matching index  $n_c$  between transverse reinforcement strength and concrete strength. Based on the limited experimental results of this study, a stricter  $n_c$  value of 12 is recommended to ensure that columns can provide the deformation capacity required for the 3% radian drift ratio.

**Table 2.** Strength and deformation test results for all specimens

| Spec | $n_{ca}$ | $DCR_a$ | $V_{max}^+$<br>$V_{max}^-$<br>(kN) | $V_{max}$<br>(kN) | $DR_{80}^+$<br>$DR_{80}^-$<br>(%) | $DR_{80}$<br>(%) |
|------|----------|---------|------------------------------------|-------------------|-----------------------------------|------------------|
| C1   | 8.9      | 1.39    | 1907<br>1904                       | 1906              | 3.54<br>3.00                      | 3.27             |
| C2   | 13.9     | 1.39    | 1729<br>1710                       | 1720              | 2.28<br>2.28                      | 2.28             |
| C3   | 7.7      | 1.00    | 2453<br>2292                       | 2373              | 4.27<br>3.94                      | 4.11             |
| C4   | 12.7     | 1.00    | 2426<br>2338                       | 2382              | 3.42<br>3.31                      | 3.37             |
| C5   | 4.7      | 1.00    | 2828<br>2846                       | 2837              | 6.00<br>6.00                      | 6.00             |
| C2A  | 19.2     | 1.00    | 1495<br>1469                       | 1482              | 2.33<br>2.11                      | 2.22             |
| C2B  | 18.4     | 1.00    | 1576<br>1520                       | 1548              | 2.22<br>1.98                      | 2.10             |

Furthermore, the axial compressive loads applied to specimens C1 and C2 were 1.39 times the axial load demand calculated by Eq. (3). Specifically, the experimental result of specimen C1, which had an  $n_{ca}$  value of 8.9, shows that its  $DR_{80}$  reached a

deformation capacity of 3.27% rad. Therefore, for columns with a lower  $n_{ca}$  value (approximately 9), the transverse confinement reinforcement requirements in Eq. (3) for high axial compressive loads tend to be slightly conservative.



**Fig. 1.** Relationship between experimental  $n_c$  value and  $DR_{80}$  for all specimens.

#### 4. Conclusions and Recommendations

Based on the experimental results regarding the seismic deformation capacity of RC columns subjected to high axial compressive loads discussed in this study, the following conclusions and recommendations can be drawn.

1. Experimental results indicate that when RC columns are designed according to the seismic design requirements of ACI 318-19, and the amount of transverse confinement reinforcement determined by Eq. (3), the required 3% radian drift ratio capacity cannot be guaranteed. The strength matching is a crucial factor, and a stricter  $n_c$  value of 12 is recommended to ensure that the columns can provide the deformation capacity required for the 3% radian drift ratio.
2. Although the applied axial compressive load on specimen C1 was increased to 1.39 times the required axial load based on Eq. (3), its ultimate deformation,  $DR_{80}$ , still reached 3.27% radian. This demonstrates that the transverse confinement reinforcement requirements of Eq. (3) exhibit a conservative trend.

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# Study on Anchorage and Bond Development Performance of High-Strength Beam Main Bars in Beam–Column Joints

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## Abstract

This study investigates the seismic behavior of reinforced concrete (RC) beam–column joints through a series of joint tests, focusing on the influence of anchorage detailing of beam longitudinal reinforcement within the joint core. The objective is to clarify the relationship between beam bar anchorage length and the overall deformation capacity of structural members. Seven RC beam–column joint specimens were designed and tested. The primary variables were anchorage type (standard hooks or headed bars), anchorage length of beam longitudinal bars, and the presence or absence of transverse reinforcement within the joint core and the beam plastic hinge region. Anchorage lengths were determined according to current seismic design provisions and an empirical formula proposed by the research team for validation purposes. Test results indicate that when the anchorage of beam longitudinal reinforcement within the joint core satisfies seismic design requirements, both standard hook and headed bar anchorage details can sustain beam-end deformations up to an inter-story drift ratio of 4%. Furthermore, the proposed empirical equation for anchorage length demonstrates adequate accuracy in predicting the required strength and deformation capacity. Under comparable detailing conditions, the presence of transverse reinforcement within the beam plastic hinge region has a negligible influence on failure mode, strength, and deformation capacity. By contrast, the absence of transverse reinforcement within the joint core has a more pronounced effect. Specimens without joint transverse confinement exhibited a transition to joint shear failure after beam yielding. Nevertheless, even under such conditions, the specimens were still able to achieve flexural strength and deformation capacity exceeding design expectations at an inter-story drift ratio of 4%.

**Keywords:** Beam–column joint, anchorage length, headed bars, transverse reinforcement

## Research Motivation and Objectives

In recent years, to address the issues of reinforcement congestion and construction difficulty within reinforced concrete (RC) beam–column joint regions, the research team has adopted headed bars as an alternative to conventional standard hooked bars for beam reinforcement anchorage. Based on a series of previous experimental studies on the seismic performance of RC beam–column joints (Kao, 2012; Hsu, 2014; Chao, 2016; Liu, 2019; Chien, 2023), it has been demonstrated that, when both the joint shear strength design and the tensile anchorage length of headed beam reinforcement satisfy the seismic design provisions specified in the current code (Ministry of the Interior, 2023), beam members subjected to cyclic

loading are capable of effectively developing flexural strength and forming plastic hinges, with deformation capacities exceeding an inter-story drift ratio of 4%. In addition, pullout behavior studies comparing standard hooks and headed reinforcement (Tsao, 2023) have confirmed that headed bars exhibit superior anchorage performance in concrete relative to hooked bars.

The aforementioned studies primarily focused on beam longitudinal reinforcement of sizes D25 (#8) and D32 (#10). There is limited experimental data for larger diameters, particularly D36 (#11) and above. Therefore, this study investigates the anchorage length and performance of SD 550W D36 reinforcement, considering both hooks and headed bar details. In the beam plastic hinge region, current design provisions

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require transverse reinforcement or ties to provide lateral support to longitudinal bars at each corner and at specified intervals. However, such detailing can cause reinforcement congestion and construction difficulties. This study also evaluates the influence of lateral support conditions for longitudinal reinforcement in the plastic hinge region. Additionally, the effects of transverse confinement reinforcement within the beam–column joint core on seismic performance and failure modes are examined. The findings are intended to inform future code development and design practice.

## Seismic Design for Beam–Column Joints

According to current seismic design provisions, beam–column connections in special moment-resisting frames must meet the strong-column, weak-beam requirement. Specifically, the flexural strength ratio  $R_m$  between columns and beams at the joint must exceed 1.2. This ensures that, during an earthquake, beams yield before columns and can fully develop plastic hinges. The flexural strength ratio is defined in Eq. (1).

$$R_m = \frac{\sum M_{nc}}{\sum M_{nb}} \quad (1)$$

$\sum M_{nc}$  and  $\sum M_{nb}$  denote the sum of nominal flexural strengths at the ends of the columns and beams, respectively, framing into the joint in the direction of loading.

To prevent joint shear failure before beam plastic hinges fully develop, the design must ensure that the factored joint shear strength,  $\phi V_{jh,n}$  ( $\phi = 0.85$ ), exceeds the joint shear demand  $V_{jh,u}$ . In this study, the ratio of joint shear demand to capacity is defined as  $V_{jh,u}/V_{jh,n}$ . The joint shear demand  $V_{jh,u}$  is the horizontal shear force transferred into the joint core when plastic hinges form at both beam ends; it is calculated using Eq. (2). The joint shear strength capacity  $V_{jh,n}$  is determined from Eqs. (3) and (4).

$$V_{jh,u} = 1.25A_s f_y - V_{col} \quad (2)$$

$$V_{jh,n} = \gamma \sqrt{f'_c} b_j h_c \quad (3)$$

$$b_j = \min \begin{cases} b_b + 2x \\ b_b + h_c \\ b_c \end{cases} \quad (4)$$

In these expressions,  $A_s$  is the area of top and bottom longitudinal reinforcement at the beam end;  $f_y$  is the design yield strength of longitudinal reinforcement;  $V_{col}$  is the column shear corresponding to the development of beam flexural strength;  $\gamma$  is the joint shear strength coefficient;  $b_j$  is the effective joint shear width, as defined in Eq. (4);  $b_b$  is the beam width;  $x$  is the distance from the beam edge to the nearest column face;  $b_c$  is the column width;  $h_c$  is the joint depth in the direction of shear; and  $f'_c$  is the specified compressive strength of concrete.

Regarding reinforcement anchorage length, this study follows code requirements and uses empirical formulas developed by the research team based on previous experiments and desired deformation performance to design the anchorage lengths of 90° hooked bars and headed bars within joints. The adopted equations are shown in Eqs. (5) through (8): Eqs. (5) and (8) correspond to the seismic anchorage length provisions for hooked bars and headed bars specified in the domestic RC design code, respectively.

$$\ell_{dh} \text{ of hooked bars (mm)} \quad \ell_{dh,3\%} = 0.185 \frac{f_y}{\sqrt{f'_c}} d_b \quad (5)$$

$$\ell_{dh,4\%} = 0.231 \frac{f_y}{\sqrt{f'_c}} d_b \quad (6)$$

$$\ell_{dt} \text{ of headed bars (mm)} \quad \ell_{dt,3\%} = 0.145 \frac{f_y}{\sqrt{f'_c}} d_b \quad (7)$$

$$\ell_{dt,4\%} = 0.185 \frac{f_y}{\sqrt{f'_c}} d_b \quad (8)$$

In these expressions,  $\ell_{dh,3\%}/\ell_{dh,4\%}$  represent the required anchorage lengths of hooked bars to achieve beam ultimate rotations corresponding to 3% and 4% drift ratios, respectively. Similarly,  $\ell_{dt,3\%}/\ell_{dt,4\%}$  denote the required anchorage lengths of headed bars for the same deformation targets. Here,  $f_y$  is the design yield strength of longitudinal reinforcement,  $f'_c$  is the specified compressive strength of concrete, and  $d_b$  is the diameter of reinforcement.

## Experimental Plan for RC Joints

In this study, seven RC beam–column joint specimens were designed and tested. The concrete compressive strength for all specimens was 70 MPa. The main variables in the experimental program were the anchorage type of beam longitudinal reinforcement, anchorage length, and the presence or absence of transverse reinforcement in the joint core and beam plastic hinge region.

For the column members, the longitudinal reinforcement consisted of SD 550W D36 bars. The transverse reinforcement was arranged using one set of SD 420W D16 (#5) outer hoops combined with four sets of SD 420W D13 (#4) inner ties. Within the joint core region, high-strength transverse reinforcement provided by Tokyo Tekko Co., Ltd. was adopted, consisting of USD 785 D16 outer hoops and USD 785 D13 inner ties, with a spacing of 100 mm. It should be noted that specimen JE4W was intentionally designed without transverse reinforcement in the joint core region; for the beam members, the longitudinal reinforcement consisted of SD 550W D36 bars. Among the specimens, JE1, JE1C, and JE2 employed different anchorage details for the top and bottom longitudinal reinforcement, including 90° hooks and headed bars. The remaining four specimens JE3, JE4, JE4C, and JE4W adopted headed bars for all beam longitudinal reinforcement anchorage. Regarding



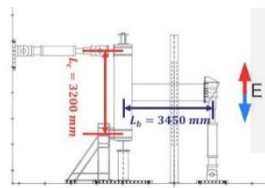
transverse reinforcement in the beam, five specimens (JE1, JE2, JE3, JE4, and JE4W) were designed with one set of SD 420W D16 outer hoops at a spacing of 100 mm. The other two specimens (JE1C and JE4C) were provided with one set of SD 420W D16 outer hoops combined with one set of SD 420W D13 inner ties, spaced at 150 mm. The detailed design parameters of all specimens are summarized in Table 1.

**Table 1.** Design Parameters for RC Joint Specimens

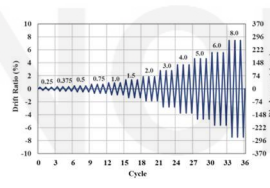
| Specimen                                | JE1                                                                               | JE1C                                                                              | JE2                                                                               | JE3                                                                               | JE4                                                                               | JE4C                                                                              | JE4W                                                                              |
|-----------------------------------------|-----------------------------------------------------------------------------------|-----------------------------------------------------------------------------------|-----------------------------------------------------------------------------------|-----------------------------------------------------------------------------------|-----------------------------------------------------------------------------------|-----------------------------------------------------------------------------------|-----------------------------------------------------------------------------------|
| $f_y$                                   | 550 (MPa)                                                                         |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |
| $f'_c$                                  | 70 (MPa)                                                                          |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |
| $P_u/A_g f'_c$                          | 0.05                                                                              |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |
| Beam                                    | 550×650 mm                                                                        |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |
|                                         | 5-#11                                                                             |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |
|                                         | 5-#11                                                                             |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |
|                                         | Stirrups<br>+Crossies                                                             | #5@100<br>mm                                                                      | #5@100<br>+ #4@150<br>mm                                                          | #5@100<br>mm                                                                      | #5@100<br>+ #4@150<br>mm                                                          | #5@100<br>mm                                                                      | #5@100<br>mm                                                                      |
|                                         | Cover                                                                             | 40 (mm)                                                                           |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |
| Column                                  | 600×700 (mm)                                                                      |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |
|                                         | 12-#11                                                                            |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |                                                                                   |
| Connection Type                         |  |  |  |  |  |  |  |
| $l_{dt,emb}$                            | 440                                                                               | 440                                                                               | 550                                                                               | -                                                                                 | -                                                                                 | -                                                                                 | -                                                                                 |
| $l_{dt,emb}$                            | 350                                                                               | 350                                                                               | 440                                                                               | 340                                                                               | 440                                                                               | 440                                                                               | 440                                                                               |
| Steel Net Spacing                       | $1.5d_b$                                                                          | $1.5d_b$                                                                          | $1.5d_b$                                                                          | $1.5d_b$                                                                          | $1.5d_b$                                                                          | $1.5d_b$                                                                          | $1.5d_b$                                                                          |
| $\rho$                                  | 0.016                                                                             | 0.016                                                                             | 0.016                                                                             | 0.016                                                                             | 0.016                                                                             | 0.016                                                                             | 0.016                                                                             |
| $M_r = \frac{\sum(M_n)_c}{\sum(M_n)_b}$ |                                                                                   |                                                                                   |                                                                                   | 2.37                                                                              |                                                                                   |                                                                                   |                                                                                   |
| $V_{jh}/V_h$                            |                                                                                   |                                                                                   |                                                                                   | 0.81                                                                              |                                                                                   |                                                                                   |                                                                                   |

All seven joint specimens were designed in accordance with the provisions of the *Concrete Structural Design Code for Buildings*, ensuring compliance with the seismic design requirements of the strong column and weak beam mechanism, and adequate joint shear strength.

Under loading, the inflection points of flexure in typical frame systems are located at the midspan of beams and the mid-height of columns. Therefore, this study uses an exterior beam-column joint subassembly, representing the region between the beam midspan and column mid-height in a frame, as the test specimen. The experimental setup is shown in Fig. 1. A constant axial load of  $0.05A_g f'_{ca}$  was applied to the column of the joint specimen. Cyclic vertical displacements with increasing amplitude were then imposed at the beam ends using hydraulic actuators, simulating the effects of lateral forces on beam-column joints. The vertical displacement at the beam ends was controlled based on the inter-story drift ratio of the moment frame. The loading protocol followed a predefined displacement history, as shown in Fig. 2.



**Fig. 1.** Experimental Setup of Specimen.



**Fig. 2.** Loading Protocol in Joint Test.

## Experimental Results and Discussion

The key results, including strength, deformation,

and ductility-related parameters, are summarized in Table 2. In this study, ductility is defined based on the drift ratio corresponding to a reduction of beam end moment capacity to 80% of the maximum measured strength, denoted as  $DR_{80\%}$ , which is obtained from the first-cycle envelope curve.

The test results indicate that all specimens could develop flexural strengths exceeding the nominal flexural strength ( $M_{na}$ ) calculated using the actual material properties. The ratio of measured strength to nominal flexural strength,  $M_{test}/M_{na}$ , ranged approximately from 1.09 to 1.17. Fig. 3 presents the hysteretic responses of beam end moment versus drift ratio for all specimens. The results show that, before reaching an inter-story drift ratio of 4%, no significant strength degradation was observed. Additionally, all specimens exhibited stable stiffness characteristics and satisfactory energy dissipation capacity.

**Table 2.** Experimental Results for Joint Specimens

| Spe. | $f_{ca}$ | $f_{yb}$ | $M_{na}^+$ | $M_{na}^-$ | $M_{max}^+$ | $M_{max}^-$ | $R_o^+$ | $R_o^-$ | $R_{o,avg}$ | $DR_{max}^+$ | $DR_{max}^-$ | $DR_{80\%}^+$ | $DR_{80\%}^-$ |
|------|----------|----------|------------|------------|-------------|-------------|---------|---------|-------------|--------------|--------------|---------------|---------------|
|      | (MPa)    |          | (kN-m)     |            |             |             |         |         |             |              |              |               |               |
| JE1  | 72.8     | 592      | 1583       | -1583      | 1854        | -1735       | 1.17    | 1.10    | 1.13        | 6.00         | -6.00        | 6.00†         | -6.00†        |
| JE1C | 91.3     |          | 1610       | -1610      | 1822        | -1757       | 1.13    | 1.09    | 1.11        | 5.00         | -5.00        | 5.59          | -5.42         |
| JE2  | 69.6     |          | 1577       | -1577      | 1944        | -1734       | 1.23    | 1.10    | 1.17        | 5.99         | -6.01        | 6.00†         | -6.00†        |
| JE3  | 89.9     |          | 1608       | -1608      | 1902        | -1747       | 1.18    | 1.09    | 1.13        | 6.00         | -6.01        | 6.00†         | -6.00†        |
| JE4  | 79.0     |          | 1594       | -1594      | 1776        | -1700       | 1.11    | 1.07    | 1.09        | 5.00         | -4.01        | 5.34          | -5.40         |
| JE4C | 90.3     | 615      | 1609       | -1609      | 1823        | -1722       | 1.13    | 1.07    | 1.10        | 5.00         | -4.97        | 5.35          | -5.34         |
| JE4W | 84.1     |          | 1659       | -1659      | 1910        | -1777       | 1.15    | 1.07    | 1.11        | 5.01         | -4.00        | 5.73          | -5.37         |

A comparison of the strength and deformation hysteretic behavior among specimens JE1, JE2, and JE1C shows that, when the top and bottom longitudinal reinforcement were anchored using 90° hooks and shorter headed bars, respectively, the bottom headed bars provided better anchorage and exhibited less strength degradation than the top hooked bars. Comparing specimens JE1 and JE1C, which differ only in the presence of transverse ties within the beam plastic hinge region, both achieved the target strength and deformation capacity, exceeding a drift ratio of 4%. This suggests that transverse ties in the plastic hinge region had a limited effect on overall seismic performance under the tested conditions. In addition, comparisons among specimens JE3, JE4, JE4C, and JE4W, where both the top and bottom longitudinal reinforcement were anchored using headed bars, demonstrate comparable seismic performance before a drift ratio of 4%. All specimens achieved the expected strength and deformation capacity. However, specimen JE4W, which lacked transverse reinforcement within the joint core, exhibited joint shear failure after the beam flexural yielding. The failure modes of all seven joint specimens are summarized in Fig. 4.

With respect to anchorage length design, the specimens designed based on Eqs. (5) through (8) consistently achieved strength, deformation capacity, and ductility exceeding the target performance levels, thereby confirming the conservativeness and reliability of the proposed empirical formulations.

## Conclusion and Suggestions

- (1) Experimental results indicate that when beam longitudinal reinforcement is anchored within the joint core using either standard hooks or headed bars, the anchorage length is designed according to Eqs. (5)–(8), the beams achieve ductility at drift ratios greater than 5%.
- (2) Comparisons among specimens JE1, JE2, and JE1C, which include 90° hooked and headed bars, show that headed bars with shorter anchorage lengths can provide better anchorage performance and strength degradation characteristics than those of 90° hooked bars.
- (3) In this case, comparisons between specimens with and without transverse ties in the beam plastic hinge region (JE1 vs. JE1C, and JE4 vs. JE4C) indicate that their overall seismic performance is similar.
- (4) Comparisons between specimens with and without transverse reinforcement in the joint core (JE4 vs. JE4W) show that both configurations can achieve the target strength and deformation capacity. However, specimen JE4W experienced joint shear failure after the beam flexural yielding, highlighting the importance of joint confinement in preventing undesirable failure modes.

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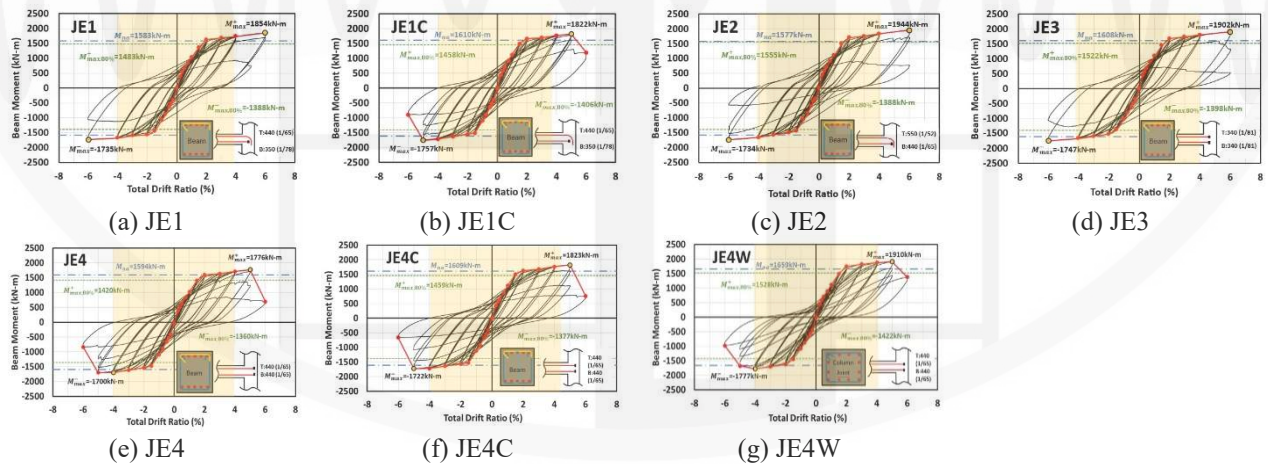


Fig. 3. Hysteretic Behavior of Strength and Deformation of Joint Specimens.

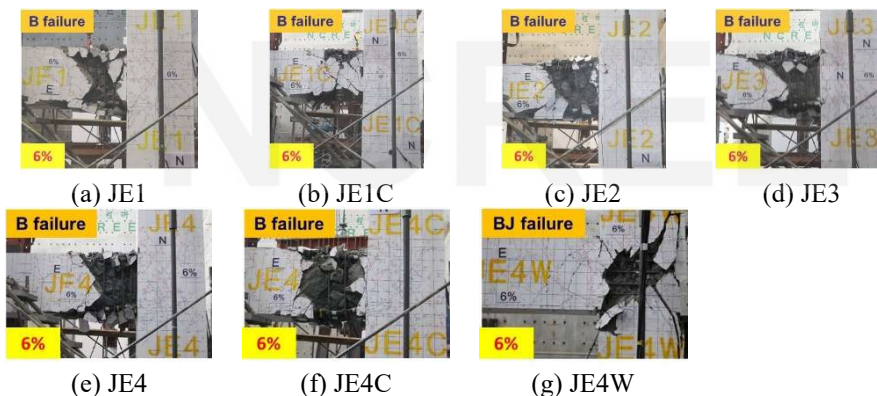


Fig. 4. Failure Modes of Joint Specimens.

# Deep Learning–Based Prediction of Structural Nonlinear Response History for Base-Isolation Configuration and Cross-Section Design Optimization

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## Abstract

In recent years, artificial intelligence technologies such as machine learning and deep learning have advanced rapidly, offering fast computation and strong generalization capabilities. These features have led to widespread adoption across various fields. In structural analysis and design, however, current practical methods face significant challenges. One major issue is the use of nonlinear response-history analysis, which, while more accurate than static or response spectrum analyses, is much more time-consuming. Another problem is the heavy reliance on engineers' experience, which limits innovation in design solutions. To address these challenges, this study integrates a previously developed surrogate model for nonlinear response-history prediction with a multi-objective optimization algorithm and reinforcement learning to develop optimization models for base-isolation configuration in reinforced concrete structures and cross-sectional design in steel structures. The results show that these models, are highly accurate and efficient, indicating strong potential for practical use in structural engineering.

**Keywords:** deep learning, nonlinear response-history analysis, base isolation, steel structures, design optimization

## Introduction

In earthquake engineering, nonlinear response-history analysis provides the most accurate structural analysis by capturing the full nonlinear behavior of structures under seismic loading and is essential for performance-based design. However, a single nonlinear response-history analysis is computationally expensive, creating a bottleneck for iterative design optimization and limiting its practical application (De Luca & Guidi, 2019; Zakian & Kaveh, 2024). Recently, deep learning–based on surrogate models has emerged as a promising solution to this challenge.

One of the core technologies of this study is the Graph Neural Network (GNN) (Scarselli et al., 2009). Unlike traditional neural networks that require fixed-dimensional inputs, graph data are represented by

nodes and edges, and their relationships and dependencies are captured through a message-passing mechanism (Gilmer et al., 2017), enabling flexible prediction of node-, edge-, and graph-level targets across graphs of varying sizes (Battaglia et al., 2018). Since structural systems naturally exhibit graph-based topology through joints and members, and the message-passing mechanism can be interpreted as the transmission of internal forces, GNNs have been widely applied to simulate structural static responses with high accuracy (Chang & Cheng, 2020; Chou et al., 2024a).

To better simulate the time-dependent dynamic responses of structures under seismic loading, this study integrates GNNs with Long Short-Term Memory (LSTM) networks (Hochreiter &

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Schmidhuber, 1997) to develop surrogate models for nonlinear response-history prediction in both reinforced concrete (RC) base-isolated and steel structures. The Graph-LSTM model for RC base-isolated structures (Kuo et al., 2024), trained on 4,650 nonlinear response-history analyses, achieves prediction accuracy ( $R^2$ ) exceeding 0.95 on the test set and improves computational efficiency by over 460× compared to finite element analysis. A surrogate model for steel structures has also been developed, which predicts the full dynamic responses of three-dimensional steel frames under bidirectional seismic excitations (Chou et al., 2024b).

Building on previously developed surrogate models, this study shifts its focus to design optimization, using the computational efficiency of surrogate models to enable automated structural design. The research has two main components: (1) multi-objective optimization of RC base-isolated structures, where the trained Graph-LSTM surrogate model is integrated with the NSGA-II algorithm (Deb et al., 2002) to simultaneously optimize isolation system configuration, superstructure cross-sectional design, and material cost, with a code-compliance module embedded in the optimization loop to ensure constructability; and (2) reinforcement learning-based cross-sectional design optimization for steel structures, where a pretrained nonlinear response-history surrogate model is incorporated into the reinforcement learning (RL) framework (Hayashi & Ohsaki, 2022), allowing the agent to account for nonlinear dynamic responses and to improve the accuracy of seismic performance evaluation.

## Multi-Objective Optimization of Base-Isolated Reinforced Concrete (RC) Structures

### Framework Introduction

As shown in Fig. 1, the workflow comprises four iterative stages: initialization, fitness evaluation, selection, and generation of new candidates. For each design, peak story acceleration ( $f_1$ ) and isolation-layer displacement ( $f_2$ ) are rapidly predicted using the Graph-LSTM surrogate model, while total material cost ( $f_3$ ) is calculated directly from the design parameters. Each design is evaluated for code compliance, and noncompliant designs are penalized. The NSGA-II algorithm ranks the population using non-dominated sorting, and the next generation is produced through crossover and mutation, with the process repeated until convergence.

### Optimization Problem Formulation

**Design Variables:** A total of 35 discrete design variables are defined, covering the superstructure (28 variables: concrete strength, column and beam cross-

sectional dimensions, and reinforcement ratios, specified for three height zones: 1–5F, 6–8F, and 9–11F) and the isolation system (7 variables: post-yield stiffness ( $K_2$ ) and characteristic strength ( $Q_d$ ) of three sets of rubber bearings, as well as the damper velocity exponent ( $\alpha$ ) and damping ratio ( $\xi$ )). These 35 design variables constitute an extremely large design space on the order of  $10^{21}$ .

**Objective Functions:** Three objectives are minimized simultaneously: (1) peak floor acceleration ( $f_1$ ), indicating the effectiveness of base isolation; (2) peak isolator displacement ( $f_2$ ), representing the stroke demand on the bearings; and (3) total material cost ( $f_3$ ), including concrete, reinforcement for the superstructure, and the costs of isolation bearings and dampers. These objectives inherently conflict, so single-objective optimization cannot fully capture their trade-offs.

**Constraints:** A two-tier handling mechanism is used: (1) Construction Feasibility Correction automatically adjusts unrealistic vertical member size distributions (such as larger sections above smaller ones) and reduces the three types of lead rubber bearings (LRBs) to two; (2) Structural Code Compliance Check verifies member strength (D/C ratio), inter-story drift angles, overturning moments, and isolation system performance (total shear strain, sliding stability, and buckling stress). Designs that fail any check receive significant penalty values.

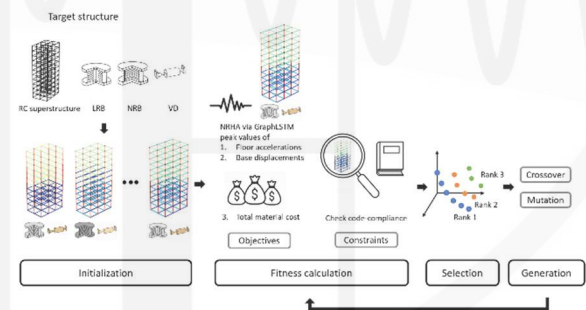


Fig. 1. Optimization problem formulation of base-isolated RC structures.

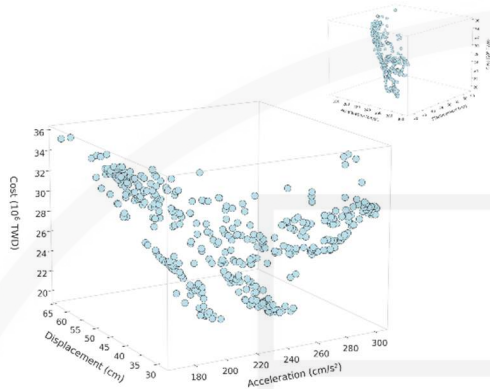
### Optimization Results

Under Maximum Considered Earthquake (MCE) conditions for Zone 3 of the Taipei Basin, optimization used 11 pairs of INMOST (local Taiwan earthquake) records. The algorithm generated 200 offspring per generation and retained 400 elite solutions for 288 generations. Convergence was confirmed by the Hypervolume (HV) indicator ( $HV \approx 0.95$ ).

The resulting 400 Pareto-optimal designs show a wide range of objective values (Fig. 2): total cost (19.4–35.7 million TWD), peak floor acceleration (168–303  $\text{cm/s}^2$ ), and isolation layer displacement (28–64 cm). These distributions highlight the trade-



offs among the three objectives. Monte Carlo validation indicates that 99.4% of randomly sampled feasible solutions (469 out of 472) are strictly dominated by the Pareto set, confirming that the framework has converged to the true efficiency frontier.



**Fig. 2.** Three-dimensional visualization of final 400 Pareto-optimal solutions.

## Cross-Section Design Optimization of Steel Structures

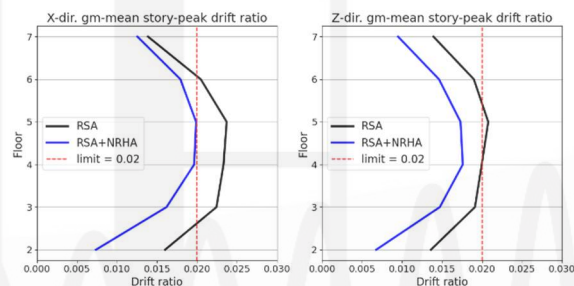
As a deep learning method, RL allows an agent to gain experience by interacting with an environment and to learn to make decisions that maximize or minimize specific objectives. Hayashi et al. (2022) developed a graph-based RL framework for optimizing the cross-sectional design of two-dimensional steel frames. This framework offers higher generalization capability than conventional optimization methods.

### Model Construction and Design Objective

This study extends the method to three-dimensional steel moment-resisting frames and develops an AI-assisted cross-sectional design optimization framework called Graph-RL. The framework optimizes cross sections for structures with varying numbers of stories (4–7), spans (2–6), and bay widths (6–8 m), aiming to minimize total member volume (material usage). Structural strength checks are conducted under two scenarios during the design process: (1) linear dynamic analysis based on Taiwanese regulations, using the open-source finite element engine OpenSees to evaluate inter-story drift ratio and strong-column–weak-beam compliance under lateral shear; and (2) nonlinear dynamic analysis based on U.S. regulations, using the previously developed Graph-LSTM surrogate model to assess inter-story drift ratio and the number of plastic hinges under earthquake time histories. The reinforcement learning algorithm is a Deep Q-Network (DQN), which is suitable for discrete action spaces (Mnih et al., 2013).

## Comparison of Model Design

To evaluate the impact of nonlinear response-history analysis on structural design, this study considered two training scenarios: (1) using only linear dynamic analysis and (2) using both linear and nonlinear dynamic analyses. Graph-RL models trained under these scenarios were applied to the same structure, producing two distinct sets of cross-sectional designs. Nonlinear dynamic analyses were then performed with finite element software to determine maximum floor displacements under several earthquake time histories. As shown in Fig. 3, Scenario 1 (black line) exceeds regulatory limits (red line) in both horizontal directions, whereas Scenario 2 (blue line) meets all regulatory requirements. This demonstrates that integrating nonlinear dynamic analysis into the design process can significantly affect the final design and improve structural safety.



**Fig. 3.** Comparison of inter-story drift in two training scenarios.

## Conclusion and Future Work

This study aims to optimize structural design using the fast and accurate nonlinear response-history analysis surrogate model, Graph-LSTM. The approach has been applied to base-isolated RC and steel structures, enabling multi-objective optimization for isolation system configuration and single-objective optimization for cross-sectional design. However, current models are limited to these structural types. Future work should extend the framework to a wider range of materials and structural forms, thereby improving its applicability and further streamlining seismic engineering analysis and design.

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# Smart Bridge Inspection and Assessment Based on 3D Reconstruction and Defect Quantification

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嚴寬<sup>1</sup>、張家銘<sup>2</sup>

## Abstract

Bridge integrity is critical for infrastructure safety, yet traditional visual inspections are labor-intensive, time-consuming, and highly subjective. Although the use of unmanned aerial vehicles (UAVs) and computer vision has improved inspection efficiency, relying only on two-dimensional image recognition often lacks the spatial context necessary to accurately quantify structural deterioration. To address this, this study proposes a smart bridge inspection and assessment framework that integrates three-dimensional (3D) digital twin and deep learning technologies. First, UAV-captured images are processed with a Mask R-CNN instance segmentation model to detect and outline common structural defects, such as concrete spalling, exposed reinforcement, and water seepage. Next, Structure from Motion and Multi-View Stereo techniques are used to reconstruct a high-precision 3D dense point-cloud digital twin of the bridge from the UAV image sequences. Geometric accuracy assessments show that this digital twin achieves high realism, with a cumulative average relative error of only 2.80%. By projecting the detected pixel masks into this 3D space, the system enables accurate measurement of physical geometric parameters (e.g., the actual area of exposed rebar and spalling) for objective defect quantification. This framework not only improves inspection precision but also integrates quantitative defect data into the current Degree, Extent, Relevancy, and Urgency assessment framework, reducing subjective judgments by inspectors and providing a reliable basis for bridge maintenance and management decisions.

**Keywords:** Unmanned aerial vehicles, Bridge inspection, Deep learning, 3D reconstruction, Structure from Motion, Defect measurement.

## Introduction

Bridge structural integrity is a critical safety concern in infrastructure management, as both natural disasters and structural aging can cause bridge deterioration and compromise public safety. With approximately 29,000 bridges in Taiwan, regular inspection and condition assessment are essential. However, traditional visual inspections rely on manual labor, which is time-consuming and inefficient and exposes personnel to hazardous environments, especially when inspecting hard-to-access components, such as river-crossing piers and the underside of bridge decks. Additionally, current bridge inspections commonly use the Degree, Extent, Relevancy, and Urgency (D.E.R.&U.) assessment criteria, which depend heavily on the experience and subjective judgment of in-

spectors. This reliance makes it difficult to ensure consistency and achieve objective measurement when facing large-scale inspection demands.

In recent years, the use of Unmanned Aerial Vehicles (UAVs) has significantly improved the efficiency and safety of bridge inspections, enabling engineers to capture images of otherwise inaccessible components. To further minimize manual labor, this study proposes a UAV-assisted smart bridge inspection workflow that incorporates artificial intelligence (AI) for automated defect detection. However, earlier AI image recognition approaches have been limited to two-dimensional (2D) analysis. This restriction makes models vulnerable to background interference and changes in lighting. More critically, it does not provide the spatial context necessary to accurately measure the physical dimensions of defects (e.g., crack length or

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spalling area), making it unsuitable for direct integration with the quantitative requirements of the D.E.R.&U. system.

To address the limitations of 2D image recognition, this study presents a systematic smart bridge inspection framework that combines three-dimensional (3D) reconstruction and deep learning. The framework uses UAVs to capture high-resolution images and applies a mask region-based convolutional neural network (Mask R-CNN) model to segment common structural defects, such as concrete cracks, spalling, exposed reinforcement, and water seepage. Structure from Motion (SfM) is then used to generate a high-precision 3D point cloud model from the UAV images, extracting depth and normal maps. Integrating this spatial information with the original images during post-processing enhances defect detection accuracy and maps identified defects into 3D space, enabling objective measurement of their geometry. This method allows quantified defect characteristics to be incorporated into the current bridge assessment system, providing real-time, scientific, and reliable support for bridge safety evaluation and maintenance management.

### Smart Bridge Inspection Framework Based on 3D Reconstruction

The smart bridge inspection framework proposed in this study includes data collection, 2D image segmentation, 3D point cloud reconstruction, multimodal data fusion, and defect quantification. To improve the deep learning model's adaptability to different bridge types and environments, this study incorporated high-resolution UAV images, historical inspection photos from engineering consulting firms, and public datasets such as Mapillary Vistas. For common deterioration types such as concrete cracks, spalling, exposed reinforcement, and water seepage, polygonal bounding boxes were used for detailed manual annotation. This process resulted in a high-quality training dataset of more than 10,000 images with pixel-level annotations.

During the image recognition phase, the framework employs Mask R-CNN as the core defect detection model, as shown in Fig. 1. It integrates advanced backbone architectures, such as ResNet and Swin-Transformer, for feature extraction. To overcome the interference caused by subtle concrete surface textures and background noise, data augmentation techniques (e.g., random rotation, contrast adjustment, and noise addition) were introduced. Combined with transfer learning to optimize model parameters, this approach enables the system to accurately localize deteriorated regions and generate irregular pixel-level masks.

To overcome the inherent lack of spatial geometric information in purely 2D images, the system incorporates SfM technology as shown in Fig. 2. Utilizing

highly overlapping image sequences captured by UAVs from diverse angles, SfM reconstructs a high-precision 3D point cloud model of the bridge while simultaneously generating corresponding depth maps and normal maps. By fusing this multimodal spatial geometric data with the original RGB images as a post-processing mechanism not only provides the necessary spatial context for defect analysis but also significantly enhances overall detection accuracy.

Assisted by 3D geometric data, the framework accurately projects defect pixel masks identified on 2D images into vector polygons in real 3D space. Using the known dimensions of bridge components for baseline calibration, the system calculates the physical size of defects, such as the length of cracks and the area of concrete spalling. This geometric quantification enables objective data to be integrated into the "Extent" metric of the current D.E.R.&U. assessment criteria, reducing the subjectivity of traditional manual visual evaluations and providing a scientific basis for bridge safety and maintenance decisions.

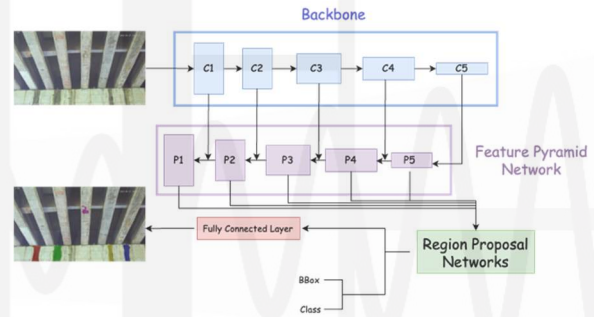


Fig. 1. MaskR-CNN network architecture.

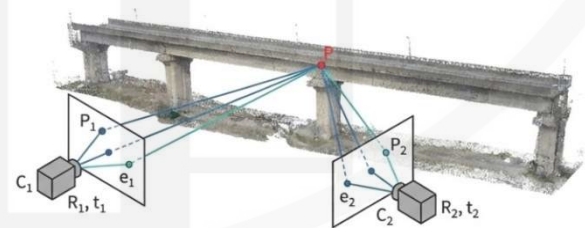


Fig. 2. Structure from Motion.

### Field Inspection Test and Defect Measurement Results

In practical field applications, this study used UAVs to inspect and collect images of selected test bridges, successfully verifying the performance of the proposed automated inspection framework. First, for the evaluation of the Mask R-CNN defect detection model, the system was tested on four types of structural damage: cracks, concrete spalling, exposed reinforcement, and water seepage. The results showed an overall recall of approximately 83.5% and an overall



precision of about 67.2%. Notably, for defects with more distinct visual features, such as exposed reinforcement and water seepage, the recall rates reached 96.8% and 90.1%, respectively. Although there are still some misjudgments and omissions in identifying, especially with fine cracks, the overall system performance demonstrates initial practical feasibility and significantly reduces the workload for engineers analyzing large volumes of inspection images.

$E_r$  Regarding the construction and geometric accuracy validation of 3D spatial data, this study used UAVs to capture 2D images of bridge components, which were then processed with SfM and multi-view stereo (MVS) techniques to generate a 3D dense point cloud model. To evaluate the accuracy of the 3D data generated by the digital twin, CloudCompare software was used to calculate geometric parameters, as shown in Fig. 3. A comparative analysis was conducted between the bridge dimensions in the architectural plans and those in the 3D point cloud model. Geometric accuracy was assessed using the relative error ( $E_r$ ) statistical method, defined by the following equation:

$$E_r = \frac{|M - T|}{T}$$

where  $M$  represents the measured value and  $T$  represents the true value. The measurement results determined that none of the measured spacing lengths exhibited a relative error surpassing 4%, with the cumulative average error recorded at a mere 2.80%, as shown in Table 1. This significantly low error rate demonstrates that the virtual reconstruction developed through this methodology possesses both realism and practical applicability concerning precision, providing a reliable spatial foundation for subsequent defect quantification.

Based on the high-precision 3D geometric models and depth maps, the framework accurately projected and converted defect masks identified in 2D images into vector polygons in the actual 3D space. Through spatial calibration, the system precisely calculated the length of cracks and the area of concrete spalling, marking a shift from visual recognition to physical quantification. These objective geometric defect data can be directly correlated with quantitative metrics for "Extent" in the current D.E.R.&U. assessment criteria. This demonstrates the feasibility of the proposed framework in engineering applications and provides a reliable scientific basis for bridge safety assessment and maintenance decision-making.

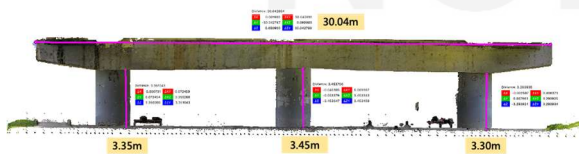


Fig. 3. CloudCompare Bridge Point Cloud Model

Table 1. Crossbeam length deviation in a bridge point cloud model

| Measurement (cm)  | Relative Error (%) |
|-------------------|--------------------|
| 77.05             | 3.69%              |
| 77.02             | 3.73%              |
| 81.06             | 1.33%              |
| 77.26             | 3.42%              |
| 77.42             | 3.23%              |
| 81.91             | 2.39%              |
| 79.15             | 1.06%              |
| 77.17             | 3.54%              |
| Average: 2.80 (%) |                    |

## Conclusions

This study has developed and validated an advanced bridge inspection framework that combines UAV inspections, deep learning, and 3D reconstruction technologies, significantly improving traditional manual visual inspection methods.

Regarding the research conclusions, the system utilizes Mask R-CNN combined with SfM and MVS techniques to construct a high-precision 3D point cloud digital twin model of bridges. Geometric validation results show that the cumulative average relative error between the 3D model dimensions and the actual structure is only 2.80%, confirming the digital twin's geometric accuracy. By projecting 2D defect recognition results into the high-precision 3D model, the system can precisely quantify the actual area of defects. This quantitative data can be directly integrated into existing D.E.R.&U. assessment criteria, providing measurements for evaluating "Extent" and "Degree," such as the specific dimensions, lengths, and widths of detected defects. For example, the system clearly illustrates the impact of combined defects such as concrete spalling, exposed reinforcement, and water seepage on overall structural durability. This approach reduces the subjectivity of traditional manual inspections, enabling management authorities to more accurately determine maintenance urgency.

In terms of future directions, the quantitative inspection framework proposed in this study has significant potential for engineering applications. First, to overcome the challenge of incomplete 3D model reconstruction in partially occluded structural areas (such as beneath bridge decks or between piers), future research can integrate UAV autonomous obstacle avoidance and smart flight path planning to capture bridge imagery without blind spots. Second, the defect image database can be continuously expanded, and more advanced deep learning models can be adopted to further improve the detection accuracy of fine cracks. Ultimately, the goal is to apply this high-precision digital twin technology to the full life-cycle management of bridges, using artificial intelligence to develop predictive maintenance models and provide

comprehensive, long-term assurance for the operational safety of national infrastructure.

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# Nonlinear Pushover Analysis of Micropile in Liquefied Ground

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## Abstract

This study uses borehole data from a five-story building with a shallow foundation located on soil with high liquefaction risk as a case study. Nonlinear pushover analysis is performed to determine the capacity curves of micropiles in liquefiable ground. The nonlinear behavior of the pile foundation is modeled by incorporating nonlinear soil springs and a distributed plastic hinge model. Based on this approach, the seismic performance of the piles under lateral loads is assessed, and their effectiveness in reducing liquefaction-related impacts is evaluated. The results are compared with the requirements specified in the current Design Code for Building Foundation Structures.

**Keywords:** liquefaction, micropile, nonlinear pushover analysis, reduction factor

## Introduction

With the recent updates to Taiwan's seismic design code and the publication of soil liquefaction potential maps, numerous shallow-foundation buildings in plain areas have been identified as being located within liquefaction-prone zones. A review of past practice, however, reveals that domestic anti-liquefaction measures have relied on large-scale ground improvement techniques requiring heavy machinery, which are ill-suited to small-scale sites such as individual or row-type shallow-foundation buildings. Consequently, there is a pressing need within the domestic engineering community to develop anti-liquefaction designs that are both economically viable and capable of ensuring the safety of such vulnerable building stock. For conventional buildings, bridges, and critical infrastructure, pile foundations constitute a widely adopted countermeasure against liquefaction. Nevertheless, comprehensive and representative studies addressing the constructability, anti-liquefaction effectiveness, and seismic performance of anti-liquefaction piles remain lacking. To develop small-diameter anti-liquefaction piles that are both applicable to general buildings and economically favorable, the present study is part of a multi-year research program aimed at establishing the fundamental knowledge and design methodology for small-diameter anti-liquefaction

piles for building structures.

Seismic performance assessment of structures involves analyzing structural responses under various levels of seismic demand (such as minor, design-basis, or maximum considered earthquakes), or developing simplified models with performance-based evaluation methods (such as nonlinear pushover analysis) to verify whether the structure meets the intended performance objectives. When the superstructure is subjected to seismic loading, the pile foundation transfers the imposed loads to the surrounding ground, and the interaction between the soil and piles enables the foundation to provide lateral and flexural capacity to the superstructure. Evaluating the lateral load-bearing performance of piles under short-term loading requires establishing their lateral capacity curves, which characterize the behavior of the piles throughout the loading process. Because the lateral load-bearing performance curve of a pile foundation is nonlinear, linear elastic analysis using elastic beam elements with constant soil spring stiffness in the Winkler foundation model is not appropriate. Instead, the analysis must incorporate nonlinear constitutive behavior for both the pile and the soil, and should be performed using numerical methods such as pushover analysis.

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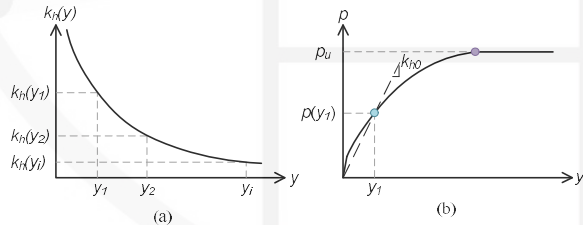


## Analysis Procedure

This section describes the analytical components involved in establishing the nonlinear soil springs and configuring the plastic hinges for simulating the nonlinear behavior of the pile material, as detailed below.

### (1) Nonlinear soil supports

The nonlinear behavior of soil can be characterized by  $p$ - $y$  curves, which describe the nonlinear variation of the coefficient of subgrade reaction. Widely adopted  $p$ - $y$  curve models for sandy soils, such as those proposed by Reese et al. (1974), the American Petroleum Institute (API, 2005), Det Norske Veritas (DNV, 2011), and the Architectural Institute of Japan (AIJ, 2001), have been formulated as semi-empirical  $p$ - $y$  curves based on theoretical considerations in conjunction with in-situ test data. To reasonably estimate the lateral load-displacement behavior of piles, nonlinear pushover analysis at the pile head can be conducted by establishing  $p$ - $y$  curves for the analyzed strata using site-specific geotechnical investigation data. In the present study, the semi-empirical  $p$ - $y$  curve model for sandy soils recommended by the Architectural Institute of Japan (AIJ, 2019) is adopted to simulate the soil reaction along the pile shaft (i.e., the lateral soil springs). For the lateral analysis of piles, the relationships between the coefficient of horizontal subgrade reaction, the ultimate soil reaction, and the horizontal displacement at the pile head, as recommended by AIJ (2019), are illustrated in Fig. 1.



**Fig. 1.** AIJ (2019) recommendations: (a) pile-head load-displacement curve; (b) relationship between stiffness and displacement.

The AIJ further defines the coefficient of horizontal subgrade reaction corresponding to a pile-head horizontal displacement of  $y_1 = 1$  cm as the reference coefficient of horizontal subgrade reaction, denoted  $k_h(y_1)$ . The relationship between the coefficient of horizontal subgrade reaction and the pile-head displacement can be expressed as follows:

$$k_h(y_1) = \alpha \cdot \xi E_0 \cdot (D/0.01)^{-0.75}$$

$$k_h(y) = 3.16 \cdot k_h(y_1) \quad \text{for } 0 \leq y/y_1 \leq 0.1$$

$$k_h(y) = k_h(y_1) [y/y_1]^{-0.5} \quad \text{for } y/y_1 > 0.1$$

where  $\alpha = 80$  for sandy soils and  $\alpha = 60$  for cohesive soils;  $\xi$  is the influence factor, taken as  $\xi = 1$  for a single pile;  $E_0$  is the deformation modulus of the soil ( $\text{kN/m}^2$ ); and  $D$  is the pile diameter (m).

### (2) Reduction of soil springs in liquefiable

In evaluating the seismic performance of pile foundations under liquefaction, it is necessary to account for the potential decrease in soil strength and stability after liquefaction during seismic events. The geotechnical parameters used in seismic design should be reduced accordingly. In practice, simplified liquefaction assessment methods (e.g., the HBF method) can be used to identify layers susceptible to liquefaction. A liquefaction-induced reduction factor  $D_E$  is then applied to decrease the ultimate lateral soil reaction  $p_u$  and the coefficient of horizontal subgrade reaction  $k_h$ . Recommended values for these reduction factors in liquefiable sandy layers are provided in the Design Code for the Foundation Structures of Buildings (2023).

### (3) Nonlinear properties of pile materials

Once the soil enters the nonlinear regime, the lateral displacement of the pile foundation also increases nonlinearly until the pile yields. Pushover analysis, therefore, requires nonlinear beam elements, typically modeled using the plastic hinge method, where hinges are assigned to beam or column elements to simulate nonlinear structural behavior. Because pile sections generally fail in flexure, flexural hinges are assigned to beam elements to capture moment-induced cracking, yielding, and failure of the pile cross-section. For reinforced concrete piles, the effects of concrete confinement and the nonlinear seismic response of concrete members must be considered. Sectional properties are determined using established concrete constitutive models (such as the Mander model), which account for the influence of longitudinal and transverse reinforcement on concrete strength. The resulting moment-curvature curve, obtained from the stress-strain relationship of the pile material, is often simplified into a bilinear or trilinear form for practical engineering applications.

Plastic hinge configurations are classified as either concentrated or distributed. The concentrated hinge model assumes all plastic deformation occurs at a single location, which is appropriate when yielding happens at the member ends, such as under fixed-head boundary conditions where the plastic hinge is located at the point of maximum moment. For free-head piles, however, the point of maximum moment changes with the applied lateral load and the extent of soil plastic deformation. The distributed hinge model (Fig. 2(a)) is therefore more suitable for nonlinear pushover analysis, as multiple hinges can be assigned to potential plastic zones to represent the development of plastic deformation. The flexural hinge is defined by a



moment–rotation ( $M-\theta$ ) relationship, where the plastic rotation is calculated by multiplying the curvature from the moment–curvature ( $M-\phi$ ) relationship by the plastic hinge length  $l_p$ . However, Chiou et al. (2009) demonstrated that this approach overestimates pile rotation and deformation and proposed a modified plastic hinge method for more accurate estimation. The modified plastic curvature and the corresponding moment–plastic rotation relationship are shown in Figs. 2(b) and 2(c).

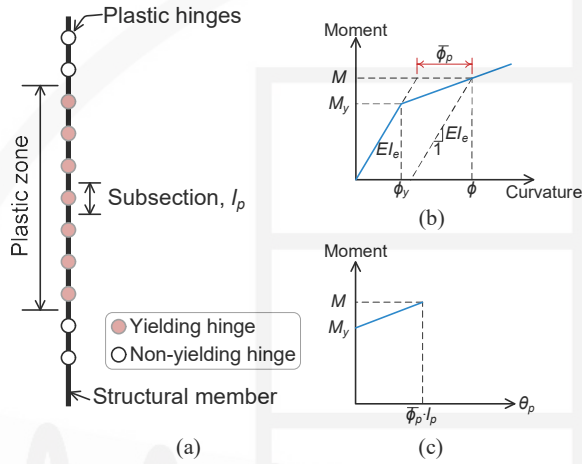


Fig. 2. (a) Distributed hinge model; (b) modified plastic curvature; (c) modified plastic rotation (after Chiou et al., 2009).

## Pushover Analysis Case Study

Fig. 3 shows the superstructure model used for the foundation pushover analysis. The short-term design seismic load, which considers soil liquefaction, is based on an earthquake with a 475-year return period. According to code-specified vertical distribution, seismic forces are applied to each story and the rooftop of the building. These forces determine the loading conditions at the top of each foundation, which are then used as input for the foundation pushover analysis. Fig. 4 presents the numerical models of the micropile for both non-liquefied and liquefied conditions. Fig. 5 shows the nonlinear soil springs along the pile shaft ( $p-y$  curves) established according to geotechnical conditions derived from borehole data. The strength and stiffness of the lateral soil springs are reduced by the soil parameter reduction factor  $D_E$ , from the liquefaction assessment results (Table 1). Flexural plastic hinges are introduced into the structural members to simulate nonlinear behavior, including cracking, yielding, and failure of the pile section under bending moments. Based on the micropile specifications and material parameters used in the design example, the moment–curvature curve for the pile section is also shown in Fig. 5.

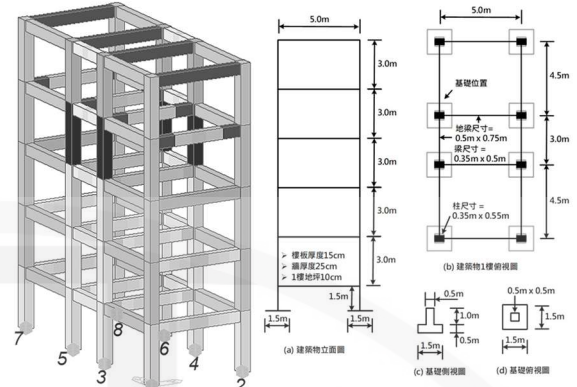


Fig. 3. Schematic of the analytical model for the five-story structure with shallow foundations.

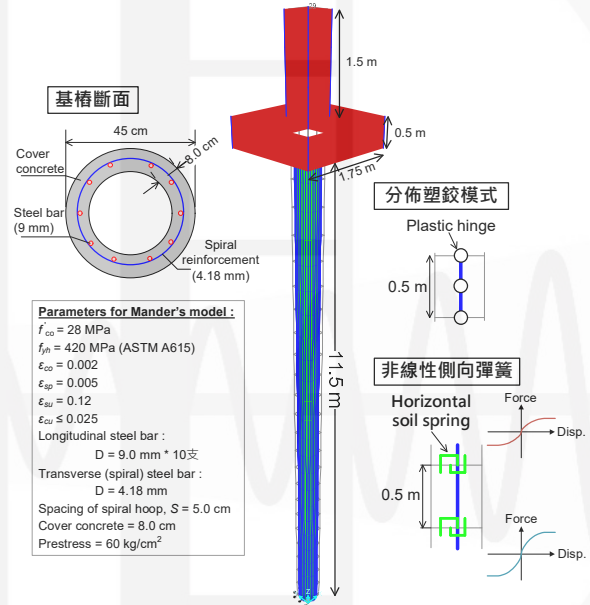


Fig. 4. Numerical model used for pushover analysis.

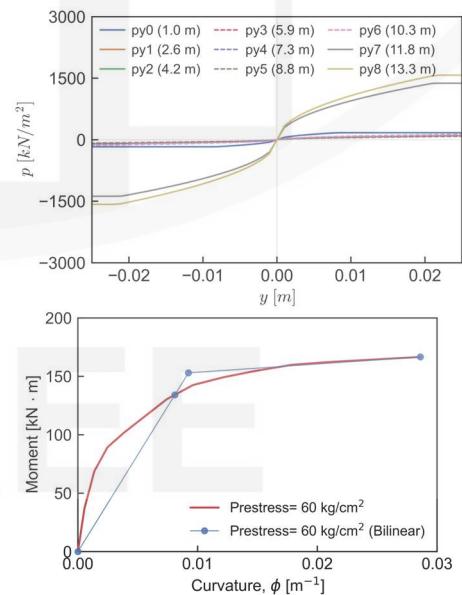


Fig. 5.  $p-y$  curves and moment–curvature relationship for the pile section

Table 1 Soil profile conditions and reduction factors

| Layer | USCS | $(N_i)_{60cs}$ | $FS$ (HBF) | $D_E$ |
|-------|------|----------------|------------|-------|
| 1     | ML   | 9.183          | 0.709      | 0.05  |
| 2     | ML   | 3.208          | 0.505      | 0.00  |
| 3     | ML   | 2.997          | 0.416      | 0.00  |
| 4     | CL   | 2.738          | 3.000      | 1.00  |
| 5     | CL   | 2.484          | 3.000      | 1.00  |
| 6     | CL   | 4.559          | 3.000      | 1.00  |
| 7     | CL   | 4.179          | 3.000      | 1.00  |
| 8     | SM   | 24.488         | 1.237      | 1.00  |
| 9     | SM   | 24.621         | 1.300      | 1.00  |

## Analysis Results

### (1) Lateral load–displacement curves of the pile

The seismic capacity curves of the pile under lateral loading, obtained from nonlinear pushover analyses for both non-liquefied and liquefied conditions, are presented in Fig. 6. As shown, the overall allowable lateral bearing capacity of the foundation under the non-liquefied condition is 164.7 kN, corresponding to a lateral displacement of 1.5 cm. Under the liquefied condition, the overall allowable lateral bearing capacity is 92.8–109 kN, corresponding to lateral displacements of 1.5–5 cm.

### (2) Performance verification of the piles

A comparison between the bending moments along the pile shaft and the cracking moment  $M_{cr}$ , yield moment  $M_y$ , and ultimate moment  $M_u$  of the pile section (Fig. 7(a)) shows that, under non-liquefied conditions, the bending moments along the shaft of the pile subjected to the maximum horizontal force (Foundation No. 4) remain below the cracking moment  $M_{cr}$ , indicating that all piles remain within the elastic range. Under liquefied conditions, the bending moments along the shafts of the piles beneath Foundation No. 4 are also below the cracking moment  $M_{cr}$ , indicating that the piles have not entered the ductile range. Because the applied moments remain below  $M_{cr}$  the cracking moment, the loading condition is considered safe and within the allowable limits. As shown in Fig. 7(b), the shear forces in the piles under external loading are substantially lower than the nominal shear strength, confirming that shear failure is prevented. The pile displacement results (Fig. 7(c)) show that, under non-liquefied conditions, the pile-head displacement (0.42 cm) is below the recommended allowable lateral displacement of 1.5 cm specified in the Design Code for the Foundation Structures of Buildings for short-term loading. Under liquefied conditions, the pile-head displacement is approximately 2.14 cm; although this exceeds the 1.5 cm allowable lateral displacement for short-term loading, it is within the 5.0 cm allowable lateral displacement corresponding to the ultimate limit state and therefore satisfies code requirements.

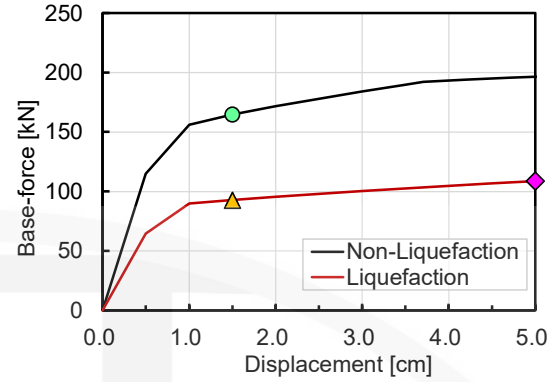


Fig. 6. Seismic response curves of the pile under lateral loading.

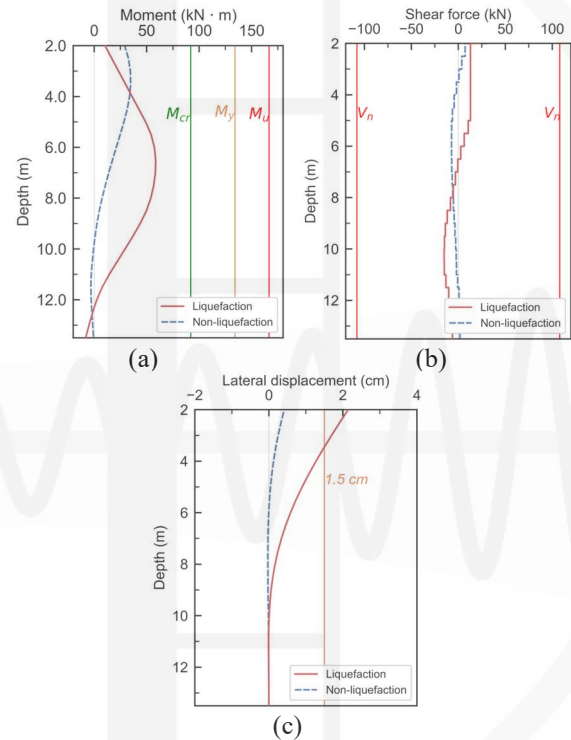


Fig. 7. Performance verification of the pile: (a) bending moment; (b) shear strength; (c) deflection.

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# Effect of Backfill Material on Buried Pipelines Subjected to Fault Displacement

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## Abstract

This study presents an experimental investigation of the behavior of embedded pipes subjected to fault displacement. The tests were conducted using a uniaxial shear box on a shaking table. Pipes of different diameters and embedment conditions were tested. Seismic motions of various intensity levels were applied. Observations and comparisons of these cases are informative and offer valuable insights into the influence of pipe size, the backfill condition of the pipe trench, shaking intensity, and fault displacement on the behavior of buried pipelines. The results of this study are helpful for enhancing seismic design and mitigating damage to buried pipelines subjected to seismically induced ground displacement.

**Keywords:** fault displacement, buried pipeline, backfill

## Introduction

Earthquake events such as fault movement, soil failure, or liquefaction occurring in proximity to fault zones can easily result in the destruction of buried pipelines. Protection measures have been discussed worldwide. Gantes and Melissianos (2016) discussed protection methods for buried pipelines subjected to fault rupture. These methods include reducing friction at the pipe-soil interface and increasing pipe strength. The friction-reduction measures discussed were placing the pipeline within concrete culverts without backfill material, increasing trench dimensions, or backfilling the trench with geofoam. That study stated that pipeline strain is reduced when the trench is backfilled with foam. Laboratory tests of pipeline response to earthquake-induced ground rupture have been conducted. Trautmann et al. (1985) conducted an experimental study to discuss the pipeline force-displacement response, the effect of pipeline burial depth, and soil types. Multiple split-box tests were carried out to study the response of buried pipelines subjected to fault movement. The effects of strike-slip faulting were presented in the studies of O'Rourke et al. (2008, 2010) and Demirci et al. (2021). Jalali et al. (2016, 2018) and Demirci et al. (2018) presented findings on the effects of reverse faults on buried pipelines. Different burial depths and backfilled sand were considered in their experimental studies. Quasi-static testing was

performed in addition to numerical simulation in most past research to capture the behavior of pipelines subjected to fault movement.

This paper presents the design concept and results of a shaking table test that captures the dynamic response of buried pipelines subjected to ground displacement due to seismic motion. According to the literature, the material surrounding a pipeline could affect the pipe response and is considered a protective measure, and pipe strength also affects pipeline performance. Therefore, different materials surrounding the pipeline were tested in this experimental study, as well as different pipe diameters, which represent the effect of pipeline strength. The results of this experimental study offer insights into the dynamic characteristics of buried pipelines during seismic motion, the effect of different pipe diameters on pipeline response, and the effect of different backfill materials on pipeline protection.

## Experiment Arrangements

The experimental configuration is shown in Fig. 1. A uniaxial shear box is located on the shaking table. The frame to the left of the shear box is used to mount the Temposonics sensors, which measure the relative displacements between the shear box sliding frames

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and the shaking table. The internal dimensions of the shear box sliding frames are 3.6 m by 1 m by 2.5 m in length, width, and height, respectively. The upper and lower portions of the shear box sliding frames were constrained separately as two levels. The shear box was designed to move separately at the upper and lower levels. The input motion was applied in the direction along the length of the shear box.

Copper pipes with diameters of 2.5 in and 4 in and a length of 2.6 m were the subjects of the experiment. Four copper pipes were positioned vertically in a diamond configuration, as shown in Fig. 2. One 2.5-in pipe and one 4-in pipe were located along the input motion direction and surrounded by Vietnamese quartz sand. Each of the other two 2.5-in pipes was placed in a box filled with polyurethane foam. This setup represents a pipeline trench backfilled with materials other than sand to discuss the possible protection provided by such material. The foam thickness was 30 cm for one 2.5-in pipe and 60 cm for the other 2.5-in pipe to discuss the effect of foam thickness.

Strain gauges were attached to two opposite sides of each of the four pipes. The configuration on one side of the strain gauges starts from the center, denoted as 0 cm, with 5-cm spacing to the locations of  $\pm 15$  cm and 15-cm spacing to the locations of  $\pm 105$  cm. This side is denoted as P. On the other side, denoted as the B side, the strain gauges start at 0 cm with 5-cm spacing to the locations of  $\pm 15$  cm, 15-cm spacing to  $\pm 30$  cm, and 30-cm spacing to  $\pm 90$  cm.

## Experiment Measurements

Strain gauge measurements of the input motion with a peak ground acceleration (PGA) of 0.05 g are shown in Fig. 3 to Fig. 6. Fig. 3 and Fig. 4 show the measurements of the 2.5-in and 4-in pipes surrounded by sand. Fig. 5 and Fig. 6 show the measurements of the 2.5-in pipes with 30 cm and 60 cm foam, respectively. The notation of the strain gauges indicates, first, the sensor type; then the pipe diameter, followed by the surrounding material type; and finally, the strain gauge location. Taking S2.5-S+B60 as an example, the first letter, S, indicates that it is a strain gauge, followed by the number 2.5, meaning that it is a 2.5-in pipe. The second S indicates that the pipe is surrounded by sand; lastly, +B60 indicates the strain gauge location, where the letter B identifies one of the two sides on which the strain gauges are located. For the cases with foam, the notation 30F and 60F is used, representing 30 cm foam and 60 cm foam, respectively. This notation replaces S, which stands for sand, as mentioned above.

Measurements collected from the 0.05 g PGA case were quite small. The 0.1 g PGA case shows similar results, producing small strains. The 4-in pipe case produces a smaller strain range compared with the

other three 2.5-in pipes. When the input motion PGA increased to 0.2 g, the 2.5-in pipe with sand had higher strain and a larger strain range compared with the other three pipes, as shown in Fig. 12 to Fig. 14. The two 2.5-in pipes with foam had similar performance, and the 4-in pipe with sand had the smallest strain level and strain range.

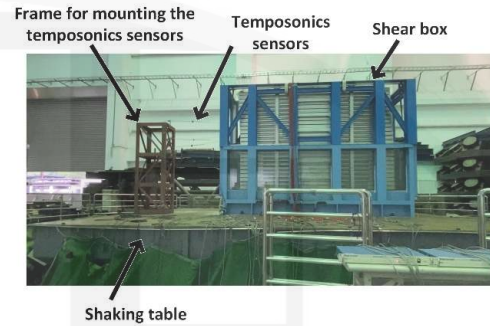


Fig. 1. Experiment configuration

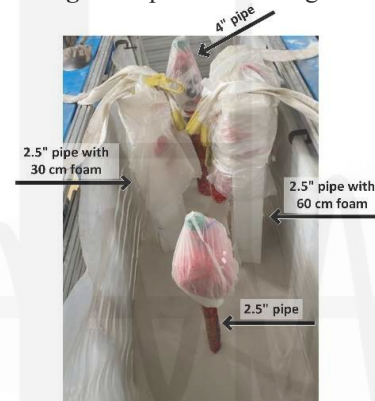


Fig. 2. Layout of the tested pipes

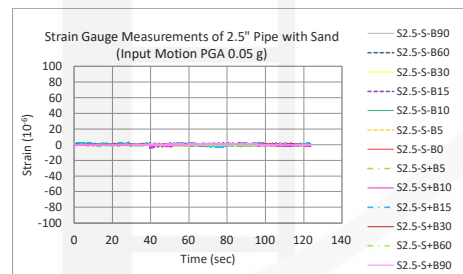


Fig. 3. Strain gauge measurements of the 2.5-in pipe with sand for the input motion with a PGA of 0.05 g.

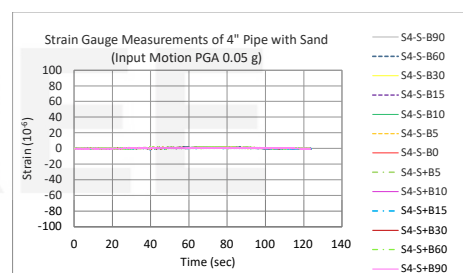
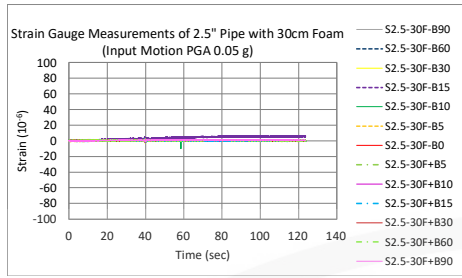
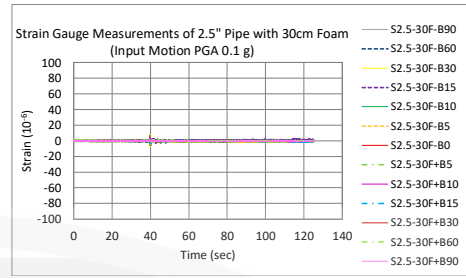


Fig. 4. Strain gauge measurements of the 4-in pipe with sand for the input motion with a PGA of 0.05 g.

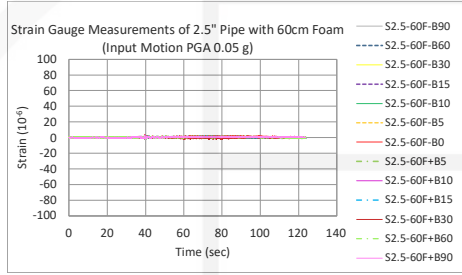




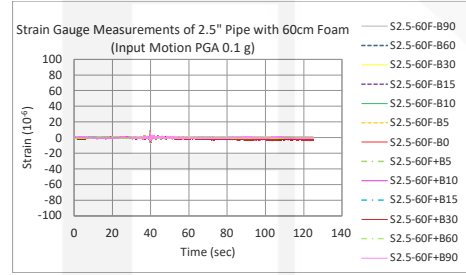
**Fig. 5.** Strain gauge measurements of the 2.5-in pipe with 30 cm foam for the input motion with a PGA of 0.05 g.



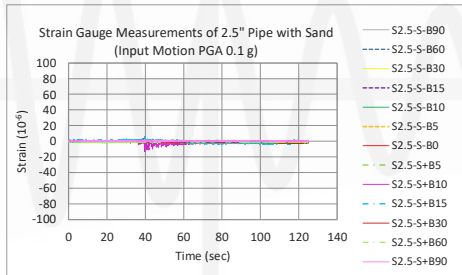
**Fig. 9.** Strain gauge measurements of the 2.5-in pipe with 30 cm foam for the input motion with a PGA of 0.1 g.



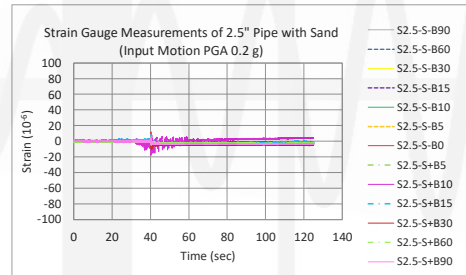
**Fig. 6.** Strain gauge measurements of the 2.5-in pipe with 60 cm foam for the input motion with a PGA of 0.05 g.



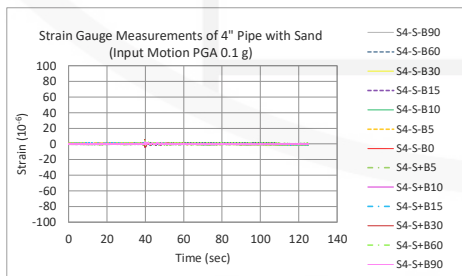
**Fig. 10.** Strain gauge measurements of the 2.5-in pipe with 60 cm foam for the input motion with a PGA of 0.1 g.



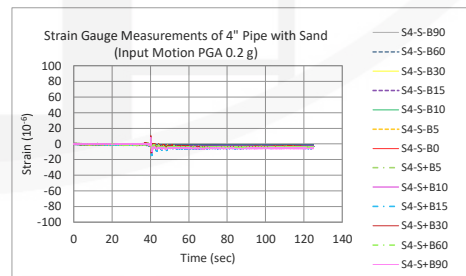
**Fig. 7.** Strain gauge measurements of the 2.5-in pipe with sand for the input motion with a PGA of 0.1 g.



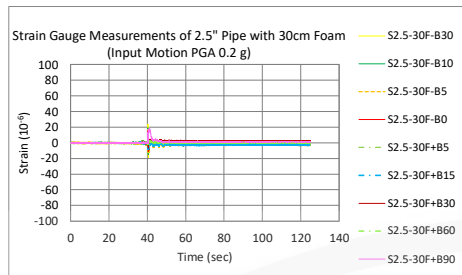
**Fig. 11.** Strain gauge measurements of the 2.5-in pipe with sand for the input motion with a PGA of 0.2 g.



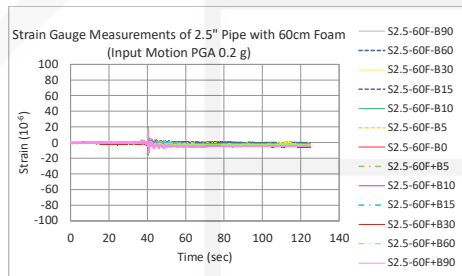
**Fig. 8.** Strain gauge measurements of the 4-in pipe with sand for the input motion with a PGA of 0.1 g.



**Fig. 12.** Strain gauge measurements of the 4-in pipe with sand for the input motion with a PGA of 0.2 g.



**Fig. 13.** Strain gauge measurements of the 2.5-in pipe with 30 cm foam for the input motion with a PGA of 0.2 g.



**Fig. 14.** Strain gauge measurements of the 2.5-in pipe with 60 cm foam for the input motion with a PGA of 0.2 g.

## Conclusions

An experimental study was conducted to observe the dynamic behavior of copper pipes subjected to earthquake-induced fault displacement. Four pipes were tested. One 2.5-in pipe and one 4-in pipe were surrounded with sand, while the other two 2.5-in pipes were embedded in pipe trenches backfilled with 30 cm and 60 cm foam. Records from station TCU068 during the 1999 Chichi earthquake were used as the input motion in this study. By scaling the PGA to 0.05 g, 0.1 g, and 0.2 g, three input motion levels were applied. The effects of input motion intensity, pipe diameter, and surrounding embedment material are discussed by comparing the strain gauge measurements of the four tested pipes.

Results show that pipe strains are small for the PGA 0.05 g and 0.1 g cases. Comparing the 2.5-in and 4-in pipes surrounded by sand, the strains in the 4-in pipe are much smaller than those in the 2.5-in pipe. The strains in the two 2.5-in pipes embedded in foam are smaller than those in the 2.5-in pipe surrounded by sand, but the reduction is not as great as in the case of the 4-in pipe surrounded by sand. The effect of the foam appears to be reducing the strain. The two 2.5-in pipes embedded in foam have similar behavior.

This study provides insight into the dynamic response of buried pipelines subjected to earthquake-induced ground displacement. The effects of pipe size and pipe trench material are noticeable. Such effects should be considered in pipeline system design and

damage mitigation.

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# Development of a Software-Hardware Framework for an AI Assistant in Disaster Prevention and Emergency Response

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楊承道<sup>1</sup>、陳蓮安<sup>2</sup>、黃士軒<sup>2</sup>

## Abstract

Large-scale disaster response requires integrating information from multiple sources, including emergency reporting systems (e.g., 119), databases, geographic information systems (GIS), and communication platforms. However, when managing large volumes of unstructured data such as incident reports, news articles, and real-time updates require manual interpretation and organization, which remain heavily relied upon, often causing information delays, inconsistent assessments, and coordination difficulties among agencies during major events. To address these challenges, this study presents the development of an AI-driven disaster prevention and emergency response system, in which large language models (LLMs) serve as the core analytical engine. The system integrates LLMs with databases, GIS, and communication platforms to support semantic analysis, event categorization, disaster-site identification, and real-time information sharing. A hybrid cloud architecture balances computational performance, real-time processing, and data security. Without changing existing operational workflows, the proposed system improves disaster-information processing and decision support, reduces manual workload, and enhances emergency-response efficiency and system resilience.

**Keywords:** AI Assistant, Large Language Models (LLMs), Disaster Information Integration, Integrated Software and Hardware Framework, Hybrid Cloud Architecture

## Introduction

With the rapid advancement of artificial intelligence technologies, the integration of autonomous agents and large language models (LLMs) have become increasingly sophisticated [1]. Since the development of systems such as AutoGPT, complex workflows can be performed with minimal human input through tool use and multi-step task planning. However, open-ended agent architectures continue to face practical challenges, including limited transparency in decision-making, high computational costs, and security risks in cross-system integration, especially for domains related to public safety and critical infrastructure.

Disaster prevention and emergency response operations require rapid action and coordination across multiple agencies. Information comes from varied and mostly unstructured sources, including incident re-

ports, news stories, and communication logs. Conventional methods depend on manual data collection and rule-based systems, resulting in delayed information, inconsistent interpretations, and increased workload. Advances in generative AI, such as LLMs, have enhanced semantic analysis and information extraction, offering potential to improve disaster information processing.

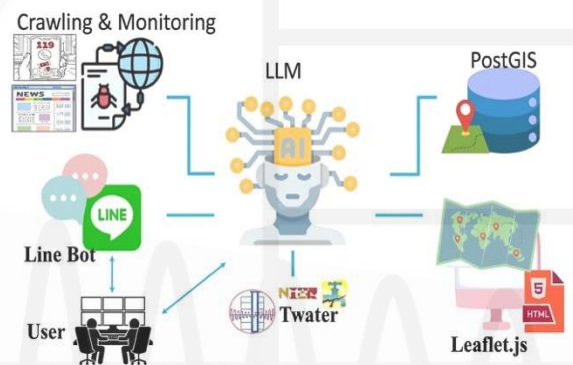
As shown in Fig. 1, this study focuses on the Disaster Prevention and Emergency Operation Center of the Taipei Water Department and addresses key requirements, including disaster information integration, disaster-site localization, and post-earthquake pipeline damage estimation. Accordingly, a software-hardware framework for an AI assistant in disaster prevention and emergency response is developed. The framework integrates multiple data sources, including news data, incident reports, and earthquake early warning information, and employs LLMs to extract disaster types, locations, and event summaries, which are further

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transformed into geospatial information for spatial decision support. In addition, the Twater model is incorporated to estimate the spatial distribution of pipeline damage following earthquakes.

A database-oriented agent layer is introduced, using the database as an intermediary between the AI system and external services to enable controlled data exchange and access management. This design improves data traceability and reduces security risks associated with open tool invocation. Disaster-response tasks are decomposed into key stages: event classification, location extraction, summarization, and GIS-based analysis. This approach allows for the integration of language models and engineering models, transforming manual workflows into a systematic and real-time decision-support process.



**Fig. 1.** Framework of the AI Assistant for Disaster Prevention and Emergency Response

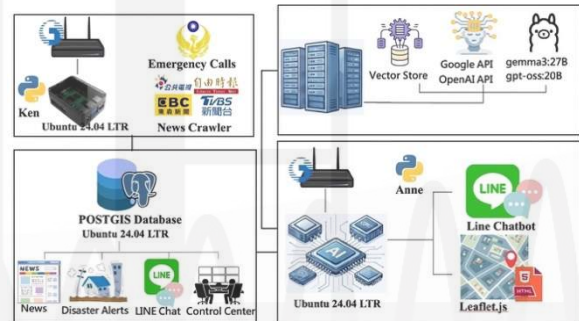
## Software–Hardware Framework

As shown in Fig. 2, a distributed deployment architecture is adopted to integrate server computing nodes, database systems, and lightweight communication interfaces, forming a disaster information processing platform with high availability and real-time responsiveness. The hardware layer consists of multi-node configurations, including high-performance servers and edge computing devices, with services deployed on the Ubuntu operating system.

At the software level, PostgreSQL/PostGIS [2] serves as the central database, integrating news, emergency dispatch, earthquake data, and user interaction records. Database triggers and a LISTEN/NOTIFY event-driven mechanism deliver data updates to external applications in real time, eliminating the delays and processing costs associated with traditional polling. Web crawlers gather information from various news sources, while connections to the emergency dispatch system supply time-critical event data. Together, these sources provide both historical and real-time information, forming a comprehensive disaster-information platform.

In the intelligence layer, LLMs and a vector database are integrated to design a retrieval-augmented generation (RAG) framework. Information from multiple sources is converted into structured knowledge through scheduled and real-time updates, enabling dependable and context-aware decision support across various time frames. Data provided to the RAG framework is primarily prepared through Python scripts, including preprocessing tasks such as cleaning, coordinate conversion, and event classification.

At the application layer, GIS and communication platforms are combined to provide real-time visualization of disaster information. A LINE Bot serves as the main interface between users and the system, supporting event-triggered notifications and proactive message delivery. This creates a comprehensive workflow for data collection, analysis, spatial visualization, and real-time notification.



**Fig. 2.** Architecture of the Software–Hardware Framework

## Disaster Point Localization

Rapid disaster-location identification and geospatial integration are critical for decision support. A framework combining semantic analysis and GIS is developed to process multi-source unstructured disaster information. LLMs are used to extract addresses and event information, which are converted into spatial coordinates through geocoding for further analysis.

Spatial analysis methods, including point-in-polygon analysis and buffer analysis, are applied to identify service zones and nearby critical facilities. Spatial indexing techniques (e.g., R-tree) are employed to improve computational efficiency and ensure real-time performance.

Multi-source data are reclassified and standardized to improve quality and consistency. The processed data are combined with geographic layers and visualized using interactive maps built with Folium, Leaflet, and Flask. A LINE interface enables real-time querying and notifications.



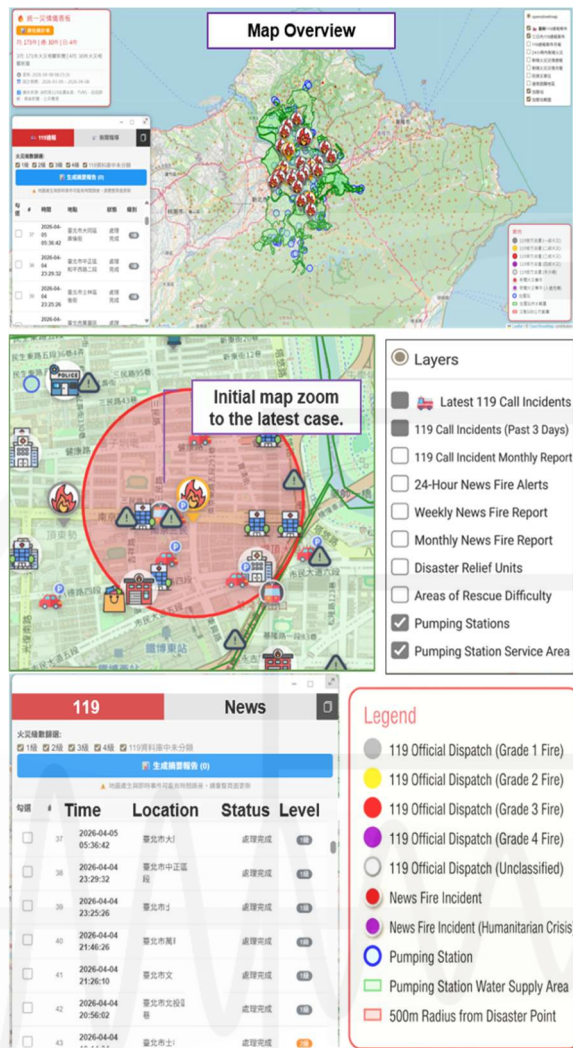


Fig. 3. Geospatial Analysis of Disaster Point Localization

## Pipeline Damage Estimation

The proposed framework further integrates Twater [3-4], the early earthquake loss assessment framework developed by the National Center for Research on Earthquake Engineering, National Institutes of Applied Research (hereinafter referred to as “NCREE”), with the pipeline database of the Taipei Water Department. Upon receiving earthquake early warning reports from the Central Weather Administration, key parameters are used to rapidly match candidate simulated earthquake scenarios, providing a basis for subsequent damage analysis.

Following the earthquake event, the system incorporates the Twater model to estimate pipeline damage based on pipeline characteristics such as seismic resistance level, diameter, and material properties. Damage rates are calculated using established empirical formulas to estimate the probability of pipeline failure and further aggregated to quantify the number of damaged pipelines across different regions.

The estimated damage outcomes are transformed into geospatial information and visualized through interactive GIS-based maps, highlighting damage distribution and potential hotspots. These results are disseminated in real time via communication platforms, allowing users to query earthquake events and corresponding pipeline damage estimations through the LINE interface. This approach enables rapid situational awareness of spatial damage patterns and supports timely, informed decision-making in post-earthquake emergency response.

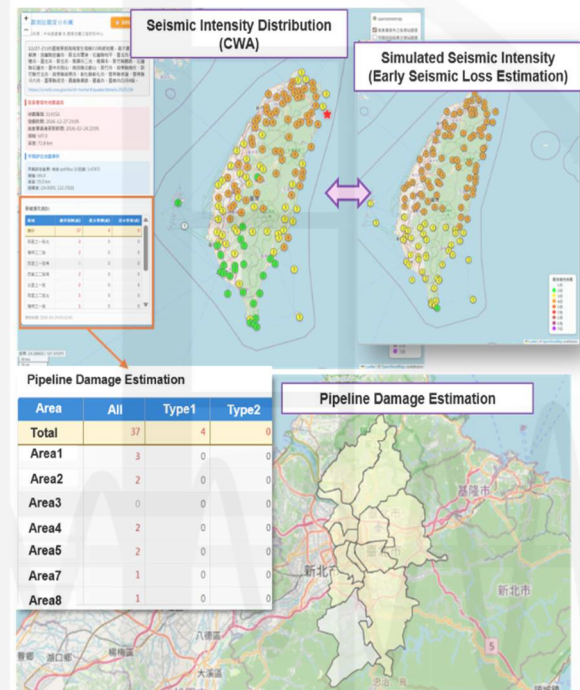


Fig. 4. Estimation and Spatial Distribution of Pipeline Damage After Earthquakes

## Conclusions and Future Work

A software and hardware framework for an AI assistant in disaster prevention and emergency response was developed and implemented at the Taipei Water Department. By integrating LLMs, databases, GIS, and earthquake damage assessment models, the platform enables data processing, semantic analysis, spatial integration, and real-time notifications. This framework replaces manual workflows with a systematic, real-time decision-support process, enhancing efficiency and coordination.

Future work includes integrating model fine-tuning and domain knowledge to enhance accuracy and reliability, broadening data sources and application areas, and increasing system scalability. The proposed framework could serve as a core element for smart-city disaster management and strengthen urban resilience and public safety.

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NCREE

# Applications of Artificial Intelligence Technologies in Disaster Prevention and Rescue

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## Abstract

This report presents eight research outcomes and practical applications in AI-driven disaster prevention and rescue, developed by the NCREE-NTU Joint Artificial Intelligence Research Center (AI Center). The scope covers the entire disaster management cycle, from pre-disaster risk assessment to post-disaster recovery. Key projects include an AI decision support platform that integrates multi-source data using a 3D ontology framework; large-scale post-disaster damage assessment combining remote sensing and deep learning; autonomous UAV inspection and crack quantification; coordinated automated inspection using multiple vehicles for construction engineering; indoor assessment tools that merge augmented reality (AR) with machine learning; AI-assisted rapid seismic screening using image recognition; a disaster response decision engine that integrates large language models (LLMs) with Geographic Information Systems (GIS); and a multi-speaker speech recognition and semantic data integration system. Each project has been validated or applied in real-world scenarios, demonstrating the feasibility and effectiveness of AI technologies in disaster prevention.

**Keywords:** AI, disaster prevention and rescue, deep learning, large language models

## Research Background and Objectives

In response to the growing frequency of natural disasters and the varied risks associated with climate change, artificial intelligence (AI) plays an increasingly vital role in disaster monitoring, emergency response, and recovery. Recent advancements in deep learning and large language models (LLMs) have enabled new approaches for disaster prevention, including

improved image recognition, natural language processing, and the integration of multiple data types.

However, significant challenges persist in moving AI technologies from research to operational use. These include scattered and inconsistently formatted disaster prevention data, which hinders interoperability across systems; limited generalizability and reliability of AI models trained on insufficient or imbalanced datasets; and the need for real-time, low-latency performance in disaster scenarios, where traditional

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cloud-based deployment may be inadequate. Effectively translating academic research into deployable disaster prevention tools is now a pressing concern.

The NCREE-NTU Joint Artificial Intelligence Research Center (hereafter, the AI Center) has focused on researching, developing, and applying AI in disaster prevention and rescue. This report presents eight research outcomes from the AI Center and its partners, covering AI-based decision support platforms, large-scale post-disaster building and road damage assessment, intelligent UAV structural inspection, automated construction management, indoor building assessment, seismic risk screening, integration of LLMs with Geographic Information Systems (GIS), and multi-speaker speech recognition with semantic extraction. These topics span the entire disaster management cycle, including pre-disaster risk assessment, disaster response, and post-disaster recovery and reconstruction.

The AI Center held the “Forum on Frontier Applications of AI in Disaster Prevention and Rescue” on January 9, 2026, bringing together research, government, and industry teams to share the latest outcomes and application cases and to promote cross-disciplinary collaboration. Commissioner Shih-Lung Chao of the Taipei City Department of Information Technology and Commissioner Huai-Chu Mo of the Fire Department also attended and delivered remarks, expressing their expectation that AI technologies will continue to be applied in urban disaster prevention, providing important support for the practical advancement of this research series (Fig. 1).



**Fig. 1.** Group photo from the Forum on Frontier Applications of AI in Disaster Prevention and Rescue.

## AI Disaster Prevention Research Outcomes

### Taipei Lens: An AI-Native Decision Support Platform for Emergency Response

This research developed an AI decision platform based on the Palantir Ontology architecture, integrating diverse data sources, including asset management, geographic information, weather models, and SCADA

grid sensing, into an Ontology knowledge graph, with a 3D visualization interface for urban disaster situational awareness. The platform serves as an “AI decision mediation layer,” connecting data and AI models with decision logic, rather than replacing existing data or prediction models. Its AI engine processes citizen reports, consolidates intelligence periodically, supports natural language inference and querying, and dynamically tracks monitoring indicators. These capabilities help commanders improve disaster awareness and convert data into actionable decision recommendations.

In practice, the system has been used for road tree incident dispatching in Taipei City, where up to 70 cases per hour can occur during typhoons. It automatically generates action directives, such as “dispatch one vehicle and three personnel from a contractor,” which improves decision speed and quality [1].

### AI Applications for Post-Disaster Large-Scale Building and Road Damage Assessment

This research integrates large-scale remote sensing imagery with AI to identify post-disaster damage to buildings and roads. The system uses a stacking ensemble architecture that incorporates YOLOv8 for building detection, MobileNetv3 for image classification, and Transformer models to fuse features from optical and Synthetic Aperture Radar (SAR) imagery. To address various disaster types and damage levels, the system classifies building damage and evaluates road passability, producing damage distribution maps. The research also examines the challenge of class imbalance in disaster datasets and the generalizability of deep learning models, aiming to provide timely large-scale damage information to support government agencies and disaster response teams in emergency rescue and recovery planning [2].

### Intelligent UAV Inspection and Crack Quantification

This research presents an automated structural health diagnosis system using UAVs as the primary platform. The system leverages Deep Reinforcement Learning (DRL) to drive AI models that integrate crack identification with exploration strategies, enabling drones to autonomously capture all potential damage zones. The hardware includes a Jetson Nano onboard computer for real-time inference and five high-precision obstacle avoidance sensors. The software uses AI to detect and analyze crack images and identify crack centerlines, employing U-Net-ResNet-34 for image segmentation, 3D reconstruction, and plane fitting. Projection matrix correction addresses 3D distortion caused by non-perpendicular camera angles, allowing accurate quantification of crack widths and lengths and the automatic generation of structural health diagnosis reports. The system has been validated through flight tests and is suitable for both pre-



disaster preventive monitoring and post-disaster damage assessment, significantly reducing labor costs and safety risks [3].

### **Automated Management and Inspection for Construction Engineering and Disaster Prevention**

This research aims to improve automated production and inspection efficiency in construction engineering by replacing the uncertainty and risk of manual inspection with autonomous sensing and data analysis. The system combines Building Information Modeling (BIM) and point cloud processing. It uses multi-modal visual data to interpret site conditions, quantify progress, verify safety equipment compliance, monitor worker behavior, and detect hazardous zone intrusions. Deep learning methods enhance defect detection and assessment consistency, supporting cost, schedule, and risk management.

The research expands the inspection system to enable multi-vehicle coordination, integrating autonomous navigation and path planning. This allows multiple robotic platforms to cover large sites simultaneously while conducting real-time monitoring and information sharing. The technology can also be applied to post-disaster rapid inspection, hazard detection, and structural resilience assessment [4].

### **Indoor Building Assessment Methods and Tools**

This research introduces an indoor building assessment system that combines augmented reality (AR) and machine learning. Based on ARKit and RoomPlan, the system reconstructs 3D indoor spaces in real time and uses a YOLOv8 object detection model to automatically identify structural defects, such as wall cracks, concrete spalling, and exposed reinforcement, during scanning. Inspectors can annotate and measure defect dimensions directly through AR overlays, with data automatically mapped to 2D floor plans and compiled into standardized reports. Field tests show that the system reduces inspection time by 30–50%, increases measurement accuracy, lowers labor demands, and improves data integration and documentation efficiency [5].

### **AI-Assisted Rapid Seismic Risk Screening of Buildings**

To address the large inventory of aging buildings constructed before the 1999 Chi-Chi earthquake, whose seismic capacity requires efficient evaluation, this research developed an AI image recognition system capable of processing tens of thousands of digitized building management blueprints. Using computer vision to automatically identify structural floor plans, the system extracts key parameters and calculates the basic seismic performance E-value and seismic capacity index  $I_s$ . For example, in a two-story townhouse, the column-wall area computation time is reduced from 90 minutes to 1 minute, and the total assessment

time from 4.5 hours to approximately 1 hour. The AI system can assess approximately eight buildings per day, compared to two by manual review, which is a fourfold improvement in efficiency.

This technology has been used to screen 1,144 buildings for seismic risk in Hualien City. Results were classified into red, yellow, and green risk categories and analyzed with the TELES earthquake risk assessment system to assess seismic damage and road blockage risks. The findings support government agencies in disaster prevention, urban renewal, and structural reinforcement planning [6].

### **Building an Intelligent Disaster Response Brain: When Large Language Models (LLMs) Meet GIS**

This research introduces the concept of an “AI Disaster Response Assistant” that integrates LLMs with GIS and damage estimation models to create a disaster response decision engine. The system can identify high-risk areas and potential disaster points in the early stages of an event, automatically classify and geolocate disaster reports, and convert unstructured notification data into integrated visualization maps for decision-making. Using the “Poseidon Project” for water monitoring at the Taipei Water Department as a case study, the system employs a LINE chatbot interface and can automatically generate seismic intensity and pipeline damage risk distribution maps. It also connects with the 119 dispatch system and performs automated fire news analysis. By leveraging LLM-based automatic analysis and geolocation, the system guides disaster response actions, reduces personnel mobilization costs, and improves overall response efficiency [7].

### **Multi-Speaker Speech Recognition, Semantic Extraction, and Data Integration**

Speech is the most common and accessible form of information in daily life. This research shows how AI speech recognition and generative AI technologies can transform unstructured voice data into structured information. The system uses the open-source multi-speaker automatic speech recognition model WhisperX for speech-to-text conversion, and LLMs for semantic analysis, topic classification, and information extraction. The system converts conversational content from meetings and post-disaster care interviews into structured data compatible with existing documentation and reporting systems. In post-disaster care interviews, for example, the system helps aging volunteer teams by addressing the challenge of conducting interviews and keeping records simultaneously. By incorporating Retrieval-Augmented Generation (RAG) technology, the system provides real-time interactive suggestions during interviews, reducing the need for manual documentation and improving information transfer efficiency. Disaster response personnel, social

workers, and volunteers can all benefit from this technology [8].

## Conclusions and Future Outlook

The eight research outcomes in this report cover the entire disaster management cycle, from pre-disaster risk assessment and response to post-disaster recovery. Several technologies have progressed from proof-of-concept to field deployment, demonstrating that AI disaster prevention technologies are moving from research to practical use. Three common trends and challenges emerge across all outcomes:

- (1) Multi-modal data fusion is a core technique, with studies consistently highlighting the integration of diverse data types to enhance interpretation accuracy and system resilience;
- (2) Edge deployment and autonomy are essential for practical implementation, meeting the need for low-latency, real-time inference and autonomous decision-making in disaster prevention;
- (3) Cross-system data interoperability and standardization remain unresolved bottlenecks. Establishing effective data standards and exchange mechanisms will determine whether AI technologies can scale from isolated applications to integrated systems.

Furthermore, validating model reliability, ensuring training data quality, and developing technology transfer mechanisms are challenges shared across all research efforts. In response, the AI Center will continue to focus on:

- (1) Establishing data standards and exchange protocols across systems;
- (2) Strengthening model generalizability and reliability validation across various disaster scenarios;
- (3) Promoting more field pilots and practical deployments, while facilitating collaboration between R&D teams and disaster prevention agencies through forums, workshops, and other exchange mechanisms, to accelerate the deployment of AI disaster prevention technologies.

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# 5D Smart City Disaster Prevention and Response Platform: Earthquake Intelligent Disaster Prevention Technology

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## Abstract

In alignment with the development trends of digital twin-based disaster prevention and response technologies, the National Center for Research on Earthquake Engineering, National Institutes of Applied Research (hereinafter referred to as “NCREE”) has continuously advanced the 5D SmartES platform by integrating 3D visualization technologies for road blockage analysis.

Using Kaohsiung City as a case study, road blockage scenarios and emergency routes are displayed on the 5D SmartES platform. Developing road blockage and emergency route technologies requires comprehensive data collection on buildings, bridges, and roads in Kaohsiung City. The analysis can determine which roads are likely to be blocked during various earthquake scenarios and automatically generate optimal emergency routes.

In addition, the platform provides detailed information about buildings, such as age and structural type (e.g., reinforced concrete or masonry), which improves situational awareness. These features help local governments conduct disaster prevention exercises and scenario simulations, such as the Wan-An Exercise.

This project aims to serve local governments by facilitating digital transformation in disaster preparedness and emergency response. Through the successful implementation of 5D digital twin technology, the platform enables rapid identification of affected areas after disasters, remote assessment of surrounding environments, and efficient post-disaster response and recovery operations.

**Keywords:** 5D SmartES, road blockage, emergency route

## Introduction

With the increasing frequency of extreme weather events and natural disasters, urban infrastructure faces greater risks. Traditional monitoring and management methods rely on isolated systems, resulting in fragmented data and poor integration. This limits real-time response and comprehensive decision-making. During earthquakes, combining multiple datasets and achieving real-time awareness of urban operations remain key challenges for smart city development.

In recent years, digital twin technology has been increasingly applied in urban governance. By

integrating 3D building models with related datasets, it enables the creation of a virtual urban environment linked to physical systems. Combined with artificial intelligence, digital twin technology improves disaster prediction, risk assessment, and decision-making. This integration allows monitoring data from infrastructure systems, such as buildings and roads, to be visualized and analyzed, enabling authorities to quickly access critical information during disasters and enhance response efficiency.

This project supports digital twin-based disaster prevention and response technologies and aims to expand the capabilities of the 5D Smart City Disaster Prevention and Response Platform. By integrating

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building data and emergency route information, the project creates an intelligent disaster management system with early warning and decision-support features. The platform strengthens the effectiveness of local government emergency operations centers, improving urban resilience and public safety.

The primary elements of this project are:

1. **Multi-level 3D Building Information Visualization:** This function involves collecting illegal construction data and establishing fundamental building information. Through multi-layer 3D visualization, disaster-related information can be displayed at the individual building level within a 3D GIS environment, including attributes such as illegal constructions and old buildings.
2. **Emergency Route Information Visualization:** This function further expands advanced disaster management visualization capabilities by displaying emergency routes. Through multi-layer mapping, the platform can present detailed information on emergency routes to support efficient emergency response plans.

## 5D Smart City Disaster Prevention and Response Platform

This project develops the 5D Smart City Disaster Prevention and Response Platform (Fig. 1), which is based on a 3D urban model generated from aerial imagery. By incorporating data from buildings, roads, and other essential infrastructure, the platform creates a digital twin framework for urban disaster prevention and response.

The platform integrates multiple data sources and displays the city's operational status using 3D models, allowing management authorities to access key information in real time and support emergency response and decision-making. The platform's main functions include:

1. Establishment of a 3D urban model and digital twin environment;
2. Disaster risk analysis and early warning mechanisms;
3. Decision support and emergency response assistance for disaster management.

In addition, the platform integrates multiple disaster prevention technologies into a single system, enhancing data sharing, improving emergency response efficiency, and strengthening decision-making capabilities.

The developed system has been successfully implemented in Kaohsiung City's smart city governance framework, demonstrating the feasibility

and practical value of digital twin technology in disaster prevention and response. Overall, the platform improves urban disaster monitoring, early warning, and emergency response, while strengthening public safety and urban resilience.



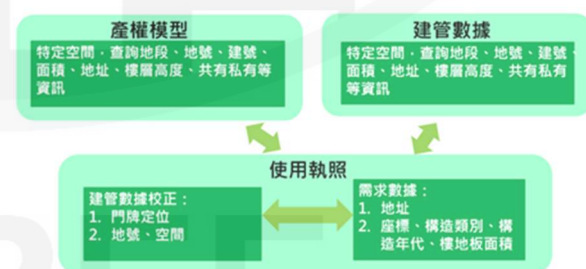
**Fig. 1.** 5D Smart City Disaster Prevention and Response Platform (Source: Public Works Bureau, Kaohsiung City Government)

## Platform Application Case

### 1. Multi-level 3D Building Information Visualization Function

To enhance the integration and application of urban spatial information, this project develops a multi-level 3D building information visualization function, which integrates building geometric models with attribute data and presents them through the platform in a visualized format.

The platform incorporates data from the Kaohsiung City Government, including building management records and usage permits, to create comprehensive datasets of building attributes. These datasets contain details such as main and secondary building types, height, floor area, and address. After importing, users can query and access detailed information for each building (Fig. 2).



**Fig. 2.** Relationship diagram of building attribute data sources

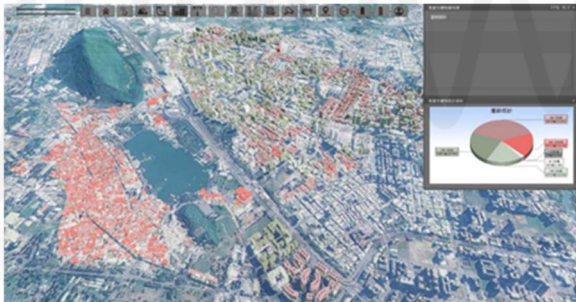
The platform enables real-time querying of building attributes. Users can select a building in the 3D environment to instantly access relevant information, improving the intuitiveness and efficiency of data retrieval (Fig. 3).





**Fig. 3.** Building attribute data query interface (Source: Kaohsiung City Public Works Bureau)

In addition, the platform enhances its functional framework by integrating building permit data from the Kaohsiung City Government's Building Administration Office with building age data from NCRE. These datasets are incorporated into the platform, enabling users to select 3D building models and query basic building information. The platform also displays all buildings within a specified area as 3D models, providing details such as location, structural materials, and construction period. Different colors represent building age categories, allowing users to easily identify the distribution of newer and older buildings within the area (Fig. 4).



**Fig. 4.** Map of building age distribution (Source: Public Works Bureau, Kaohsiung City Government)

Through this platform, building model attribute data can be effectively integrated, enhancing the platform's information visualization capabilities and supporting subsequent disaster prevention analysis and urban governance applications.

## 2. Emergency Route Information Visualization

### A. Earthquake Scenario and Road Blockage Analysis

To improve the assessment of transportation disruption risk and the evaluation of emergency routes after earthquakes, this project develops a methodology for analyzing road blockage risk during earthquake scenarios. The study examines the 12 administrative districts of Kaohsiung City as the research area, providing a foundation for planning emergency routes.

The platform establishes a workflow for earthquake scenario simulation and road blockage analysis (Fig. 5), integrating spatial data on buildings, roads, and bridges. By applying earthquake damage assessment and road network analysis methods, a comprehensive analysis is performed. The main steps include:

1. Development of a seismic attribute database for buildings: Statistical and spatial analyses of building inventory and seismic characteristics are performed to improve the accuracy of post-disaster building damage estimation;
2. Application of the Taiwan Earthquake Loss Estimation System (TELES), developed by NCRE, to evaluate seismic damage to buildings and road systems;
3. Utilization of the Tgbs module (a subsystem of TELES) to estimate building damage;
4. Application on highways for bridge and road damage estimation, along with the Rescue Road Planning and Seismic Blockage Risk Assessment Technology (ROSA) for road blockage risk assessment.

Through this analytical workflow, the platform enables a comprehensive assessment of road blockages and traffic accessibility risks after an earthquake, providing essential support for emergency route planning and disaster response decisions.



**Fig. 5.** Workflow of earthquake simulation and road blockage analysis

In earthquake loss assessment in Taiwan, the choice of analytical units significantly affects the accuracy and relevance of the results. Traditional methods usually use administrative units (such as villages or neighborhoods) or 500-meter grid cells for analysis. Administrative units help local governments assess challenges within their jurisdictions and develop disaster prevention strategies, but they cannot capture differences in population density or efficiently incorporate information about nearby emergency routes. Grid cells are useful for combining with other hazard datasets, but they must be translated into administrative boundaries for decision-making and do

not directly connect with emergency route data.

This study uses urban blocks and buffer zones around emergency routes as the main units of analysis. By integrating spatial data on buildings, population, and road networks, this approach produces results that more accurately represent real urban structures. Analyzing at the block level aligns well with existing administrative boundaries and helps reveal spatial relationships among population density, buildings, and roads. Additionally, evaluating buffer zones along emergency routes supports more precise assessments for emergency planning and response.

### **B. Emergency Route Plan and Alternative Path Analysis**

Following the earthquake simulation and road blockage analysis, this project develops emergency route plans by integrating seismic damage assessments with road analysis methods to design post-disaster rescue routes.

The platform incorporates the ROSA technology developed by NCREE. It performs integrated analyses of building damage, population impacts, and road blockage conditions caused by earthquakes, supporting disaster management agencies in evaluating and improving strategies for pre-disaster preparedness, emergency response, and post-disaster recovery.

From a methodological perspective, the system identifies road blockage locations and their impact areas under various disaster scenarios, then determines optimal emergency routes and alternative paths. These findings offer essential guidance for emergency response teams in resource allocation and route planning during disaster operations.

In addition, the platform incorporates data on population distribution, building clusters, and the accessibility of emergency routes to assess rescue priorities, thereby improving disaster response efficiency.

## **Conclusions and Future Work**

This project successfully developed the 5D Smart City Disaster Prevention and Response Platform, integrating data such as bridge information, building records, and usage permits to create a comprehensive database of building spatial and attribute details. Using the digital twin platform, these datasets are visualized to improve the integration and application of urban spatial information.

In terms of applications, this project developed analytical and decision-support functions for earthquake disaster emergency routes. By integrating earthquake loss assessment with road analysis

techniques, the system performs road blockage risk assessment and emergency route planning. The analytical results are visualized and integrated within the platform, enabling management authorities to effectively understand post-disaster road accessibility and support timely decision-making.

Overall, the outcomes of this project improve the assessment of road risks and increase rescue efficiency during earthquakes. By integrating and visualizing building attribute data, the platform further enhances disaster information visualization and supports decision-making.

For future work, the platform can be enhanced by expanding building datasets and improving data update processes, as well as strengthening real-time data integration and analysis to increase data timeliness and accuracy. In addition, integrating multi-hazard scenario analysis and cross-system data applications will further improve the precision of emergency route planning and response, while broadening the use of digital twin technology in smart city management.

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# Water Environment Intelligent Disaster Prevention Application Platform

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## Abstract

In recent years, more frequent extreme rainfall events have increased flood and landslide risks in Taiwan. However, monitoring data remain fragmented across multiple ministries and in varying formats, while current disaster prevention systems lack cross-domain integration and real-time visualization. To address these gaps, this study developed the Water Environment Intelligent Disaster Prevention Application Platform, using the Yanshui River basin in Tainan and the Pu-Tun-Pu-Nas River basin in Kaohsiung as demonstration sites. The platform integrates flood simulation and landslide susceptibility prediction subsystems. Built with the ArcGIS Maps SDK for JavaScript (hereafter, ArcGIS Maps SDK), the platform generates 3D scenes that combine flood simulation results from the D-Flow Flexible Mesh (D-Flow FM) hydrodynamic model and landslide probabilities calculated by machine learning models (XGBoost, Random Forest, Gradient Boosting, and Logistic Regression). Key features include conditional layer triggering, IoT sensor data comparison, and machine learning-based prediction, along with interactive water depth and rainfall hyetograph charts. The platform provides a cross-scale visualization and decision support tool for water-related disasters.

**Keywords:** Flood simulation, landslides, machine learning, ArcGIS

## Introduction

Between July and August 2021, continuous heavy rainfall from southwest airflows caused severe flooding in the Yanshui River basin in Tainan, exposing deficiencies in the real-time information integration and visualization capabilities of existing disaster monitoring systems. Water environment issues involve complex, cross-agency, and multi-temporal disasters, including upstream landslide predictions, mid- and downstream flood simulation, and rainfall forecasting. Relevant data are scattered across the following six government agencies: the Ministry of the Interior, the Water Resources Agency, the Central Weather Administration, the Central Geological Survey, the Soil and

Water Conservation Agency, and the Taiwan Agricultural Research Institute. These datasets cover 23 dynamic and static factors across four major categories (geomorphological, locational, geological, and dynamic) but exist in disparate formats and lack a unified display platform, creating an urgent need for cross-domain integration to develop an intelligent decision support tool.

This paper focuses on the design and implementation of the frontend visualization system, which is based on the D-Flow Flexible Mesh (D-Flow FM) hydrodynamic model [1] simulation results and data from 88 IoT water level sensors, using the ArcGIS

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Maps SDK [2] to construct 3D WebScenes with interactive charts. The system comprises the following three pages: a Hub homepage, a Flood Simulation Dashboard, and a Landslide Risk Prediction interface. Section 2 describes the system architecture, Sections 3 and 4 introduce the flood simulation and landslide prediction subsystems, respectively, and Section 5 concludes with future development directions.

## System Architecture

The platform uses a three-tier architecture: a frontend presentation layer, a backend service layer, and a data layer (Fig. 1). The frontend includes three pages built with the ArcGIS Maps SDK [2] to create 3D WebScenes and interactive charts for water depth time series and rainfall hyetographs. The backend employs an asynchronous framework to deliver unified flood and landslide APIs, integrating data from NetCDF, Shapefile, CSV/Excel, and GeoJSON formats. It also hosts four machine learning models for landslide prediction.

The data layer includes D-Flow FM simulation outputs, observation comparison tables, QPESUMS rainfall grids, and IoT station locations. Data governance uses a three-tier bronze/silver/gold framework that progressively standardizes heterogeneous data from various ministries into Analysis Ready Data (ARD). This data is indexed via the SpatioTemporal Asset Catalog (STAC) standard, allowing the frontend to query cross-ministry spatio-temporal data through a unified interface, which improves integration efficiency and scalability. The backend provides OGC API interfaces and ArcGIS Server map image services to support dynamic loading of multi-source layers. The system is deployed as a single application; users authenticate via ArcGIS Portal OAuth and can access either the flood simulation or landslide prediction subsystem from the Hub homepage.

The system uses native JavaScript with an asynchronous module loading mechanism to load the ArcGIS Maps SDK directly, eliminating the need for additional build toolchains. This reduces deployment complexity in high-performance computing environments and allows direct access to rendering resources for improved display performance. The interface features a dark theme with semi-transparent panels designed for command center applications. For data integration, the frontend connects to backend services built on FastAPI using asynchronous requests, with error interception and state handling mechanisms to ensure that time-series data transmission does not block the user interface. After retrieving time-series JSON data, the frontend applies custom algorithms for pre-processing (such as nearest time-step search and sliding window slicing) before passing the processed data to chart components.

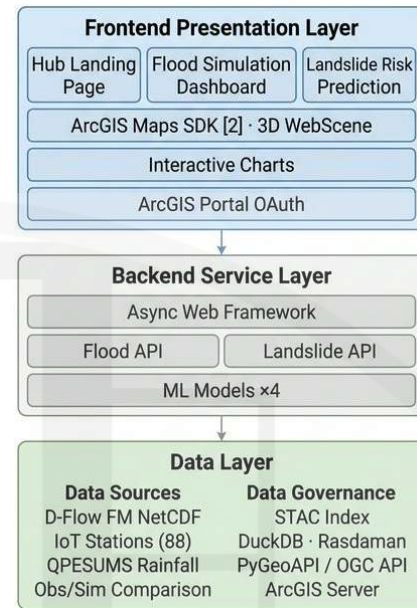


Fig. 1. System architecture

## Flood Simulation Subsystem

The Flood Simulation Subsystem interface is shown in Fig. 2. Based on D-Flow FM simulation results, the subsystem provides time-series animation and validation analysis for historical rainfall events. The study area's D-Flow FM model consists of 2,736,630 two-dimensional mesh faces and 3,314,131 nodes, with 73 time steps, and solves shallow water equations on an unstructured mesh. The system uses a conditional triggering mechanism: users control the display of flood depth layers, legends, time sliders, and water depth and rainfall panels through two dropdown menus for “analysis model” and “event date.” The initial view displays only the basemap and navigation bar; users enable additional analytical functions as needed. Currently supported analysis models include DFlow and WFlow, with three historical rainfall events: 20210604, 20210730, and 20210805. This conditional design effectively prevents visual overlap between analysis models and rainfall events, improving usability.

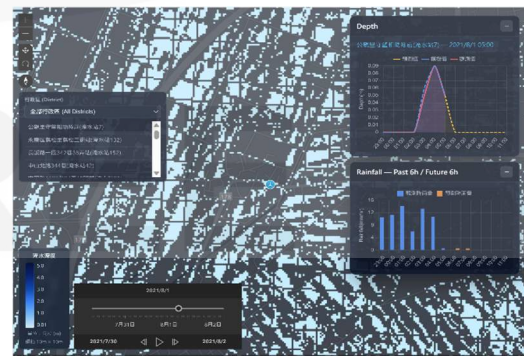


Fig. 2. Flood simulation dashboard interface.



The frontend loads a Portal WebScene basemap using ArcGIS SceneView, which includes time-aware flood depth layers published as Mosaic Datasets. The layer production workflow is as follows: offline scripts read D-Flow FM unstructured mesh NetCDF output files, interpolate water depth values to a regular grid for each time step, and convert them to GeoTIFF format (Web Mercator projection, ~10 m resolution). These GeoTIFFs are then compiled into a time-aware Mosaic Dataset and automatically published to ArcGIS Portal via ArcPy scripts, preserving the temporal dimension for frontend queries. The time slider operates in real-time mode with 73 time steps and supports looped playback. The frontend tracks time extent changes and updates the displayed flood rasters accordingly, showing the dynamic process from inundation to recession.

The map displays 88 IoT water level sensors using custom icons with a green indicator light by default. When a sensor is selected, its icon switches to an enlarged orange version for visual feedback. The system allows users to filter stations by administrative district; clicking a station pans the view and initiates relevant queries. The right panel features two chart groups: the water depth panel shows observed values (red solid line), simulated values (blue dashed line), and predicted values (yellow dashed line) within a  $\pm 6$ -hour sliding window. The frontend determines the display range based on the current time slider position. The three curves meet at the current time point, allowing users to quickly compare model outputs with observed data.

The rainfall panel displays QPESUMS observed and forecast rainfall as a bar hyetograph. The backend uses nearest-neighbor queries to identify grid points and extracts six hours of data before and after each event. The rainfall data source can connect to short-term precipitation prediction models integrated in the backend, expanding the system's use from historical event analysis to real-time early warning.

For validation, we compared flood sensor observations with model simulations across three historical rainfall events in 2021. Some stations were offline or had incomplete data during certain events, so the number of stations available for comparison ranged from 74 to 75 out of 88 deployed. Results show that, for the 20210604 event, 82.7% (62/75 stations); for the 20210730 event, 72% (54/75 stations); and for the 20210805 event, 93.2% (69/74 stations) had a maximum error of  $\leq 10$  cm, all within acceptable tolerance.

### Pu-Tun-Pu-Nas River Landslide Susceptibility Prediction Subsystem

The Landslide Susceptibility Prediction Subsystem interface is shown in Fig. 3. Using machine learn-

ing methods, the subsystem performs landslide probability prediction for slope units. The Pu-Tun-Pu-Nas River basin is geologically fragile, representing a case area prone to compound disasters. The model integrates spatial data from six ministries totaling 14 factors: geomorphological factors (elevation, slope, aspect, terrain roughness, profile curvature, Normalized Difference Vegetation Index (NDVI)), locational factors (distance to streams, roads, faults, building area percentage), geological factors (sensitive area indicator, folding degree, dip slope indicator), and dynamic factors (rainfall, Interferometric Synthetic Aperture Radar (InSAR) displacement gradient), using the Ministry of the Interior's 20 m digital terrain model and slope units as the unified spatial baseline for basin landslide prediction.

During data preprocessing, the system converts formats and aligns spatial units of raw data from various ministries, standardizes projection coordinate systems, and calculates representative values for each factor within slope unit boundaries using spatial statistics.

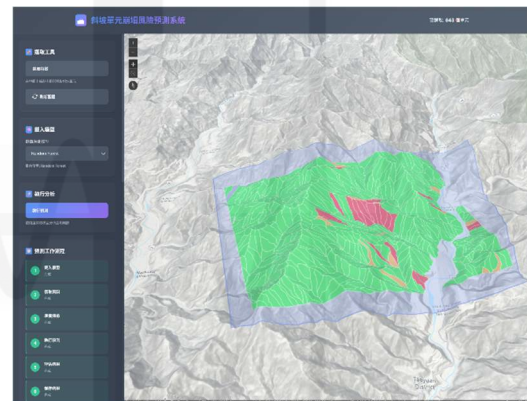


Fig. 3. Interface for predicting landslide susceptibility.

The frontend creates a 3D WebScene using the ArcGIS Maps SDK [2]. Users can click or draw polygons to select slope units, and the system returns identifier lists and updates selection counts. The side control panel allows slope unit selection, model choice (XGBoost [3], Random Forest, Gradient Boosting, or Logistic Regression), analysis execution, and result export. The backend receives identifier lists and model names, loads the corresponding serialized models, and outputs landslide probabilities and prediction labels for each slope unit as JSON. Validation results show the model achieved a producer accuracy of approximately 85%, effectively distinguishing between landslide and non-landslide areas. Single predictions complete in 2–4 seconds.

Prediction results are classified into three risk levels: high ( $>70\%$ , red), medium ( $30\text{--}70\%$ , yellow), and low ( $<30\%$ , green). The frontend displays slope units with color-coded grading. The side panel shows statistical cards with bar charts indicating distribution proportions and lists the ten highest-risk units; clicking

a row moves the camera and highlights the unit. Results can be exported as CSV or GeoJSON for use on GIS platforms and field patrols.

## Conclusions

The platform's frontend visualization system integrates flood simulation and landslide susceptibility prediction, providing interactive spatio-temporal disaster analysis through 3D WebScenes and conditional layer triggering mechanisms. The flood subsystem converts D-Flow FM simulation results into time-aware raster layers, combined with IoT sensor observations for step-by-step comparison and QPESUMS rainfall hyetographs, supporting dynamic retrospection and validation across multiple historical events. The landslide subsystem integrates 14 factors from six ministries with four machine learning models for slope unit landslide probability prediction and three-level risk classification. Both subsystems are deployed through a unified architecture, balancing operational simplicity with extensibility.

Future work will focus on four areas: (1) integrating additional rainfall events and hydrological models, such as WFlow, to expand spatio-temporal coverage and enable multi-scenario comparisons; (2) incorporating real-time rainfall forecasting and water level prediction to upgrade the system from historical analysis to real-time early warning; (3) strengthening standardized interfaces at the data governance layer by using STAC indexing and OGC API for automated data ingestion and querying across ministries; and (4) improving landslide model interpretability and developing mobile-friendly interfaces to support better on-site decision-making.

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# A Hybrid Simulation with Mixed Control Modes Using a General-purpose Software Framework for Quasi-static Structural Testing

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## Abstract

Hybrid simulation (HS) integrates structural testing and numerical simulation to effectively investigate the time-history responses of large or full-scale structures under dynamic loading. However, its implementation is complex, requiring expertise in structural analysis, experimental facility control, computer programming, and network communication. The National Center for Research on Earthquake Engineering, National Institutes of Applied Research (hereinafter referred to as “NCREE”) has developed several general-purpose software programs to simplify the planning and execution of HS. This article demonstrates the application of these software programs in conducting HS on a two-bay, three-story steel buckling-restrained frame subjected to earthquake excitation.

**Keywords:** earthquake engineering, large-scale structural testing, experimental methods, numerical simulation, hybrid simulation

## Test Requirement

To study the seismic response of a three-story, two-bay steel buckling-restrained brace (BRB) frame, a hybrid simulation (HS) was conducted on a full-scale beam-column-BRB subassembly. Fig. 1 shows the PISA3D model of the structure, with the experimental parts shown in color. Note that only the first-story central column (C2) and the BRB were selected as the physical substructure (PS) (shown in blue) in the HS. Other components in the experimental part (shown in red) were regarded as the fixture intended to provide a more realistic boundary condition for the PS.

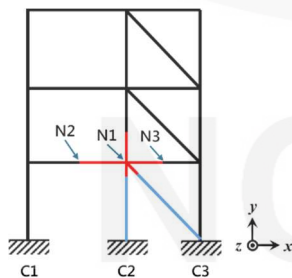


Fig. 1. The PISA3D model of the BRB frame

Fig. 2 shows the experimental setup. The control targets include: (1) horizontal displacement of node N1, (2) vertical displacement at nodes N2 and N3, and (3) vertical force at node N1. The vertical displacements of nodes N2 and N3 were the average displacements of columns C1 and C2 and columns C2 and C3, respectively. Due to the difficulty of accurately controlling the rotational degree-of-freedom (DOF) at node N1, this DOF was not explicitly controlled after the validity of this decision was verified by thorough numerical simulation. The shortening of the three first-story columns (obtained by direct measurement for C2, and by ABAQUS offline static simulation results for C1 and C3) was required to be enforced in the PISA3D simulation in the HS.



Fig. 2. Experimental setup

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## Software Architecture

The National Center for Research on Earthquake Engineering, National Institutes of Applied Research (hereinafter referred to as “NCREE”) has worked to develop and improve software and computer programs to support HS and other types of structural testing. These programs include:

- (1) An object-oriented C++ software framework “Software Framework for Quasi-static Structural Testing” (SFQSST);
- (2) An application communication protocol “Remote Experimental Control and Data Exchange” (Recdex) that facilitated data exchange across the Internet to support geographically distributed structural testing;
- (3) A general-purpose facility control program (Flex-Control.exe), which was built using SFQSST and supports Recdex communication;
- (4) The structural analysis program PISA3D, which was also extended to support Recdex.

To fulfill the test requirement described above, it was necessary to simultaneously run two instances of PISA3D (PisaHS and PisaRef) in the HS. Both PISA3D instances used experimentally measured specimen responses (C2 shear, moment, and axial force, and BRB axial force). However, PisaHS additionally applied a pull-down force technique to enforce displacement compatibility for columns C1, C2, and C3 in the numerical simulation. PisaRef was run alongside PisaHS because calculating the pull-down force in PisaHS required knowledge of the column shortening in the scenario where vertical displacement compatibility was not enforced. A detailed derivation of the pull-down force calculation is provided in Wang et al. (2023). Fig. 3 shows the software architecture and the quantities exchanged between programs.

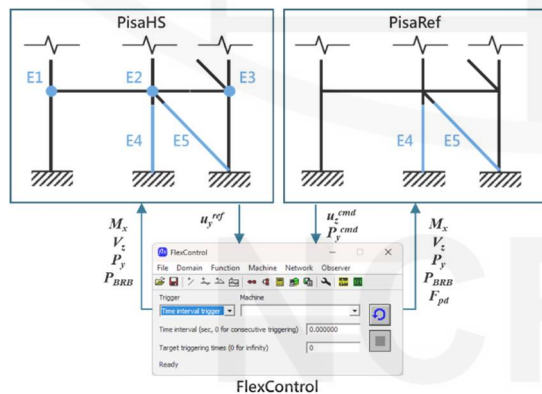


Fig. 3. Software architecture used for the HS

## PISA3D Models

As shown in Fig. 3, five and two experimental elements (shown in blue) were used in PisaHS and

PisaRef, respectively. E1, E2, and E3 were the one-node experimental elements used to introduce the pull-down force into the PISA3D step-by-step integration, while the two-node E4 and E5 were used to transmit the measured responses of column C2 and the BRB. To achieve maximum flexibility, the current design of the Recdex command and feedback data packets contains displacement and force data for all DOFs of all nodes of an experimental element, although in HS practice, it is unlikely that all these fields would be used. Those two instances of PISA3D were set as Recdex servers in the HS, meaning that in each execution step, they received a RECDEX\_EXECUTE packet containing the measured specimen response corresponding to step  $i$  from the remote client (FlexControl), and then responded with a RECDEX\_EXECUTE\_DONE packet containing the displacement and force command values corresponding to the next step,  $i + 1$ .

## Settings of the Facility Control Program

A SequentialComposite machine (named “TestSeq” as shown in Fig. 4) was defined to include 11 component machines, which performed specific tasks in an execution step to support the HS. In each execution step, the “Trigger” object asked the “TestSeq” object to perform its defined task, which involved making sequential calls to each of the 11 component machines to perform their defined tasks.

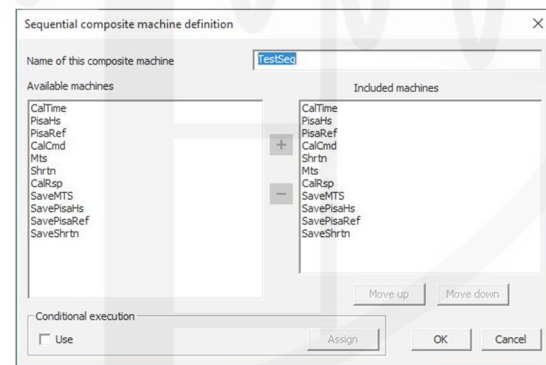


Fig. 4. Component machines for the HS

The SimpleCalculator machines “CalTime,” “CalCmd,” and “CalRsp” were used to calculate the required signals that represent the integration time, the vertical force command  $P_y^{cmd}$  for the specimen, and the pull-down force and the C2 axial restoring force, respectively.

The machines “PisaHS” and “PisaRef” were instances of the RecdexRHAClient machine type, which acted as clients in Recdex communication. In each execution step, the machine first sent a RECDEX\_EXECUTE packet containing the measured specimen response to the remote Recdex servers, then received a RECDEX\_EXECUTE\_DONE packet with the displacement and force commands from the servers. As shown in Fig. 5, FlexControl provided a



graphical user interface (GUI) that allowed the user to easily assign a specific control system signal to each required field in the RECDEX\_EXECUTE packet.

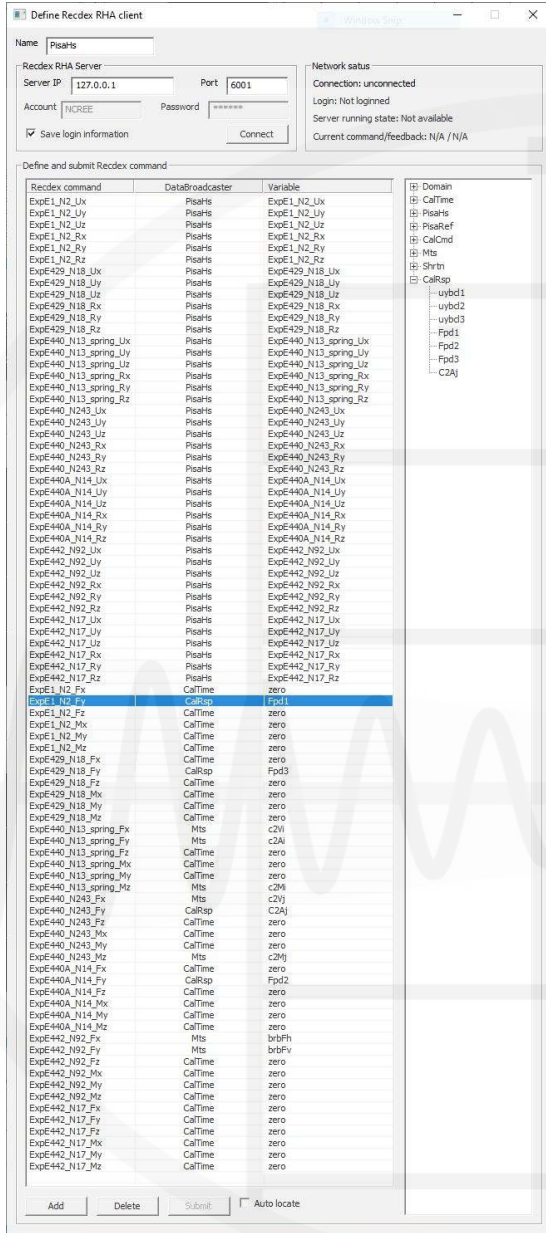


Fig. 5. Definition of each field in the RECDEX\_EXECUTE packet

As mentioned above, at each execution step, both PISA3D instances sent their nodal displacement values to the Recdex client, FlexControl. In addition to all displacement commands, PisaHS was also configured to send the axial restoring force of the BeamColumn element (E422), which represented the second-story central column. This restoring force was sent because the vertical force command for the specimen can be calculated by the following equation:

$$P_y^{cmd} = -F_{y, E422, N14} - 121.3 \quad (1)$$

where  $F_{y, E422, N14}$  is the restoring force at the y-DOF at node N14 of element E422. The second term on the right-hand side of Eq. (1), -121.3, was the constant

gravity load at the node N14. A SimpleCalculator machine “CalCmd” was used to perform the calculation of  $P_y^{cmd}$  before it could be used by the “MTS” machine (detailed below) to impose the vertical force on the specimen.

The “Shrtn” machine was a CommandGenerator, which implemented the strategy design pattern, enabling the user to select one of several algorithms for generating commands to be executed by other Machine objects in FlexControl. In this particular HS testing project, the “FileCommand” algorithm was chosen. At the start of the test, the “Shrtn” machine read the time histories of the vertical displacement of columns C1 and C3 from an input file, which had been obtained from an offline ABAQUS static analysis. During each execution step after the test began, it broadcasted these values, allowing the “MTS” machine to obtain the vertical displacements of columns C1 and C3 as two of its required input parameters.

The “MTS” machine is a software agent that communicates with the MTS actuator controller (FT100). After its creation, the agent interacts with the MTS control software “Station Manager” to retrieve details about the available control channels, including their quantity, names, and associated control modes. It then displays this information and allows the user to specify which control channels are controlled by FlexControl, set tolerance for control error for each channel during tests, and select the command source for each channel, as shown in Fig. 6.

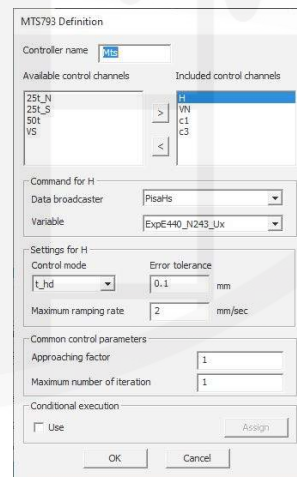


Fig. 6. Definition of the MTS agent

Any numeric-type signal can be selected as the command source for a control channel. This is how SFQSST transmits data values generated by external programs to the MTS control system.

In this HS, six hydraulic actuators (“H”, “VN”, “VS”, “25t\_N”, “25t\_S”, “50t”) were used to impose displacement or force on the specimen. Two virtual control channels (“c1” and “c3”) were also configured to receive the column-shortening magnitudes obtained from the offline ABAQUS static analysis. As shown in Fig. 6, FlexControl sent command values to control

four of these control channels: “H”, “VN”, “c1”, and “c3”. “H” was responsible for delivering the command value to impose the horizontal displacement of node N1 (see Fig. 1). “VN” sent the command value to impose the vertical force on node N1. “c1” and “c3” transmitted the shortening magnitudes of columns C1 and C3 to the FT100.

The other control channels were controlled by the “Station Manager” to achieve the desired control for this testing project. For example, “VS” was set to follow the displacement achieved by “VN”; “25t\_S” was set to follow the displacement achieved by “25t\_N”. “25t\_N” was set to achieve the displacement that was the average of “c1” displacement command (set by the FlexControl) and measured vertical displacement of node N2. Similarly, “50t” was set to achieve the displacement that was the average of the “c3” displacement command (set by FlexControl) and the measured vertical displacement of node N2.

Once the actuator motion control was finished (the “MTS” machine finished its work) in an execution step, the “TestSeq” asked the next machine “CalRsp” to work. It was of the SimpleCalculator type, and its responsibility was to calculate some of the specimen responses (the axial force of column C2) and the pull-down force for the purpose of enforcing displacement compatibility of node N1 in PisaHS. Fig. 7 shows the defining window for this machine. The user is allowed to create any number of virtual signals by supplying a name and a mathematical formula for each virtual signal. As can be seen in Fig. 7, the mathematical formula is simply a text string. The formula can contain names of real or other virtual signals in the system, including signal values obtained from the MTS control system, and any numeric values sent by external programs such as PisaHS and PisaRef. Commonly used mathematical operation symbols such as “+”, “-”, “\*”, “/”, “%”, “>”, “<”, “>=”, “<=”, “=”, “&&”, “||”, “^”, and functions such as “sin(x)”, “cos(x)”, “exp(x)”, “log(x)” are supported.

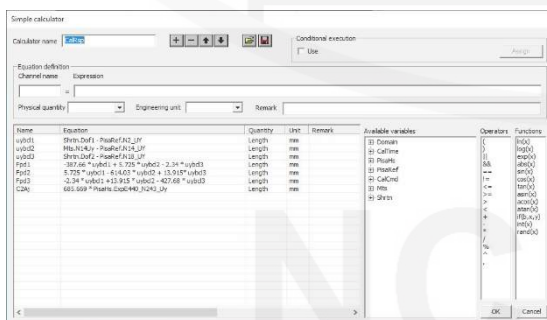


Fig. 7. CalRsp window responsible for calculating some of the required specimen responses in HS

SimpleCalculator machines are useful in that most of the required transformations in an experimental project can be implemented smoothly without the need to modify the source code. It should be noted that the MTS-supplied software “Station Manager” also has this feature of creating and defining

virtual signals.

Finally, the “TestSeq” also contained four DataFileSaver machines. They were used to save the experimental data to disk files during the tests.

All the specimen responses corresponding to the command values received by “PisaHS” in this execution step were not sent to the remote PISA3D analyses until the subsequent execution step. In the subsequent execution step, these measured specimen responses were sent via a RECDEX\_EXECUTE packet.

## Conclusions

This paper describes the procedure for preparing an HS using the NCREE-developed software. The test requirements were complex, involving actuator motion control with mixed control modes, integration of data generated by external applications (such as ABAQUS), transformation of commands and measurements according to the experimental setup, detailed network communication between programs, specific generation method of signal values related to the simulation technique (for example, the vertical force command for the subassembly), and enforcement of displacement compatibility for the DOF under force control. Despite these complexities, this HS could be set up using general-purpose testing software without hardcoding, demonstrating the capability of the software to support versatile structural testing.

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# Seismic Risk Assessment Model for Commercial Insurance

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## Abstract

In response to the rapid growth of Taiwan's high-tech precision industries and the island's location in a seismically active region, earthquake insurance and business interruption insurance have become essential tools for enterprises to transfer catastrophe risk and manage losses effectively. The National Center for Research on Earthquake Engineering has integrated current seismic observation data and probabilistic seismic source models, along with related research, to develop a localized Seismic Risk Assessment Model for Commercial Insurance. This model evaluates potential damage to various insured assets and estimates potential business interruption durations. In accordance with actual claims practices, indemnity terms may adopt co-insurance, first loss, or layered indemnity mechanisms. Combined with deductibles, coverage limits, and aggregate limits that may apply to multiple policies across different locations within a corporate group, the model calculates insured losses for real property, movable property, and business interruption. By incorporating probabilistic seismic source models, the system produces probabilistic risk assessment results, including average annual loss, loss standard deviation, and coefficient of variation. It can also generate the occurrence exceedance probability curve and aggregate exceedance probability curve for annual maximum losses. These outputs provide valuable reference data for non-life insurers in evaluating financial capacity, setting premium rates, reviewing risk diversification strategies, and developing risk retention or risk transfer plans; thereby, improving the efficiency of earthquake risk transfer and strengthening the economic resilience of Taiwan's industries.

**Keywords:** Earthquake Insurance, Business Interruption Insurance, Seismic Risk Assessment, TERA

## Introduction

In recent years, Taiwan's advanced manufacturing industries have experienced rapid growth. Due to Taiwan's location in an earthquake-prone region, earthquakes present significant risks to high-value production facilities and supply chains. Consequently, earthquake insurance and business interruption insurance have become critical tools for companies to manage seismic risk and implement effective loss control strategies.

In 2023, the National Center for Research on Earthquake Engineering, National Institutes of Applied Research (hereinafter referred to as "NCRE") completed the Taiwan Earthquake Risk Assessment (TERA) system. TERA integrates Taiwan's historical earthquake records and active fault data, and includes

seismic source models calibrated for Taiwan. It can be used with damage assessment models for various asset types to perform probabilistic seismic risk analyses.

Building upon this foundation and based on the Residential Earthquake Insurance Risk Evaluation Model (TREIF-ERA) developed for the Taiwan Residential Earthquake Insurance Fund, NCRE updated key databases, analytical modules, and parameter settings in 2024 to meet operational requirements. This led to the development of the Taiwan Earthquake Insurance Risk Assessment System (TERA-ins). The system produces comprehensive risk metrics, including average annual total loss, insured loss, and its standard deviation, occurrence exceedance probability (OEP) and aggregate exceedance probability (AEP) curves of insured loss, probable maximum loss (PML)

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for single events, and layer-specific performance indicators for risk transfer mechanisms such as average annual loss, standard deviation, attachment probability, and exhaustion probability. These outputs support the review of premium rates, policy conditions, risk layering structures, and coverage limits for residential earthquake insurance.

TERA-ins is a probabilistic earthquake insurance risk assessment platform based on scenario simulation. Its analytical framework consists of three core databases: (1) Exposure Database, (2) Stochastic Events, and (3) Event Loss Table. It also consists of five analytical modules: (1) Exposure Analysis, (2) Hazard Analysis, (3) Vulnerability Analysis, (4) Financial Loss Analysis, and (5) Probabilistic Risk Assessment.

To further expand the service scope and applications of earthquake insurance risk assessment, NCREE completed the development of the Seismic Risk Assessment Model for Commercial Insurance in 2025. Incorporating claims conditions into the model estimates insured losses arising from real property damage, movable property losses, and business interruption. The model is designed to support applications by non-life insurers, reinsurers, and reinsurance brokers in underwriting, pricing, and risk transfer strategy formulation, thereby enhancing the efficiency of seismic risk transfer and strengthening the resilience of Taiwan's industrial sector.

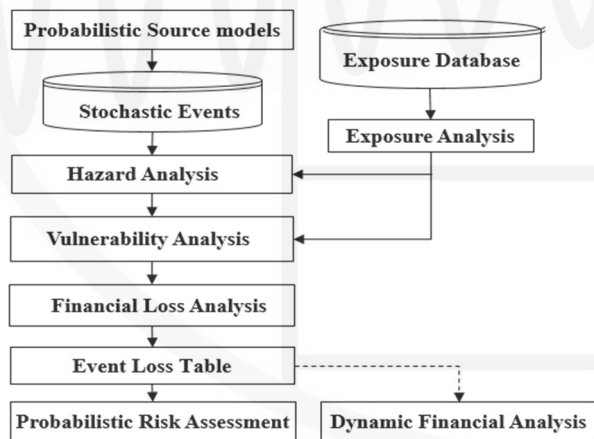


Fig. 1. TERA Analysis Workflow

## Commercial Earthquake Insurance Exposure Analysis

Commercial earthquake insurance and business interruption insurance are both add-ons to commercial fire insurance, which is the main policy. Under standard commercial fire insurance policy terms, insured property is classified as either real property or movable property. Real property refers to buildings and tenant improvements, whereas movable property includes business equipment, machinery, and inventory within

the building.

Business interruption insurance covers losses resulting from operational shutdown caused by property damage. To obtain coverage for business interruption losses caused by earthquakes, the insured must purchase both earthquake insurance and business interruption insurance, in addition to a commercial fire policy. This ensures protection for both real and movable property. Risk assessment for business interruption insurance is therefore based on property insurance exposure data.

Policy exposure analysis involves transforming “policy data” into “individual policy records,” which are subsequently aggregated into “village-level classified policy data” for seismic loss assessment. The policy data, maintained by users and imported into the TERA commercial earthquake insurance module, include not only policy identification information but also building characteristics, property insurance details, claims conditions, and business interruption coverage terms. The required data fields are summarized in Table 1.

After transformation, individual policy records are enhanced through the system’s geocoding feature to obtain latitude and longitude coordinates. Data such as construction year, building type, and number of stories are used to determine the building’s structural classification and seismic capacity, which are key parameters for earthquake damage assessment.

Coverage for real property includes both buildings and tenant improvements. Because earthquake damage affects these components differently, losses are evaluated separately and then calculated based on policy terms and insured amounts. The extent of tenant improvements depends on building usage, so the ratio of building structure to tenant improvements is estimated based on building usage. Similarly, coverage for movable property includes machinery, equipment, and inventory. Because policy data often lack detailed quantities for each category, TERA estimates their distribution based on building occupancy type and evaluates seismic damage separately to support insured loss estimation. The adopted building occupancy classification is presented in Table 2.

Considering uncertainties in seismic source characteristics, hazard potential, and damage assessment models, probabilistic earthquake risk assessment uses the village as the statistical unit. Policies within the same village that share similar structural type, seismic capacity, claims conditions, and business category are combined into village-level classifications. Using the village as the geographic unit for loss assessment captures regional risk differences and significantly increases computational efficiency.



**Table 1. Commercial Earthquake Insurance Data Fields**

| Data Category                               | Policy Fields                                                                                         |
|---------------------------------------------|-------------------------------------------------------------------------------------------------------|
| General Information                         | Company Code; Policy Number; Policy Liability Limit                                                   |
| Building Information                        | Structural Type (RC, SRC, Steel, etc.)                                                                |
|                                             | Number of Floors; Year of Construction                                                                |
|                                             | Building Use (Residential, Commercial, etc.)                                                          |
|                                             | Address; Postal Code                                                                                  |
| Property Insurance Information              | Building (Replacement Cost of Basic Structure)                                                        |
|                                             | Insured Amount – Building                                                                             |
|                                             | Insured Amount – Movable Property                                                                     |
|                                             | Additional Coverage Expenses (including temporary accommodation, etc.)                                |
|                                             | Basis of Claims Settlement (Scheduled Indemnity / Actual Loss Indemnity / Replacement Cost Indemnity) |
|                                             | Scheduled Indemnity Terms                                                                             |
|                                             | Actual Loss Indemnity Terms: Deductible and Deductible Percentage                                     |
| Business Interruption Insurance Information | Occupancy Type (Residential, Factory, Office, Cinema, etc.)                                           |
|                                             | Insured Amount – Business Interruption                                                                |
|                                             | Deductible or Waiting Period (days)                                                                   |
|                                             | Indemnity Period (days)                                                                               |

**Table 2. Building Occupancy Classification**

| Category | Building Occupancy Type                                                                                   |
|----------|-----------------------------------------------------------------------------------------------------------|
| A        | Residential                                                                                               |
| B        | Government Offices, Office Buildings, Cultural Museums, and Tutorial Centers                              |
| C        | Retail Shops with Attached Storage                                                                        |
| D        | Vacant Buildings, Electrical Rooms, and Cogeneration Facilities (excluding general and textile factories) |
| E        | Public Warehouses and Storage Facilities                                                                  |
| F        | Open Storage Tanks, Stockpiles, and Equipment                                                             |
| G        | Public Assembly Areas                                                                                     |
| H        | Cinemas                                                                                                   |
| I        | Buildings Under Construction or Renovation                                                                |
| J        | Markets and Shopping Centers                                                                              |
| K        | Parking Lots, Passenger and Freight Terminals, and Gas Stations                                           |
| L        | Broadcasting Stations                                                                                     |
| M        | Oil Facilities                                                                                            |
| N        | General Factories                                                                                         |
| O        | Warehouses, Storage Facilities, and Open-Air Equipment within General Factories                           |
| P        | Auxiliary Buildings within General Factories (excluding textile factories)                                |
| Q        | Textile Factories                                                                                         |
| R        | Warehouses, Storage Facilities, and Open-Air Equipment within Textile Factories                           |
| S        | Auxiliary Buildings within Textile Factories                                                              |
| T        | Spontaneous Combustion and Explosion Hazards                                                              |
| U        | Computers and Peripheral Equipment                                                                        |

## Financial Loss Analysis

The financial loss analysis is based on estimated damage to real property and movable property, as well as the projected duration of business interruption.

These results are combined with the policy's agreed indemnity terms to estimate insured losses separately for real property, movable property, and business interruption coverage. TERA supports three main indemnity structures:

1. **Step Policy Structure:** Losses are divided into several tiers according to the loss rate, with each tier corresponding to a fixed percentage of the insured amount payable as indemnity. In other words, compensation is triggered based on the damage state (or associated loss ratio) of the insured property, and a predefined proportion of the insured amount is paid for each tier. This structure inherently incorporates the concepts of attachment thresholds and deductibles.
2. **First Loss Basis:** In a first-loss structure, the insurer does not use the total value of the insured property to determine compensation. Instead, payment is based on the actual loss, up to the policy limit, after subtracting any agreed deductible. For example, assuming no deductible applies, if the insured amount is \$30 billion, but the actual loss from the earthquake is \$30 million, the insurer would pay \$30 million.
3. **Co-insurance Mechanism:** With co-insurance, the total value of the insured property is taken into account. Compensation is calculated based on the proportion of the insured amount to the property's total value, and the deductible is applied afterward. If the property is underinsured, the claim payment is reduced proportionally. For example, disregarding the deductible: Assuming an insured amount of \$30 billion and an actual loss of \$30 million, if the property's total value is \$100 billion, the insurer would pay \$10 million, calculated as follows:

$$\text{Compensation} = \text{Actual Loss} \times \frac{\text{Insured Amount}}{\text{Total Property Value}} \quad (1)$$

Under both first-loss and co-insurance structures, the agreed deductible applies. Deductibles are either a fixed monetary amount or a percentage of the loss. When estimating insured losses, TERA evaluates both types together. For smaller losses, the fixed deductible is used; for larger losses, the deductible is calculated as a percentage of the loss.

## Business Interruption Loss Model

Business interruption insurance covers losses incurred when a company is unable to operate normally due to an insured event. The time required for a business to recover from an operational interruption after such an event is influenced by numerous factors. For example, if business interruption is caused by building

damage, operations may resume once the building is repaired. Certain industries may rent alternative premises or implement other measures to resume operations without waiting for full restoration of the original building. In contrast, some industries may experience prolonged interruption due to damage to precision equipment that is difficult to repair. Therefore, the duration of business interruption is closely related to the specific industry.

This study assumes that the duration of business interruption matches the time needed for building repair. Because labor, equipment, construction conditions, sequencing, and other relevant factors cannot be precisely predicted, probability theory is used to estimate the expected repair time, as shown in the following formula:

$$T_{\text{repair}} = \sum_{i=2}^5 (p_i \cdot t_i) \quad (2)$$

The damage states  $i = 2, 3, 4, 5$  correspond to slight damage, moderate damage, severe damage, and complete damage, respectively. The probability of each building damage state can be obtained from the building damage assessment model. The required repair time for each damage state should account for variations across business categories. When the level of building damage renders repair economically unfeasible, the duration of business interruption is assumed to be equivalent to the time required for reconstruction. Based on the estimated business interruption duration, business interruption loss can be calculated as follows:

$$\text{Business Interruption Loss} = \text{business interruption duration} \times \frac{\text{Insured Amount}}{\text{indemnity period}} \quad (3)$$

where the indemnity period represents the maximum compensation period specified in the insurance policy. The ratio of the insured amount for business interruption to the indemnity period represents the daily compensation amount for business interruption.

### Group-Level Aggregate Limits and Sub-limits

Commercial earthquake insurance policies typically provide coverage for multiple insured items, including real property, movable property, and business interruption. The indemnity payable for each insured item (i.e., real property, movable property, and business interruption) is capped at its respective insured amount. In addition, the aggregate indemnity payable for all insured items combined shall not exceed the total policy liability limit specified in the policy. For example, consider a policy with a total insured amount

of NT\$19 billion and an overall liability limit of NT\$2 billion. The insured amounts for real property, movable property, and business interruption are NT\$3 billion, NT\$15 billion, and NT\$1 billion, respectively. Each item has its own insured amount serving as the maximum indemnity. However, when calculating the final claim payment, the combined indemnity for all three insured items shall not exceed the overall liability limit of NT\$2 billion.

Large corporate groups often own multiple factories or business locations, with different sites insured under separate policies. These policies may be subject to a shared aggregate liability limit at the group level. When a major earthquake occurs, causing damage to multiple locations within the group and triggering claims under multiple policies, the system will consolidate the losses across all affected sites. If the total losses claimed under all policies exceed the group's agreed aggregate liability limit, the final indemnity payable will be capped at that group-level limit.

### Conclusions

NCRE has developed a commercial earthquake insurance loss model, which has been integrated into the TERA-ins earthquake insurance risk assessment system. Commercial policy and claims data from recent major earthquakes will be used to calibrate model parameters, improving its reliability and robustness. TERA-ins is based exclusively on domestic data and locally developed models. Ongoing review and validation of results after significant earthquakes will help produce more accurate estimates of insurance losses, including probable losses and average annual losses.

In the future, we will seek collaboration with property insurance companies and reinsurance brokers to promote the application of TERA-ins in commercial earthquake insurance loss assessment. The objective is to assist the industry in estimating insurance losses, formulating reasonable premium rates, and reviewing risk diversification strategies. The system is expected to serve as a technical reference for the insurance industry when developing earthquake-related insurance products, thereby supporting the implementation of earthquake risk transfer and disaster risk management strategies across Taiwan's industries.

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# A Dynamic Network-Based Resilience Assessment Framework for Post-Earthquake Emergency Healthcare Systems

Chi-Hao Lin<sup>1</sup> Shih-Liang Chen<sup>2</sup>

林祺皓<sup>1</sup>、陳世良<sup>2</sup>

## Abstract

Earthquakes simultaneously cause surges in medical demand and disrupt healthcare system capacity, creating complex and time-dependent challenges for emergency response. Traditional evaluation methods, which rely on static measures such as hospital bed counts or nominal capacity, cannot capture the dynamic and interconnected nature of healthcare systems after disasters. This study introduces a dynamic resilience assessment framework for regional emergency healthcare systems following earthquakes. The framework defines healthcare resilience as a time-dependent property resulting from the interaction of varying medical demand, multiple resource constraints, spatial redistribution of patients, and phased recovery processes. By modeling hospitals as throughput-limited systems connected by patient flows, the framework captures cascading congestion and resource shortages that spread throughout the regional network. This approach allows for a trajectory-based evaluation of system performance and introduces a System Resilience Index to measure both the severity and duration of service disruptions. The framework offers a systematic basis for identifying critical vulnerabilities and supports decision-making for disaster preparedness and emergency medical resource allocation.

**Keywords:** earthquake, emergency healthcare systems, resilience, dynamic modeling

## Introduction

Earthquake disasters place multiple stresses on healthcare systems by increasing medical demand and reducing service capacity. After a major earthquake, hospitals must care for a large number of injured patients while dealing with disruptions caused by damaged buildings, broken equipment, interrupted utility services, and staff shortages. This combined pressure leads to complex and dynamic system behavior that cannot be adequately captured using traditional static indicators.

Traditional methods for evaluating healthcare performance often use facility-level metrics, such as bed capacity, physician staffing, or distance to care. However, these measures are inadequate after earthquakes, when healthcare systems function as interconnected regional networks. Disruptions at one facility can lead to patients being redirected to other hospitals, causing congestion throughout the network. Healthcare delivery is also limited by interdependent resources, where bottlenecks may restrict patient flow.

Furthermore, system capacity changes over time due to ongoing damage and gradual restoration, making resilience a dynamic property.

Despite these complexities, existing studies often focus on accessibility or total capacity without fully accounting for the interplay among varying demand, limited resources, spatial shifts, and recovery over time. To address this gap, this study presents a dynamic resilience assessment framework for post-earthquake healthcare systems. The framework defines resilience as a time-varying system characteristic resulting from severity-based demand, multiple resource limitations, spatial distribution, and changes in capacity over time. By integrating these factors, the proposed approach supports a trajectory-based evaluation of system performance and offers a foundation for analyzing healthcare resilience following seismic events.

## Conceptual Framework

This study defines regional emergency healthcare

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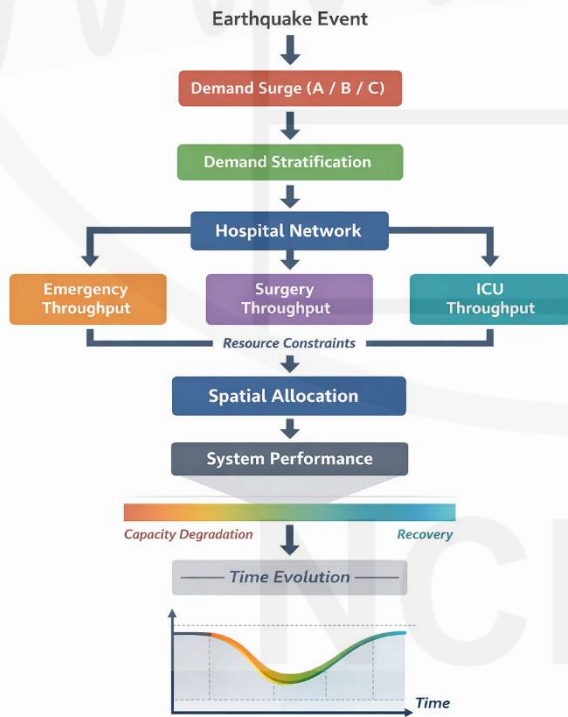
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resilience as a dynamic system property resulting from the interaction between changing medical demand and limited service capacity during earthquake-induced disruptions. Unlike traditional methods that rely on fixed indicators, the proposed framework focuses on system-level interactions, spatial connections, and temporal changes. At the regional level, the healthcare system is modeled as a network of hospitals connected by patient flows, where disruptions at one facility can affect the entire system.

The overall structure of the framework is illustrated in Fig. 1. After an earthquake, medical needs are assessed and categorized by severity before entering the regional hospital network. Within this system, treatment capacity is constrained by several interconnected resources, and patients are assigned to hospitals based on proximity and available capacity. The relationship among patient needs, resource limitations, and hospital assignments affects system performance over time.

A key feature of the framework is its explicit representation of temporal dynamics. Hospital capacity undergoes degradation immediately after the earthquake and gradually recovers, resulting in time-varying system performance that can be interpreted as a resilience trajectory. Overall, the framework integrates demand stratification, multi-resource constraints, spatial allocation, and capacity degradation and recovery, forming the basis for subsequent model formulation.



**Fig. 1.** Conceptual model of the dynamic healthcare resilience framework

## Demand Stratification and Care Pathways

Post-earthquake medical demand is inherently heterogeneous and must be differentiated according to severity and resource requirements. In this study, demand is categorized into three levels corresponding to distinct care pathways. Critically injured patients require immediate surgical intervention followed by intensive care, imposing simultaneous demands on emergency services, operating rooms, and ICU capacity. Moderately injured patients do not require surgery but still depend on intensive emergency treatment and may, under high-demand conditions, require access to intensive care resources. Mildly injured patients primarily rely on basic emergency treatment and short-term observation, imposing comparatively lower resource demands.

To enable quantitative analysis, total medical demand is divided into severity-specific components based on predefined proportions. Let total demand at a given location  $i$  and time  $t$  be denoted as  $D_{i,t}$ . The demand for each category is calculated as:

$$D_{i,t}^A = p_A D_{i,t}, D_{i,t}^B = p_B D_{i,t}, D_{i,t}^C = p_C D_{i,t}$$

where  $p_A$ ,  $p_B$ , and  $p_C$  represent the proportions of critical, moderate, and mild cases, respectively, with  $p_A + p_B + p_C = 1$ .

In subsequent modeling, moderate and mild cases are grouped into a non-surgical pathway, as both primarily rely on emergency and observation resources. However, their resource use differs: moderate cases place greater demand on emergency services and may be limited by ICU capacity, while mild cases mainly use lower-intensity emergency resources. This approach maintains model simplicity while capturing differences in system load across patient categories.

## Multi-Resource Throughput Constraints

Hospital capacity is defined as treatment throughput, limited by several interdependent resources rather than a fixed number. Emergency treatment is the main entry point and is restricted by triage, physician availability, and diagnostic services:

$$C_{ED} = \min(C_{\text{triage}}, C_{\text{physician}}, C_{\text{lab}})$$

To distinguish between demand and resource load, the model differentiates between patient counts and effective resource consumption. Different patient categories are modeled using distinct mechanisms reflecting their treatment characteristics.

For critically injured patients (Category A), treatment involves sequential movement through emergency, surgical, and intensive care stages. The effective capacity is therefore limited by:

$$C_A = \min(C_{ED}, C_{OR}, C_{ICU})$$

This formulation reflects the multi-stage dependency structure of critical care, where the most limited resource determines overall throughput.

For non-surgical patients (Categories B and C), treatment capacity depends on emergency and observation resources:

$$C_{Non} = C_{ED} + C_{obs}$$

However, moderate and mild patients differ in resource consumption intensity. To capture this difference, resource weighting is introduced to convert demand into equivalent emergency load,  $D_{i,t}^{B,*} = w_{ED,B} \cdot D_{i,t}^B$  and  $D_{i,t}^{C,*} = w_{ED,C} \cdot D_{i,t}^C$ , where  $w_{ED,B} > w_{ED,C}$ , indicating that moderate patients impose higher demand on emergency resources.

In addition, moderate cases may require intensive care resources during periods of high demand. To account for this, a system-level ICU capacity constraint is introduced. Let  $ICU_{j,t}$  represent ICU capacity at hospital  $j$  at time  $t$ . The total ICU capacity available is  $\sum_j ICU_{j,t}$ . This system-level aggregation reflects the sharing and allocation of ICU resources across hospitals within the regional healthcare network.

A fraction of this capacity is allocated to moderate cases,  $C_{ICU,t}^B = \theta_{ICU,B} \cdot \sum_j ICU_{j,t}$ , where  $\theta_{ICU,B}$  represents the proportion of ICU capacity potentially utilized by moderate patients.

Let  $S_{B,t}^{raw}$  denote the unconstrained service volume for moderate patients, determined by allocation and non-surgical capacity. The final served volume is defined as:

$$S_{B,t} = \begin{cases} S_{B,t}^{raw}, & S_{B,t}^{raw} \leq C_{ICU,t}^B \\ S_{B,t}^{raw} \cdot \frac{C_{ICU,t}^B}{S_{B,t}^{raw}}, & S_{B,t}^{raw} > C_{ICU,t}^B \end{cases}$$

This proportional scaling mechanism models competition for ICU resources under capacity limits, enabling gradual service reduction instead of sudden discontinuation.

Finally, resource competition across patient categories is explicitly modeled. Let  $S_A$  denote the number of critical cases served. The available capacity for non-critical patients is adjusted as:

$$C_{Non}^{adj} = C_{Non} - w_{ED,A} \cdot S_A$$

This formulation captures competition among categories for shared emergency resources and enables the model to represent system-wide congestion.

## Spatial Allocation under Capacity Constraints

Patient allocation is modeled as a dynamic process influenced by travel limitations and available hospital capacity. Let  $i$  denote demand locations and  $j$  denote hospitals, with travel time  $d_{ij}$ . Hospitals are considered eligible if they meet a maximum travel time requirement, forming an eligible set  $J_i$ . For critically injured patients, allocation prioritizes nearby hospitals to address urgent treatment needs. Demand is sequentially assigned to the closest eligible hospitals within capacity limits:

$$x_{ij,t} = \min(D_{i,t}^A, C_{A,j,t})$$

where  $x_{ij,t}$  denotes assigned demand and  $C_{A,j,t}$  represents available capacity for critical care at hospital  $j$ .

For non-critical patients, allocation is determined by a probabilistic distribution based on travel time, reflecting more flexible healthcare-seeking behavior. Allocation weights are defined using a distance-decay function:

$$w_{ij} = e^{-\beta d_{ij}}, p_{ij} = \frac{w_{ij}}{\sum_{k \in J_i} w_{ik}}$$

The allocated demand is then given by  $x_{ij,t} = \min(p_{ij} D_{i,t}^{Non}, C_{Non,j,t})$ , where  $D_{i,t}^{Non} = D_{i,t}^B + D_{i,t}^C$  denotes the total demand from moderate and mild cases. This approach models the redistribution of demand among multiple facilities, accounts for capacity limits, and reflects realistic variations in patient preferences.

Importantly, allocation is recalculated in each time window, allowing the model to incorporate feedback between changes in available resources and redistribution patterns. As hospitals reach full capacity, excess demand is redirected to other facilities, causing congestion to spread throughout the regional network. As capacity increases over time, redistribution patterns shift, linking allocation decisions to system recovery.

## Capacity Degradation and Recovery Dynamics

Healthcare system capacity is modeled as a function of time and multiple factors, including facility stability, staff availability, infrastructure reliability, and medical supplies. Instead of assuming that the system is either fully operational or not, the model treats capacity loss as a proportional decrease in service delivery. The effective capacity of a resource  $k$  at time  $t$  is defined as:

$$C_{k,t} = C_{k,0} \cdot F_{struct,t} \cdot F_{staff,t} \cdot F_{infra,t} \cdot F_{supply,t}$$

where  $C_{k,0}$  denotes baseline capacity, and each

functionality factor ranges between 0 and 1.  $F_{\text{struct},t}$  represents the level of structural integrity,  $F_{\text{staff},t}$  reflects available medical personnel,  $F_{\text{infra},t}$  captures the functionality of supporting infrastructure such as utilities and equipment, and  $F_{\text{supply},t}$  denotes the availability of medical supplies.

Recovery is modeled as a gradual restoration process across sequential time intervals. As functionality improves, system throughput increases, affecting resource allocation and overall performance. The interplay between recovery and redistribution creates feedback mechanisms that continuously alter system behavior.

This representation enables the identification of critical periods during which system performance is most vulnerable, often occurring when demand remains elevated while capacity has not yet fully recovered. Such temporal dynamics are essential for capturing resilience trajectories and evaluating recovery-sensitive system performance.

## Resilience Metrics and Trajectory-Based Evaluation

Resilience is assessed as a dynamic trajectory over time, rather than as a single static value, reflecting how system performance changes during disruption and recovery. System performance in each time interval  $t$  is defined as the fraction of demand that is met:

$$R_t = \frac{S_t}{D_t}$$

where  $S_t$  and  $D_t$  denote the served and total demand, respectively. To capture pathway-specific performance, category-level indicators are further defined as:

$$R_{A,t} = \frac{S_{A,t}}{D_{A,t}}, R_{\text{Non},t} = \frac{S_{\text{Non},t}}{D_{\text{Non},t}}$$

which differentiate between emergency and routine care pathways and expose variations in system vulnerability.

To quantify overall system resilience across the disruption–recovery period, the System Resilience Index (SRI) is defined as the average performance over all time intervals:  $\text{SRI} = \frac{1}{T} \sum_{t=1}^T R_t$ . In addition, cumulative resilience loss is expressed as:  $\text{Loss} = \sum_{t=1}^T (1 - R_t)$ , which captures both the depth and duration of service degradation. With this trajectory-based approach, resilience is understood as the dynamic change in system functionality over time. This method enables the identification of key vulnerability periods, especially when demand remains high but recovery is not yet complete, and reveals pathway-specific weaknesses that may be

missed by aggregate performance measures.

## Conclusion

This study presents a dynamic network-based framework for assessing post-earthquake emergency healthcare resilience from a system-level perspective. By integrating demand variability, multiple resource constraints, spatial allocation, and capacity degradation and recovery, the framework captures the dynamic interactions that govern system performance under seismic disruption. It enables a systematic analysis of how inter-hospital connectivity, resource bottlenecks, and recovery processes influence resilience, and facilitates the identification of critical vulnerability periods during high demand and partial recovery. Compared to conventional static methods, the proposed framework provides a more realistic representation of post-disaster healthcare system behavior and supports decision-making for resource allocation, patient redistribution, and recovery planning. Future work may integrate this framework with the Taiwan Earthquake Loss Estimation System (TELES) to support regional healthcare network resilience analysis and enhance its applicability in disaster preparedness and operational planning.

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# Preliminary Investigation of Vulnerable Components in Hydrogen Energy Storage and Transportation Facilities

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## Abstract

With the advancement of the energy transition, hydrogen energy has become a key technology for integrating renewable energy and supporting decarbonization. As a result, the safety of hydrogen infrastructure is receiving increased attention. This study examines vulnerabilities and risks in hydrogen energy storage and transportation facilities by analyzing cases of earthquake damage and hydrogen-related accidents. The findings show that pipelines, valves, flange joints, and anchoring systems are particularly prone to damage. Material degradation, design flaws, and operator error are common causes of accidents. Additionally, earthquakes and vibrations can worsen component failure and raise the risk of hydrogen leaks. Overall, the cases reviewed demonstrate that the reliability of individual components and effective system-level safety management are critical for hydrogen energy storage and transportation facilities.

**Keywords:** hydrogen storage and transportation infrastructure, renewable energy, seismic resilience, decarbonization technologies

## Introduction

As the global energy system accelerates its transition toward low-carbon and renewable energy, hydrogen has gradually emerged as a key energy carrier, widely applied in transportation, industry, and energy storage. Hydrogen holds significant potential for integrating intermittent renewable energy sources such as solar and wind power. Through Power-to-Gas (P2G) technology, surplus electricity can be converted into hydrogen for storage and later reconverted into electricity during peak demand periods, thereby enhancing grid stability and increasing renewable energy penetration. Green hydrogen produced from renewable energy can also serve as a clean fuel or feedstock for hard-to-electrify, high carbon-emitting industries, facilitating deep decarbonization of the energy system. As hydrogen infrastructure rapidly expands, its safety and resilience under seismic conditions have become critical factors affecting overall energy system reliability.

Hydrogen's properties pose significant engineering challenges for storage, transportation, and

material selection. Under dynamic loading, such as during earthquakes, embrittlement can reduce material strength and promote crack propagation, compromising the reliability of critical equipment. Additionally, hydrogen facilities are composed of interconnected mechanical systems that display complex coupled dynamic behavior during seismic events, which can result in resonance amplification, stress concentrations, and interface failures. These effects are difficult to capture using conventional static analysis. Current seismic design practices, largely adapted from petrochemical and general industrial standards, do not adequately address the unique material degradation mechanisms, dynamic responses, and accident risks specific to hydrogen systems. Therefore, there is an urgent need for more realistic analytical models and risk-informed design approaches, supported by experimental validation and component-level testing, to improve the safety and seismic resilience of hydrogen facilities.

To investigate the seismic performance of key components in hydrogen infrastructure, this study compiles damage data to identify vulnerabilities in

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hydrogen storage and transportation systems. These findings inform subsequent seismic analysis of critical components.

### Seismic Damage Cases of Hydrogen Storage and Transportation Facilities

Based on a preliminary review of available damage data, no large-scale cases of hydrogen storage or transportation facilities being damaged during earthquakes have been identified to date. However, comparable industrial disasters and recent hydrogen-related incidents still offer important risk warnings and useful insights. The 1995 Great Hanshin (Kobe) Earthquake in Japan caused severe damage to many chemical and gas processing plants, including tank buckling, pipeline rupture, and anchorage failure. The design of these key components closely resembles those in hydrogen systems, making these cases valuable references.

The 2011 Great East Japan Earthquake further highlighted the importance of seismic safety in energy facilities. Following the earthquake, the failure of emergency response systems prevented effective suppression of reactor core temperatures, leading to overheating of fuel rods and the generation of hydrogen subsequently mixed with air and triggered explosions, resulting in containment failure and the release of radioactive materials. In the aftermath, Japan has actively promoted the development of hydrogen infrastructure such as the Fukushima Hydrogen Energy Research Field (FH2R) and has incorporated seismic resilience as a key consideration in design.

In addition, hydrogen-related industrial accidents occurred during the earthquake. At a petrochemical plant in Fukushima Prefecture operated by the KUREHA Group (株式会社クレハ), pipelines connected to a hydrogen storage tank were damaged under a ground acceleration of approximately 310 Gal, causing an explosion (see Fig. 1). The incident damaged nearby buildings and injured personnel (Wada et al., 2013).

Another incident occurred at a factory in Tochigi Prefecture, where equipment inside a soundproof enclosure installed on a steel frame approximately 2.5 m above the fourth-floor roof was damaged. The system included a hydrogen compressor and motor supported by air-spring vibration isolators, while an adjacent buffer gas cylinder was anchored to the base of the enclosure (see Fig. 2). Due to local conditions, a ground acceleration of approximately 541.9 Gal was recorded, and amplification effects from the rooftop and elevated support structure were likely. This led to failure of the vibration isolators, excessive displacement of the compressor, and anchorage failure of the gas cylinder, causing rupture of the connected

flexible hoses and outlet pipelines. Hydrogen accumulated within the enclosure, and failure of the ventilation system due to power outage triggered an explosion (High Pressure Gas Safety Institute of Japan, 2013).



Fig. 1. Seismic damage to hydrogen pipelines in a petrochemical facility (Wada et al., 2013).

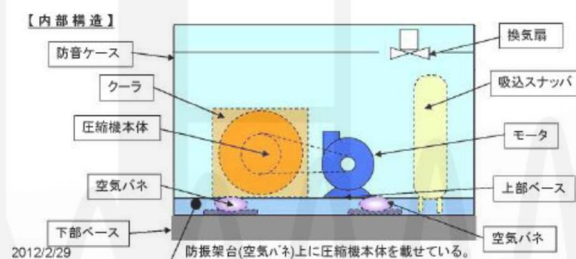


Fig. 2. Schematic diagram of a hydrogen compressor (High Pressure Gas Safety Institute of Japan, 2013).

### Non-Seismic Damage to Hydrogen Storage and Transportation Facilities

In addition to the earthquake-induced damage cases, most accidents involving hydrogen storage and transportation facilities are caused by component failures resulting from material defects or human factors, which subsequently lead to hydrogen leakage, explosions, or fires.

In 2012, a hydrogen refueling station for buses in Emeryville, California experienced an incident involving a pressure relief valve nozzle (Fig. 3). The nozzle, made of stainless steel unsuited to hydrogen environments, suffered from hydrogen embrittlement, leading to reduced strength and ductility. Machining tolerance issues further contributed to the failure. Within eight months of operation, a leak and small fire occurred. Emergency evacuation contained the situation, underscoring the importance of selecting appropriate materials for hydrogen service (Harris et al., 2012).

In 2015, a safety accident occurred at the Formosa Plastics Corporation plant in Mailiao, Taiwan, when a hydrogen leak caused an explosion (see Fig. 4).

The incident was caused by fractured stainless steel bolts, whose strength had declined due to prolonged corrosion from sea winds. Despite leak detection warnings, the accident was not prevented, resulting in localized damage to nearby areas (Cheng et al., 2015).

In 2019, a major explosion occurred at a P2G demonstration site in Gangneung, South Korea, where hydrogen was produced by water electrolysis (see Fig. 5). The accident was suspected to be caused by electrolyzer membrane failure, which allowed oxygen to enter the hydrogen storage tank. The mixture was ignited by a static spark, causing a rapid energy release under high pressure (approximately 1 MPa), resulting in two fatalities and damage to buildings within a 50 m radius (Kim & Kim, 2019).

In the same year, an explosion and fire occurred at a hydrogen refueling station in Sandvika, Norway (see Fig. 6). The incident was caused by improperly torqued bolts at the flange of a high-pressure hydrogen cylinder, resulting in a significant hydrogen leak and ignition (Løkke, 2019).

Additionally, in 2019, a hydrogen trailer leak occurred during transfer operations in Santa Clara, California (see Fig. 7). The incident was caused by improper operation, leading to an explosion and fire that damaged multiple trailers (Hydrogen Safety Panel, 2021).

From the above cases, it can be concluded that interface components such as pipeline flanges, valves, and various types of joints represent critical vulnerable points within the system. Improper design, material selection, or installation and operation can easily lead to hydrogen leakage, which may further trigger explosions or fires. These cases also demonstrate that hydrogen storage and transportation facilities are highly sensitive to detailed design and construction quality, where even minor defects may result in severe consequences.

Moreover, stresses and displacement effects caused by earthquakes or vibrations may increase these risks, underscoring the urgent need to improve the safety of hydrogen infrastructure. However, because there are few documented cases of seismic damage, experimental research on the seismic performance of hydrogen infrastructure is limited. Most current studies rely on data from nuclear or petrochemical facilities, demonstrating that further investigation is needed in this field.



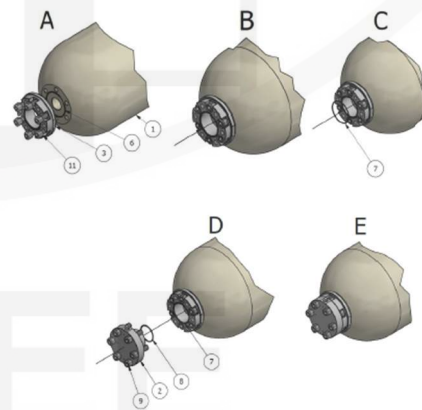
**Fig. 3.** Failed pressure relief valve component at the Emeryville bus hydrogen refueling station (Harris et al., 2012).



**Fig. 4.** Hydrogen explosion at a chemical plant (Cheng et al., 2015).



**Fig. 5.** Hydrogen storage tank accident (Kim & Kim, 2019).



**Fig. 6.** Assembly of flange connections for high-pressure cylinders in Europe (Løkke, 2019).





**Fig. 7.** Hydrogen trailer accident (Hydrogen Safety Panel, 2021).

## Conclusions

By analyzing earthquake damage cases and recent hydrogen-related accidents, it is evident that, although there are currently no direct reports of large-scale hydrogen storage or transportation facilities being damaged by earthquakes, experiences from seismic damage in chemical and gas facilities, as well as multiple hydrogen leakage and explosion incidents, have already revealed structural and equipment vulnerabilities in hydrogen systems. In particular, key interface components are often the main sources of failure and disaster initiation. Material degradation (e.g., hydrogen embrittlement), poor design, and human error also contribute significantly to accidents. Additionally, dynamic effects from earthquakes or vibrations can increase relative displacements and stress concentrations among components, raising the risk of leakage and failure. This suggests that the safety of hydrogen storage and transportation facilities depends not only on the performance of individual components but also on the overall behavior of the system.

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# Establishment of Environmental Monitoring Stations and Data Collection in Geothermal Potential Areas

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謝宏灝<sup>1</sup>、林世榮<sup>1</sup>、黃有志<sup>2</sup>、李宥葭<sup>3</sup>

## Abstract

This study focuses on high geothermal potential areas and aims to establish microseismic and geochemical gas monitoring stations through site selection and deployment, while developing baseline data to support environmental assessment before geothermal resource development. Because these areas are typically located in volcanic or tectonically active regions, the study integrates geological conditions, geothermal manifestations, gas emission characteristics, ambient noise levels, and construction feasibility to develop site selection criteria and guide monitoring station deployment.

By the end of 2025, an integrated microseismic and geochemical gas monitoring station was established in the Gengziping area of New Taipei City, along with a data acquisition and transmission system. Preliminary monitoring data have been collected and are being used for system testing and baseline analysis.

The primary objective of this study is to develop a standardized workflow and technical framework for deploying monitoring stations in areas with geothermal potential, and to characterize site environmental conditions and observational data features. The results provide a foundation for future station expansion, long-term data collection, assessment of environmental variability, and support for environmental monitoring and risk management during geothermal development.

**Keywords:** geothermal potential areas, environmental monitoring, microseismic monitoring, geochemical gas monitoring

## Introduction

In line with the national policy goal of achieving net-zero emissions by 2050, green energy is a central element of future energy transition strategies. Among renewable sources, geothermal energy offers stable, low-carbon, locally available power. Unlike intermittent sources such as wind and solar, geothermal energy is unaffected by weather and can deliver continuous baseload electricity.

Taiwan, situated along the Pacific Ring of Fire, has significant geothermal resources, especially in the Tatun volcanic area. Geothermal development alters subsurface heat flow, geological structures, and hydro-geological systems. There are ongoing public concerns

about environmental impacts, particularly whether development may cause abnormal changes. Establishing scientific monitoring systems and baseline data is essential to differentiate natural variability from development-related changes and provide objective references for future geothermal projects.

This study aims to deploy microseismic and geochemical gas monitoring stations and develop baseline data in areas with high geothermal potential, creating a technical framework for long-term environmental monitoring and variability assessment.

## Monitoring Station Deployment

The objective of this project is to establish a monitoring framework for microseismic and geochemical

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gas observations and to develop standardized procedures for site selection and station deployment. Existing monitoring stations and networks were reviewed to assess spatial coverage and data complementarity.

Site selection was based on geological conditions, topography, and environmental constraints. The main factors included:

- (1) surface gas emissions and geothermal manifestations;
- (2) ambient noise levels influencing microseismic observations;
- (3) construction feasibility and maintenance accessibility; and
- (4) data transmission and power supply conditions.

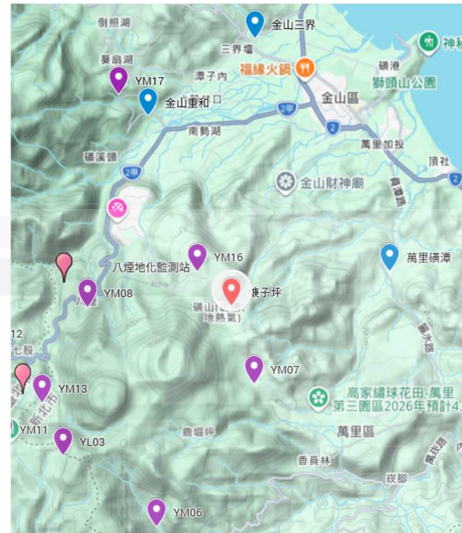
Additional considerations specific to volcanic environments such as gas corrosion, ground instability, and high temperature and humidity were incorporated to ensure long-term operational stability.

Based on these criteria, a monitoring site was selected in the Gengziping area, considering nearby fumarolic features (Fig. 1) and the distribution of existing monitoring stations (Fig. 2). The selection also accounted for the spatial relationship between geothermal features and established observation points to ensure that the new station can effectively capture representative signals of geothermal activity.

The monitoring station was completed by the end of 2025 and includes microseismic instruments and geochemical gas monitoring modules. The system integrates data acquisition and remote transmission, enabling real-time data access and preliminary quality control (see Figs. 3 and 4).



**Fig. 1.** Fumarolic features in the Gengziping area



**Fig. 2.** Distribution of monitoring stations near the Gengziping area



**Fig. 3.** Monitoring station completed by the end of 2025 in the Gengziping area



**Fig. 4.** Interior layout of the monitoring station in the Gengziping area



To optimize monitoring performance, station placement also considers spatial complementarity, avoiding redundancy and improving regional resolution for future event location and activity analysis. High-sensitivity seismometers are used to detect weak signals, while the gas monitoring system initially focuses on soil CO<sub>2</sub> flux measurements and will be further configured according to site-specific gas emission characteristics.

In addition, the configuration of monitoring instruments considers local environmental conditions, including high humidity, elevated temperatures, and potential corrosive gases, which are common in geothermal and volcanic areas. Protective measures and appropriate installation methods are adopted to ensure stable instrument performance under such conditions. The system design also emphasizes long-term reliability, including continuous data acquisition, stable power supply, and robust data transmission capabilities to support uninterrupted monitoring.

Furthermore, considerations for operation and maintenance are incorporated into the system design, such as accessibility for periodic inspection and equipment servicing, as well as flexibility for future upgrades or expansion of monitoring components. The overall system is therefore designed not only for initial deployment but also for sustainable long-term operation under field conditions.

## Baseline Data Development

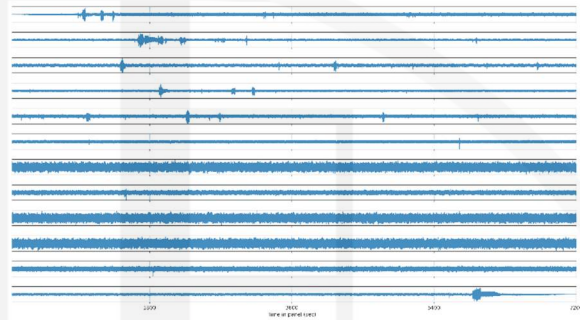
Following station installation, continuous data acquisition and preliminary processing were performed to establish baseline environmental data. Monitoring data consist of continuous microseismic waveforms and geochemical gas measurements, which are managed through an integrated data system to ensure integrity, consistency, and traceability.

Quality control procedures include waveform continuity checks, anomaly detection, and instrument status verification for microseismic data, ensuring stable operation and reliable background noise characterization. In addition, time synchronization and data completeness are also examined to minimize potential errors during data acquisition and transmission. Geochemical gas data are subjected to preliminary validation, including range checks and basic consistency evaluation, to identify abnormal values and ensure data reliability.

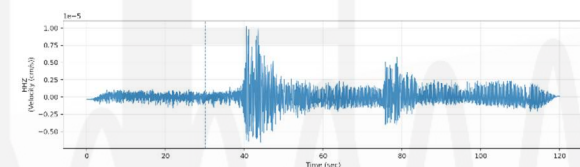
At this stage, the datasets are primarily used for system testing, evaluating monitoring conditions, and verifying the performance of instruments and data transmission systems. Preliminary processing also includes segmenting the data and comparing different time periods to assess the stability of background conditions. These procedures offer an initial understanding of site characteristics and establish a foundation for

long-term monitoring and advanced data analysis.

Approximately three months of continuous microseismic data were collected. Fig. 5 displays continuous waveform data, demonstrating stable recording conditions and consistent background vibrations. Fig. 6 presents recorded microseismic events, confirming that the station effectively captures seismic signals.



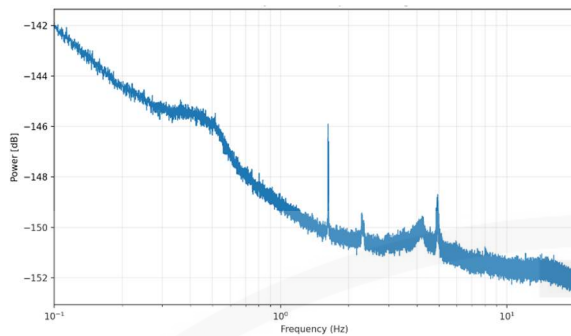
**Fig. 5.** Continuous waveform data from the monitoring station (single component, one-day record)



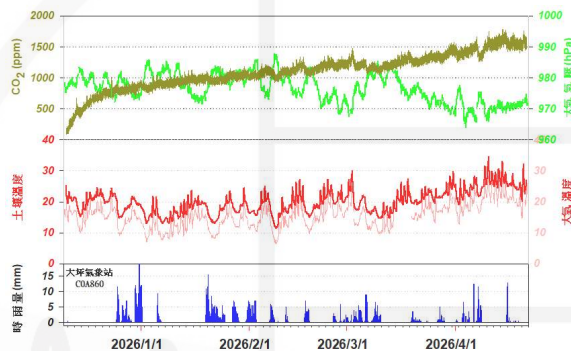
**Fig. 6.** Example of microseismic signals recorded at the monitoring station

Power Spectral Density (PSD) analysis was conducted to assess frequency-domain characteristics (Fig. 7). The spectrum decreases with increasing frequency, indicating greater energy at low frequencies and more stable behavior at higher frequencies. Local spectral variations appear in the 1–6 Hz range, with a distinct peak around 1–2 Hz, which may be attributed to environmental or anthropogenic factors. Overall, the PSD results suggest stable data quality and consistent spectral characteristics, supporting long-term monitoring and data comparison.

Soil CO<sub>2</sub> flux measurements are shown in Fig. 8. These data are compared with nearby atmospheric observations, and periodic components are filtered to assess correlations and evaluate long-term trends. The primary objective at this stage is to confirm system stability and data reliability, and to provide an initial characterization of site environmental conditions. Ongoing monitoring will support long-term baseline development and analysis of environmental variability.



**Fig. 7.** Power Spectral Density (PSD) of background noise recorded at the monitoring station



**Fig. 8.** Time series of soil CO<sub>2</sub> flux and atmospheric data

## Conclusions and Future Work

This study completed the site selection and deployment of an environmental monitoring station in an area with high geothermal potential, establishing a microseismic and geochemical gas monitoring system. A practical workflow for station deployment was developed, including site evaluation, instrumentation, and system integration, along with preliminary characterization of environmental conditions and data quality.

The current datasets represent an initial stage and are primarily used for system testing and establishing baselines. However, the monitoring framework has been validated, and the data acquisition and transmission systems have demonstrated stability.

Future work will focus on long-term data collection, baseline establishment, and environmental variability analysis. Additional monitoring stations are planned to expand the network and improve spatial resolution. The integration of microseismic and gas monitoring will further enhance the capability to assess environmental changes associated with geothermal development.

The methodology and experience gained in this study can serve as a reference for environmental monitoring in other geothermal areas and support the development of standardized monitoring practices.

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# Technology Survey and Application Potential Assessment of Low-Carbon and Recycled Materials for Building Structures

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## Abstract

In response to the global trend toward net-zero carbon emissions and Taiwan's 2050 net-zero target, decarbonizing construction materials and promoting resource circulation have become key priorities in the building and civil engineering sectors. This study systematically reviews low-carbon and recycled materials for structural applications in buildings and evaluates their current domestic technological maturity, industrial supply conditions, and feasibility for engineering implementation. Major categories of materials available in Taiwan are identified and classified, including low-carbon concrete, blended cementitious systems, recycled aggregate concrete, novel binders, and natural materials. These materials are further categorized based on their application potential and level of technological maturity. Subsequently, a localized database of low-carbon and recycled materials is established, integrating life-cycle information such as material sourcing, manufacturing, engineering application, and recycling/reuse, thereby providing a basis for material screening and decision-making. Furthermore, this study evaluates material availability, supply stability, and the practical challenges associated with engineering adoption. A set of application screening indicators is proposed, encompassing carbon reduction benefits, structural performance, construction quality, and compatibility with existing codes and standards. Finally, essential verification items are identified to ensure that these materials meet fundamental performance requirements and maintain rational applicability in structural engineering practice. The results indicate that while certain low-carbon materials have reached a level of maturity suitable for practical application, the overall development remains in a transitional stage. It is recommended that a systematic and institutionalized evaluation framework be established to facilitate the rational and effective implementation of low-carbon and recycled materials in the construction industry.

**Keywords:** Low-carbon materials, recycled materials, material databases, life cycle

## Research Motivation and Objectives

Driven by global climate change and net-zero carbon emission policies, the building and construction sector has shifted its carbon reduction strategies from focusing mainly on operational energy efficiency to managing and reducing embodied carbon in materials. Major construction materials are inherently energy- and carbon-intensive during production, significantly contributing to the life-cycle carbon emissions of buildings. Therefore, developing alternative materials and resource recycling methods to reduce carbon emissions has become a key focus in construction engineering.

Low-carbon and recycled materials represent one of the most promising technological pathways in this

regard. Low-carbon materials aim to reduce emissions through modifications to material composition or improvements in manufacturing processes, such as low-clinker cement and high-supplementary-cementitious-material concrete. Recycled materials, on the other hand, utilize recovery and reuse mechanisms to convert construction waste and industrial by-products into engineering materials, achieving dual benefits of reducing natural resource extraction and minimizing waste disposal.

However, the use of low-carbon and recycled materials in structural engineering involves more than simple material substitution. It requires consideration of several interrelated factors, including material properties, design methods, seismic performance, construction conditions, supply chain reliability, and

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compliance with current codes and standards. Without a comprehensive evaluation framework, these materials may introduce new risks, affect construction quality, and limit wider adoption. Additionally, Taiwan's location on the seismically active Pacific Ring of Fire and its subtropical climate characterized by high temperatures, humidity, and exposure to chloride-induced deterioration places strict demands on the durability and long-term performance of construction materials. Therefore, while promoting low-carbon materials, it is essential to ensure their fundamental performance and long-term reliability are maintained.

Based on the above considerations, the objectives of this study are as described in the following sections:

## Review of Domestic Material Technologies and Current Status

At present, low-carbon and recycled materials in Taiwan encompass a wide range of types, exhibiting varying levels of technological maturity and application scope. A detailed description is provided as follows:

### (1) Low-Carbon Concrete

Low-carbon concrete reduces emissions by decreasing clinker content or altering binder composition (e.g., adding limestone powder). It is the most advanced low-carbon material in Taiwan, matching the performance of conventional concrete and already used in general structural components. It can be produced with current supply systems, enabling high feasibility and flexible supply.

### (2) Ultra-High-Performance Concrete (UHPC)

UHPC has very high strength and a dense microstructure, achieved through low water–binder ratios and fiber reinforcement. Although UHPC has a higher unit carbon footprint, its capacity to reduce member size and material usage offers life-cycle carbon benefits. In Taiwan, UHPC is primarily used in research and pilot projects, including bridges and precast components.

### (3) Recycled Aggregate Concrete

Recycled aggregate concrete uses recycled materials, including demolition waste and industrial by-products, instead of natural aggregates. This approach promotes resource conservation and reduces waste. In Taiwan, research demonstrates that recycled aggregate concrete can meet essential performance standards when properly managed, but it is mainly applied to non-structural projects. Due to inconsistent material quality, its use remains cautious, and improved quality control is needed.

### (4) Geopolymer Concrete

Geopolymer concrete uses aluminosilicate

materials activated by alkaline solutions, reducing or eliminating the need for cement. It offers high design flexibility and strong low-carbon potential. In Taiwan, development remains at the research and experimental stage, with emphasis on mix design and material performance.

### (5) Natural Materials

Natural materials such as timber (cross-laminated timber [CLT] and glued-laminated timber [GLT]) and bamboo sequester carbon and enable low-carbon construction. Engineered wood is widely applied internationally, offering benefits for lightweight structures and prefabrication. Bamboo, with its high strength-to-weight ratio, also shows promise. In Taiwan, these materials are still in early development.

## Material Availability and Feasibility of Engineering Implementation

In addition to technological maturity, the application of construction materials is strongly influenced by factors such as material availability, supply stability, and the feasibility of engineering implementation. Different material types exhibit significant variations in supply sources, constructability, and associated engineering risks. A systematic evaluation of these aspects is therefore essential. The key observations are summarized as follows:

(1) Low-Carbon Concrete: Low-carbon concrete is currently one of the most feasible materials for engineering implementation. It relies primarily on existing cement production and ready-mixed concrete supply systems, requiring only adjustments in mix design and binder composition to achieve carbon reduction. Because its construction methods are consistent with those of conventional concrete, the barriers to implementation are low, with minimal impact on construction processes and equipment. This makes it suitable for widespread application in general building structures. In terms of supply stability, the key factor lies in the availability of alternative cementitious materials, which is currently considered stable overall. In summary, this material falls into the category of high availability with low implementation difficulty.

(2) UHPC: UHPC production requires high-quality cementitious materials, fine aggregates, and specialized additives such as steel fibers. These materials are less readily available and have stricter quality requirements than those used in conventional concrete. While Taiwan can supply these materials, they are generally sourced for specific projects rather than through standardized market supply. In terms of engineering implementation, UHPC also demands precise mix control and rigorous construction management,

with higher standards for construction quality. It is primarily used in precast components or specialized structural applications, resulting in a high implementation threshold. Overall, UHPC has moderate material availability and high implementation difficulty, making it most suitable for high-value-added engineering projects.

- (3) **Recycled Aggregate Concrete:** Recycled aggregate concrete primarily utilizes construction and demolition waste as well as industrial by-products. Its availability is regionally dependent, influenced by the presence of recycling facilities and the volume of waste generated in each area. As a result, supply stability and material quality may vary across different regions. In terms of engineering implementation, variability in material sources and quality must be carefully considered, often requiring additional quality control measures and mix design adjustments. Its application is prioritized for non-structural or secondary structural components to mitigate engineering risks. Overall, this material type can be classified as having moderate availability with moderate implementation difficulty, with its successful promotion relying on the establishment of stable supply systems and robust material grading and quality control mechanisms.
- (4) **Geopolymer Concrete:** Geopolymer concrete uses materials such as fly ash, ground granulated blast-furnace slag, and alkaline activators. Although aluminosilicate sources are relatively available in Taiwan, the cost and supply of alkaline activators may restrict wider adoption. The binding mechanism differs from conventional cement, creating engineering challenges, particularly due to the lack of comprehensive design codes and construction standards. As a result, practical use often relies on project-specific solutions and experimental validation, contributing to high implementation difficulty. Overall, geopolymer concrete is moderately available but difficult to implement, and remains in the development and validation stage.
- (5) **Natural Materials:** The availability of natural materials is influenced by forestry resources and supply chains. In Taiwan, structural timber depends on imports, making supply stability susceptible to fluctuations in international markets. Although bamboo offers advantages as a locally available resource, its application still requires further development in processing techniques and standardization. In terms of engineering implementation, natural materials necessitate specialized design approaches and construction techniques and are subject to regulatory constraints. The level of implementation difficulty is therefore relatively high, particularly with respect to structural design provisions and fire safety requirements. Further development of a

comprehensive technical framework is required to support their widespread application. Overall, natural materials can be classified as having moderate availability with moderate-to-high implementation difficulty.

## Life-Cycle Perspective of Materials

From a broad perspective, the use of low-carbon and recycled materials in construction should be assessed from a life-cycle standpoint to effectively achieve carbon reduction goals. The material life cycle is a closed-loop system that includes raw material sourcing, production, application in construction, service phase, demolition, recycling, and reuse. The following is a brief description of each stage:

At the raw material sourcing stage, materials come from either natural resources or recycled sources, such as construction and demolition waste or industrial by-products. Using more recycled materials reduces the need for virgin resource extraction and lowers waste disposal requirements.

During material production, carbon emissions can be reduced by optimizing processes and substituting materials, such as reducing cement content or using recycled aggregates.

- (1) In the engineering application stage, materials are utilized in structural or non-structural components according to their performance characteristics and technological maturity. Design approaches and construction methods employed at this stage are critical in determining the overall effectiveness of carbon reduction.
- (2) In the service stage, materials must meet structural requirements, ensure durability, and allow for efficient maintenance, all of which affect their long-term performance and sustainability.
- (3) Finally, at the end-of-life stage, materials can be recovered and reintroduced into the material cycle through recycling and reusing mechanisms, thereby completing the circular system.

In summary, the application of low-carbon and recycled materials should be comprehensively assessed from a life-cycle perspective. By integrating material substitution and resource circulation strategies, it is possible to achieve both environmental sustainability and practical engineering applicability.

## Local Database of Low-Carbon and Recycled Materials

Based on the review and classification of materials, this study establishes a localized database of low-carbon and recycled materials. The database framework consists of four key elements: (1) material

category, (2) representative materials, (3) application potential, and (4) technological maturity, summarized in Table 1. This localized material database is intended to serve as a reference for future development directions and to support material selection and decision-making in design and construction practices.

**Table 1.** Local Database of Low-Carbon and Recycled Materials Used in Construction.

| Material Category                      | Representative Materials                                              | Application Potential | Maturity Level |
|----------------------------------------|-----------------------------------------------------------------------|-----------------------|----------------|
| Low-Carbon Concrete                    | Slag, fly ash, limestone powder, supplementary cementitious materials | Primary structural    | High           |
| Ultra-High Performance Concrete (UHPC) | Mortar matrix, advanced cementitious materials, steel fibers          | Primary structural    | High           |
| Recycled Aggregate Concrete            | Recycled concrete, recycled aggregates, industrial by-products        | Secondary structural  | Medium         |
| Novel/Geopolymer Concrete              | Fly ash, ground granulated blast-furnace slag, alkaline activators    | Secondary structural  | Medium-Low     |
| Natural Materials                      | CLT/GLT timber, bamboo                                                | Secondary structural  | Medium-Low     |

## Material Application Screening Criteria

The preliminary material application screening criteria developed in this study focus on engineering feasibility and consider environmental benefits and material characteristics. The criteria include five main aspects:

- (1) Carbon Reduction Benefits: Assessment of the material's carbon reduction performance compared to conventional materials, including the accuracy and reliability of its carbon emission data.
- (2) Material Properties: Evaluation of mechanical properties such as strength, elastic modulus, and ductility ensures compliance with structural design standards.
- (3) Durability Performance: Consideration of the material's long-term performance stability under typical service environments, including properties such as permeability resistance and volumetric stability.
- (4) Construction Quality: Evaluation of constructability and consistency in construction quality, aiming to prevent adverse impacts on the

project schedule and overall construction quality.

- (5) Standards and Code Compliance: Assessment of the material's adherence to current standards and design codes, or confirmation that suitable verification and certification processes are established.

## Required Material Verification Items

To ensure materials meet the requirements for design and engineering applications, the verification framework should provide a practical and reliable evaluation mechanism that satisfies essential performance criteria. The verification should include the following items:

- (1) Mechanical Performance Verification: Evaluation of fundamental mechanical properties, including compressive strength, tensile strength, and deformation characteristics, to ensure that the material satisfies structural design requirements.
- (2) Material Property Verification: Assessment of characteristics such as volumetric stability, durability, and workability, which are key indicators of suitability for engineering applications.
- (3) Environmental Impact Verification: Assessment of environmental benefits through carbon footprint analysis or carbon emission estimation to quantify the material's contribution to carbon reduction.

## Conclusions

This study presents a preliminary review of domestic technologies, material classifications, and assessments of application potential for low-carbon and recycled materials. Overall, Taiwan has established a diverse technological foundation for these materials, with many technologies currently undergoing development and implementation.

From the perspectives of material supply and engineering adoption, notable differences exist among various material types regarding availability, constructability, and compatibility with existing codes and standards. These differences highlight the importance of considering factors such as supply chain stability and quality control mechanisms. Furthermore, the material database and application screening criteria developed in this study serve as preliminary tools for organization and evaluation. Future work should incorporate additional experimental data to refine and enhance their applicability. In addition, there is continuous potential for improvement and adjustment in the verification and evaluation frameworks associated with these materials, to better support their practical implementation in engineering applications.



# Uniaxial Tensile Response on AASHTO T 397

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吳振維<sup>1</sup>、歐昱辰<sup>2</sup>、陳俊仲<sup>3</sup>

## Abstract

Ultra-high-performance concrete (UHPC) achieves a dense microstructure through optimized particle packing, a low water-to-binder ratio, and high-performance admixtures. When combined with high-strength steel fibers, UHPC demonstrates tensile strengths of 6–12 MPa, significant strain-hardening behavior, and high ductility, greatly surpassing the performance of conventional concrete. Traditionally, UHPC tensile properties have been assessed using indirect tensile tests, which tend to overestimate tensile strength. In contrast, direct tensile tests more accurately capture elastic, strain-hardening, and strain-softening responses. However, due to the complexity of materials and testing setups, standardized methods were lacking for many years. Recently, the Federal Highway Administration (FHWA) promoted related research, resulting in the development of AASHTO UHPC 2024 specifications and the AASHTO T 397 standard test method. Three performance indicators are defined: effective cracking strength ( $f_{t,cr}$ ), localization cracking strength ( $f_{t,loc}$ ), and localization cracking strain ( $\epsilon_{t,loc}$ ). This study follows AASHTO T 397 to conduct uniaxial tensile tests on UHPC produced by the Ruentex Group. The effects of fiber dosage, fiber length, and curing conditions are investigated to provide a foundation for localized UHPC design.

**Keywords:** UHPC, tensile, AASHTO UHPC 2024, AASHTO T 397

## 1. Introduction

Ultra-high-performance concrete (UHPC) demonstrates high compressive strength (over 120 MPa) and excellent tensile performance (6–12 MPa), exhibiting strain-hardening behavior and high ductility that significantly exceed those of conventional concrete. Therefore, accurate characterization of its tensile behavior is essential for structural design. Indirect tensile methods, such as four-point bending and splitting tests, are commonly used but tend to overestimate strength due to complex stress distributions. Direct tensile testing offers a more realistic assessment of material behavior; however, it has been rarely implemented because of testing challenges and a lack of standardized procedures. With substantial support from the FHWA [1][2], direct tensile testing of UHPC has been standardized in AASHTO T 397-24 [3], facilitating comprehensive evaluation of tensile behavior. AASHTO UHPC 2024 [4] specifies that UHPC tensile behavior must meet three minimum performance requirements: (1) Effective cracking strength ( $f_{t,cr}$ )  $\geq 5.17$  MPa (0.75

ksi); (2) Localization cracking strength ( $f_{t,loc}$ )  $\geq$  effective cracking strength ( $f_{t,cr}$ ); (3) Localization cracking strain ( $\epsilon_{t,loc}$ )  $\geq 2.5 \times 10^{-3}$ . This study utilizes the standard method to examine the effects of fiber length, fiber dosage, and curing conditions on UHPC tensile performance for localized design applications.

## 2. Test method

AASHTO T 397-24 [3] uniaxial tensile test employs a prismatic specimen with a constant cross-section. A uniaxial tensile load is applied to assess the mechanical response and strain behavior, enabling determination of tensile properties. For fiber lengths not exceeding 20 mm, a square cross-section of 50 mm  $\times$  50 mm is recommended, with dimensions at least 2.5 times the fiber length to ensure uniform fiber distribution and minimize stress concentration. In this study, specimens with a length of 430 mm and a 50 mm  $\times$  50 mm cross-section were used.

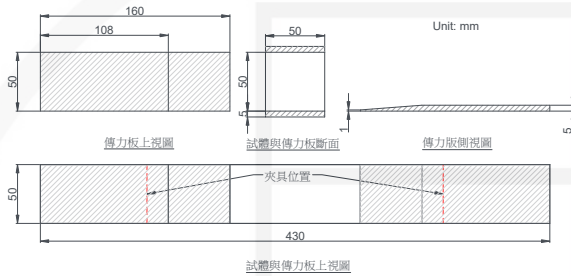
To prevent premature failure caused by stress concentration at the gripping ends, load-transfer plates

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were installed. Aluminum alloy plates were selected for their sufficient stiffness, which allows for even distribution of local stresses and enhances stress uniformity. This configuration significantly increases the test success rate and ensures that cracking occurs within the gauge length rather than near the grips. The dimensions, bonding location of the load-transfer aluminum plates, and the fixture positions are presented in Fig. 1. The plates were bonded using a high-strength, high-stiffness epoxy (Sikadur-31+). During installation, the plates were firmly pressed against the specimen surface until fully cured, ensuring test reliability.



**Fig. 1.** The dimensions of the load-transfer aluminum plates and their bonding locations on the specimen.

### 3. Test Results

The experimental program included three variables: fiber length, fiber volume fraction, and curing method, resulting in eight mix combinations. Fiber lengths of 13 mm and 20 mm were used. For 13 mm fibers, volume fractions of 2.0% and 2.5% were adopted; for 20 mm fibers, 1.5% and 2.0% were used. Additionally, each combination was tested under both water curing and steam curing conditions.

Table 1 presents the three tensile performance parameters for the eight mix proportions. Based on the experimental results, stress-strain curves, and subsequent analysis, the effects of various test parameters on the tensile behavior of UHPC are summarized as follows.

First, regarding the effect of fiber length: At a constant fiber volume fraction (2.0%), increasing the fiber length from 13 mm to 20 mm resulted in only a slight increase in strength (about 1.1 times) ( $f_{t,cr}$ ,  $f_{t,loc}$ ). However, ductility improved significantly, with localization cracking strain ( $\epsilon_{t,loc}$ ) increasing by 2 to 4 times. This improvement is attributed to the enhanced anchorage and pullout resistance of longer fibers, which increases energy dissipation.

Second, fiber volume fraction had the most significant impact on tensile strength. Increasing the dosage from 1.5% to 2.0% enhanced both effective cracking strength ( $f_{t,cr}$ ) and localization cracking strength by more than 1.2 times. Further increasing the dosage from 2.0% to 2.5% resulted in a strength increase of more than 1.5 times. This demonstrates that the strength of the fiber-bridging effect is highly

dependent on fiber content. Higher fiber content increases the likelihood of cracks being bridged, thereby effectively suppressing crack initiation and propagation. However, the improvement in localization cracking strain ( $\epsilon_{t,loc}$ ) was less significant. Although higher fiber content increases crack-bridging probability, factors such as fiber distribution and matrix density also affect ductility.

Curing conditions had a minor effect on strength and ductility. However, water curing exhibited greater variability, whereas steam curing resulted in more consistent stress-strain curves and performance parameters. This suggests that steam curing enhances test repeatability and reliability, which is advantageous for engineering applications.

**Table 1.** Experimental results of tensile performance parameters of UHPC

| Fiber length (mm)                | 13    |       |       |       | 20    |       |       |       |
|----------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|
| Fiber volume fraction (%)        | 2.0   | 2.5   | 1.5   | 2.0   | 2.0   | 2.5   | 1.5   | 2.0   |
| Curing condition                 | water | steam | water | steam | water | steam | water | steam |
| $f_{t,cr}$ (MPa)                 | 5.46  | 6.88  | 8.52  | 8.51  | 3.95  | 5.21  | 5.79  | 7.17  |
| $f_{t,loc}$ (MPa)                | 5.93  | 7.63  | 8.94  | 9.63  | 5.67  | 5.23  | 6.42  | 7.97  |
| $\epsilon_{t,loc}$ ( $10^{-3}$ ) | 1.97  | 1.30  | 2.85  | 2.77  | 3.24  | 2.77  | 2.58  | 4.15  |

### 4. Conclusions and Outlook

This study systematically evaluates the tensile performance of locally produced UHPC (Ruentex Group) using the AASHTO T 397-24 uniaxial tensile test method. The effects of fiber length, fiber dosage, and curing conditions are examined, and the results are compared with AASHTO UHPC 2024 requirements to assess compliance and engineering applicability.

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# Integration and Initial Study of Long-term Structural Vibration Monitoring Data

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## Abstract

This study focuses on long-term acceleration monitoring data of bridge deck vibrations. The data comprise continuous vibration signal time histories sampled at 200 Hz over six months. This paper addresses the data compilation and inventory process. To ensure the integrity of the monitoring data, a systematic check was performed to identify data defects, for which corresponding hypotheses and solutions are proposed. Challenges and trade-offs encountered during the compilation of long-term monitoring data are discussed. A preliminary exploration of the signals at different time scales was conducted to investigate correlations with real-world environmental phenomena, such as traffic variations between weekdays and weekends, as well as monthly changes. Baseline analysis (detrending moving average) was applied to extract the baseline components of long-term signals for observation. Short-time Fourier transform (STFT) spectrograms were utilized to present time-frequency content and capture time-varying phenomena of characteristic frequencies. Based on these results, the concepts of analyzing long-term structural vibration monitoring data and the extracted information are discussed. In addition, the practical experience gained from data compilation serves as a reference for hardware specifications and post-processing in long-term monitoring.

**Keywords:** bridge monitoring, long-term acceleration monitoring, baseline analysis, time-frequency analysis

## Introduction

Bridges are essential components of transportation infrastructure, and their structural safety is crucial for public safety and economic development. As bridges age and are exposed to extreme climatic conditions, real-time information on their performance has become necessary for effective management. Traditional manual inspection methods are insufficient to meet the requirements for real-time and comprehensive monitoring.

This study explores long-term acceleration vibration data from bridge decks for structural health monitoring (SHM). Common vibration parameters identified in these signals include natural frequency, mode shapes, and damping ratios. Standard techniques for analysis include modal analysis methods such as

Frequency Domain Decomposition (FDD) and Stochastic Subspace Identification (SSI). Time-frequency analysis methods, such as the Short-time Fourier transform (STFT), effectively present time-varying frequency phenomena. More advanced methods, including the Wavelet Transform and Hilbert–Huang Transform, further enhance the representation of time-frequency domain contents.

Bridge structures possess unique characteristics. They primarily carry traffic loads, including vehicles and their associated vibrations, which are wide-ranging and diverse. Additionally, they are subjected to load events such as natural disasters and traffic accidents. Bridges are also large-scale geometric structures, often spanning rivers with foundations located near or directly within riverbeds. The behavior of such massive structures is challenging to address

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using simplified concepts. Long-term monitoring of actual bridges yields data for in-depth analysis, reflecting real structural behavior and providing a reliable foundation for maintenance and management.

Comprehensive long-term monitoring data enable an understanding of bridge behavior from multiple perspectives. This study examines long-term acceleration response data at bridge deck expansion joints to observe real bridge behavior and characteristics.

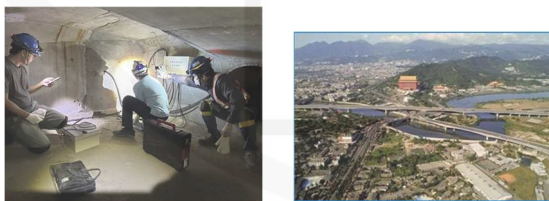
## Monitoring Overview

This section introduces the monitoring targets and equipment. The target description provides an overview of the observed structure and the monitoring locations. This content is primarily based on the project work report.

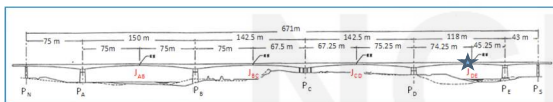
### Monitoring Target

**Structural Overview:** The observation target is the Yuanshan Bridge on National Highway No. 1 (Zhongshan Highway) (see Fig. 1, left). This bridge is a segmentally cantilevered articulated bridge. Each span is connected centrally by a male–female mortise-and-tenon hinge. The bridge is located at mileage 23K+876.809, spanning the Keelung River, with a total length of 671 m. It was constructed in December 1977.

**Observation Locations:** Acceleration measurements were obtained at the northbound articulated joint JDE. The installation site is shown in Fig. 1 (right). The location relative to the overall structure is marked with a star in Fig. 2.



**Fig. 1.** (Left) Accelerometer installation site; (right) exterior view of Yuanshan Bridge.



**Fig.2.** Structural dimensions of Yuanshan Bridge.

### Monitoring Equipment

The equipment used is the SARA Electronic Instrument SA10/1D single-axis force-balance

accelerometer (Fig. 3), which has a dynamic range greater than 165 dB. It operates in conjunction with the Sanlien Technology DMS24 data logger (Fig. 4), a four-channel, 24-bit logger with a 120 dB dynamic range and NTP time synchronization. The sampling rate is set at 200 Hz.



**Fig. 3.** SARA SA10 accelerometer.

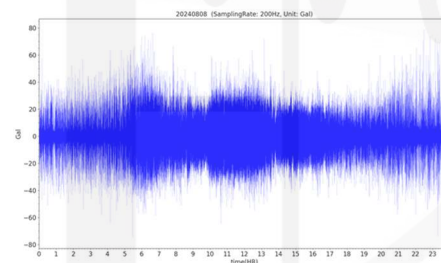


**Fig. 4.** Data Logger DMS24.

## Monitoring Results and Analysis

### Monitoring Results

After data review and compilation, this study obtained complete monitoring data from 2024/8/7 13:30 to 2025/4/17 23:30, covering a total of 252 days. To examine the dataset, both daily and annual scales (using all available data) were employed to represent time histories and observe relevant phenomena.



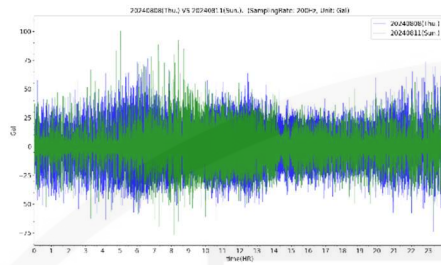
**Fig. 5.** Daily-scale acceleration record on August 8, 2024.

### Data Analysis

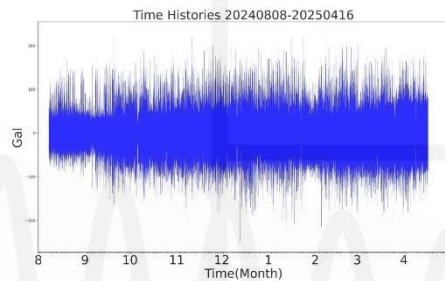
**Daily Scale:** Observations indicate that bridge deck acceleration signals are strongly correlated with traffic flow. As shown in Fig. 5, amplitudes are higher between 5:00–8:00 and 10:00–13:00, likely corresponding to commuting and lunch-hour traffic. Fig. 6 reveals a clear distinction between weekdays (blue) and holidays (green): holiday peak vibrations occur later and do not exhibit the midday surge, consistent with typical travel patterns.

**Annual Scale:** Fig. 7 displays the complete record. Signals before mid-September exhibit clear daily

amplitude cycles and baseline shifts. After mid-September, additional signal components reduce the clarity of these phenomena.

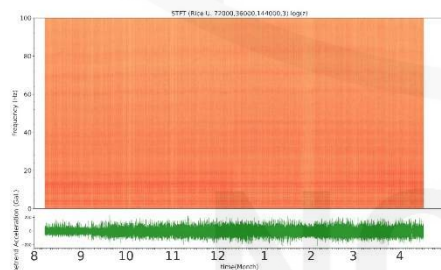


**Fig. 6.** Comparison of daily-scale acceleration between weekdays (August 8, 2024) and weekends (August 11, 2024).



**Fig. 7.** Acceleration histories at the annual scale (complete dataset).

Time-Frequency Analysis: Using STFT (Fig. 8), as described by Allen (1977) and Minh (2014), horizontal dark lines indicate prominent characteristic frequencies, while minor fluctuations reflect frequency changes over time. Vertical patterns represent energy variations; the reduced energy observed around February corresponds to decreased traffic during the Lunar New Year holiday.



**Fig. 8.** Spectrum obtained from STFT.

## Conclusion

This study has completed data compilation and preliminary exploration. Missing data were

temporarily handled by “zero-padding” to maintain continuity. Observations in the time domain indicate that amplitude changes correlate with traffic patterns (weekdays versus holidays). STFT results further confirm the relationship between vibration energy and traffic flow, particularly during holiday periods.

Future work will include:

- observing signal phenomena across different time scales.
- using data fusion to integrate traffic and other monitoring data for structural analysis.
- Long-term data accumulation will be essential for clarifying bridge characteristics and supporting maintenance and operations.

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# Study on Nonlinear Finite-Element Analysis Model of Single-Column PCI Girder Bridges Considering Near-Fault Effects

Fang-Yao Yeh<sup>1</sup>

葉芳耀<sup>1</sup>

## Abstract

The present study develops a nonlinear finite-element model of a single-column PCI girder bridge and investigates the seismic response and failure mechanisms of such bridges under near-fault earthquakes, as well as evaluates their seismic capacity. The findings are as follows: (1) Using commercial structural-analysis software, bridge columns can be modeled with either a fiber-section plastic-hinge formulation or a concentrated plastic hinge, both of which reproduce the nonlinear hysteretic behavior of reinforced-concrete columns under seismic loading. (2) Comparing the bridge's nonlinear behavior under far-field and near-fault ground motions shows that, when a velocity pulse occurs in a near-fault earthquake, the column-base bending moment-rotation relationship and the top-node displacement of the column change rapidly within a short period, generating a jump-type hysteresis loop. (3) Seismic-capacity assessment using incremental dynamic analysis reveals that near-fault ground motions produce the maximum displacement ratio of the column at relatively low PGA levels, causing earlier onset of longitudinal-reinforcement buckling and concrete crushing failure.

**Keywords:** nonlinear finite-element model, hysteretic behavior, near-fault earthquake, incremental dynamic analysis

## Introduction

In recent years, severe earthquakes have occurred consecutively worldwide, such as the 1994 Northridge earthquake in the United States, the 1995 Kobe earthquake in Japan, and the 1999 Chi-Chi earthquake in Taiwan. All these events occurred in regions close to active faults and resulted in significant casualties and substantial property loss. Because near-fault earthquakes pose a serious threat to structures, lives, and assets, research on their effects has attracted increasing attention from scholars both domestically and internationally. Historical seismic records show that ground motions observed near a fault rupture differ markedly from those recorded farther from the fault, particularly with respect to surface displacement and velocity pulses, which most clearly reveal their characteristic features. In near-fault zones, large-magnitude, unidirectional ground displacement and high-velocity pulses can occur within a short time span, often causing severe bridge damage and endangering the safety of road users.

This study develops a three-dimensional nonlinear finite-element model of a single-column PCI (precast segmental concrete) girder bridge, analyzes

the seismic response and failure modes of the bridge under near-fault earthquake excitations, and performs a seismic-capacity assessment to evaluate the bridge's seismic performance.

## Finite-Element Model of the Target Bridge

The target bridge spans the Wugu, Sanchong, and Luzhou districts of New Taipei City and is in seismic zone 1 of Taipei. The total bridge length is 290 m, with the span arrangement (north to south) as  $4 \times 37 \text{ m} + 2 \times 37 \text{ m} + 2 \times 34 \text{ m}$ , divided into two vibration units. The first vibration unit comprises spans 1–4, totaling 148 m, while the second vibration unit consists of spans 5–8, with a total length of 142 m. The structural system is a four-span continuous PCI simply-supported girder bridge, and artificial rubber bearing pads are used between the superstructure and the bridge piers. The bridge contains nine piers, utilizing four cross-section types: a circular column with a diameter of 300 cm, elliptical columns with major/minor axes of  $350 \times 200 \text{ cm}$ ,  $350 \times 250 \text{ cm}$ , and  $400 \times 250 \text{ cm}$ . Pier heights range from 7.47 m to 10.92 m.

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The structural characteristics show that, as ground acceleration increases, the horizontal forces generated in the superstructure by the earthquake also increase and are transmitted to the base of the piers. When the bending moment at a pier base exceeds the pier's yield moment, the pier enters a nonlinear stage, resulting in the development of a plastic hinge. Based on the bridge's nonlinear behavior, pier plastic hinges are modeled using a fiber-section model to simulate the pier's hysteretic response. A finite-element analysis model is constructed, as illustrated in Fig. 1.

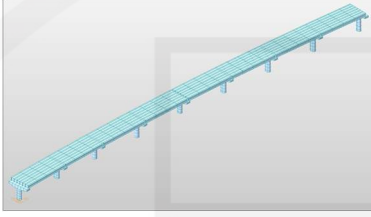


Fig. 1. Target bridge finite-element model.

### Nonlinear Model – Pier Plastic-Hinge Definition

This study utilizes the commercial structural analysis software SAP2000 to simulate the nonlinear hysteretic behavior of bridge piers [1]. The pier plastic hinges are represented by a fiber-section model for hysteresis-loop simulation, and the concrete fiber constitutive model adopts the compressive stress-strain relationship proposed by Mander et al. [2]. The concrete stress-strain curve is illustrated in Fig. 2(a), and the corresponding equation is provided as follows:

$$f_c = \frac{f'_{cc} x r}{r - 1 + x^r} \quad (1)$$

$$f'_{cc} = f'_{co} \left( -1.254 + 2.254 \sqrt{1 + \frac{7.94 f'_l}{f'_{co}}} - 2 \frac{f'_l}{f'_{co}} \right) \quad (2)$$

In which  $x = \varepsilon_c / \varepsilon_{cc}$ ,  $\varepsilon_{cc} = \varepsilon_{co} [1 + 5(f'_{cc} / f'_{co} - 1)]$ ,  $E_{sec} = f'_{cc} / \varepsilon_{cc}$ ,  $r = E_c / (E_c - E_{sec})$ . Here  $f_c$  = the axial stress of the concrete;  $f'_{co}$  = the compressive strength of unconfined concrete;  $f'_{cc}$  = the compressive strength of confined concrete;  $\varepsilon_c$  = the axial strain of the concrete;  $\varepsilon_{cc}$  = the concrete axial strain corresponding to  $f'_{cc}$ ;  $\varepsilon_{co}$  = the concrete axial strain corresponding to  $f'_{co}$ ;  $E_c$  = the elastic modulus of the concrete; and  $E_{sec}$  = the secant modulus of the confined concrete.

The constitutive model for reinforced steel fiber incorporates the strain-hardening effect, as shown in Fig. 2(b). The strain-hardening segment is represented by a parabolic curve, with its vertex located at the ultimate strain.

$$\text{elastic portion } (\varepsilon_s \leq \varepsilon_y) \quad f_s = E_s \varepsilon_s \quad (3)$$

$$\text{yielding portion } (\varepsilon_y < \varepsilon_s \leq \varepsilon_{sh}) \quad f_s = f_y \quad (4)$$

$$\text{strain-hardening portion } (\varepsilon_{sh} < \varepsilon_s \leq \varepsilon_{su})$$

$$f_s = f_{su} - (f_{su} - f_y) \left( \frac{\varepsilon_{su} - \varepsilon_s}{\varepsilon_{su} - \varepsilon_{sh}} \right)^2 \quad (5)$$

where  $E_s$  = the elastic modulus of the reinforcement;  $f_s$  = the axial stress of the reinforcement;  $f_{su}$  = the ultimate strength of reinforcement;  $f_y$  = the yield stress of the reinforcement;  $\varepsilon_s$  = the strain of the reinforcement;  $\varepsilon_{sh}$  = the strain at the onset of strain hardening;  $\varepsilon_{su}$  = the ultimate strain of the reinforcement; and  $\varepsilon_y$  = the yield strain of the reinforcement.

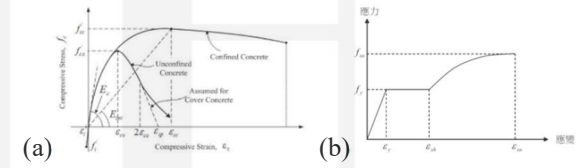


Fig. 2. Constitutive material models of the fiber-section model: (a) concrete model; (b) reinforced steel model.

To save analysis time and computational resources, only the primary nonlinear components governing the failure mode are modeled using a nonlinear fiber-section model, while the remaining bridge piers are represented by concentrated plastic hinges as developed in prior research [3]. Following the detailed seismic capacity assessment procedures in Chapter 4 of the Highway Bridge Seismic Evaluation and Retrofit Design Code [4], the moment-rotation relationship of the backbone curve is defined, and the plastic hinges are modeled with the Pivot hysteresis model. The  $\alpha$  and  $\beta$  parameters of the Pivot model for flexural-failure columns are set according to Equations (6) and (7), based on the findings of Ling et al. [5].

$$\alpha = 0.30 \times \left( \frac{N}{A_g f'_c} \right)^{-2.8} \times \left( \frac{\rho_l f_{yl}}{f'_c} \right)^{1.0} \times \left( \frac{\rho_{sh} f_{yt}}{f'_c} \right)^{0.65} + 2.20 \leq 10 \quad (6)$$

$$\beta = 0.62 \times \left( \frac{N}{A_g f'_c} \right)^{0.10} \times \left( \frac{\rho_l f_{yl}}{f'_c} \right)^{0.10} \times \left( \frac{\rho_{sh} f_{yt}}{f'_c} \right)^{-0.15} \leq 1 \quad (7)$$

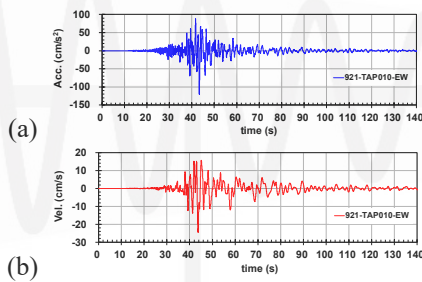
where  $N$  = the axial force of the member;  $A_g$  = the total cross-sectional area of the column;  $f'_c$  = the concrete compressive strength;  $\rho_l$  = the longitudinal reinforcement ratio;  $f_{yl}$  = the yield strength of the longitudinal reinforcement;  $\rho_{sh}$  = the shear-reinforcement volume ratio; and  $f_{yt}$  = the yield strength of the transverse reinforcement.

### Comparison of Bridge Nonlinear

## Behavior under Far-Field and Near-Fault Earthquakes

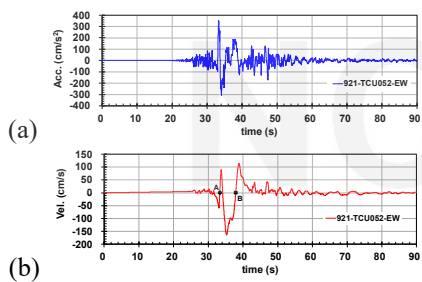
The selection of ground-motion time histories is a critical step in conducting nonlinear time-history analysis, as the chosen records dictate the strength demands on the structural system. In this study, far-field and near-fault ground-motion records were screened using the website tool developed by the National Center for Research on Earthquake Engineering, the Input Motion Selection of Taiwan (INMOST) [6], and the Database of Near-Fault Strong Motions with Pulse-like Velocity [7], respectively. The target bridge is in seismic zone 1 of Taipei and has a fundamental period of 0.676 s. The screening process resulted in the selection of three far-field records – 921-TAP003, 921-TAP010, and 331-TAP016 – and one near-fault record, 921-TCU052.

Fig. 3 shows the east-west ground-motion time history recorded at the TAP010 station in seismic zone 1 of Taipei during the 1999 Chi-Chi earthquake. The peak ground acceleration (PGA) is  $99.4 \text{ m/s}^2$ , and the peak ground velocity (PGV) is  $25.8 \text{ m/s}$ . No distinct velocity pulse is observed in the velocity time history.



**Fig. 3.** Seismic record of the Chi-Chi earthquake at the TAP010 station: (a) acceleration time history; (b) velocity time history.

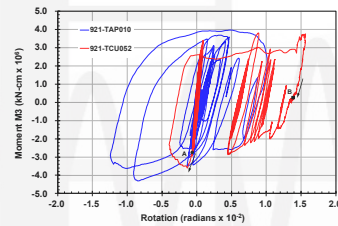
Fig. 4 shows the east-west ground-motion time history recorded at the TCU052 station in Beitun District, Taichung City. The shortest distance from the station to the fault-rupture surface is 1.03 km. The PGA is  $348 \text{ m/s}^2$ , and the PGV is  $183 \text{ m/s}$ . A distinct velocity pulse is observed between points A and B in the velocity record, with a horizontal pulse period of 12.29 s.



**Fig. 4.** Seismic record of the Chi-Chi earthquake at the TCU052 station: (a) acceleration time history; (b) velocity time history.

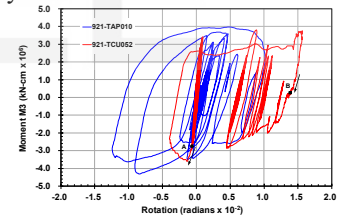
The target bridge was analyzed using a nonlinear

finite-element model, incorporating the ground-motion records 921-TAP010 and 921-TCU052. Both records were scaled so that their PGA corresponded to 300 gal, and then subjected to nonlinear dynamic time-history analysis. The results are presented in Figs. 5 and 6. Fig. 5 illustrates the moment-rotation relationship at the base of pier PU12Q. Under the 921-TAP010 record, the moment and rotation initially exhibit a linear trend and gradually transition into the nonlinear range, producing relatively regular hysteresis loops. The maximum positive and negative moments are  $3.97 \times 10^6 \text{ kN}\cdot\text{cm}$  and  $-4.31 \times 10^6 \text{ kN}\cdot\text{cm}$ , respectively, while the maximum positive and negative rotations are  $1.02 \times 10^{-2} \text{ rad}$  and  $-1.24 \times 10^{-2} \text{ rad}$ . Under the 921-TCU052 record, the moment-rotation behavior similarly evolves from linear to nonlinear. When the velocity pulse occurs, the response rapidly transitions from point A to point B in a short time, forming a jump-type hysteresis loop. The maximum positive and negative moments are  $3.81 \times 10^6 \text{ kN}\cdot\text{cm}$  and  $-3.60 \times 10^6 \text{ kN}\cdot\text{cm}$ , respectively, and the maximum positive and negative rotations are  $1.56 \times 10^{-2} \text{ rad}$  and  $-3.95 \times 10^{-3} \text{ rad}$ .



**Fig. 5.** Moment-rotation relationship of the target bridge during seismic response.

Fig. 6 shows the displacement of the top of pier PU12Q. Under the 921-TAP010 ground motion record, the displacements in the two horizontal directions exhibit a nonlinear elliptical relationship: the maximum positive and negative X-direction displacements are 12.90 cm and  $-15.77 \text{ cm}$ , respectively, while the maximum positive and negative Y-direction displacements are 12.57 cm and  $-11.40 \text{ cm}$ , respectively. Under the 921-TCU052 ground motion record, the top node displacements also trace a nonlinear elliptical path. When the velocity pulse occurs, the response rapidly transitions from point A to point B, producing a jump-type elliptical loop. The maximum positive and negative X-direction displacements are 19.76 cm and  $-5.85 \text{ cm}$ , respectively, and the maximum positive and negative Y-direction displacements are 10.27 cm and  $-27.40 \text{ cm}$ , respectively.

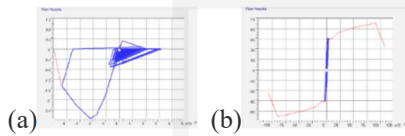


**Fig. 6.** Top displacement of the target bridge pier during seismic response.



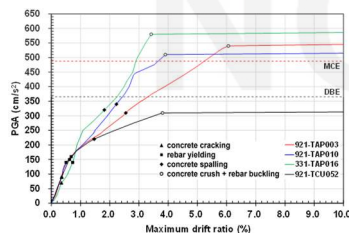
## Bridge Failure Modes and Seismic-Capacity Assessment

Incremental Dynamic Analysis (IDA) was employed to evaluate the seismic capacity of the bridge. Based on the multiple ground-motion records selected in the previous section to match the site characteristics, each record was scaled by its PGA and subjected to nonlinear dynamic analysis using the direct-integration method to obtain the bridge's seismic response and nonlinear behavior. Because the analysis utilizes a fiber plastic-hinge model, the study fully captures the stress-strain behavior of the concrete fibers in the component cross-section (Fig. 7(a)) and the reinforcement fibers (Fig. 7(b)), enabling objective assessment of damage states such as concrete cracking, steel yielding, concrete spalling, main-reinforcement buckling, and concrete crushing.



**Fig. 7.** Stress-strain hysteresis relationships of the bridge-section fibers under seismic loading: (a) concrete fiber; (b) steel reinforcement fiber.

Using IDA to evaluate the bridge's seismic capacity, the largest moment during shaking occurs in pier PU12Q, where the reinforcement yields first. The resulting IDA curve for the bridge is presented in Fig. 8. For the far-field earthquakes (921-TAP003, 921-TAP010, and 331-TAP016), the analysis shows that at a PGA of 70-90 gal, the pier exhibits concrete cracking; at a PGA of 140-160 gal, reinforcement yielding occurs; at a PGA of 310-340 gal, concrete spalling is observed; and at a PGA of 510-580 gal, the pier experiences main-bar buckling and concrete crushing. The corresponding maximum drift ratios for these damage states are 6.061 %, 3.905 %, and 3.419 %, respectively. For the near-fault earthquake (921-TCU052), the analysis indicates that at a PGA of 90 gal, concrete cracking occurs; at a PGA of 150 gal, reinforcement yielding is reached; at a PGA of 220 gal, concrete spalling occurs; and at a PGA of 310 gal, the pier undergoes main-bar buckling and concrete crushing, with a maximum drift ratio of 3.803 %. These results illustrate the progressive damage mechanisms and drift capacities of the bridge under increasing seismic intensity, as assessed by the IDA approach.



**Fig. 8.** IDA curve of the target bridge indicating damage states.

## Conclusions

This study develops a nonlinear finite-element model of a single-column PCI girder bridge to examine its seismic response, failure mechanisms, and seismic performance under near-fault earthquakes. The results indicate that:

1. Using commercial structural analysis software, the pier plastic hinge can be represented by either a fiber-section model or a concentrated plastic hinge. Both approaches can simulate the nonlinear hysteretic behavior of reinforced concrete piers under seismic loading.
2. A comparison of the nonlinear behavior of bridges under far-field and near-fault ground motions shows that, when a velocity pulse occurs in a near-fault earthquake, the base moment and rotation of the pier, as well as the top-node displacement, change rapidly within a short time, resulting in a jump-type hysteresis loop.
3. Seismic capacity assessment using IDA indicates that near-fault ground motions can induce the maximum drift ratio and result in main-bar buckling and concrete crushing at relatively low PGA, occurring earlier than for far-field ground motions.

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# Vertical Track Geometry Monitoring During Construction

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## Abstract

Track geometry is a core indicator for evaluating train ride comfort and operational safety. Civil engineering structures, such as embankments, viaducts, and tunnels, serve as the foundation of railway tracks and are key factors affecting the stability of track geometry. When these structures are subjected to earthquakes or extreme climatic conditions, the terrain may undergo abrupt or gradual changes, which can compromise train comfort and operational safety. To ensure long-term performance and safe railway operation, railway authorities often undertake reinforcement and rehabilitation of civil structures. During such construction periods, maintaining normal railway operations while providing effective early warning and real-time feedback on track conditions relies heavily on stable and reliable monitoring technologies. Compared with conventional highway-bridge monitoring, railway monitoring presents greater challenges. For example, sensors installed in embankment environments must withstand intense solar exposure and high temperatures, limited installation space, stringent reliability requirements, short allowable installation windows, and long maintenance intervals. This study presents technical improvements to a fiber Bragg grating-based differential settlement monitoring system and introduces a practical application case in railway embankment ground-improvement construction.

**Keywords:** railway embankment, ground improvement, track geometry, differential settlement, fiber-optic monitoring

## 1. Introduction

Vertical profile variation is a key monitoring item during railway construction because it directly affects train ride comfort and safety. A smooth and stable track profile is also an important indicator of the effectiveness of civil structure improvements. For embankment and viaduct sections, Taiwan's transportation regulations specify allowable vertical deformation limits for railway tracks. According to Ref. [1], the alert threshold is set at one-third of the allowable limit, while the action threshold is set at one-half. These thresholds serve as the basis for routine safety monitoring and remind engineering personnel to monitor structural conditions and implement appropriate responses when necessary.

## 2. Relationship Between Wavelength Response and Settlement

Fig. 1 illustrates the initial equilibrium state of a series-connected fiber Bragg grating differential settlement monitoring system (FBG-DSM). A total of

**m identical sensors** are connected via water-filled tubes without air bubbles. Each instrument includes a suspended cylindrical mass attached to a fiber Bragg grating. The cylinder density is slightly higher than water (e.g., acrylic density  $\approx 1.15$ ) to provide sufficient preload.

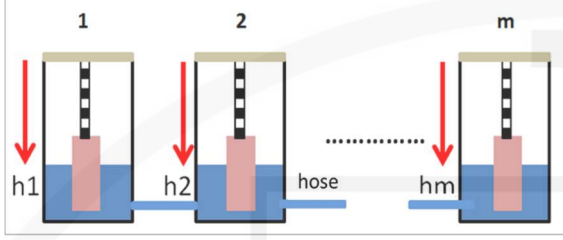
The reflected wavelength of a fiber Bragg grating changes with water level and is also influenced by temperature. In the following derivation, however, temperature effects are temporarily disregarded. According to the principle of communicating vessels, the water surface elevations in all instruments shown in Fig. 1 must be equal, as the free surface of static water forms a gravitational equipotential plane. (Temperature gradients within the water column are beyond the scope of this study.)

Due to installation constraints during construction, the base elevations of the instruments are not necessarily the same. Consequently, the distances between the instrument covers and the water surface differ. These distances are denoted as  $h_i$ , where  $i = 1$  to  $m$ , representing the initial state. When differential

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settlement occurs, water flows within the system until a new equilibrium is reached, at which point the water levels in all  $m$  instruments become equal again, as illustrated in Fig. 2. This condition is referred to as the post-equilibrium state. The distances from the instrument covers to the water surface in the initial and post-equilibrium states are expressed in vector form as



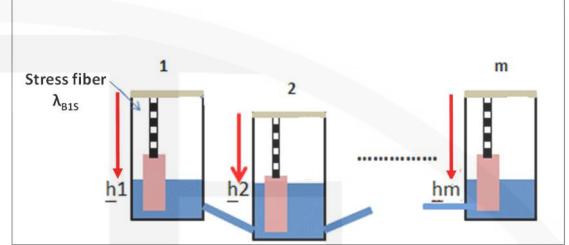
**Fig. 1.** Initial equilibrium state of the FBG-based DSMs

According to Archimedes' principle of buoyancy, and assuming the grating material behaves elastically within the linear range governed by Hooke's law, the relationship between water level change and wavelength variation can be expressed, based on Ref. [2], as  $\Delta h = C(\Delta\lambda_B/\lambda_B)/r^2$ , where  $\lambda_B$  denotes the Bragg wavelength before stretching,  $r$  is the radius of the cylindrical mass, and  $C$  is a material constant. The constant  $C$  incorporates factors such as liquid density, gravitational acceleration, Young's modulus of the optical fiber, cross-sectional area, and the photoelastic coefficient.

Equations (1) and (2), respectively.

$$\mathbf{H} \equiv \langle h_1, h_2, \dots, h_m \rangle \quad (1)$$

$$\underline{\mathbf{H}} \equiv \langle \underline{h}_1, \underline{h}_2, \dots, \underline{h}_m \rangle \quad (2)$$



**Fig. 2.** Post-equilibrium state: changes in the distance between the top cover and the water surface, with corresponding tension changes in the FBGs

Numerically, it is reasonable to assume that Equation (3) holds, and since the radius  $r$  is a predetermined parameter, subtracting Equation (1) from Equation (2) yields the water level change vector  $\underline{\mathbf{H}} - \mathbf{H}$ , as expressed in Equation (4), where  $K$  is a constant,  $K \equiv C/\lambda_{B1}/r^2$ . By defining the first instrument as the reference point, the relative height change vector of the remaining instruments with respect to this reference can be expressed by Equation (5). In this notation,  $R$  denotes "relative,"  $H$  denotes "height," and WOT indicates "without considering temperature effects."

$$1/\lambda_{Bi} \approx 1/\lambda_{Bj} \quad 1 \leq i, j \leq m \quad (3)$$

$$\underline{\mathbf{H}} - \mathbf{H} \equiv \langle \Delta h_1, \Delta h_2, \dots, \Delta h_m \rangle = K \langle \Delta\lambda_{B1}, \Delta\lambda_{B2}, \dots, \Delta\lambda_{Bm} \rangle \quad (4)$$

$$\mathbf{RH}_{\text{WOT}} \equiv \underline{\mathbf{H}} - \mathbf{H} - K\Delta\lambda_{B1} \langle 1, 1, \dots, 1 \rangle \quad (5)$$

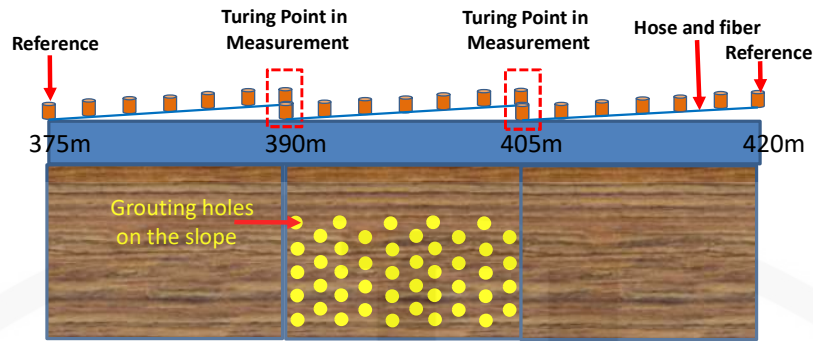
### 3. Monitoring of Track Improvement in an Embankment Section

#### 3.1 Layout of Monitoring Instruments

The application case presented in this study involves an embankment reinforcement project in a hilly area of central Taiwan. The ground-improvement construction zone extends from chainage 390 m to 405 m along the railway alignment. Fig. 3 shows a side view of the track and adjacent slope, with the central section representing the construction area on the embankment slope. The yellow circular symbols indicate the locations of drilling and grouting holes. FBG-DSM sensors were installed at 2.5 m intervals, covering a monitoring range from chainage 375 m to 420 m, which fully encompasses the construction

influence zone. The deployment density of the FBG-DSM system is sufficient to capture vertical track profile variations within the construction area. Because the track alignment has a longitudinal gradient, connecting all instruments with a single communicating tube would result in some sensors being installed at excessive heights, potentially interfering with train operations. To address this, a three-segment communicating-tube configuration was adopted, as shown in Fig. 3. This arrangement provides segmented measurement with data linkage at transition points. By applying cumulative calculations and adjustment computations between adjacent reference points, the relative elevation changes of each monitoring point with respect to a designated reference point can be determined. Here, "adjacent reference points" refers to the two bounding points, which are treated as fixed reference points.





**Fig. 3.** Side view of the railway embankment ground-improvement area and layout of monitoring instruments. Three communicating-tube systems were deployed, each with seven FBG-DSM sensors (21 sensors in total). Two sensors installed at different elevations within the red dashed box serve as measurement transition points.

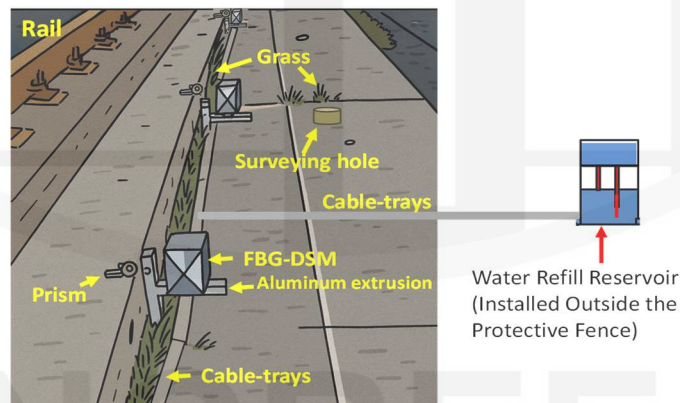
### 3.2 Sensor Installation

The instrument installation is shown in Fig. 4. The FBG-DSM sensors were anchored to the vertical wall surface of the concrete slab track. The installation height of this vertical surface was approximately 30–35 cm; exceeding this height would interfere with train operations. The gray cable trays shown in Fig. 4 were used to house the optical fiber transmission lines and water tubes. Outside the protective fence of the track area, automatic water refill reservoirs were installed to mitigate water evaporation and reduce the frequency of maintenance refilling.

In addition to the FBG-DSM sensors, two types of manual measurement methods were used on site: prisms and measurement holes. Prisms were installed on the track slab by the contractor as temporary facilities. During construction, leveling instruments were set up outside the protective fence, and personnel

conducted leveling surveys at least twice per working day, with increased frequency during the grouting stage. Measurement holes were installed on both the inner and outer sides of the rails. These are permanent facilities maintained by the railway authority, with short steel rod bases inside the holes to support invar leveling staff. The railway authority also maintains its own survey team, which performs track alignment surveys in the construction area approximately once per month during nighttime maintenance windows, typically between midnight and 4:00 a.m.

Although these two manual surveying methods are time-consuming and labor-intensive, and their operation is heavily constrained by weather and scheduling limitations, making them less efficient and less real-time than automated monitoring systems, the railway authority adopts a cautious approach by cross-validating multiple measurement datasets to ensure operational safety and construction quality.



**Fig. 4.** Slab track system with installed FBG-DSM sensors, surveying prisms, measurement holes, and automatic water refill reservoirs

## 4. Monitoring During Construction

### 4.1 Drilling Stage

The day on which the fiber-optic instruments were fully installed and began recording was defined

as Day 0, representing the initial baseline condition. During the drilling stage, the contractor erected working platforms and drilling equipment on the embankment slope, as shown in Fig. 5, to drill the grouting holes that had been planned (see Fig. 3) for subsequent grouting operations. The excavated soil samples were then jointly evaluated with the railway

authority to assess soil quality and determine the appropriate mix design parameters for the ground-improvement grout, including workability, initial setting time, and compressive strength. In this phase of construction, continuous disturbance from the drilling equipment to the embankment slope inevitably led to increased settlement. As shown in Fig. 6, the track profile exhibited progressive downward deformation, making the drilling stage a period of particular concern.

#### 4.2 Grouting Stage

After the drilling stage was completed, the project proceeded to the grouting stage. Fig. 7 and Fig. 8 show the changes in the vertical track profile during grouting operations. As illustrated in Fig. 7, the track profile within the grouted zone displayed gradual, alternating uplift from side to side during construction, likely due to staged or alternating grouting procedures. After 58 days of grouting, a localized depression persisted on the left side of the alignment on Day 129.

To address this small-area settlement, the railway authority requested the contractor to perform micro-grouting to raise the depressed section and further optimize the track profile. The monitoring results during this process are shown in Fig. 8. The localized grouting successfully filled the depressed zone, resulting in a smoother overall track profile. Finally, the differences in vertical track profile before and after construction, including measurements from the railway authority's survey team, contractor surveys, and fiber-optic monitoring, are presented in Fig. 9. The results from the different measurement methods show general agreement.



Fig. 5. Scaffolding work platforms and drilling equipment installed on the embankment slope.

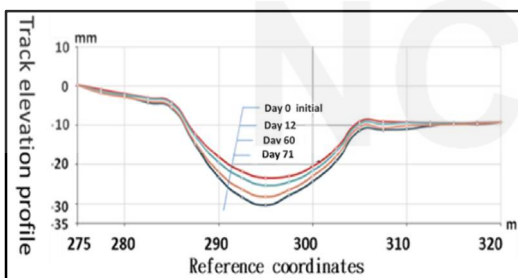


Fig. 6. Settlement increased steadily during the drilling stage, reaching a peak on Day 71.

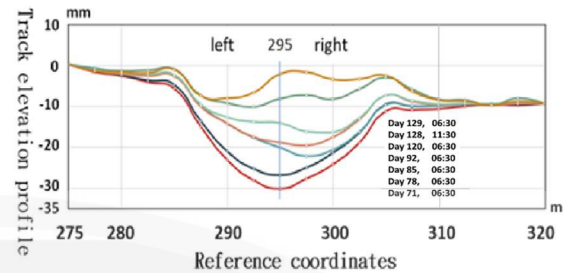


Fig. 7. Time history of the track profile during the grouting stage

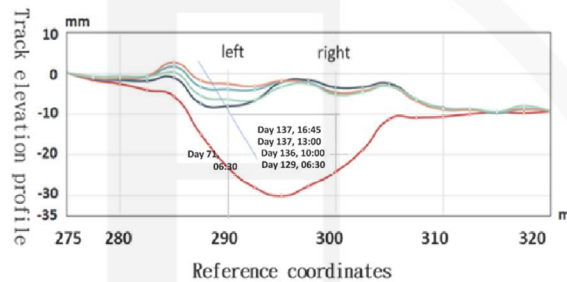


Fig. 8. Time history evolution of the track profile during the localized grouting stage

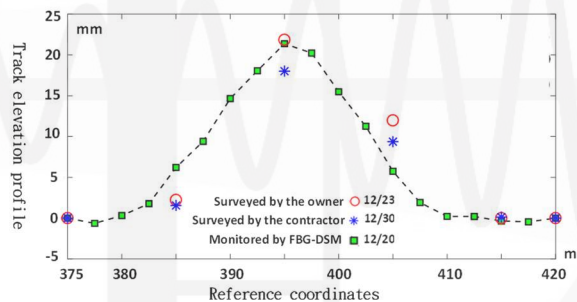


Fig. 9. Comparison of track profile variations measured by different teams before and after construction

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# Experimental and Numerical Evaluation of Full-Scale Viscoelastic Dampers under Damage Conditions

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林旺春<sup>1</sup>、汪向榮<sup>2</sup>

## Abstract

This study investigates the seismic performance of a full-scale viscoelastic (VE) damper subjected to transient earthquake loading. Previous research has examined the design, beyond-design, and residual performance of VE dampers through sinusoidal reversal tests, along with fractional derivative models to capture their hysteretic behavior. However, the damaged and post-damage behavior of VE dampers under seismic excitations has not been experimentally verified. In this study, seismic responses were obtained from numerical analyses of a viscoelastically damped structure and applied in dynamic tests of a full-scale VE damper. Results show that the damper remained intact and maintained stable design performance up to about 700% shear strain, surpassing the approximately 600% damage limit previously identified under harmonic loading in identical specimens. Comparisons between experimental responses and model predictions verified the accuracy of the fractional derivative model at the design stage. Furthermore, by introducing appropriate switching rules among design, beyond-design, and residual models, the seismic responses of the VE damper at and after damage were reproduced satisfactorily. Numerical results using a VE stick model further confirmed the applicability of the proposed approach, providing a framework for predicting the performance of VE dampers across different damage states under seismic loading.

**Keywords:** Viscoelastic damper, seismic loading, design-level performance, extreme-event performance, residual functionality, switching rules

## Introduction

Viscoelastic (VE) dampers have been widely studied and used in earthquake engineering because they provide both stiffness and energy dissipation. These devices are typically made of polymer materials with VE properties, allowing them to deform under seismic loads while dissipating energy. Previous studies have primarily investigated undamaged VE dampers under harmonic loading, focusing on how their hysteretic response depends on shear strain, excitation frequency, and ambient temperature. However, actual seismic events often cause transient responses, loads beyond design limits, and potential material damage. Understanding how VE dampers perform after sustaining damage is therefore essential for both design and retrofit applications. This paper addresses this gap by experimentally testing a full-scale VE damper under

earthquake loading, comparing its performance with fractional derivative models, and proposing switching rules for more accurate prediction of damper behavior during design-level, extreme, and post-event conditions.

## Experimental Study

The experimental program was conducted at the National Center for Research on Earthquake Engineering, National Institutes of Applied Research (hereinafter referred to as “NCREE”) Taipei laboratory using a high-performance damper testing facility. A full-scale VE damper specimen composed of ISD111 material layers was mounted to the horizontal actuator [1-3]. The setup ensured uniaxial shear deformation while controlling boundary conditions such as temperature and loading path. The test program

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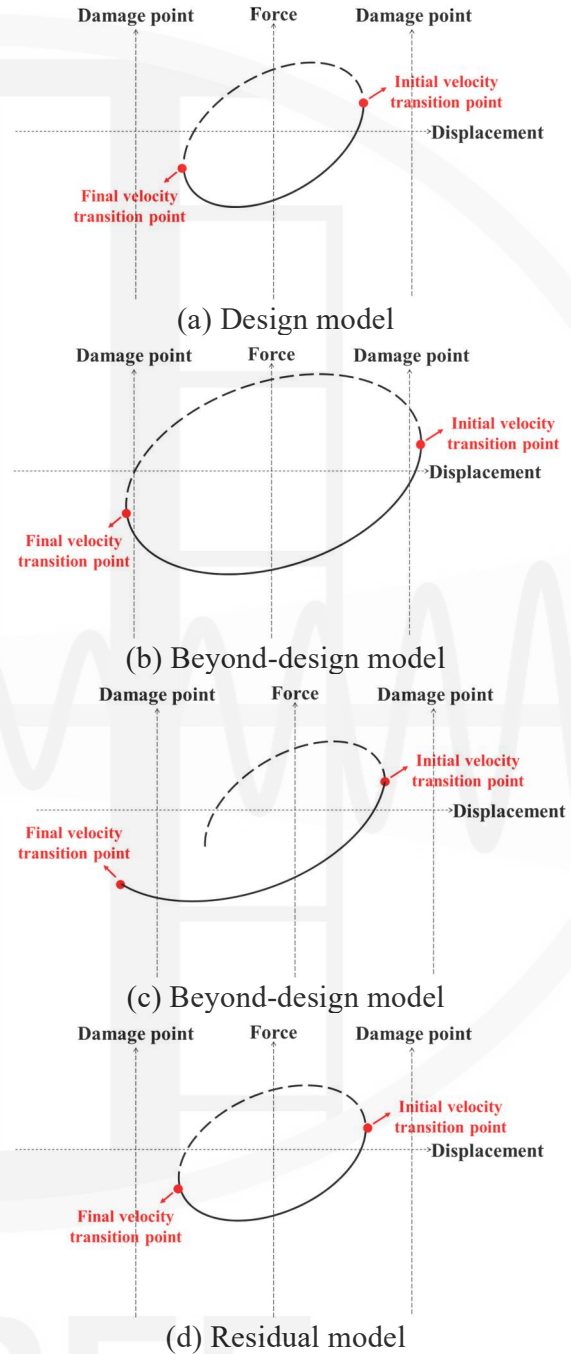


included four levels of seismic loading, derived from numerical simulations of a 26-story steel building equipped with VE dampers. Ground motions were selected from the Chi-Chi earthquake records (TCU017 and TCU089) and scaled to represent different seismic loading levels, as detailed below and summarized in Table 1. Level I targeted shear strains below the damage threshold to validate the design model [1]. Level II was intended to first exceed the 600% damage threshold observed in harmonic loading, allowing for observation of damper failure initiation. Level III focused on residual behavior once the damper had already been damaged, while Level IV subjected the specimen to large shear strains once more to validate the proposed switching rules. Results showed that the VE damper could sustain up to 700% shear strain before significant visible tearing and stiffness degradation occurred, suggesting improved resilience under seismic loading compared to harmonic excitation. To more accurately capture VE damper behavior across different performance stages, switching rules are proposed. When shear strain is below the damage threshold (about 600%), the design model is used; once the threshold is exceeded, the shear stress decreases with a negative slope before transitioning to the beyond-design model. For damaged dampers under seismic loading, the beyond-design model is applied if the velocity transition points in a half cycle exceed the damage threshold; otherwise, the residual model is used. These switching rules ensure numerical stability and reasonably reflect the transition of VE damper performance from intact to damaged states under seismic loading, as shown in Fig. 1.

**Table 1. Test program**

|                  | Excitation frequency (Hz) | Shear strain amplitude (%) | Ambient temperature (°C) | Number of cycles         |
|------------------|---------------------------|----------------------------|--------------------------|--------------------------|
| Harmonic loading | 0.1, 0.2, 0.3, 1.0, 2.0   | 100, 300, 480              | 25, 30 (±2°C)            | 6                        |
| Seismic loading  | Ground motion             | Scale (%)                  | Ambient temperature (°C) | Maximum shear strain (%) |
|                  | TCU017-DBE                | 33                         |                          | 207.84                   |
|                  | TCU017-DBE                | 100                        |                          | 422.56                   |
|                  | Level I TCU089-DBE        | 33                         | 3F                       | 295.02                   |
|                  | TCU089-DBE                | 100                        |                          | 470.72                   |
|                  | Level II TCU017-DBE       | 100                        | 4F                       | 894.54                   |
|                  | TCU017-DBE                | 33                         |                          | 207.84                   |
|                  | Level III TCU017-DBE      | 100                        | 3F                       | 422.56                   |
|                  | TCU089-DBE                | 33                         | 15F                      | 276.38                   |
|                  | TCU089-100%               |                            |                          | 479.56                   |

| DBE   |            |          |        |
|-------|------------|----------|--------|
| Level | TCU089-DBE | 150% 3F  | 722.82 |
| IV    | TCU089-DBE | 190% 15F | 702.22 |

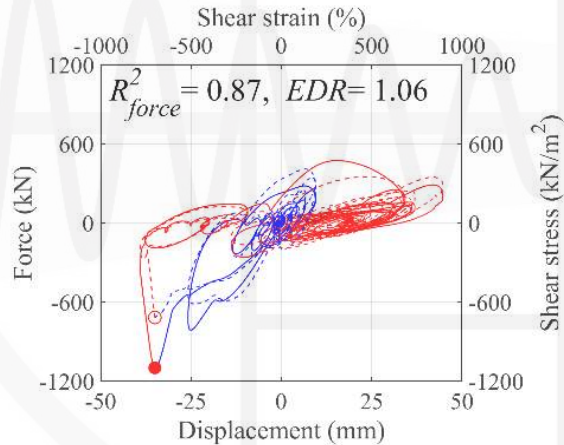


**Fig. 1. Illustrations of switching-rule conditions for various fractional derivative models**

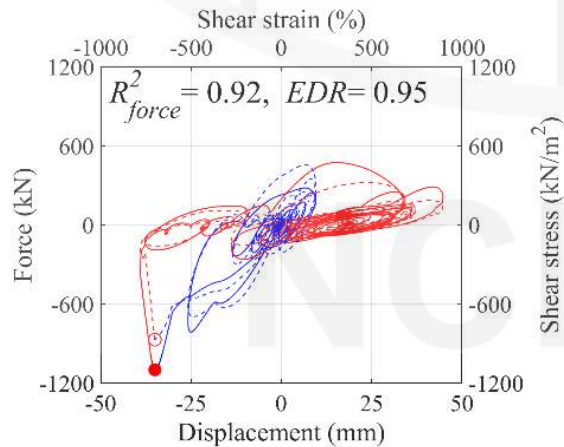
## Comparison and Discussion

The seismic test results were compared with predictions from fractional derivative models previously calibrated under harmonic loading. The design model accurately captured the hysteretic loops

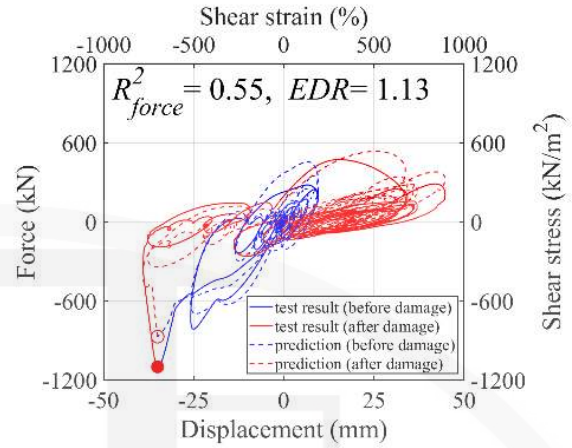
of intact dampers. Under Level I loading, the design model accurately represented the pre-damage performance of the VE damper. At Level II loading, both the combined design and beyond-design models and the combined design, beyond-design, and residual models achieved similar accuracy, outperforming the combined design and residual model. This suggests that the beyond-design model more realistically describes post-damage behavior, as shown in Fig. 2. For Level III loading, either the beyond-design model or the residual model alone could represent the damaged responses, though differences in accuracy highlight the relative suitability of each model, as shown in Fig. 3. Under Level IV loading, both the combined beyond-design and residual models and the beyond-design model alone aligned well with the experimental results, while the residual model alone showed lower accuracy, as shown in Fig. 4. Overall, when the VE damper experiences shear strains beyond the damage threshold, the beyond-design model and residual model can reasonably predict post-damage performance using switching rules. However, since only one specimen was tested, further validation with additional experiments is necessary.



(a) Prediction by the combined design, beyond-design, and residual model

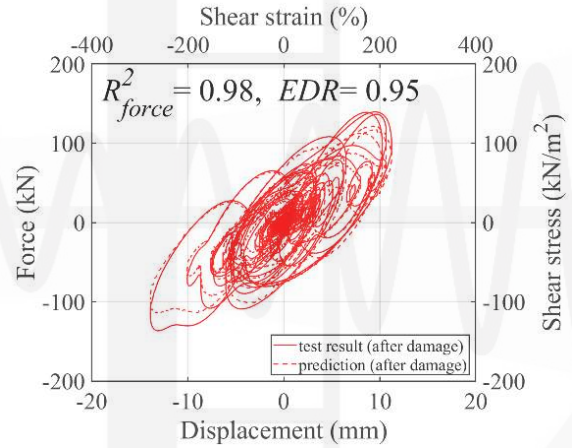


(b) Prediction by the combined design and beyond-design model

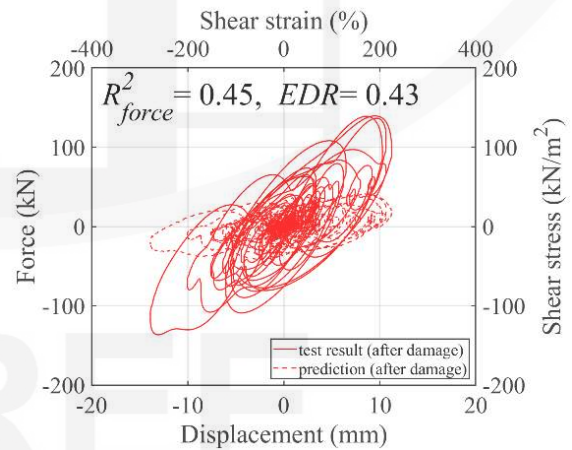


(c) Prediction by the combined design and residual model

**Fig. 2.** Experimental and predicted performance of the VE damper before and after damage under TCU017-DBE-100%-4F (Level II seismic loading)

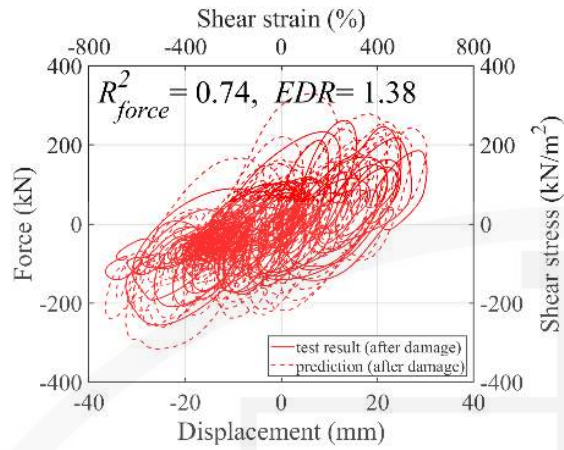


(a) Prediction by the residual model

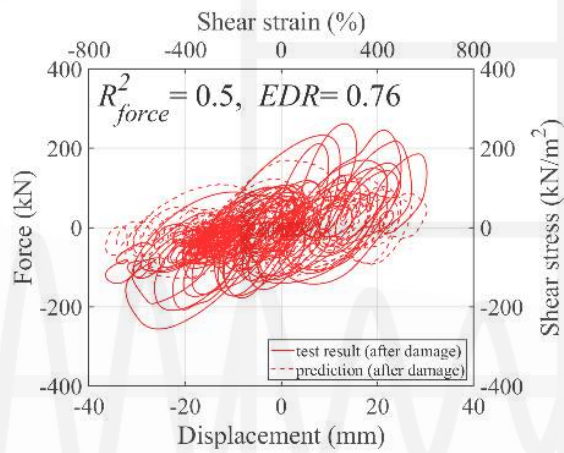


(b) Prediction by the beyond-design model

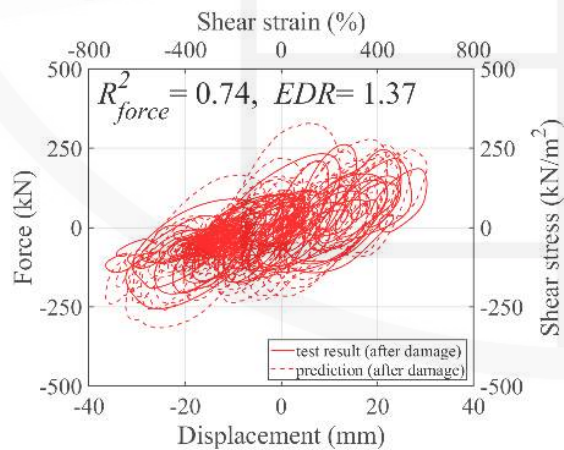
**Fig. 3.** Experimental and predicted post-damage performance of the VE damper under TCU089-DBE-33%-15F (Level III seismic loading)



(a) Prediction by the combined beyond-design and residual model



(b) Prediction by the beyond-design model only



(c) Prediction by the residual model only

**Fig. 4.** Experimental and predicted post-damage performance of the VE damper under TCU089-DBE-150%-3F (Level IV seismic loading)

hardening, the pre-damage performance of VE dampers can be accurately simulated under both harmonic and seismic loading. The identified damage threshold for VE dampers under seismic loading, which is a shear strain of 700% and is slightly higher than that identified under harmonic loading (i.e., shear strain of 600%), indicates that VE dampers have a greater damage capacity during seismic events than during harmonic excitation. Furthermore, by employing fractional derivative models for beyond-design and residual performance, combined with a rational and feasible switching strategy, the true mechanical performance of damaged VE dampers under seismic response histories can be predicted with higher accuracy. This capability is particularly important for conservatively assessing the seismic mitigation effectiveness of structures equipped with VE dampers.

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## Conclusions

By using fractional derivative models that account for excitation frequency, ambient temperature, internal damper temperature, and material softening and



# Feasibility Study on Discontinuous Inerter Mass Dampers

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## Abstract

Taiwan is located at an active tectonic plate boundary with frequent earthquakes. As urban development progresses, the increasing number and density of high-rise buildings make structural control technologies essential for ensuring seismic safety. Although the tuned mass damper (TMD) is widely used, its performance strongly depends on parameter tuning and requires large mass and stroke capacity. To overcome these limitations, this study proposes a discontinuous inertial mass damper (DIMD), with two configurations based on inertial mass placement: the centralized type (M-DIMD) and the dual-sided type (DS-DIMD), along with corresponding analytical models. The effects of key design parameters, including the inertial mass ratio and the segmented length of the driving inertial mass, are investigated using frequency response and inertia force-displacement relationships. The proposed systems are then applied to a single-degree-of-freedom (SDOF) structure to evaluate their feasibility for vibration control. Numerical simulations of free vibration are conducted to examine energy distribution, modal energy transfer, and equivalent damping. Based on SDOF and two-degree-of-freedom (2DOF) analyses, the mechanical characteristics of DIMD are identified and compared with those of a conventional TMD in terms of structural response and damper stroke. Results show that although DIMD provides slightly lower control performance than TMD, it effectively reduces damper stroke while maintaining good stability and design flexibility, offering a practical alternative for mitigating the limitations of traditional TMD systems.

**Keywords:** near-fault ground motions, quasi-dynamic model, variable rupture velocities, 1992 Landers Earthquake

## Introduction

Traditional passive tuned mass dampers (TMDs) are widely used for wind and vibration control in high-rise buildings and bridges, such as Taipei 101 and the Tokyo Skytree, due to their simple design, ease of construction, and low maintenance cost. However, the control performance of TMDs is highly dependent on parameter tuning, and performance may significantly degrade when design parameters deviate from optimal conditions. To address this issue, researchers have proposed using multiple TMDs to distribute control demand and reduce reliance on a single damper, as well as semi-active control strategies that adjust damping or stiffness to respond more effectively to seismic excitation.

Conventional TMDs also require large mass and displacement space, creating practical challenges for

structural applications. To reduce additional structural demand, some studies have utilized large rooftop water tanks as dampers. Other studies have explored rotational mechanisms to achieve efficient control with smaller masses. Malcolm C. Smith proposed the concept of the inerter, which generates rotational inertia through a flywheel driven by a rack-and-pinion mechanism. Ioan Lazar further combined the inerter with dampers to enhance structural vibration control performance. Building upon these developments, this study extends the design of discontinuous inertial components and compares their performance with conventional TMDs to evaluate their potential and effectiveness for structural vibration mitigation.

## Analytical Model of TMD cooperated with Discontinuous Inerter

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This study proposes a discontinuous inerter mass damper (DIMD). Based on the placement of the discontinuous inerter, the system can be classified into the middle-type DIMD (M-DIMD) and the dual-side-type DIMD (DS-DIMD). The two types of dampers are installed on the roof of a two-story, two-degree-of-freedom (2DOF) primary structure, as illustrated in Fig. 1 and Fig. 2. Here,  $\delta$  denotes the segmented length of the damper, which controls the activation range of the inerter mechanism. For the M-DIMD, the inerter rotates when the damper stroke remains within the segmented length. By contrast, for the DS-DIMD, the inerter rotates only when the stroke exceeds the segmented length. The rotational inertia generated by the inerter contributes to the mass term in the structural equations of motion, and the corresponding equivalent mass ratio is defined as the inerter ratio ( $\beta$ ). The discontinuous interaction between the damper and the inerter is simulated based on the following assumptions: (1) when the damper and inerter are separated, the inerter continues moving at a constant velocity; and (2) when contact occurs, a perfectly inelastic impact is assumed, such that both components move together immediately after impact at the same velocity.

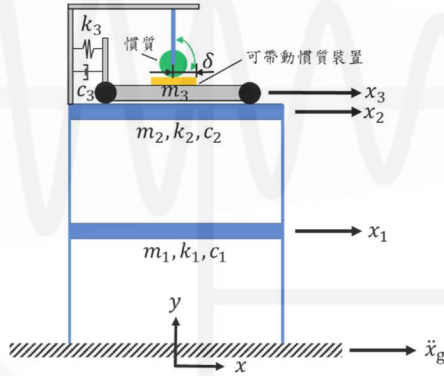


Fig. 1. M-DIMD schematic diagram

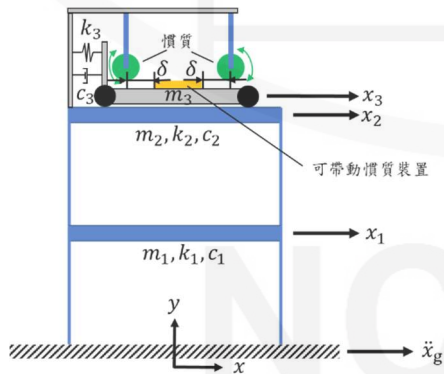


Fig. 2. DS-DIMD schematic diagram

The numerical models shown in Fig. 1 and Fig. 2 represent 2DOF primary structures equipped with M-DIMD and DS-DIMD, respectively. The complete system is therefore a three-degree-of-freedom (3DOF) system, and its equation of motion can be expressed as

Eq. (1):

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

According to the operating concepts of the two inerter mechanisms, the mass-switching logic associated with the segmented length  $\delta$  can be determined by Eqs. (2a) and (2b), respectively. Here,  $M_{int}$  represents the condition in which the inerter effect is activated; otherwise, only the original structural mass matrix is considered.

$$\mathbf{M}_\beta = \begin{cases} \mathbf{M}_{int}, & \text{if } |u_3| \leq \delta \\ \mathbf{M}_s, & \text{if } |u_3| > \delta \end{cases} \quad (2a)$$

$$\mathbf{M}_\beta = \begin{cases} \mathbf{M}_s, & \text{if } |u_3| \leq \delta \\ \mathbf{M}_{int}, & \text{if } |u_3| > \delta \end{cases} \quad (2b)$$

In Eq. (2), due to the switching behavior of the inerter mechanism, the mass matrices with and without the inerter effect can be calculated using Eq. (3).

$$\mathbf{M}_{int} = \begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3(1 + \beta) \end{bmatrix} \quad (3a)$$

$$\mathbf{M} = \begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} \quad (3b)$$

Furthermore, the damping matrix and stiffness matrix in Eq. (1) can be calculated using Eqs. (4) and (5), respectively.

$$\mathbf{C} = \begin{bmatrix} c_1 + c_2 & -c_2 & 0 \\ -c_2 & c_2 + c_3 & -c_3 \\ 0 & -c_3 & c_3 \end{bmatrix} \quad (4)$$

$$\mathbf{K} = \begin{bmatrix} k_1 + k_2 & -k_2 & 0 \\ -k_2 & k_2 + k_3 & -k_3 \\ 0 & -k_3 & k_3 \end{bmatrix} \quad (5)$$

## Characteristics and Frequency Response of Discontinuous Inerter

To investigate the mechanical characteristics of the DIMD, this section employs the single-degree-of-freedom (SDOF) equation of motion of the DIMD together with the assumption of a perfectly inelastic collision. A discrete-time state-space method is adopted to simulate the effects of the inerter ratio and segmented length on the dynamic behavior of the DIMD. The SDOF equation of motion is obtained by reducing the 2DOF of the primary structure from Eq. (1), as expressed in Eq. (6):

$$m_{int} \ddot{x}_3 + c_3 \dot{x}_3 + k_3 x_3 = -m_3 \ddot{x}_g \quad (6)$$

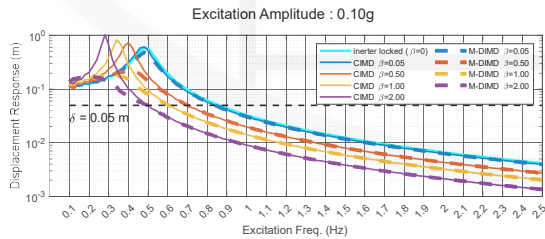
The mass-switching conditions corresponding to the inerter effects of the M-DIMD and DS-DIMD are expressed by the following equations, respectively.

$$m_{\text{int}} = \begin{cases} m_3(1 + \beta), & \text{if } |u_3| \leq \delta \\ m_3, & \text{if } |u_3| > \delta \end{cases} \quad (7a)$$

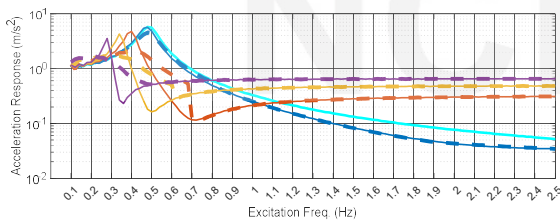
$$m_{\text{int}} = \begin{cases} m_3, & \text{if } |u_3| \leq \delta \\ m_3(1 + \beta), & \text{if } |u_3| > \delta \end{cases} \quad (7b)$$

A DIMD system with a mass, damping coefficient, and stiffness of 2 kg, 1.057 N·s/m, and 18.974 N/m, respectively, is subjected to sinusoidal excitation to obtain the frequency responses of steady-state displacement and absolute acceleration. By varying the inerter ratio ( $\beta$ ) and comparing the results with those of a continuous inerter mass damper (CIMD), the characteristics of the two discontinuous inerter systems are investigated. The inerter ratios are set to 0.05, 0.5, 1.0, and 2.0, while the segmented length of both the M-DIMD and the DS-DIMD is fixed at 0.05 m. Numerical simulations are conducted using sinusoidal excitations with different frequencies and a constant amplitude of 0.1g.

First, the simulation results of the M-DIMD system are shown in Fig. 3. The excitation frequency corresponding to the peak frequency response,  $\Omega_{\text{peak}}$ , decreases as the inerter ratio increases, and its value lies between 0.49 Hz and the  $\Omega_{\text{peak}}$  of the CIMD with the same inerter ratio (Fig. 3(a)). In addition, when the displacement response approaches the segmented position, a distinct transition can be observed in the absolute acceleration response near the corresponding excitation frequency (Fig. 3(b)). This phenomenon occurs because once the damper displacement becomes smaller than the segmented length, the system behavior changes from that of the M-DIMD to that of the CIMD. The frequency response curves of the M-DIMD in the high-frequency range also nearly overlap with those of the CIMD. Furthermore, the absolute acceleration frequency response shows that the acceleration response gradually approaches a constant value as the excitation frequency increases.

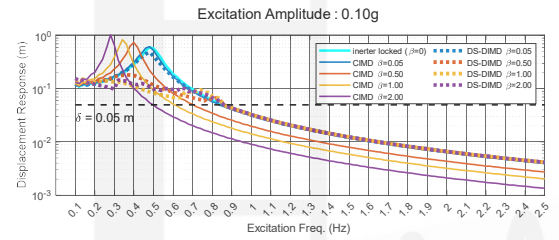


**Fig. 3(a).** M-DIMD frequency response of displacement

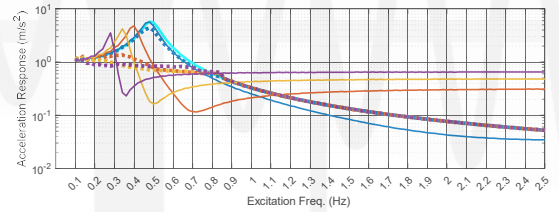


**Fig. 3(b).** M-DIMD frequency response of absolute acceleration

For the DS-DIMD, dynamic behavior is more complex because the inerters on both sides move independently. As a result, fluctuations appear in some frequency-response simulations, making  $\Omega_{\text{peak}}$  difficult to identify clearly, especially when the displacement response approaches the segmented position. In addition, when the excitation force is small, the differences in frequency response among different inerter ratios are less significant. As the excitation amplitude increases, the system exhibits more frequent switching, leading to greater differences in the frequency responses. Finally, like the M-DIMD, when the displacement of the DS-DIMD becomes smaller than the segmented length, the system transforms into a continuous system without inerter effects. Therefore, its frequency-response curves almost overlap with those of the locked-inerter case ( $\beta = 0$ ).



**Fig. 4(a).** DS-DIMD frequency response of displacement



**Fig. 4(b).** DS-DIMD frequency response of absolute acceleration

## Seismic Performance

Following the parametric analysis of the discontinuous inerter system in the SDOF model, numerical simulations based on the equation of motion in Eq. (1) were further conducted to verify the feasibility of applying discontinuous inerters for structural vibration control. M-DIMD and DS-DIMD systems with a mass, damping coefficient, stiffness, and segmented length ( $\delta$ ) of 2 kg, 1.057 N·s/m, 18.974 N/m, and 0.05 m, respectively, were installed on the roof of a two-story 2DOF primary structure, forming an overall 3DOF system. The mass of each floor in the primary structure is 100 kg.

Fig. 5 presents the results of the 2DOF primary structure equipped with the M-DIMD under an excitation amplitude of 0.1g. For the primary structure with a first-mode frequency of 0.2 Hz, the M-DIMD with an inerter ratio of 2.0 exhibits the lowest absolute acceleration response at the roof level among all



configurations, indicating favorable interaction between the M-DIMD and the primary structure. When the first-mode frequency increases to 0.4 Hz, the minimum absolute acceleration response occurs at an inerter ratio of 1.0, whereas for a first-mode frequency of 0.6 Hz, the smallest response is obtained with an inerter ratio of 0.05. These results indicate that the optimal inerter ratio of the M-DIMD varies with the frequency of the primary structure and tends to decrease as the structural frequency increases. For the primary structure with a first-mode frequency of 0.8 Hz, the roof absolute acceleration responses under the four investigated inerter ratios are identical to those of the uncontrolled structure, suggesting that the M-DIMD has limited effectiveness in reducing absolute acceleration for high-frequency structures. On the other hand, regarding damper stroke, higher inerter ratios lead to reduced stroke under the same structural frequency. This effect is particularly significant when the first-mode frequencies are 0.4 Hz and 0.6 Hz, demonstrating that the inerter effectively suppresses the stroke demand of the M-DIMD.

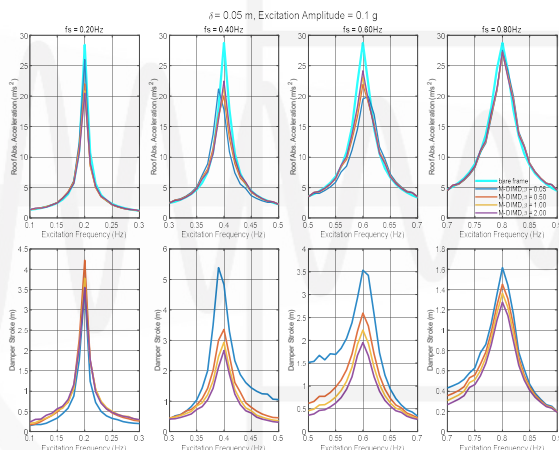


Fig. 5. FRF of structure with M-DIMD

Fig. 6 shows the results of the 2DOF primary structure equipped with the DS-DIMD under an excitation amplitude of 0.1 g. Overall, smaller inerter ratios are more suitable for higher-frequency structures, which is consistent with the behavior observed in the M-DIMD system. However, under the same structural frequency, the DS-DIMD achieves greater reduction in absolute acceleration response than the M-DIMD, particularly when the first-mode frequencies of the primary structure are 0.2 Hz and 0.4 Hz. In contrast to the M-DIMD, the DS-DIMD exhibits a different trend in damper stroke behavior. When the primary structural frequency is 0.2 Hz, larger inerter ratios lead to increased damper stroke, even exceeding the corresponding stroke of the M-DIMD. For the other three structural frequencies, however, larger inerter ratios reduce the damper stroke, showing a trend like that of the M-DIMD.

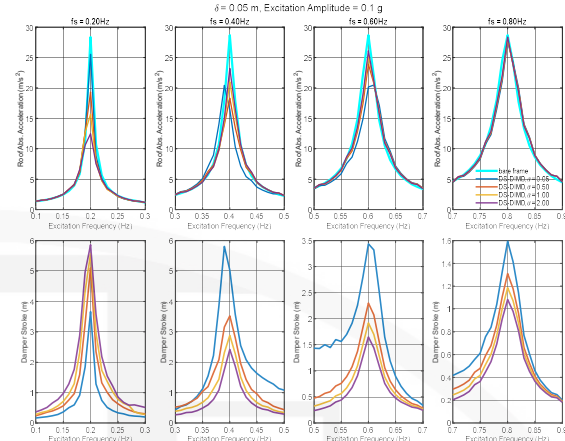


Fig. 6. FRF of structure with DS-DIMD

## Conclusions

This study investigates the dynamic characteristics and vibration control performance of the DIMD. Two configurations, namely the M-DIMD and the DS-DIMD, are proposed based on the placement of the inerter, and corresponding mathematical models are established to evaluate their effects on frequency response, energy behavior, and structural response under different parameter settings. By utilizing rotational inertia generated by the inerter, the DIMD can amplify the equivalent system mass, thereby reducing the required physical mass of the damper while maintaining comparable control performance. In addition, the incorporation of discontinuous behavior with the inerter mechanism helps limit energy transfer between systems and effectively reduce damper stroke. Overall, compared with conventional mass dampers such as the TMD, the DIMD offers advantages in lightweight design and space efficiency, providing greater flexibility for practical structural applications.

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# Application of Particle Swarm Optimization of Fundamental Solutions to Solve External Flow Field Problems for Offshore Wind Turbine Components

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古孟晃

## Abstract

This paper develops a boundary-type meshless numerical model to simulate the external flow field for wind turbine components under different conditions and calculates the forces acting on the components. It is expected to provide a reference for future wind turbine component design. The study mainly aimed to develop innovative numerical methods, including the particle swarm optimization (PSO) algorithm combined with the method of fundamental solutions (MFS) to construct a potential flow model. The MFS is a boundary-type meshless numerical method, requiring only points set at the boundary for simulation, thus avoiding the need for mesh construction and related numerical integration procedures. It can be considered a simple and computationally accurate innovative method. Since the MFS previously relied heavily on suitable source point locations to obtain high-precision solutions, a highly efficient PSO algorithm is used to determine the optimal source point locations, ensuring the accuracy of the numerical simulation results. After the external flow field model is developed, the external flow field of wind turbine components, such as the tower and blades, will be calculated, and the forces acting on them will be directly and precisely calculated. Current wind turbine component design typically employs blade element momentum theory for calculations, which is limited to the laws of energy and mass conservation. Therefore, modifications are often necessary to accommodate the flow field characteristics at different component locations. This proposed approach is based on potential flow theory and considers both mass and momentum conservation. Thus, calculations can be performed simply by changing the shape of the wind turbine components. This avoids pitfalls that cannot be addressed using the blade element method and is expected to provide a more effective tool for domestic wind turbine component design.

**Keywords:** Meshless, Method of Fundamental Solutions, Particle Swarm Optimization

## I. Introduction

Due to the changes and developments in all aspects of human society over the past century, the political and economic strategies of countries around the world have been constantly changing. In 2015, the United Nations published the “2030 Sustainable Development Goals” [1], which listed 17 important sustainable development goals (SDGs) and 169 detailed indicators, 1352 documents, etc. The 17 SDGs are shown in Fig. 1. At that time, it was expected that the 17 important SDGs could be achieved before 2030. Among them, the 7th of the 17 goals, “Affordable and Clean Energy,” is regarded by countries as one of the important directions for development. In 2022, the

National Development Council, together with seven ministries, including the Ministry of Economic Affairs and the Ministry of Science and Technology of the Executive Yuan, also published the “Taiwan 2050 Net Zero Emission Path and Strategy General Explanation.” The 2050 net zero emission path will be based on four types of transformation: “energy transition,” “industrial transition,” “lifestyle transition,” and “social transition.” It is expected to be based on “technology research and development” and “climate legislation” as governance foundations, supplemented by “twelve key strategies” [2], as shown in Fig. 2. Energy transition and the development of related clean energy have become important and urgent issues of concern to various units in Taiwan. As Taiwan is an

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island with a small land area and is surrounded by the sea, there are limits to the development of alternative energy solutions on land. However, Taiwan is surrounded by wind fields, so it is very suitable for taking advantage of the marine environment to develop related clean energy technologies. Therefore, the development of marine energy in the waters around Taiwan has an inherent advantage that cannot be underestimated, and the relevant planning of Taiwan's net zero emission path also indirectly points out that Taiwan should take advantage of these advantages to develop clean energy technologies.

In 2018, Taiwan Power Company released a feasibility assessment report on offshore wind power generation [3]. Based on early research and investigation of offshore wind power generation, the Bureau of Standards, Metrology and Inspection of the Ministry of Economic Affairs issued the Technical Guidelines for Site Investigation and Design of Offshore Wind Power Generation [4], Technical Guidelines for Power Generation Operation and Maintenance [5], and Technical Guidelines for Manufacturing and Construction [6] in 2023. These guidelines are used to help establish a foundation for Taiwan's wind turbine design, development, and maintenance. Due to Taiwan's geographical characteristics, the key analysis technology of offshore wind power generation systems cannot be separated from stress analysis to design stable devices that can resist natural phenomena such as wind, waves, and currents. In recent years, OpenFAST (Open-source Simulation Tool for Advanced Wind Turbine Systems), an open-source analysis tool supported by the Wind Energy Technologies Office of the U.S. Department of Energy [7,8], has been widely favored by users from all parties. OpenFAST provides a modular method for simulating the dynamic behavior of wind power generation device components under various conditions. Because OpenFAST allows for flexible modular selection and combination of physical models and numerical methods to meet specific design requirements, it provides users with flexibility and scalability to select appropriate models and parameters. It can simultaneously consider factors such as structural mechanics, aerodynamics, control systems, and power systems. This enables relevant users to fully understand the performance, response, and load characteristics of wind power generation systems and thus conduct appropriate design and evaluation. Similar well-known analysis tools include QBlade [9], developed by the team at the Technical University of Berlin. In 2020, the National Renewable Energy Laboratory (NREL) of the U.S. released the IEA 15 MW reference offshore wind turbine open-source design under the name of the International Energy Agency (IEA) to help manufacturers evaluate performance and cost before developing prototypes. During the development of the IEA 15 MW, OpenFAST was heavily relied upon. OpenFAST uses blade element momentum (BEM) theory for wind load

calculation [10,11,12,13]. BEM theory is based solely on the laws of conservation of mass and energy in classical mechanics. After many simplifications, it is used to estimate the forces on local areas of wind turbine components. Due to these simplifications, BEM theory often needs to be revised or corrected using experimental data, such as tip loss and hub loss [14]. Therefore, many researchers have resorted to using software to solve the viscous flow field to calculate the relevant forces. However, these techniques all originate from computational fluid dynamics (CFD), and their foundations are based on numerical methods such as the finite difference method (FDM) [15], the finite volume method (FVM) [16], and the finite element method (FEM) [17].

Numerical methods can be divided into two main categories: (1) domain-type methods and (2) boundary-type methods. The so-called FDM, FVM, and FEM are all domain-type numerical methods based on grid computing. The force problem of fluid flowing through the relevant components of a wind power generation device can be regarded as an external flow problem. If a domain-type numerical method is used to solve an external flow problem, problems such as excessive grid generation and computational load are often encountered. Boundary-type numerical methods can be further subdivided into mesh-dependent and meshless methods according to spatial discretization. Common global meshless methods such as smoothed particle hydrodynamics (SPH) [18], radial basis function collocation method (RBFCM) [19], and generalized finite difference method (GFDM) [20, 21] have been developed, but when facing external flow field problems, they are still limited by spatial discretization and still need to consider the entire domain, so their computational workload is still huge. The well-known boundary element method is a boundary-type numerical method that requires a mesh for spatial discretization [22]. Although the boundary element method is a boundary-type numerical method, it still requires a mesh for spatial discretization, which causes many limitations in application, such as mesh generation and numerical integration.

The proposed method of fundamental solutions (MFS) is a boundary-type meshless method, which does not need mesh generation or related numerical integration. It only needs boundary points for calculation. Therefore, the MFS has a computational advantage compared with other regional meshless methods or boundary element methods and can simultaneously achieve efficiency and accuracy. The MFS was proposed by Kupradze and Aleksidze [23] in 1964. It only requires a set of boundary points and source points to perform computer simulation analysis. The MFS assumes that the numerical solution is a linear superposition of the fundamental solution and uses the boundary points to introduce boundary conditions into the linear system. After solving this



linear system, the physical quantities and their partial derivatives at all points in the computational domain can be calculated. The numerical analysis process of the MFS is simple, the computer program is easy to write, and the simulation results are very accurate. Therefore, it has been used in various mathematical and physical problems [24, 25, 26]. Although the MFS has advantages, the accuracy of its numerical results is directly affected by the source point locations. Therefore, many scholars have proposed different methods to determine the optimal source point locations [27], but each method has its limitations. Therefore, this paper proposes the particle swarm optimization algorithm (PSO) to determine the optimal source point locations of the MFS to obtain stable and accurate simulation results, avoid the trouble of manually determining parameters, and use the MFS and PSO algorithm to jointly develop multidimensional high-efficiency models for external flow field problems.

In recent years, artificial intelligence has begun to receive attention, and many supervised learning and unsupervised learning algorithms have been proposed and applied. The PSO algorithm proposed in this case is one of the latest unsupervised algorithms, which can efficiently find the optimal solution in an infinite solution space when there is not much effective information. The PSO algorithm was first proposed by Kennedy and Eberhart in 1995 [28]. It is a biomimetic optimization algorithm proposed by observing the group movement behavior of animals. It assumes that the swarm consists of particles with no volume or mass. During the movement of the particles, the inertia effect, the cognition effect, and the social effect must be considered at the same time, and the optimal position solution is found iteratively moving through the movement of the particle swarm [29]. PSO has been applied to various fields in recent years due to its simple theory and high computational efficiency. For example, PSO has also been applied to the optimal configuration of offshore wind farm planning [30] and solving financial mathematical problems [31]. PSO has also been modified into an adaptive algorithm to accelerate its iterative convergence and avoid falling into the dilemma of local optimal solutions [32]. In 2023, MFS and PSO were combined to solve multidimensional boundary value problems [33].

According to the above literature, the PSO algorithm is a highly promising optimization algorithm. Therefore, this paper adopts the PSO algorithm to obtain the optimal source point locations of the MFS. This allows the MFS to be used in the future to simulate engineering application problems for which analytical solutions cannot be obtained. This paper anticipates using the MFS and the PSO algorithm as the main spatial discretization methods, combined with potential flow theory, to develop a high-efficiency external-flow module to study the force distribution of wind turbine components under

different wind-field conditions. It is expected that the developed numerical model can be executed on a general small workstation, and the calculation results will be compared with the OpenFAST calculation based on BEM theory to verify the feasibility of the proposed method.

## II. Governing Equation and Boundary Conditions

This paper primarily focuses on developing numerical models using the MFS as the main spatial discretization method. These models solve for the physical quantities of the entire field and then calculate the forces acting on the target object. The following briefly explains the related mathematical descriptions and the MFS and PSO algorithms used in the simulation. The paper uses the potential flow equation as the governing equation for the flow field; the following equation is the governing equation for the potential flow:

$$\nabla^2 \phi(\vec{x}) = 0 \quad \text{and} \quad \vec{x} \in \Omega,$$

where  $\phi$  is the velocity potential,  $\vec{x}$  is the vector of the space,  $\nabla^2(\ )$  is the Laplace operator, and  $\Omega$  denoted the computational domain.

Although the potential flow equation is used as the governing equation for the field, appropriate boundary conditions still need to be considered to obtain a solution. The boundary conditions required in this case are listed below.

The first-kind boundary conditions:

$$\phi(\vec{x}) = C_D(\vec{x}), \quad \text{and} \quad \vec{x} \in \partial\Omega_D,$$

The second-kind boundary conditions:

$$\frac{\partial \phi(\vec{x})}{\partial \vec{n}} = C_N(\vec{x}), \quad \text{and} \quad \vec{x} \in \partial\Omega_N,$$

where  $C(\vec{x})$  is the function of the space,  $\partial\Omega$  denoted the boundary of the computational domain, the subscripts  $D$  and  $N$  indicate the first-kind boundary condition (Dirichlet BC) and the second-kind boundary condition (Neumann BC), respectively. And  $\vec{n}$  denoted the normal vector of the boundary. Fig. 3 shows the diagram of the computational domain and boundary.



Fig. 1. Important sustainable development goals (SDGs) (<https://sdgs.un.org/goals>)



Fig. 2. General explanation of Taiwan's 2050 net-zero emissions pathway and strategy [2].

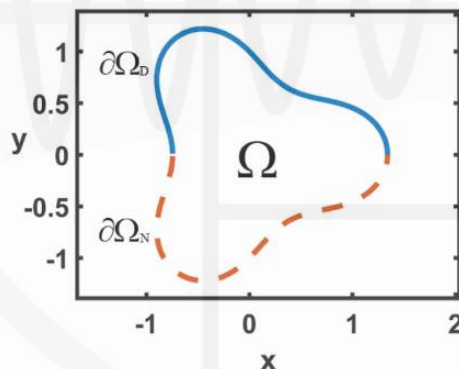


Fig. 3. Diagram of the computational domain and boundary [34].

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# Assessment of the Natural Frequency of a Monopile-Supported Offshore Wind Turbine System Considering Geological Uncertainty

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## Abstract

Monopile foundations are widely regarded as the most common substructure type, currently supporting most offshore wind turbines (OWTs) globally. The natural frequency ( $f_n$ ) is a critical design parameter that must be carefully assessed to prevent resonance with wind, wave, and rotor operational excitations. This study proposes a simplified method for deriving equivalent uniform shear wave velocity ( $V_s$ ) profiles to evaluate  $f_n$  in monopile-supported OWT systems, using Taiwan's Wind Farm No. 26 as a case study. Field data from alternating sand-clay stratigraphy demonstrate that shallow soil layers govern the system  $f_n$ . A simplified equivalent-depth approach yields  $f_n$  estimates within approximately 2% of detailed depth-dependent models, significantly improving computational efficiency in preliminary offshore wind farm design under conditions of limited geological information.

**Keywords:** monopile foundation; geological uncertainty; natural frequency; shear wave velocity

## Introduction

The rapid expansion of offshore wind energy has made offshore wind turbines a key part of the global renewable energy infrastructure. Monopile foundations support over 80% of installed offshore wind turbines (OWTs) because of their structural simplicity, construction efficiency, and cost-effectiveness (Arany et al., 2016; Sunday and Brennan, 2022). Taiwan, which has abundant offshore wind resources, has experienced substantial growth in offshore wind farm development in recent years.

The natural frequency ( $f_n$ ) is a critical design parameter that must be evaluated to prevent resonance with environmental excitations such as wind loading, wave forcing, and rotor operational frequencies (Lombardi et al., 2013; Jiang et al., 2023). Resonance can cause excessive structural vibrations, accelerate fatigue damage, and lead to structural failure. The natural frequency ( $f_n$ ) is determined by structural

properties and soil-structure interaction (SSI) at the foundation level (Álamo et al., 2018). Conventional methods require depth-dependent soil parameters, which demand substantial computational resources during preliminary design (Bouzid et al., 2018). Previous studies show that shallow soil layers largely control lateral pile stiffness (Lombardi et al., 2013), but the quantitative effects of shallow versus deep layers on the natural frequency of OWT systems remain insufficiently studied.

This study investigates the influence of monopile length and soil layer characteristics on OWT system  $f_n$  through numerical modal analysis using Taiwan's Wind Farm No. 26. The objectives are: (1) to quantify the relationship between monopile length and  $f_n$ ; and (2) to develop and validate a simplified soil parameter model to improve preliminary design efficiency.

In practice, OWT systems are typically designed to operate in the soft-stiff frequency range, which lies

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between the rotor frequency (1P) and the blade-passing frequency (3P) to avoid resonance with operational loading. Accurate prediction of  $f_n$  during the preliminary design phase is therefore critical, as it directly influences the selection of structural dimensions and foundation geometry. Errors in  $f_n$  estimation may necessitate costly design revisions or compromise long-term structural performance under cyclic loading.

## Site Description

Taiwan's Wind Farm No. 26 was selected as the case study, with average sea depths of 17–49 m. Five cone penetration tests (CPTs) were conducted during the preliminary investigation. The site consists of alternating sand and clay layers, each 1–5 m thick. Shear wave velocity ( $V_s$ ) was estimated from CPT data following Robertson (2009). Regression analysis produced an exponential relationship between shear wave velocity and depth (Fig. 1):

$$V_s = 51.89(z + 1)^{0.46} \quad (1)$$

where  $z$  is the depth below the seabed (m).

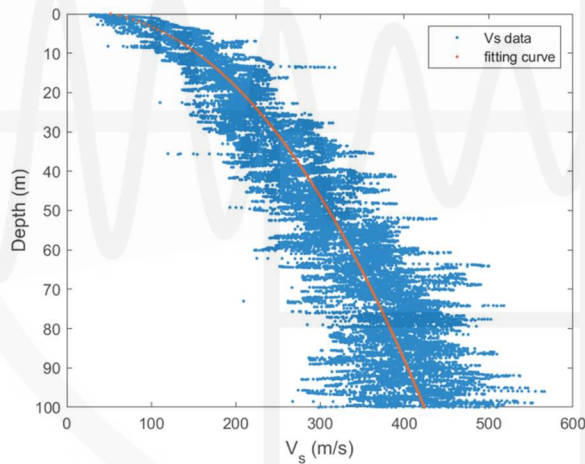


Fig. 1. Distribution of  $V_s$  at the study site and results of the regression analysis

## Development of Numerical Model

Modal analyses were performed using SAP2000 for a reference 4 MW OWT configuration. Fig. 2 presents a schematic diagram and numerical model for Case A1 analysis. The Rotor–Nacelle assembly (RNA) mass was set to 250 metric tons. The tower (90 m), transition piece (TP, 20 m), and monopile (MP, 60 m) were modeled as tapered steel tubular beam elements with 0.5 m nodal spacing (Table 1). The mean sea level and seabed surface coincide with the top and bottom of the TP, respectively. Soil–structure interaction was simulated via the Winkler spring approach; spring stiffness was evaluated using the site-specific  $V_s$

profile (Eq. 1) and the Japan Road Association (JRA, 2002) methodology, with link elements at 0.25 m spacing. The RNA was modeled as a lumped mass at the top of the tower.

Two reference boundary conditions were evaluated to bracket the system response: (i) the superstructure with a fixed base at the seabed, yielding  $f_n = 0.268$  Hz; and (ii) the full OWT system without lateral soil support, assuming a fixed pile tip, yielding  $f_n = 0.180$  Hz. These cases represent the upper and lower bounds of  $f_n$ , confirming that soil-supported conditions restore the natural frequency to approximately 96% of the rigid fixed-base value.

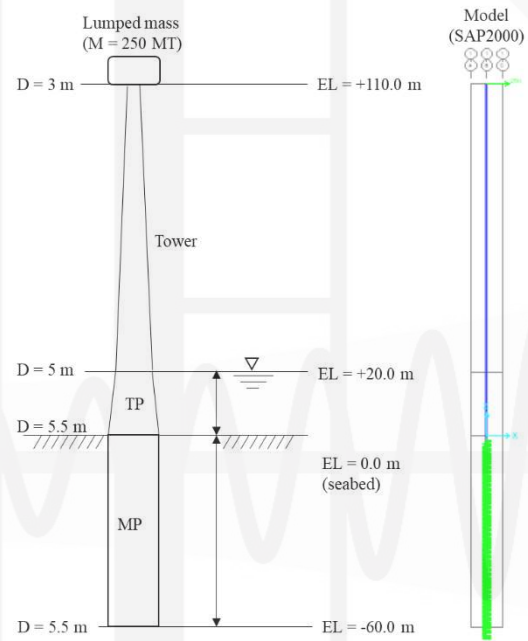


Fig. 2. Simplified schematic and numerical model for case A1.

Table 1. Structural cross-section parameters for Case A1

| Structure        | $D_{top}$ (m) | $D_{bottom}$ (m) | T (m) |
|------------------|---------------|------------------|-------|
| Tower            | 3.0           | 5.0              | 0.04  |
| Transition Piece | 5.0           | 5.5              | 0.09  |
| Monopile         | 5.5           | 5.5              | 0.09  |

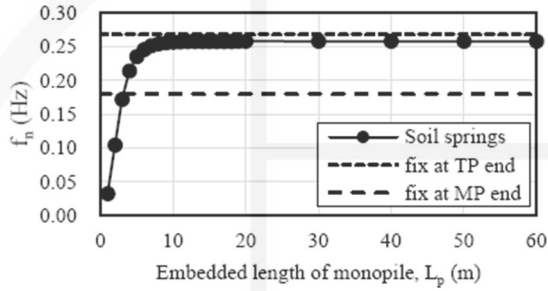
## Influence of Monopile Length on Natural Frequency

Modal analyses for Case A1 with varying embedded pile lengths reveal that  $f_n$  increases nonlinearly with pile length (Fig. 3). Initially,  $f_n$  rises linearly; the rate decelerates beyond  $L_p = 4$  m. When  $L_p$  exceeds 13 m,  $f_n$  converges to a stable plateau  $f_{n,end} = 0.258$  Hz. With soil support,  $f_n$  is approximately 96% of the fixed-base superstructure frequency (0.268 Hz);



without lateral soil support,  $f_n$  is only 67% (0.180 Hz).

The stabilization of  $f_n$  beyond a critical embedment depth can be explained by the concept of effective pile length, defined as the depth beyond which additional lateral soil resistance has little effect on foundation stiffness. When the embedded length exceeds this threshold, the pile behaves as an infinitely long structural element in its lateral response, and further embedment does not increase rotational or lateral restraint at the pile head. This mechanism accounts for the plateau in  $f_n$  observed in all analyzed cases.

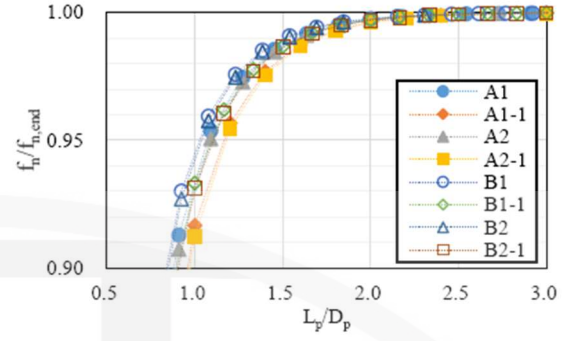


**Fig. 3.** Relationship between embedded monopile length and  $f_n$  of the OWT system for Case A1

Eight cases with varying turbine capacities and structural parameters were analyzed (Table 2). Normalizing pile length and natural frequency as  $L_p/D_p$  and  $f_n/f_{n,end}$ , respectively (Fig. 4), shows a consistent trend:  $f_{n,end}$  is attained when  $L_p > 3D_p$ . The critical pile length ( $f_n = 0.99f_{n,end}$ ) corresponds to  $L_p/D_p$  of 1.5–1.7. Typical monopile foundations ( $L_p/D_p = 3$ –6) reliably exceed this threshold, ensuring stable  $f_n$  values. For a specified turbine capacity, tower height ( $H$ ) exerts a more substantial influence on  $f_{n,end}$  than monopile diameter ( $D_p$ ), underscoring the importance of accurate shallow-soil characterization.

**Table 2.** Parameter settings and  $f_{n,end}$  analysis results for different cases

| Case | $W_{RNA}$<br>(ton) | $H$<br>(m) | $D_p$<br>(m) | $t_p$<br>(m) | $f_{n,end}$<br>(Hz) |
|------|--------------------|------------|--------------|--------------|---------------------|
| A1   | 250                | 110        | 5.5          | 0.09         | 0.258               |
| A1-1 | 250                | 110        | 5.0          | 0.09         | 0.237               |
| A2   | 250                | 120        | 5.5          | 0.09         | 0.228               |
| A2-1 | 250                | 120        | 5.0          | 0.09         | 0.219               |
| B1   | 450                | 120        | 6.5          | 0.08         | 0.213               |
| B1-1 | 450                | 120        | 6.0          | 0.09         | 0.207               |
| B2   | 450                | 130        | 6.5          | 0.09         | 0.199               |
| B2-1 | 450                | 130        | 6.0          | 0.09         | 0.192               |



**Fig. 4.**  $L_p/D_p$  versus normalized natural frequency  $f_n/f_{n,end}$  for all cases

### Simplified Method for Soil Parameters

Building on the finding that shallow soil layers dominate  $f_n$ , a simplified soil parameter method is proposed. An equivalent depth  $z_{eq}$  defines the active soil zone; the average  $V_s$  within  $z_{eq}$  is defined as  $V_{s,eq}$ :

$$V_{s,eq} = \int_0^{z_{eq}} V_s(z) dz / z_{eq} \quad (2)$$

Uniformly distributed springs computed from  $V_{s,eq}$  replace the depth-varying spring distribution, substantially reducing model complexity.

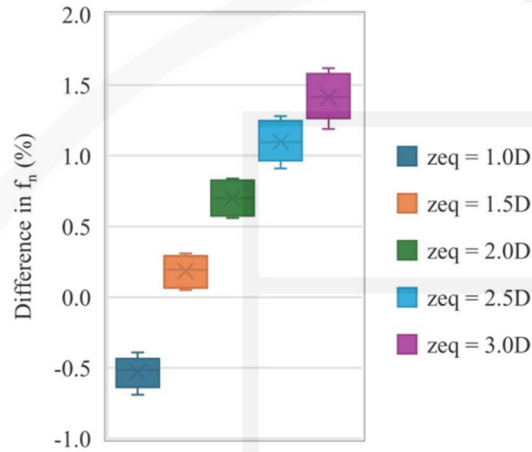
The physical rationale for the  $z_{eq}$  concept is that lateral soil springs at greater depths contribute progressively less to the rotational and lateral stiffness components that control  $f_n$ . Concentrating  $V_s$  averaging within the uppermost  $z_{eq}$  of the soil profile captures the dominant stiffness contribution while eliminating the computational overhead of a fully depth-varying spring configuration. This approach is particularly advantageous in early-stage project planning when detailed borehole data across the entire wind farm footprint remains sparse.

Five  $z_{eq}$  values ( $1.0D_p$  to  $3.0D_p$ ) were evaluated across all eight cases (Table 3). The difference in  $f_n$  between  $z_{eq} = 1.0D_p$  and  $z_{eq} = 3.0D_p$  is approximately 2%. Statistical comparison with the original depth-dependent model (Fig. 5) shows that  $z_{eq} = 1.5D_p$  provides the closest match. Increasing  $z_{eq}$  overestimates shallow soil stiffness, yielding slightly elevated  $f_n$  values. Accordingly,  $z_{eq} = 1.0D_p$  is recommended for preliminary design.

**Table 3.** Frequencies (Hz) at various equivalent depths for different cases

| Cas e | $z_{eq} = 1.0D_p$ | $z_{eq} = 1.5D_p$ | $z_{eq} = 2.0D_p$ | $z_{eq} = 2.5D_p$ | $z_{eq} = 3.0D_p$ |
|-------|-------------------|-------------------|-------------------|-------------------|-------------------|
| A1    | 0.244             | 0.245             | 0.246             | 0.247             | 0.248             |
| A1-1  | 0.235             | 0.237             | 0.238             | 0.239             | 0.240             |
| A2    | 0.227             | 0.229             | 0.230             | 0.231             | 0.231             |

|      |       |       |       |       |       |
|------|-------|-------|-------|-------|-------|
| A2-1 | 0.217 | 0.219 | 0.220 | 0.221 | 0.222 |
| B1   | 0.212 | 0.214 | 0.215 | 0.216 | 0.216 |
| B1-1 | 0.206 | 0.208 | 0.209 | 0.210 | 0.210 |
| B2   | 0.198 | 0.200 | 0.201 | 0.202 | 0.202 |
| B2-1 | 0.191 | 0.193 | 0.194 | 0.194 | 0.195 |



**Fig. 5.** Boxplot showing the differences in  $f_n$  between simplified and original models.

## Conclusions

This study examined the influence of monopile length and soil layer properties on the natural frequency of OWT systems. Three primary conclusions are:

1. The  $f_n$  of monopile-supported OWT systems increases nonlinearly with embedded pile length, reaching a stable plateau  $f_{n,end}$  when  $L_p > 3 D_p$ . The critical pile length ( $f_n = 0.99 f_{n,end}$ ) corresponds to  $L_p/D_p$  of 1.5–1.7. Typical monopile foundations ( $L_p/D_p = 3$ –6) reliably satisfy this criterion, ensuring stable  $f_n$  independent of deeper embedment.
2. A simplified method based on equivalent depth  $z_{eq} = 1.5D_p$  and equivalent shear wave velocity  $V_{s,eq}$  provides  $f_n$  estimates within approximately 2% of detailed depth-dependent models, significantly improving computational efficiency in preliminary design.
3. Shallow soil layers govern  $f_n$  of monopile-supported OWT systems, confirming the validity of the simplified approach and underscoring the importance of accurate shallow soil characterization in offshore wind farm planning.

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# Hydrodynamic Design Loads Specified in the Revised Seismic Guidelines of RC Water Tanks in Taiwan

Gee-Yu Liu<sup>1</sup>

劉季宇<sup>1</sup>

## Abstract

The Taiwan Water Corporation (TWC), the largest water utility in Taiwan, has long paid close attention to the structural design of water tanks. In response to recent revisions in relevant domestic and international design codes, TWC collaborated with the National Center for Research on Earthquake Engineering through a commissioned service project to revise its own *Seismic Design Guidelines for Water-Containing Reinforced Concrete Structures*. Major revisions related to hydrodynamic loads include: (1) adjustment of the response modification factor for the impulsive mode with respect to various tank configurations; (2) determination of modal periods for elevated water towers; (3) distribution of impulsive and convective pressures on the base slab; and (4) a new formula for computing maximum sloshing heights, together with the modified hydrodynamic loads specified in ACI 350.3-20 to address insufficient roof slab clearance.

**Keywords:** water-containing structures, reinforced concrete, seismic design guidelines

## Introduction

The Taiwan Water Corporation (TWC), the largest water utility in Taiwan, is responsible for providing a dependable, sustainable, and hazard-resilient water supply. For decades, it has addressed the improved seismic design practices of water pipelines and facilities. In the 1980s, TWC published “Water Tank Design (I): Rectangular Reinforced Concrete Water Tanks” and “Water Tank Design (II): Non-Prestressed Circular Reinforced Concrete Water Tanks.” For seismic design, TWC partnered with the National Center for Research on Earthquake Engineering (NCREE) through a commissioned service project to develop the first edition of the “Seismic Design Guidelines for Water-Containing Reinforced Concrete Structures” in 2018. This document was primarily based on the American Concrete Institute’s “ACI 350.3-06 Seismic Design of Liquid-Containing Concrete Structures and Commentary” (ACI, 2006), particularly its provisions on seismic loading from the accelerated water, and has since become a key reference for the seismic design and assessment of concrete water tank structures in Taiwan.

Although only a few years have passed since 2018, there have been significant developments regarding the relevant domestic and foreign design

codes during this period. For example, the new ACI 350.3-20 (ACI, 2020) has introduced substantial updates compared to its predecessor, ACI 350.3-06 (ACI, 2006). Furthermore, Taiwan published the revised “Building Seismic Design Code and Commentary” in 2022, and the “Concrete Engineering Design Code and Commentary” has officially become a national standard in 2023. These changes highlight that the 2018 guidelines are no longer adequate.

In response, TWC and the NCREE have once again collaborated through a new commissioned service project to revise the “Seismic Design Guidelines for Water-Containing Reinforced Concrete Structures.” The revision was released in late 2025.

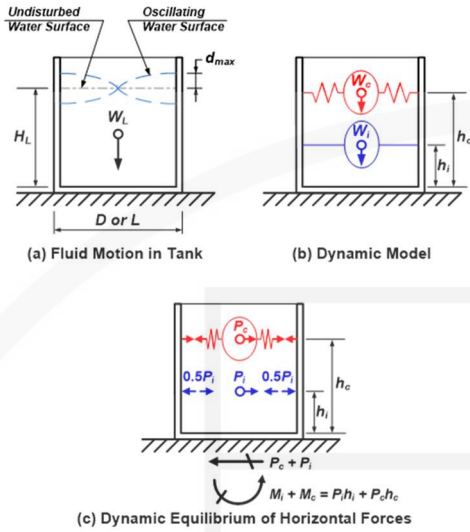
## Hydrodynamic Loading on Water Tanks During Earthquakes

The ACI 350.3 series of guidelines addresses the seismic design of water tanks by adopting G.W. Housner’s theory (Housner, 1963) to define the hydrodynamic loads induced by earthquakes. This theory categorizes fluid response into two modes: impulsive and convective. The impulsive mode arises from the portion of the liquid that moves rigidly with the tank wall, whereas the convective mode corresponds to the portion of the fluid that oscillates in

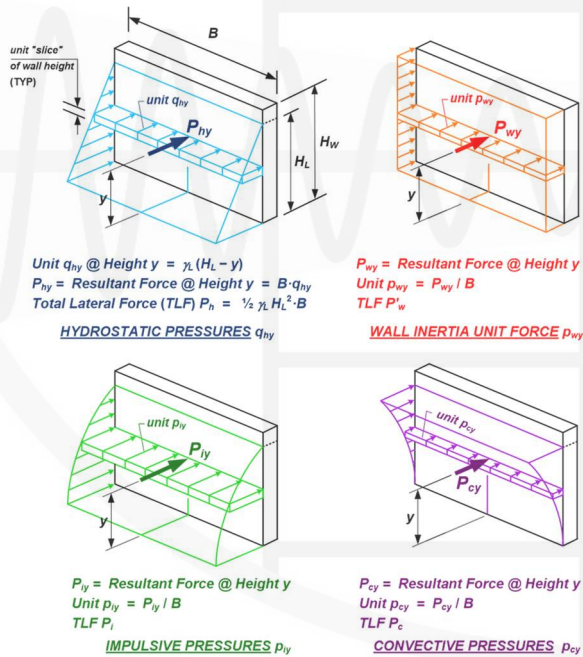
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response to seismic excitation. Both modes have corresponding analytical expressions for pressure distribution.



**Fig. 1.** Spring-mass model of a coupled water-tank system from ACI 350.3-06.



**Fig. 2.** Load distribution on the front wall of a rectangular tank, based on ACI 350.3-06.

Housner's theory can be represented by an equivalent spring-mass model, as shown in Fig. 1. The total weight of the liquid is divided into two components: the impulsive and convective components. The corresponding modal lateral forces act on the front and rear tank walls through rigid links and equivalent springs, respectively. The tank walls are thus classified as leading (facing the direction of seismic force) and trailing (opposite the direction of seismic force). The leading wall is subjected to the seismic force, whereas the trailing wall is under tension. For a rectangular tank, the load distribution on

the leading wall includes hydrostatic pressure, wall inertia, half of the impulsive force, and half of the convective force, as shown in Fig. 2.

## Modal Lateral Forces and Response Modification Factors

Analogous to the relationship among base shear, structural weight, and response spectrum coefficients in building seismic design, the impulsive and convective modal lateral forces in the seismic design of tank structures are calculated in accordance with ACI 350.3 provisions as follows:

$$P_i = C_i(T_i) \cdot I \cdot \frac{W_i}{R_i} \quad (1)$$

$$P_c = C_c(T_c) \cdot I \cdot \frac{W_c}{R_c} \quad (2)$$

where  $R_i$  and  $C_i$  are the response modification factor and period-dependent seismic response coefficient of the impulsive liquid, respectively,  $R_c$  and  $C_c$  are the corresponding values for the convective liquid.  $I$  is the importance factor of the tank. The coefficients  $C_i$  and  $C_c$  are the ordinates of the design response spectrum at periods  $T_i$  and  $T_c$ , respectively, where  $T_i$  is the fundamental period of oscillation of the tank (including the impulsive component of the contents), and  $T_c$  is the natural period of the first convective mode of sloshing.

These response modification factors play a role like the ductility reduction factor in building design, allowing for reasonable reduction of seismic demand based on the ductility, energy dissipation, and redundancy of the structure.

Compared to the previous edition, ACI 350.3-20 (ACI, 2020) slightly increases the value  $R_i$  across various tank configurations, due to more supporting empirical data. Accordingly, the new guidelines (TWC, 2025) adopt the updated  $R_i$  values from ACI 350.3-20, which slightly reduce the impulsive modal forces, enabling more economical design outcomes. The value of  $R_c$ , however, remains at 1.00. The updated values of  $R_i$  and  $R_c$  for different structural configurations are listed in Table 1.

**Table 1.** Response modification factors for tank-type structures in the new guidelines.

|       | Structural Type | ACI 350.3-20 |
|-------|-----------------|--------------|
| $R_i$ | On Grade        | 2.25         |
|       | Buried          | 3.25         |
| $R_c$ |                 | 1.00         |

Additionally, consider the tank configuration shown in Fig. 3, where the left side is buried and the right side is above ground on uneven terrain. This type

of structure is not directly addressed in the ACI 350.3 provisions. From a conservative standpoint, the entire structure could be treated as an above-ground tank, applying a lower  $R_i$  and thereby resulting in higher seismic forces. However, this may lead to an overly conservative design for the buried side of a large and tall tank.

To address this issue, the new guidelines (TWC, 2025) require that the seismic analysis apply lateral forces according to the actual condition of each wall: treat the left wall as buried and the right wall as above-ground. This approach provides a more rational and accurate basis for seismic design.



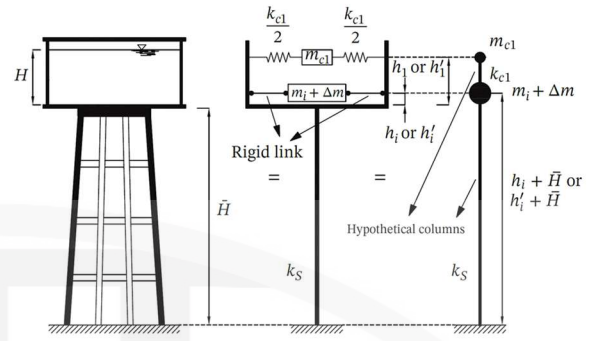
**Fig. 3.** Side view of a tank positioned on uneven terrain.

Lastly, for tank structures containing internal partition walls or baffle walls, the new guidelines (TWC, 2025) uniformly assign  $R_i = 2.25$ , treating these walls as equivalent to those of on-grade tanks. This is because such internal walls lack out-of-plane support, and their boundary conditions are identical to those of on-grade tanks.

### Modal Periods of Elevated Water Towers

The new guidelines (TWC, 2025) introduce specific requirements for the seismic design of elevated water towers, which is an area not previously addressed in either the original Taiwanese guidelines or the ACI 350.3-20 codes. However, elevated water towers are commonly required in water infrastructure projects.

The determination of modal periods for elevated water towers is based on the method proposed by Calvi and Nascimbene (2023), in which the system is modeled as a coupled two-degree-of-freedom (2DOF) system, as illustrated in Fig. 4. In this model, the structure is conceptually divided into the upper tank and the lower supporting frame. According to Housner's hydrodynamic pressure theory, the water mass can be further divided into an impulsive equivalent mass rigidly attached to the tank wall and a convective equivalent mass connected through an equivalent spring. These values can be derived from the convective mode period and mass. The equivalent stiffness of the elevated tower can be determined using mechanics of materials (if the supporting frame is simple) or by finite element analysis (if the frame is complex).



**Fig. 4.** Dynamic model of an elevated water tower.

The two natural circular frequencies for the coupled system can be analytically derived as:

$$\omega_n^2 = \frac{1}{2} \left[ \frac{k_s + k_{cl}}{m_i + \Delta m} + \frac{k_{cl}}{m_{cl}} \right] \pm \left( \frac{k_s + k_{cl}}{m_i + \Delta m} - \frac{k_{cl}}{m_{cl}} \right)^2 + 4 \frac{k_{cl}^2}{(m_i + \Delta m)m_{cl}} \quad (3)$$

Here, the "+" sign indicates the impulsive mode frequency, while the "-" sign indicates the convective mode frequency. Once the modal periods are determined, the corresponding spectral coefficients and lateral forces acting on the elevated water tower can be calculated.

### Hydrodynamic Pressure on the Base Slab

Both impulsive and convective modes generate dynamic hydrodynamic pressures on the floor slab of a tank structure. ACI 350.3-20 (ACI, 2020) requires that these base pressures be considered. According to Housner's theory, the overturning moment on the tank base equals the lateral force times the vertical distance between the centers of gravity of the lateral force distributions for the cases with and without base pressure.

As shown in Fig. 5, the base area near the leading wall experiences positive dynamic pressure, while the area near the trailing wall experiences negative pressure. Assuming a linear pressure distribution with respect to the tank's centerline, the dynamic pressures on the base at location  $x$  caused by impulsive and convective modes can be calculated as follows:

$$p_{bi}(x) = P_i \cdot (h_i' - h_i) \cdot x / I_b \quad (4)$$

$$p_{bc}(x) = P_c \cdot (h_c' - h_c) \cdot x / I_b \quad (5)$$

where

$$I_b = BL^3 / 12 \quad (\text{rectangular})$$

$$I_b = \pi D^4 / 64 \quad (\text{circular}).$$

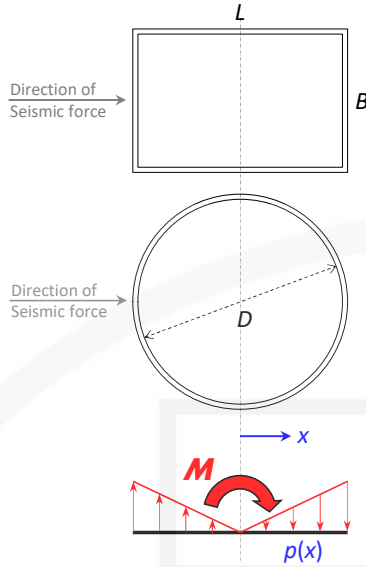


Fig. 5. Dynamic fluid pressure on the tank floor slab.

### Sloshing Height and Freeboard

The vertical distance from the tank's roof slab to the liquid surface must be greater than the maximum sloshing height  $d_{\max}$  to prevent the roof from being struck by wave action. The new guidelines (TWC, 2025) standardize the calculation using Housner's formula (Housner, 1963) as follows:

$$d_{\max} = \frac{0.264L}{\left(\frac{2g}{\omega_c^2 C_c L} - 1\right) \tanh\left(\frac{3.16H_L}{L}\right)} \quad (\text{rectangular}) \quad (6)$$

$$d_{\max} = \frac{0.204D}{\left(\frac{2g}{\omega_c^2 C_c D} - 1\right) \tanh\left(\frac{3.68H_L}{D}\right)} \quad (\text{circular}) \quad (7)$$

where  $\omega_c$  denotes the angular frequency of water's convective mode.

For rectangular tanks, Equation (7) calculates the maximum sloshing height in a single direction. In practice, the combined sloshing height in both perpendicular directions should be determined using the square-root-sum-of-the-squares (SRSS) method:

$$d_{\max} = \sqrt{d_{\max,X}^2 + d_{\max,Y}^2} \quad (8)$$

### Concluding Remarks

This article presents a summary of the revisions related to hydrodynamic loads in the new Seismic Design Guidelines for Water-Containing Reinforced Concrete Structures in Taiwan (TWC, 2025). Major revisions include: (1) adjustment of the response modification factor (ductility reduction factor) for the impulsive mode with respect to various tank

configurations; (2) determination of modal periods for elevated water towers; (3) distribution of impulsive and convective pressures on the base slab; and (4) a new formula for computing maximum sloshing heights, as well as the modified hydrodynamic loads specified in ACI 350.3-20 to address insufficient roof slab clearance.

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# Generic Nonlinear Amplification Functions for Taiwan

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王允佑<sup>1</sup>、張毓文<sup>2</sup>、許尚逸<sup>3</sup>

## Abstract

This study employs generic velocity models developed for various soft-soil sites and incorporates soil dynamic property curves to perform equivalent-linear site response analyses. The objective is to investigate nonlinear site amplification effects in Taiwan under different ground-motion scenarios and to establish a correction model for seismic hazard assessments. These generic profiles combine shallow velocity measurements and geophysical inversions, extending to geological bedrock to represent how deep structures influence surface vibrations. The proposed nonlinear model effectively captures regional characteristics in Taiwan's ground motion evaluations, offering a more comprehensive understanding of nonlinear soil behavior during major earthquake events.

**Keywords:** strong ground motion evaluation model, generic velocity profile, site response analysis, soil nonlinearity.

## Introduction

Ground motion models (GMMs), which depend on magnitude, distance, and site effects, are commonly used to evaluate seismic ground motions. For the site term, the time-averaged shear-wave velocity in the upper 30 m ( $V_{S30}$ ) is typically used as a proxy in amplification models. However,  $V_{S30}$  alone is insufficient to accurately represent site amplification functions because deeper velocity structures also significantly affect site response. If only  $V_{S30}$  is considered, GMMs may yield identical ground motion estimates for sites with similar shallow velocities but markedly different deep velocity profiles, failing to capture variations in amplification throughout the entire velocity structure. To improve accuracy, site terms in GMMs should incorporate the effects of the full velocity profile.

In earthquake engineering practice, the site amplification factor is typically defined as the sum of the intensity-independent linear amplification ( $F_{lin}$ ) and the nonlinear site response ( $F_{nl}$ ). Linear amplification can be estimated using empirical data, whereas nonlinear effects require site response analyses (SRA). This study employs generic velocity profiles for soft sites in Taiwan (Wang et al., 2024) to perform equivalent-linear site response analyses

(EQL-SRA).

By evaluating linear and nonlinear amplification across various ground motion scenarios, the aim is to develop a generic nonlinear amplification model for Taiwan, offering a reliable reference for nonlinear corrections in engineering applications.

## Taiwan Generic Velocity Profiles

Comprehensive velocity profiles extending to the geological bedrock for soft sites in Taiwan were developed by Wang et al. (2024), as shown in Fig. 1. Following the framework proposed by Kuo and Abrahamson (2023), the full profile is divided into four layers: very shallow, shallow, medium, and deep. The data for the very shallow range were collected from PS-logging measurements in the Engineering Geological Database for TSMIP (EGDT) (<https://egdt.ncree.org.tw/>). For the shallow range, velocity profiles from the Central Weather Administration (CWA) vertical array and microtremor array results were utilized, providing information on velocity characteristics within hundreds of meters to several kilometers. The medium range includes regional 1-D velocity profiles obtained from surface wave inversion or ambient noise inversion techniques.

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For the deep range, tomography models were employed. Based on the selected TSMIP station locations, 1-D deep velocity profiles were extracted to reveal the deeper geological structure.

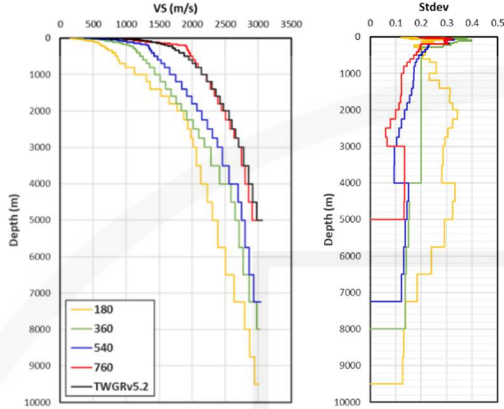


Fig. 1. Taiwan generic velocity profiles

## Site Response Analysis

EQL-SRA was used to estimate the effective dynamic behavior of soil under effective shear strain and simulate nonlinear soil responses during earthquakes. Fig. 2 illustrates the schematic of the soil layers and input motions used in the equivalent-linear analysis. The input ground motions were applied at the depth of the geological bedrock ( $V_S = 3$  km/s), assuming a point-source rupture. A total of 11 sets of input intensities were considered, with magnitudes ranging from  $M_w 4.0$  to  $M_w 9.0$ ; the specific input depths are summarized in Table 1.

Table 1. Depths of input motions for various generic velocity profiles at soft-soil sites

| $V_{S30}$ (m/s)        | 180 | 360 | 520  | 760 |
|------------------------|-----|-----|------|-----|
| $Z_{1.0}$ (m)          | 800 | 170 | 90   | 40  |
| Depth(km)<br>at 3 km/s | 9.5 | 8.0 | 7.25 | 5   |

## Analysis Settings and Modeling

Because the maximum soil depth reaches 9.5 km, measures were taken to prevent overestimation of damping effects during the iterative process. Soil layers below the bedrock depth ( $Z_{1.0}$ ) are treated as rock and assumed to exhibit linear behavior with a constant damping ratio of 0.4%, while soil layers above  $Z_{1.0}$  are modeled with nonlinear characteristics. The soil dynamic property curves for the generic site models were established using the Electric Power Research Institute (EPRI, 1993) model, with adjustments based on Hardin and Drnevich (1972) for large-strain conditions, as shown in Fig. 3. This modification was necessary because the EPRI

model shows a linearly increasing damping ratio at large strains, which diverges from empirical soil test results in Taiwan. Additionally, the unit weight of each soil layer was estimated using the empirical relationships proposed by Brocher (2005). Furthermore, to account for uncertainties, 200 sets of randomized profiles were generated based on the model's standard deviation.

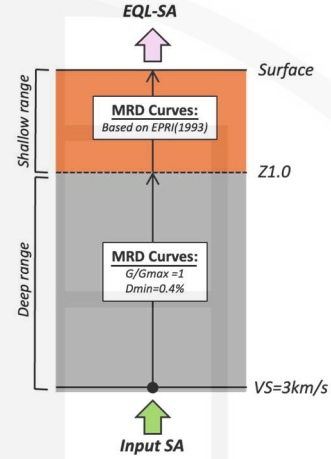


Fig. 2. Schematic of the site response analysis in this study

## Amplification Factor

The primary objective was to investigate the variations in soil properties (shear modulus and damping ratio) across alluvial regions in Taiwan under various generic conditions and seismic intensities, quantifying the impact of nonlinear site effects. This study performs EQL-SRA, obtaining surface outcrop response spectra under various ground motion intensities. Using the engineering bedrock outcrop ( $V_{S30} = 760$  m/s) as a reference, the amplification factor for individual velocity profiles is defined as the ratio of the surface response spectrum to the engineering bedrock response spectrum, as expressed in Eq. (1):

$$ampfactor = F_{lin} + F_{nl} = \frac{SA_{Pi,EQL}}{SA_{P760,EQL}} \quad \text{Eq. (1)}$$

where  $i$  represents different soil velocity profiles. The linear amplification function ( $F_{lin}$ ) is defined as the amplification factor obtained under weak ground motions ( $M_w 4.0$ ), as shown in Eq. (2):

$$linear\ term = \ln\left(\frac{SA_{Pi,EQL,M4.0}}{SA_{P760,EQL,M4.0}}\right) \quad \text{Eq. (2)}$$

Consequently, the nonlinear amplification effect caused by seismic shaking for various profiles can be determined using Eq. (3):

$$nonlinear\ term = \ln\left(\frac{ampfactor}{linear\ term}\right) \quad \text{Eq. (3)}$$

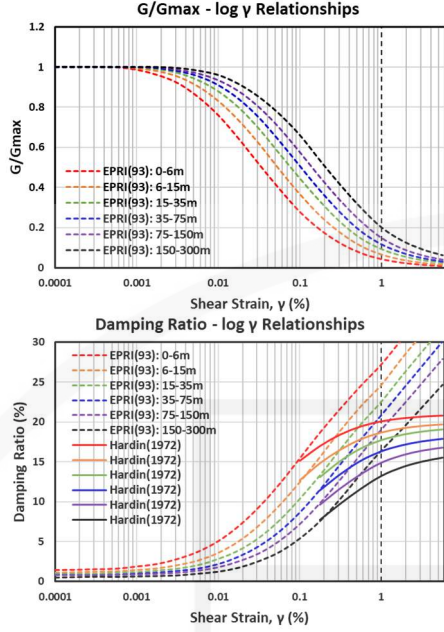


Fig. 3. Soil dynamic curves used in this study

Fig. 4 illustrates the variation of linear and nonlinear amplification effects with period for a generic profile with  $V_{S30} = 360$  m/s (P360). The results indicate that during weak earthquakes, the amplification factor remains at 1. This indicates that the  $M_w 4.0$  results represent linear amplification. As magnitude increases, de-amplification occurs, suggesting that nonlinear effects in Taiwan soils are mainly observed within the 0.05–0.5 s period range. Furthermore, the dominant period correlates with seismic intensity; stronger ground motions shift the soil frequency toward longer periods, resulting in amplification characteristics around 1.0 s. Fig. 5 displays the median nonlinear amplification distribution for 200 randomized profiles across four generic sets at periods of 0.3 s and 1.0 s. The results show that for shorter periods, nonlinear behavior becomes more pronounced in softer sites as seismic intensity increases; conversely, firmer site conditions exhibit more linear amplification even under large ground shaking.

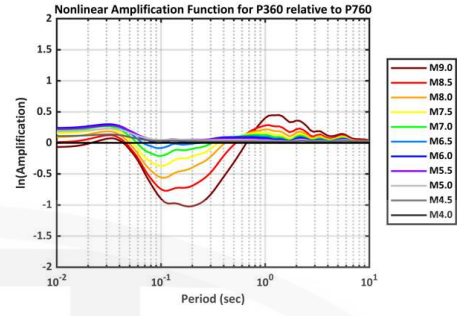
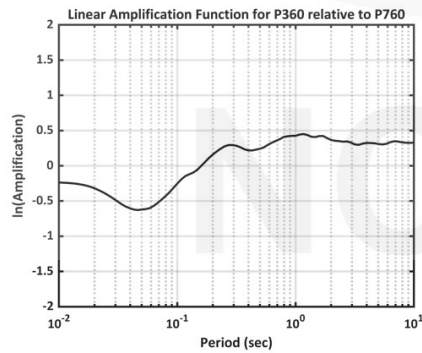


Fig. 4. Spectral ratio of linear (top) and nonlinear (bottom) amplification effects for a generic soil profile with  $V_{S30} = 360$  m/s (P360)

### Generic Nonlinear Amplification Function Model

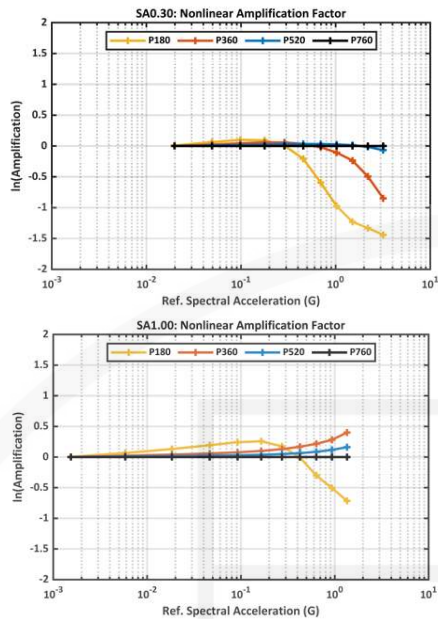
Nonlinear amplification factors for various periods were obtained across the four sets of generic soft soil profiles. These results facilitate the construction of a nonlinear amplification model correlated with reference rock ground motion levels and  $V_{S30}$ . Regression analyses were conducted for different periods to model the nonlinear effects. Using the distributions from all randomized models as the dataset, the regression followed the site-term function form based on Chao et al. (2019), as expressed in Eq. (4):

$$\ln(amp_{nlin}) = C_1 \left( -1.5 \ln \left( \frac{V_{S30}}{760} \right) + \ln \left( \frac{SA_{ref} + 2.4 \left( \frac{V_{S30}}{760} \right)^{1.5}}{SA_{ref} + 2.4} \right) \right) + C_2 \quad \text{Eq. (4)}$$

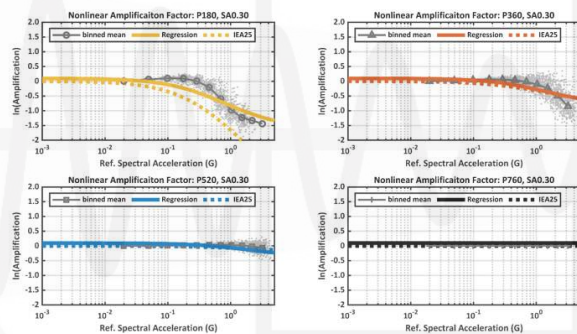
where  $C_1$  and  $C_2$  are the regression coefficients. Fig. 6 shows the nonlinear amplification factors and the corresponding regression results as a function of reference rock ground motions for various site conditions. Gray dots indicate nonlinear amplification factor data derived from SRA, while solid-colored lines represent the regression results from this study. Dashed colored lines indicate the nonlinear amplification model developed by Ilhan et al. (2025) for the western United States (WUS). The comparative results reveal that for soft soil layers, nonlinear de-amplification behavior differs between the WUS and Taiwan, especially at short periods where de-amplification is strongly affected by local soil characteristics.

Despite these regional differences, the overall amplification trends remain similar. A comparison of the WUS and Taiwan indicates that even under identical site conditions ( $V_{S30}$ ), the influence of deep velocity structures on soil amplification is significant and cannot be neglected.





**Fig. 5.** Nonlinear amplification factors for various soft sites as a function of reference rock spectral acceleration at periods of 0.3 s (top) and 1.0 s (bottom)



**Fig. 6.** Regression results of nonlinear amplification factors for various soft sites as a function of reference rock spectral acceleration ( $T = 0.3$  s).

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Wang, Y. Y., Chang, Y. W. and Hsu S. Y. (2024) Generic Amplification Functions for Taiwan. 18th World Conference on Earthquake Engineering, Milan, Italy.

# Parameters for Seismic Hazard Assessment Statistical Analysis of Geometry of the Chihshang Fault in Taiwan

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## Abstract

Defining fault geometry is a core task in probabilistic seismic hazard analysis. Fault dip is a key geometric parameter that significantly affects site-specific hazard estimates. Limited geological and geophysical data lead different teams to adopt varying uncertainty ranges, increasing hazard uncertainty for particular sites. In this study, we present the following two methods: statistical analysis of dip and a two-dimensional (2D) geometry stacking profile to address dip angle uncertainty and compare their results. The Chihshang Fault in eastern Taiwan is used as a case study.

We collected the fault dip results from geological and geophysical investigations to map the various dips along the Chihshang Fault. Normality tests (Quantile–Quantile plots and skewness) confirm that the sample dip data follow a normal distribution. Subsequently, we estimate that the mean, range, and corresponding weights for the Chihshang Fault are  $54^\circ \pm 19^\circ$  (by D10–D90) with weights of 0.3 / 0.4 / 0.3, or  $54^\circ \pm 24^\circ$  (by D5–D95) with weights of 0.2 / 0.6 / 0.2.

The 2D geometry stacking profile indicates a most common dip of  $60^\circ$ , with a range of  $\pm 15^\circ$ . In contrast, the unfiltered statistical results show a lower median ( $54^\circ$ ) and a wider range ( $\pm 19^\circ$  to  $\pm 24^\circ$ ), influenced by very low angles from subsurface borehole and trench data. After filtering, the statistical median increases to  $60^\circ$ , and the uncertainty range narrows to  $\pm 13^\circ$  (D10–D90) and  $\pm 16^\circ$  (D5–D95), differing by only  $1^\circ$ – $2^\circ$  from the 2D geometry stacking result. By combining both 2D geometry stacking and statistical analysis, the dip angles are defined as  $45^\circ$  (lower bound),  $60^\circ$  (median), and  $75^\circ$  (upper bound), with assigned weights of 0.2, 0.6, and 0.2, respectively.

**Keywords:** PSHA, dip, Chihshang fault, weights

## Introduction

When compiling Probabilistic Seismic Hazard Analysis (PSHA) models developed by different teams for specific faults, significant variation was observed in the median values, ranges, and assigned weights of fault geometry models. For example, studies of the Chihshang Fault system report dip angles of  $75^\circ$  (Shyu et al., 2020),  $70^\circ/75^\circ/80^\circ$  (NCREE, 2019),  $55^\circ/60^\circ/65^\circ$  (Hu et al., 2016), and  $60^\circ$  (Yen et al., 2020). These differences suggest that the median values and uncertainty ranges of fault geometry models depend heavily on the subjective judgment of each assessment team during the evaluation process.

This study gathered fault dip angles from geological and geophysical surveys along the Chihshang fault system, creating a fault dip database through tabulation and visualization. After verifying that the data follow a normal distribution, statistical analysis was applied to determine the median dip angle, uncertainty range, and weights derived from the distribution. Using a two-dimensional (2D) fault geometry model, the study assesses both inherent and knowledge-based uncertainties. The 2D profiles evaluate the fault's geometry from shallow to deep levels and compare these results with statistical dip data to address differences and quantify uncertainty.

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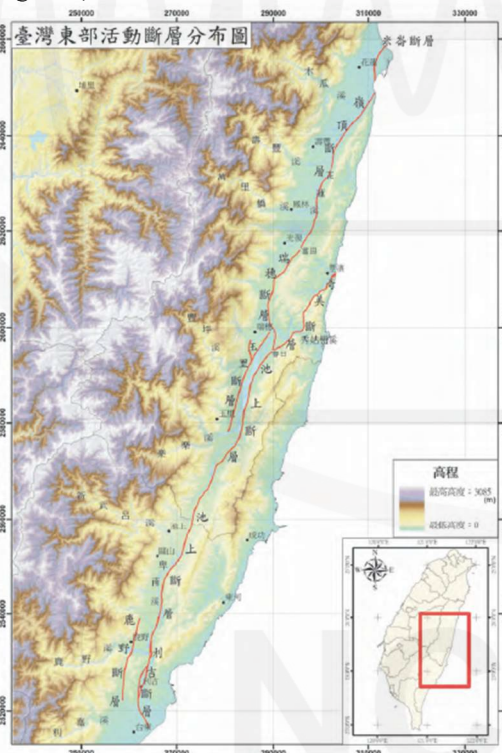
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## Chihshang Fault

The Chihshang Fault is a thrust fault with a left-lateral strike-slip component. It follows a north-northeast strike, starting from Chunri in Hualien County in the north and extending south to Ruilong in Luye Township, with a total length of approximately 67 km (Lin et al., 2009) (Fig. 1). Results from surface field surveys, geological drilling, trench excavation, and geophysical exploration (Chen et al., 2006; Dong et al., 2005; Shih et al., 2001) indicate that the dip angle of the Chihshang Fault is approximately  $45\text{--}75^\circ$ . However, at the Jinyuan (3) and Wan-an (1) trenches, the Chihshang Fault exhibits gentler dip angles of about  $15\text{--}33^\circ$ .

Historically, three earthquakes are known to have caused surface ruptures along the Chihshang Fault: the Taitung earthquake on November 25, 1951; the Chengkung earthquake on December 10, 2003; and the Chihshang earthquakes on September 17 and 18, 2022. Trench excavation surveys in the Wan-an area revealed that the Chihshang Fault has experienced 5 to 10 paleoearthquake events over the past 800 years, with a recurrence interval of 50 to 160 years (Chu, 2007). It is clearly an active fault (as of October 2025, the Geological Survey and Mining Management Center no longer classifies active faults into categories).



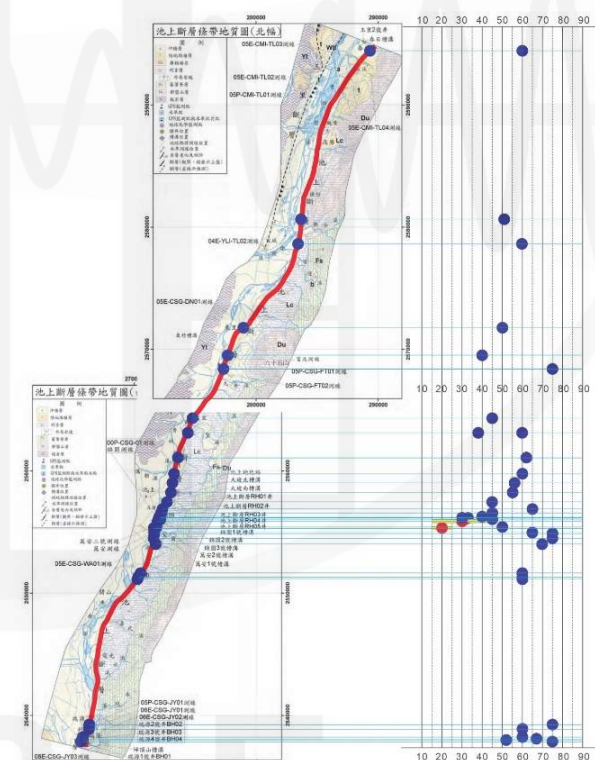
**Fig. 1.** Active fault map of Eastern Taiwan. The Chihshang fault is located in the middle of this active fault map (Lin et al., 2009).

## Data Integration

Geological data (field surveys, drilling, and trenching) and geophysical data (seismic reflection and electrical resistivity surveys) were collected along the fault. These survey results were consolidated into a fault dip parameter table (due to space constraints, please refer to the 115th year full report for the table). Furthermore, the survey points and their corresponding dip results were marked on a map (Fig. 2). This not only allows for the observation of dip variations along the fault (fault segmentation) but also facilitates subsequent statistical analysis.

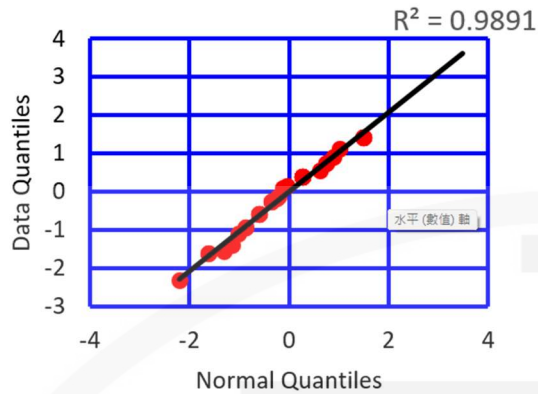
## Normality Tests

A Quantile–Quantile Plot (Q–Q Plot) is used to verify whether the statistical data follow a normal distribution. The Q–Q Plot is an XY scatter plot diagnostic where the Y-axis represents the data quantiles of the sample, and the X-axis represents the theoretical normal quantiles. If the sample data conforms to (or approximates) a theoretical normal distribution, the plotted points will fall along the  $X=Y$  straight line (Fig. 3).



**Fig. 2** Locations of study sites and dip angles along the Chihshang fault system.

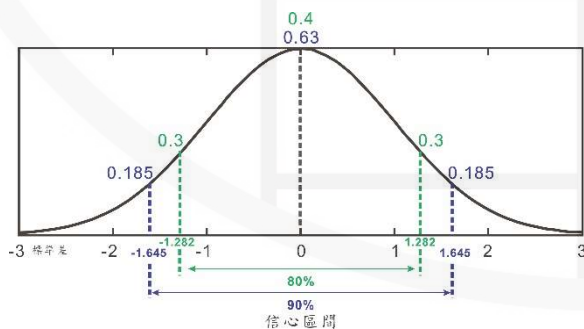




**Fig. 3** Quantile-Quantile plot of the dip angle of the Chihshang fault.

### Uncertainty Range Selection

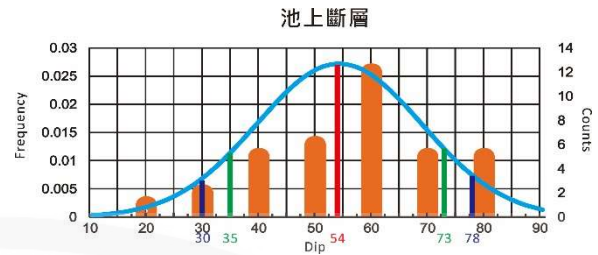
Based on previous research (Hammond and Bickel, 2013; Megill, 1984; Pearson and Tukey, 1965), the normal distribution curve of fault dip data utilizes two methods for selecting confidence ranges and corresponding weights: (a) the ESM method (Extended Swanson-Megill method), which uses the 10th, 50th, and 90th percentiles as the upper, median, and lower boundaries which are equivalent to an 80% confidence interval ( $\pm 1.282$  standard deviations); with corresponding weights of 0.3/0.4/0.3; and (b) the EPT method (Extended Pearson-Tukey method), which uses the 5th, 50th, and 95th percentiles as the upper, median, and lower boundaries that are equivalent to a 90% confidence interval ( $\pm 1.645$  standard deviations) with corresponding weights of 0.185/0.63/0.185 (commonly rounded to 0.2/0.6/0.2) (Fig. 4).



**Fig. 4.** The selected confidence intervals and their corresponding standard deviations and weights of ESM (green) and EPT (navy) method.

### Statistics Results

Based on the statistical methods described above, the pre-screening upper, median, and lower bounds for the Chihshang Fault dip angle are 35°, 54°, and 73°, with corresponding weights of 0.3, 0.4, and 0.3 (ESM method), or 30°, 54°, and 78°, with weights of 0.2, 0.6, and 0.2 (EPT method).

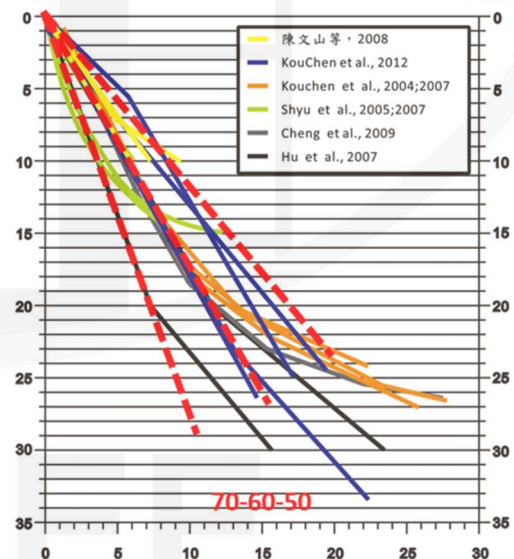


**Fig. 5.** Statistical analysis of the dip angle of the Chihshang Fault.

### 2D Fault Geometry and Discussion

Referring to the geological and geophysical results of previous studies, the 2D geometric profile of the Chihshang Fault shows a median dip angle of 60° with a range of  $\pm 15^\circ$  (45°–75°) (Fig. 6). Compared to the pre-screening statistical results, we believe the discrepancy is due to the influence of shallow dip angles (15°–30°) observed in near-surface drilling and trenching. Consequently, the pre-screening median is lower (54°) and the range is broader ( $\pm 19^\circ$  to  $\pm 24^\circ$ ). By contrast, the screened data matches the 2D geometric profile, with the same median (60°) and similar boundary ranges ( $\pm 13^\circ$  to  $\pm 16^\circ$ ) (Fig. 7), differing by only 1°–2°.

By integrating the results of the 2D geometric profile and the EPT statistical method, the estimated shallow dip angle of the Chihshang Fault is 60°, with lower and upper bounds of 45° and 75°, respectively, weighted at 0.2, 0.6, and 0.2.



**Fig. 6.** Summary of interpreted Chihshang fault structures.

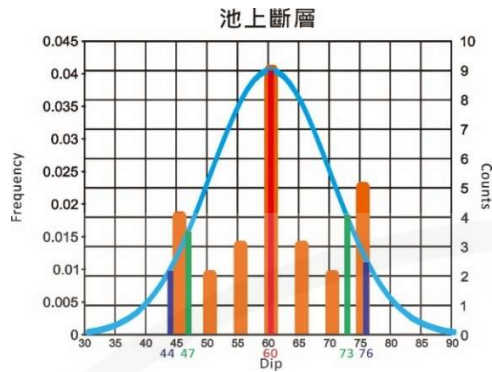


Fig.7. Statistical results of filtered dip angle of the Chihshang fault.

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# Linear and Nonlinear Correlation of Meteorological Parameters with Radon Concentration Variations

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瓦里亞<sup>1</sup>、林世榮<sup>2</sup>

## Abstract

Radon in soil gas has been widely investigated as a potential precursor for earthquake prediction. However, radon concentration is strongly influenced by meteorological parameters such as temperature, humidity, atmospheric pressure, and rainfall, making it difficult to isolate tectonic signals. This study presents a nonlinear, event-based analytical framework for evaluating radon variability and its relationship with environmental factors. Hourly radon data from the Jiaosi monitoring station in Taiwan, covering January 2021 to April 2022, were analyzed alongside meteorological data. Conventional linear correlation methods were compared with nonlinear approaches using event classification and locally weighted scatterplot smoothing (LOWESS). The results indicate that while temperature, humidity, and pressure dominate in linear correlations, rainfall emerges as a significant nonlinear driver of radon variability. A U-shaped relationship between rainfall and radon was identified, emphasizing the importance of transient environmental processes. The proposed methodology enhances the detection and interpretation of radon anomalies, contributing to more reliable earthquake precursor studies.

**Keywords:** Radon monitoring, meteorological data, linear and nonlinear correlation, Taiwan

## Introduction

The tectonic boundary between the Eurasian Plate and the Philippine Sea Plate represents one of the most seismically active regions in the world, characterized by high strain rates and complex fault geometries (Fu et al. 2019). In such environments, the development of robust earthquake precursor monitoring systems is essential for hazard mitigation and the advancement of short-term forecasting models (Walia et al. 2018; Chowdhury et al. 2025). Soil-gas radon (<sup>222</sup>Rn) has been extensively studied as a geochemical indicator because of its short half-life and high sensitivity to the mechanical deformation of the crust (King 1986). Variations in radon concentration are typically associated with micro-fracturing, alterations in rock permeability, and enhanced gas migration processes triggered by pre-seismic stress accumulation (Toutain and Baubron 1999). However, the practical application of radon as a reliable precursor is complicated by the fact that radon signals are significantly influenced by meteorological parameters, including temperature,

humidity, atmospheric pressure, and rainfall (Friedmann et al. 2011). The primary obstacle in geochemical forecasting remains the “contamination” of radon time-series data by these meteorological factors, which often mask the subtle signals associated with tectonic stress. Atmospheric changes modulate the transport and exhalation of radon from the lithosphere through mechanisms such as the “pumping effect” of barometric pressure or the thermal convection of soil air. This creates a highly nonstationary signal in which the input is modulated in both amplitude and waveform frequency, leading to potential false positives in anomaly detection. To isolate true precursory anomalies, assessing and quantifying these environmental influences with high technical precision is a major prerequisite. Conventional approaches rely primarily on linear correlation analysis or simple regression to evaluate relationships between radon and environmental variables. However, these methods are fundamentally limited in capturing the nonlinear, lagged, and transient behaviors that characterize the interaction between the lithosphere and the atmosphere (Walia et

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al. 2018). Consequently, advanced analytical techniques are required to effectively decouple meteorological noise from genuine tectonic signatures. This study introduces a nonlinear event-based framework combined with locally weighted scatterplot smoothing (LOWESS) modeling (Cleveland 1979). By using LOWESS, we performed nonparametric regression that adapts to local variations in the data, allowing for the removal of seasonal and diurnal trends without the rigid assumptions of global linear models. This methodology aims to better characterize the underlying radon variability, providing a refined dataset that improves the reliability of earthquake precursor detection in tectonically active regions.

## Study Area and Data

### Data Acquisition and Monitoring Site

The study utilizes hourly soil-gas radon concentration data collected from the Jiaosi monitoring station in Taiwan between January 2021 and April 2022. The monitoring network utilizes a continuous setup involving a RAD7 detector, gas coolers, and water traps to ensure sample integrity before transmission to a centralized database (Fig. 1). Synchronous meteorological data, including temperature, humidity, atmospheric pressure, and precipitation, were obtained from the Central Weather Administration.

### Preprocessing and Time-Series Integrity

Raw data underwent rigorous preprocessing to ensure analytical validity:

- **Data cleaning:** Removal of physically invalid values, such as negative rainfall or zero atmospheric pressure readings;
- **Normalization:** Datasets were normalized to enable comparative analysis across disparate physical units;
- **Chronological ordering:** Strict time-series integrity was maintained to support event-based differencing.

### Analytical Framework: Linear vs. Nonlinear Space

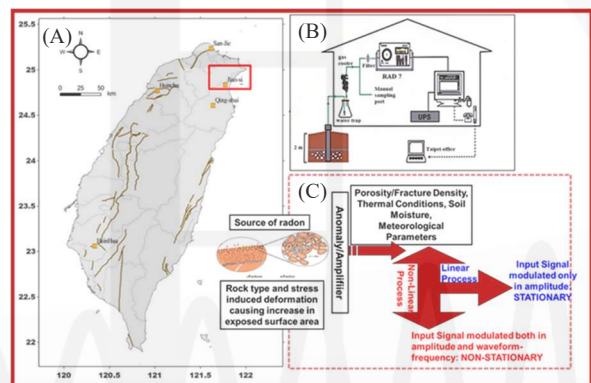
The study compares two distinct modeling approaches:

1. **Linear correlation:** Evaluation using Pearson correlation coefficients to identify long-term periodic and seasonal trends;
2. **Nonlinear event-based analysis:** Identification of transitory “hike” (rapid increase) and “drop” (rapid decrease) events through time-series differencing;

3. **LOWESS modeling:** LOWESS was employed to model the radon response to specific environmental changes. Unlike aggregated correlation studies, LOWESS captures localized, nonlinear trends that often characterize high-impact environmental events.

## Results and Discussion

Radon variability exhibits complex relationships with meteorological parameters, reflecting both steady-state environmental controls and transient nonlinear responses. Combined linear and nonlinear analyses reveal that while some parameters govern baseline radon behavior, others dominate short-term variability and extreme events.

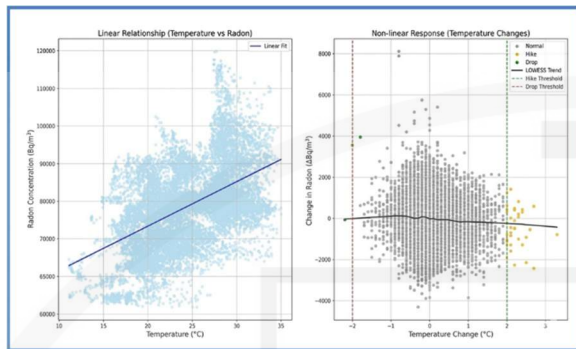


**Fig. 1.** (A) Network of soil-gas monitoring stations in Taiwan, (B) schematic of a continuous earthquake monitoring station setup, and (C) processes influencing radon generation and anomaly amplification.

### Temperature vs. Radon

The relationship between radon concentration and temperature demonstrates a clear distinction between long-term and short-term behavior. Linear correlation analysis shows a consistent positive relationship, indicating that elevated temperatures enhance radon concentration (Fig. 2). This behavior is primarily attributed to increased diffusion rates and enhanced soil gas mobility under warmer conditions, which facilitate upward radon transport. However, when temperature is analyzed in terms of short-term changes ( $\Delta T$ ), the relationship becomes weak and highly nonlinear. Minor fluctuations in temperature produce negligible changes in radon concentration, whereas larger variations (beyond approximately  $\pm 2^\circ\text{C}$ ) result in increasingly scattered and asymmetric responses. This suggests the presence of threshold-dependent mechanisms, in which only sufficiently large thermal perturbations influence subsurface gas dynamics. The observed nonlinearity may be linked to transient changes in soil-air density gradients, thermal expansion of pore gases, and localized convection processes. Additionally, temperature effects are often coupled with other environmental variables, particularly soil moisture and pressure,

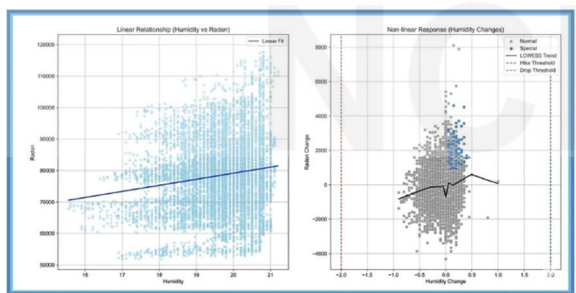
which can modulate the efficiency of radon transport. Therefore, while temperature acts as a dominant first-order control on seasonal radon variability, its role in short-term anomaly generation is limited and indirect.



**Fig. 2.** Linear and nonlinear relationships between radon concentration and temperature.

### Relative Humidity vs. Radon

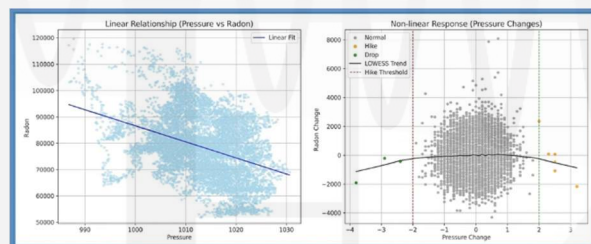
Relative humidity exhibits a weak positive correlation with radon concentration, indicating a secondary and indirect influence (Fig. 3). This relationship is likely associated with the role of moisture in modifying soil properties, such as porosity and permeability, which in turn affect radon emanation and migration. Nonlinear analysis of humidity changes ( $\Delta H$ ) reveals that small fluctuations have minimal impact on radon variability. Moderate increases in humidity are associated with slight increases in radon concentration, possibly due to enhanced emanation efficiency from partially saturated soils. However, at higher humidity levels, variability increases significantly, indicating unstable and threshold-dependent behavior. This can be explained by competing physical processes. On the one hand, increased moisture can facilitate radon release from mineral grains (emanation effect). On the other hand, excessive moisture reduces soil permeability, limiting gas transport. The interplay between these opposing mechanisms results in a weak and inconsistent overall relationship. Consequently, humidity is best characterized as a secondary modulating factor rather than a primary driver of radon variability. Its influence becomes more apparent when interacting with other parameters, particularly rainfall and temperature.



**Fig. 3.** Linear and nonlinear relationships between radon concentration and relative humidity.

### Atmospheric Pressure vs. Radon

Atmospheric pressure demonstrates one of the most physically consistent relationships with radon concentration. The observed negative correlation indicates that low-pressure conditions favor increased radon levels, while higher pressure suppresses radon exhalation (Fig. 4). This behavior is explained by the well-established barometric pumping mechanism, in which pressure gradients between the subsurface and atmosphere drive gas movement. A drop in atmospheric pressure reduces the confining force on soil gases, allowing radon to migrate upward more efficiently. Conversely, increased pressure inhibits gas release. Nonlinear analysis further confirms that pressure changes ( $\Delta P$ ) produce systematic and predictable radon responses. Small pressure variations have limited effects, but larger drops lead to pronounced increases in radon concentration. This response is more consistent and less scattered than those associated with temperature and humidity, indicating that pressure is a key driver of short-term radon dynamics. Importantly, pressure effects often act in synergy with other parameters. For example, pressure drops occurring simultaneously with rainfall-induced soil saturation can significantly amplify radon anomalies. This highlights the importance of considering combined environmental forcing rather than isolated variables.

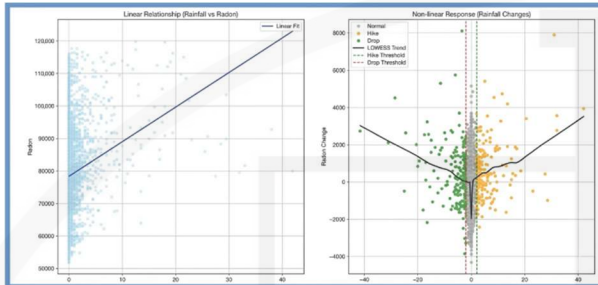


**Fig. 4.** Linear and nonlinear relationships between radon concentration and atmospheric pressure.

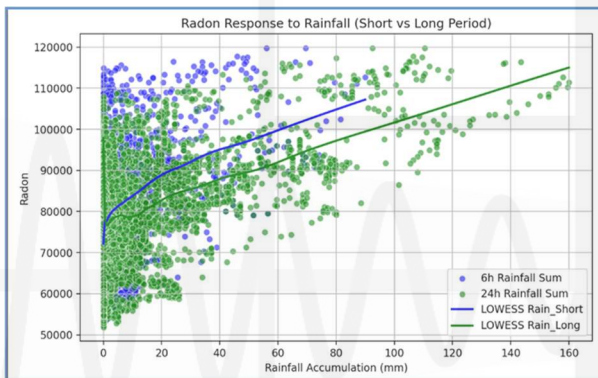
### Rainfall vs. Radon

Rainfall emerges as the most influential parameter in terms of nonlinear radon variability. Linear analysis suggests a general positive relationship between rainfall and radon concentration (Fig. 5), which can be attributed to the “capping effect,” whereby water infiltration reduces soil permeability and traps radon beneath the surface. However, the nonlinear event-based analysis provides a more detailed and revealing perspective. Radon response is highly sensitive to the rate and intensity of rainfall rather than to its cumulative amount. Rapid rainfall events (classified as “hikes”) trigger disproportionately large increases in radon concentration, whereas gradual or low-intensity rainfall produces minor effects. The LOWESS analysis reveals a distinct U-shaped (or V-shaped) relationship between rainfall change and radon variability. Within a normal range of rainfall variation, radon remains stable. Beyond this range, both rapid increases and decreases in rainfall are associated with

high variability in radon response. This behavior reflects dynamic soil processes. Intense rainfall can rapidly saturate the upper soil layer, creating a temporary impermeable barrier that prevents radon escape and leads to accumulation. When combined with pressure drops, this trapped radon is forced upward, producing sharp spikes in concentration.



**Fig. 5.** Linear and nonlinear relationships between radon concentration and rainfall.



**Fig. 6.** Comparison of the impact of short-period and long-period rainfall on radon variations.

A more detailed comparison between short-duration (6-hour) and long-duration (24-hour) rainfall highlights the importance of rainfall intensity (Fig. 6). The 6-hour accumulation shows a steeper response curve, indicating that radon reacts more strongly to rapid, high-intensity rainfall events. In contrast, the 24-hour accumulation reflects a smoother, more gradual response, even at higher total rainfall values. This indicates that prolonged rainfall allows partial equilibration of the system, reducing the magnitude of radon fluctuations. The higher sensitivity to short-term rainfall suggests that rapid hydrological changes, rather than total water input, are the primary drivers of radon anomalies. This finding has important implications for hazard assessment, as short, intense storms pose a greater risk of sudden radon surges than prolonged precipitation events.

When all meteorological parameters are considered collectively, a hierarchical control on radon variability becomes evident, reflecting the combined influence of both steady-state and transient environmental processes. Temperature primarily governs the long-term baseline behavior of radon through its control of soil gas diffusion and seasonal

trends, whereas atmospheric pressure plays a dominant role in short-term variability via barometric pumping mechanisms that regulate subsurface gas transport. Relative humidity acts as a secondary modulating factor, influencing radon indirectly through its impact on soil moisture and permeability, although its overall effect remains limited and inconsistent. In contrast, rainfall emerges as the most significant nonlinear driver, particularly during high-intensity events, when rapid soil saturation produces strong transient responses. The most pronounced radon anomalies occur when multiple environmental factors act simultaneously, especially under conditions in which intense rainfall coincides with falling atmospheric pressure. Under such circumstances, the formation of a near-surface water cap restricts radon escape, while reduced atmospheric pressure enhances upward gas migration, resulting in amplified radon concentrations. These interactions demonstrate that radon variability cannot be attributed to a single controlling parameter but is instead governed by the dynamic interplay of multiple meteorological forces. Consequently, while linear models can adequately describe general trends, they fail to capture the high-amplitude, short-duration fluctuations that are critical for interpreting radon anomalies in the context of earthquake precursor studies.

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# Influence of Superstructure Inclusion on Penetration Settlement of Raft Foundations in Liquefied Soils

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## Abstract

This study investigates the effect of superstructures on the penetration settlement of raft foundations in liquefied soil layers, while considering soil–structure interaction (SSI) effects. A numerical model was developed using PLAXIS 2D. Under identical seismic conditions, soil parameters, foundation dimensions and loading configurations, the settlement behavior of models with and without the superstructure was compared. The results indicate that the overall settlement of the model with the superstructure differs by approximately 5% from that of the simplified model, suggesting that the raft-only model still retains reasonable predictive capability. However, during seismic loading, the superstructure generates additional dynamic stresses due to inertial and kinematic effects, leading to a significant increase in both settlement and tilt. This finding suggests that the dynamic effects of the superstructure should be considered in seismic design.

**Keywords:** raft foundation, superstructure, settlement, numerical simulation, soil liquefaction

## Introduction

In earthquake-prone regions, soil liquefaction often causes a significant reduction in bearing capacity, leading to structural settlement and tilting, which threaten structural safety. Therefore, investigating the behavior of foundation systems under liquefied conditions is essential for seismic design and disaster mitigation.

Previous studies have primarily adopted simplified foundation models and have rarely examined the influence of superstructures on foundation settlement behavior, particularly during seismic loading. Consequently, the role of superstructures in soil–structure interaction (SSI) remains insufficiently understood.

This study employs the finite element software PLAXIS with interface elements and the PM4Sand soil model to simulate the nonlinear soil behavior under liquefaction conditions. The objective is to compare the settlement behavior of raft foundations with and without superstructures during both seismic loading and post-seismic consolidation. In addition, the effects on settlement time histories and structural tilt are

analyzed to assess the influence of superstructures on foundation design. The findings of this study can serve as a reference for raft foundation design and simplified modeling in liquefied soils.

## Numerical Model Description

This study is based on the raft foundation numerical model proposed by Hsu (2024), with the superstructure incorporated into the system. The dimensions of beams and columns follow the standard townhouse design provided by the National Center for Research on Earthquake Engineering, National Institutes of Applied Research (hereinafter referred to as “NCREE”). The beam dimensions are 25 cm × 60 cm, and the column dimensions are 50 cm × 60 cm. The story height is 3.5 m, and the column spacing is 5.5 m. The superstructure is centrally placed on the raft foundation to reflect typical building configurations.

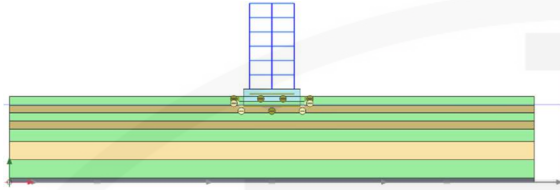
In the numerical model, the superstructure is represented using plate elements, and interface elements are introduced at the connection between the superstructure and the foundation to simulate soil–structure interaction behavior. A representative six-

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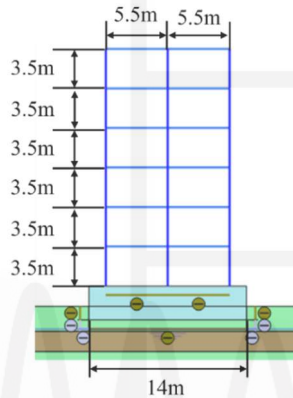
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story structure with a foundation width of 14 m is adopted as an example, as shown in Fig. 1. The detailed configuration of the superstructure is presented in Fig. 2. This model is subsequently used for dynamic analysis to evaluate the influence of the superstructure on foundation settlement.



**Fig. 1.** Numerical model of a six-story structure with raft foundation



**Fig. 2.** Detailed configuration of the superstructure model

To ensure consistent comparison between models, the contact stress is controlled through load redistribution. Specifically, the original raft foundation weight is equally divided between the raft and the superstructure, so that both components share the structural load while maintaining equivalent contact stress conditions. In addition, the soil stratigraphy and material properties follow those adopted in previous studies to maintain consistent analysis conditions.

The analysis includes a range of building heights and foundation widths, with a total of 13 models evaluated for comparison, as summarized in Table 1. The associated soil stratigraphy is presented in Table 2.

**Table 1.** Building Characteristics (Hsu, 2024)

| Number of Stories | Foundation Width |
|-------------------|------------------|
| 1-story building  | 13 m, 24 m, 38 m |
| 2-story building  | 23 m, 43 m       |
| 3-story building  | 24 m, 43 m       |
| 4-story building  | 19 m, 30 m, 42 m |
| 5-story building  | 15 m, 24 m, 35 m |
| 6-story building  | 14 m, 22 m, 32 m |

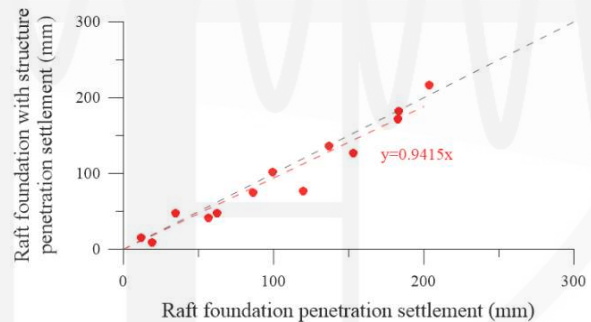
**Table 2.** Simplified Soil Stratigraphy

| Depth Start (m) | Depth End (m) | SPT-N | D <sub>r</sub> (%) |
|-----------------|---------------|-------|--------------------|
| 0               | 2.2           | -     | 75                 |
| 2.2             | 4             | 6     | 35                 |
| 4               | 6             | -     | 75                 |
| 6               | 8             | 6     | 35                 |
| 8               | 11            | -     | 75                 |
| 11              | 15.5          | 19    | 64                 |
| 15.5            | 20            | -     | 75                 |

## Numerical Results and Discussion

### (1) Overall Settlement Behavior

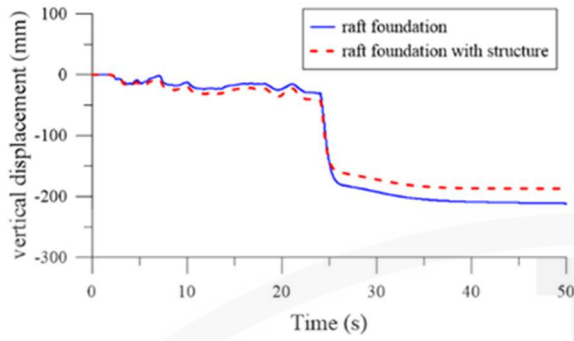
The penetration settlement of raft foundations with and without superstructures is compared, as shown in Fig. 3. The results indicate that the settlement of the model including the superstructure is approximately 0.94 times that of the model without it, corresponding to a difference of approximately 5%. This suggests that the simplified raft foundation model still provides reasonable predictive performance for engineering applications.



**Fig. 3.** Settlements of raft foundations with and without superstructures.

### (2) Settlement Behavior During and After Earthquake

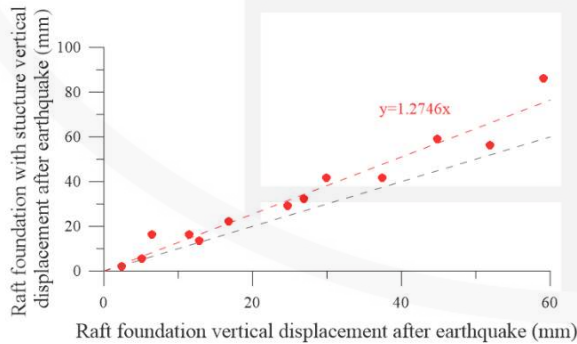
To examine the effect of the superstructure on settlement time-history behavior, the vertical displacement at the center of the raft foundation is monitored. The settlement time histories during seismic loading and the subsequent consolidation phase are compared. Fig. 4 shows results for a five-story building with a foundation width of 24 m. Compared with the model without the superstructure, the inclusion of the superstructure leads to a slight increase in settlement during the earthquake. However, during the subsequent dissipation of excess pore water pressure, the settlement becomes relatively smaller.



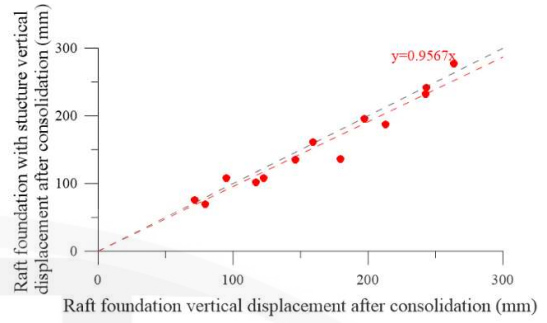
**Fig. 4.** Time-history comparison of vertical displacement for raft foundations with and without the superstructure.

Fig. 5 and Fig. 6 show the comparisons of vertical displacement immediately after the earthquake and after consolidation, respectively. Fig. 5 shows that the total settlement of the model with the superstructure is approximately 1.3 times greater than that of the model without the superstructure, indicating an amplification effect during seismic loading. This can be attributed to inertial and kinematic effects of the superstructure, which induce additional dynamic stresses and consequently increase the overall settlement.

According to Fig. 6, during the consolidation phase, the settlement of the model with the superstructure is approximately 0.96 times that of the model without it, indicating a reduced difference. This suggests that the influence of the superstructure is mainly limited to the seismic loading phase, whereas the consolidation behavior is primarily governed by soil properties.



**Fig. 5.** Comparison of vertical displacement after seismic loading for models with and without a superstructure.



**Fig. 6.** Comparison of vertical displacement after consolidation for models with and without a superstructure

### (3) Structural Tilt Analysis

Structural tilt is an important indicator of building stability. In this study, the tilt of raft foundations with and without superstructures is further analyzed. Yasuda (2014) reported that structural tilt is positively correlated with settlement.

In this study, tilt is defined as the ratio of the difference in vertical displacement between the left and right sides of the foundation to the foundation width, as shown in Equation (1) and illustrated in Fig. 7.

$$\beta = \frac{|\delta - \delta'|}{L} \quad (1)$$

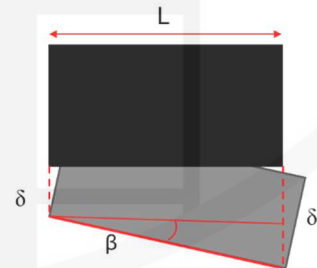
where

$\beta$  = structural tilt

$\delta$  = settlement at the left side of the foundation

$\delta'$  = settlement at the right side of the foundation

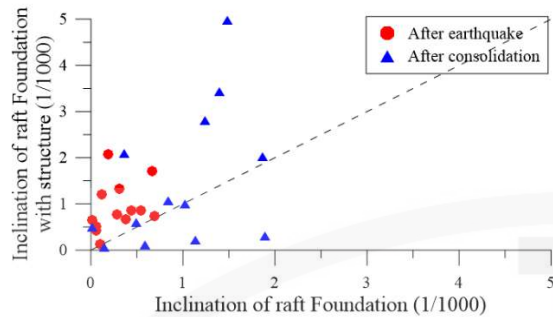
$L$  = foundation width



**Fig. 7.** Schematic illustration of structural tilt (adapted from Hsu, 2024)

Fig. 8 compares the tilt after seismic loading and after consolidation for models with and without superstructures. The results show that the presence of a superstructure increases the tilt during seismic loading, whereas its influence becomes less significant during the consolidation phase. In some cases, the tilt is even reduced. Overall, the influence of the superstructure on settlement and tilt exhibits a clear phase-dependent behavior.





**Fig. 8.** Comparison of structural tilt after seismic loading and after consolidation for models with and without superstructure

## Conclusions

This study investigates the influence of superstructures on the settlement behavior of raft foundations in liquefied soil conditions. The main conclusions are summarized as follows:

1. Significant influence during seismic loading: The presence of a superstructure amplifies both settlement and tilt by approximately 1.3 times, indicating that structural dynamic effects should be considered in seismic design.
2. Limited influence during consolidation: The settlement difference decreases to approximately 0.96 times during consolidation, indicating that long-term settlement behavior is primarily governed by soil properties, while the influence of the superstructure becomes relatively minor.
3. Applicability of simplified models: The overall difference in settlement is approximately 5 %, indicating that raft-only models can be used as a practical tool for preliminary design and rapid engineering assessment.

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# Evolution of Volatile Partitioning and Fluid Transport in a Shallow Submarine Hydrothermal System at Kueishantao Volcano, NE Taiwan (2020–2025)

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## Abstract

Long-term gas monitoring was conducted at the shallow submarine hydrothermal system of Kueishantao, northeastern Taiwan, from 2020 to 2025, establishing a time series of major gas compositions and helium isotopes. The results show that air-corrected  $3\text{He}/4\text{He}$  ratios remained stable at about 7.28–8.17 RA throughout the observation period, indicating a persistent and stable magmatic volatile source. In contrast, gas compositions exhibited pronounced temporal variations. Based on systematic changes in gas ratios, three evolutionary stages are identified. During 2020–2021, the system was characterized by  $\text{CO}_2$ -dominated emissions with limited variability, representing a stable baseline state. From 2022 to 2024, elevated  $\text{Stotal}/\text{CO}_2$  ratios accompanied by decreased  $\text{N}_2/\text{Ar}$  values suggest enhanced gas–water equilibration within the shallow hydrothermal system. In 2025, a further transition is observed, marked by significant  $\text{CO}_2$  depletion along with increased  $\text{N}_2/\text{Ar}$  ratios and helium concentrations, reflecting reorganization of shallow fluid transport pathways and preferential  $\text{CO}_2$  removal. These results indicate that, despite stable magmatic input, gas composition variations are primarily controlled by shallow hydrothermal processes. This study highlights the importance of long-term geochemical monitoring in resolving the evolution of fluid transport pathways and volatile partitioning in submarine volcanic systems.

**Keywords:** volcanic gas geochemistry; hydrothermal system evolution; Kueishantao

## 1. Introduction

Volcanic gas geochemical monitoring is a fundamental tool for assessing volcanic activity and subsurface fluid processes (Giggenbach, 1996; Fischer et al., 1998). Changes in gas composition are widely considered key indicators of shifts in volcanic systems, especially during periods of unrest or hydrothermal development. However, gas composition is influenced not only by deep magmatic volatile input but also by shallow hydrothermal processes such as gas–water interactions, fluid transport pathways, and phase partitioning (Giggenbach, 1996). This complexity can

complicate the interpretation of geochemical signals.

Kueishantao, located off northeastern Taiwan, is an active submarine hydrothermal system characterized by continuous emission of  $\text{CO}_2$ -dominated volcanic gases and vigorous intense gas–water interactions. Unlike subaerial volcanic systems, submarine hydrothermal environments are more strongly affected by surrounding water and pressure conditions, resulting in more complex fluid transport and geochemical signatures. Previous studies have mainly focused on short-term observations or discrete sampling, and comprehensive investigations of long-term compositional variations and their controlling

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mechanisms are still limited.

In this study, long-term gas monitoring data collected between 2020 and 2025 are analyzed to assess temporal variations in major gas compositions, gas ratios, and helium isotopes. The objective is to determine the relative influence of deep magmatic input and shallow hydrothermal processes on gas composition changes. By integrating multiple geochemical parameters, this study presents a multi-stage evolutionary model of the Kueishantao hydrothermal system and constrains the mechanisms controlling fluid transport and volatile partitioning.

## 2. Methods

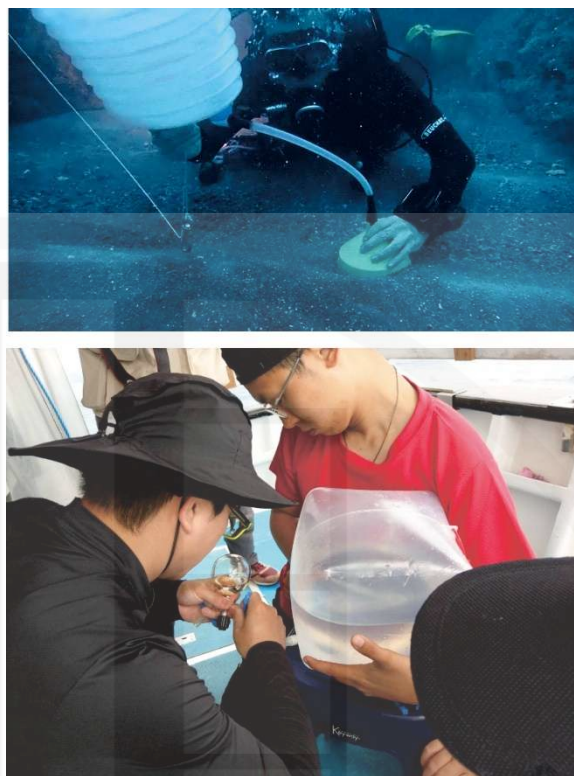
Volcanic gas samples were collected in situ by divers in the offshore area of Kueishantao, northeastern Taiwan. The main sampling site was at the White Vent field near the turtle head of the island (Fig. 1). During each campaign, gas samples were taken from two neighboring vents at a depth of about 15 m. The procedure was repeated during two separate dives to ensure spatial representation, resulting in four samples per campaign.

Submarine gases were collected using gas bags via the water displacement method. Immediately after collection, the samples were transferred onboard into Giggenbach bottles and evacuated glass flasks with double valves to minimize atmospheric contamination and chemical alteration.

Sampling was conducted twice per year, typically in April–May and September–October. The sampling procedure and onboard transfer workflow are shown in Fig. 2.



**Fig. 1.** Sampling locations of submarine hydrothermal gases at Kueishantao. White crosses indicate sites sampled during previous campaigns, while the 2025 sampling locations are highlighted to show alignment with earlier sites, thereby ruling out spatial variability as a cause of the observed compositional changes.



**Fig. 2.** Submarine gas sampling and onboard sample transfer procedures. (Top) Gas sampling at a seafloor vent using gas bags via the water displacement method. (Bottom) Onboard transfer of gas samples into Giggenbach bottles for preservation.

Gas compositions were analyzed in the laboratory. Non-condensable gases were measured using gas chromatography (GC). Chloride and sulfate concentrations dissolved in the alkaline solution were determined by ion chromatography (IC) and were used to calculate HCl and total sulfur gas concentrations ( $\text{H}_2\text{S} + \text{SO}_2$ ). Carbonate ( $\text{CO}_3^{2-}$ ) and bicarbonate ( $\text{HCO}_3^-$ ) concentrations were determined by titration to reconstruct  $\text{CO}_2$ -related components (Giggenbach, 1987). The  $\text{SO}_2/\text{H}_2\text{S}$  ratio was determined using the iodine titration method. The analyzed gas species include  $\text{CO}_2$ ,  $\text{H}_2\text{S}$ ,  $\text{SO}_2$ ,  $\text{HCl}$ ,  $\text{CH}_4$ ,  $\text{N}_2$ ,  $\text{H}_2$ ,  $\text{He}$ , and  $\text{Ar}$ .

Samples collected in evacuated glass flasks were analyzed for helium isotopes ( $^3\text{He}/^4\text{He}$ ) using a noble gas mass spectrometer. Helium isotope ratios were used to determine gas sources (Sano & Marty, 1995). Results were corrected for atmospheric contamination and reported as air-normalized, corrected ratios, denoted as  $(\text{R}/\text{RA})\text{C}$ .

## 3. Results

Time-series data of gas compositions and geochemical parameters at Kueishantao from 2020 to 2025 show clear temporal variations (Fig. 3). Air-



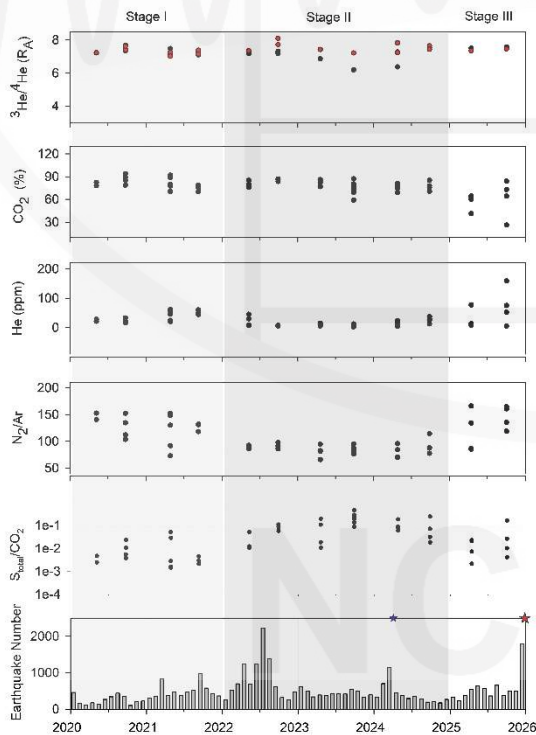
corrected helium isotope ratios ( $(R/RA)C$ ) remained relatively constant throughout the observation period ( $\sim 7.28$ – $8.17$  RA), indicating a stable magmatic volatile source. In contrast, major gas compositions and ratios display systematic changes over time.

Based on the observed compositional variations, three distinct stages can be identified (Fig. 3). During Stage I (2020–2021), gas composition was dominated by  $CO_2$  (approximately 70–90%) with limited variability.  $N_2/Ar$  ratios during this stage were generally higher than the atmospheric equilibrium value.

During Stage II (2022–2024),  $Stotal/CO_2$  ratios increased, while  $N_2/Ar$  ratios declined toward equilibrium with the atmosphere.

A further transition occurs in Stage III (2025), marked by a significant decrease in  $CO_2$  concentrations to approximately 26%, accompanied by increased  $N_2/Ar$  ratios ( $>120$ ) and increased helium levels. The sampling sites in 2025 remain consistent with those from previous campaigns (Fig. 1), confirming that these compositional changes are not attributable to spatial variability.

Overall, the observations indicate a clear temporal evolution and multi-stage variation in gas compositions. To further interpret the underlying processes, a conceptual model is presented that integrates these observations (Fig. 4).



**Fig. 3.** Time-series variations of gas compositions and geochemical parameters at Kueishantao. Variations in

helium isotopic ratios ( $^3He/^4He$ ),  $CO_2$  concentration, helium concentration,  $N_2/Ar$ , and  $S_{total}/CO_2$  from 2020 to 2025 are shown. Gray shaded areas indicate three stages (Stage I–III). The bar chart shows regional seismicity; blue stars mark the 2024 Hualien earthquake, and red stars indicate the 2025 Okinawa Trough earthquake.

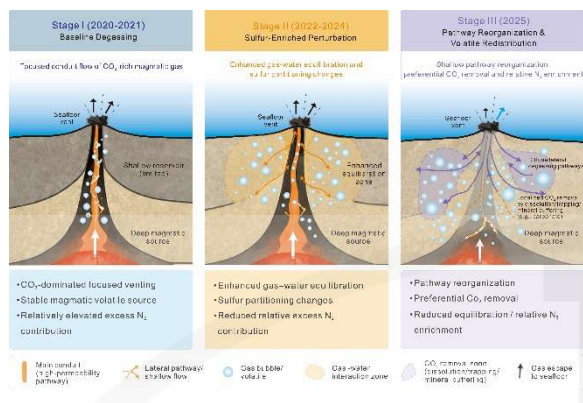
#### 4. Discussion

Based on the observations and the conceptual framework (Fig. 4), the variations in gas composition at Kueishantao are best explained by processes within the shallow hydrothermal system rather than changes in the deep magmatic source. The constant helium isotopic ratios indicate a stable magmatic volatile supply throughout the study period (Sano & Marty, 1995).

The geochemical trends observed during Stage II indicate enhanced gas–water equilibration. The rise in  $Stotal/CO_2$  and the decline in  $N_2/Ar$  toward atmospheric equilibrium suggest increased interaction between ascending gases and seawater or hydrothermal fluids (Giggenbach, 1996). These interactions promote the re-partitioning of sulfur species and alter gas compositions in the shallow system, highlighting the important role of physical conditions such as permeability and fluid residence time (Fischer et al., 1998).

The pronounced compositional shift during Stage III is characterized by a significant decrease in  $CO_2$ , accompanied by elevated  $N_2/Ar$  ratios and helium concentrations. Given the stable helium isotopic signature, this transition is unlikely to reflect changes in deep magmatic input. Instead, it is most consistent with reorganization of shallow fluid pathways. Such reorganization may lead to preferential removal of  $CO_2$  through dissolution, trapping, or mineral buffering, while reducing its transport efficiency and resulting in relative enrichment of other gases.

Taken together, these observations indicate that the temporal evolution of gas compositions is controlled by dynamic shallow processes, such as fluid pathway restructuring and gas–water interactions, rather than by changes in deep magmatic supply.



**Fig. 4.** Conceptual model illustrating the multi-stage evolution of the Kueishantao submarine hydrothermal system. The model integrates geochemical observations from 2020 to 2025 and depicts three evolutionary stages: (I) stable CO<sub>2</sub>-dominated degassing through high-permeability conduits; (II) increased gas–water interaction causing redistribution of sulfur species and system disturbance; and (III) reorganization of shallow fluid pathways, with preferential CO removal and relative enrichment of other gases, indicating changes in gas–liquid partitioning.

## 5. Conclusions

By integrating long-term geochemical observations with a conceptual model (Fig. 4), this study demonstrates that variations in gas compositions at Kueishantao are primarily controlled by shallow fluid transport and gas–water interactions, rather than changes in the deep magmatic source. With a stable supply of magmatic volatiles, changes in shallow permeability and fluid pathways can drive multi-stage compositional evolution and affect the partitioning and transport efficiency of CO<sub>2</sub>.

These findings indicate that temporal variations in gas compositions within submarine hydrothermal systems may not reflect changes in deep magmatic activity, but instead record dynamic adjustments within the shallow system. Integrating long-term monitoring with process-based models offers valuable insights into fluid behavior in volcano–tectonic environments.

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# Applications for Machine Learning in Volcano Seismic Monitoring

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## Abstract

This study develops an automated workflow for cataloging volcano-tectonic (VT) earthquakes in the Tatun Volcano Group (TVG). The workflow integrates a fine-tuned Volpick model for phase picking with PyOcto for rapid event association. Using a reference catalog of 2,294 VT events as a baseline, we evaluate 320 combinations of four key parameters and apply Pareto-front analysis to identify 43 optimal solutions. Three operating points are recommended for balanced detection, real-time monitoring, and catalog building. The probability thresholds for P and S waves are the most influential parameters; the best balanced point achieves an F1 score of 0.786, with a recall of 0.864 and precision of 0.720. This workflow can support real-time monitoring, b-value analysis, and long-term activity assessment, and can be incorporated into the routine operations of the Tatun Volcano Observatory.

**Keywords:** Tatun Volcano Group, volcano-tectonic earthquake, machine learning

## Background

The Tatun Volcano Group (TVG), located within Yangmingshan National Park in northern Taiwan, is the largest volcanic area on the island. Recent geophysical studies have confirmed its active status: Lin (2016) detected S-wave shadows and P-wave delays, providing the first seismic evidence for an underground magma reservoir in the area. Huang et al. (2021) used three-dimensional tomography to map a distinct low-velocity zone at depths of 8 to 20 km on the eastern side of the TVG, showing a P-wave velocity reduction and an estimated melt fraction of approximately 19%. These findings underscore the need for continuous and detailed monitoring of TVG seismic activity.

Volcanic seismicity can be categorized into four main types: (a) volcano-tectonic (VT) earthquakes, (b) long-period events, (c) volcanic tremors, and (d) hybrid events. VT earthquakes are the most useful for automated monitoring. Their waveforms are like those of regular tectonic earthquakes, allowing the use of existing phase-picking and location tools. Their magnitude–frequency distribution (b-value) and patterns of clustering in space and time also provide a quantitative indicator of the stress state within the deep volcanic system. Notably, rapid increases in the rate of VT earthquake swarms often precede eruptions. For example, before the Sundhnúkur fissure eruption in the Grindavík area of Iceland in 2023, intense swarm

activity began in late October and peaked in mid-November, with more than 10,500 earthquakes recorded within a few days (including several with magnitudes of 3.0 or greater), followed by a large fissure eruption on December 18 and several subsequent eruptions in 2024. Similarly, the 2024 Ruang volcano eruption in Indonesia was preceded by a magnitude 6.5 earthquake in the Molucca Sea close to the eruption time, along with a sharp increase in the local earthquake rate in the days immediately before the event. Detecting such increases in seismic activity in near real time is an important mission for volcano observatories worldwide.

The Taiwan Volcano Observatory (TVO) has upgraded the high-density broadband network and has built a long-term VT catalog. However, routine processing still relies on manual or semi-automatic picking, which becomes significantly slower during high-rate swarm periods. The traditional STA/LTA method exhibits low recall on shallow VT events affected by strong high-frequency scattering. Although mainstream deep-learning pickers such as PhaseNet and EQTransformer perform well in many tectonic settings, they were trained mostly on far-field data, which limits their performance on volcanic VT signals. A promising solution is the volcano-specific picker Volpick paired with the high-throughput associator PyOcto.

Implementing such a workflow at the TVG would

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do more than just replace the picker. A stable automated pipeline would also facilitate broader follow-up work: with more events and consistent picks, tasks such as b-value time series studies, hypocenter clustering, and array analysis become much easier to manage. This shift would allow analysts to focus less on routine catalog editing and more on unusual sequences. With these motivations, this study aims to build and validate an end-to-end VT catalog workflow using TVG data that supports both real-time monitoring and offline research.

## Objectives

The work has three goals: (i) systematically scan the main parameter space of the event associator and analyze how each combination affects catalog quality; (ii) use Pareto front analysis to identify optimal solutions, providing a quantitative basis for parameter selection in different practical scenarios; and (iii) provide reproducible documentation and open-source code, enabling integration of the workflow into routine observatory monitoring.

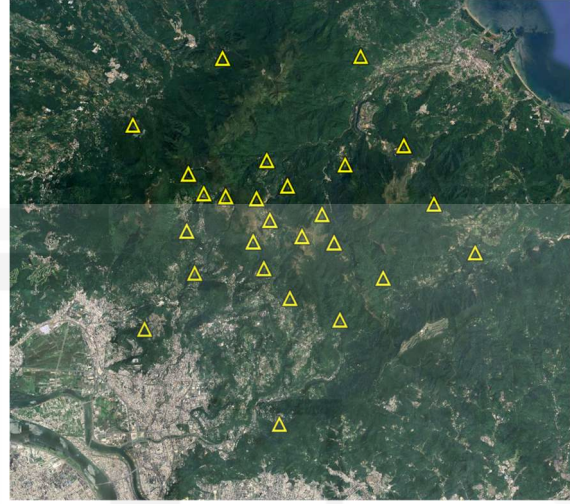
## Methods and Data

The input is the continuous waveforms recorded by stations in and around the TVG (Fig. 1). The reference catalog contains 2,294 VT events. We use SeisBench to standardize the waveform format, using the volcano-specialized Volpick as the base model, and fine-tune it on the Tatun catalog. The model outputs arrival times and probability values for the P and S phases, which are then passed to the PyOcto associator. A grid search is conducted over four key parameters (Table 1), resulting in 320 valid combinations. Performance is measured using the formulae  $\text{Precision} = \text{matched} \div (\text{matched} + \text{extra})$ ,  $\text{Recall} = \text{matched} \div 2,294$ , and  $\text{F1} = 2 \cdot \text{P} \cdot \text{R} \div (\text{P} + \text{R})$ .

Matching is based on arrival time and station identity: a model pick and a reference pick are considered matched if they are from the same station, the same phase, and their arrival times are within the tolerance window (tol). An event is matched to a reference event when at least the minimum pick fraction (frac) of its picks satisfies this per-station condition. This approach reflects both picking and association errors in the same precision–recall measure, enabling a single grid search to optimize both jointly.

**Table 1.** Systematic scan design for the key association parameters.

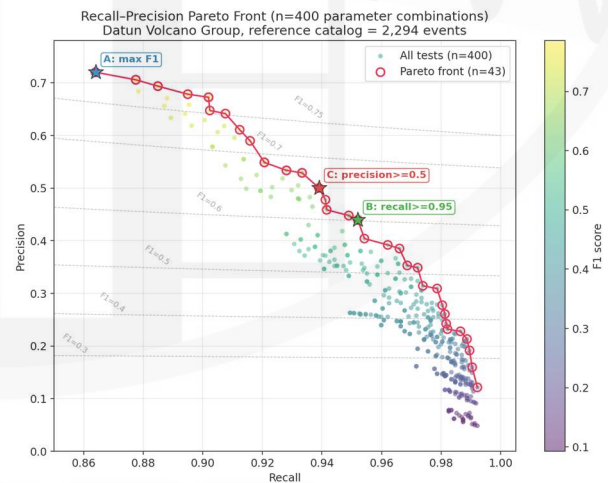
| Parameter         | Meaning            | Scan values                                      |
|-------------------|--------------------|--------------------------------------------------|
| p_prob_threshold  | P prob. threshold  | 0.1 / 0.2 / 0.3 / 0.7                            |
| s_prob_threshold  | S prob. threshold  | 0.1 / 0.2 / 0.3 / 0.45                           |
| min_pick_fraction | Min. pick fraction | 0.55–0.75 (0.05 step)<br>0.125 / 0.150 / 0.175 / |
| pick_match_toler. | Match window (s)   | 0.20                                             |



**Fig. 1.** Locations of broadband seismic stations. Yellow triangles represent station sites.

## Results

To evaluate the trade-off between recall and precision across 320 parameter sets, we compute the Pareto front, which represents the set of solutions not outperformed in both recall and precision. We identify 43 such solutions (Fig. 2). For practical application, three recommended operating points on the front are: A, which maximizes F1; B, which has the highest F1 with recall  $\geq 0.95$ ; and C, which has the highest F1 with precision  $\geq 0.50$ . The parameter sets and performance metrics for these points are shown in Table 2.



**Fig. 2.** Recall–Precision Pareto front (320 parameter sets, 43 non-dominated solutions). Color shows F1 score; dashed lines are F1 contours; stars A/B/C mark the recommended operating points.

Table 2. Parameters and performance of the three recommended operating points (A: balanced; B: monitoring with recall  $\geq 0.95$ ; C: catalog with precision  $\geq 0.50$ ).

| Point | F1    | Recall | Prec. | p / s / frac / tol         |
|-------|-------|--------|-------|----------------------------|
| A     | 0.786 | 0.864  | 0.720 | 0.70 / 0.45 / 0.75 / 0.125 |
| B     | 0.601 | 0.952  | 0.440 | 0.30 / 0.30 / 0.55 / 0.125 |
| C     | 0.653 | 0.939  | 0.500 | 0.70 / 0.45 / 0.55 / 0.150 |

The probability thresholds for P and S waves clearly dominate catalog quality. Increasing both thresholds from the lowest levels ( $P = 0.1$ ,  $S = 0.1$ ) to the highest levels ( $P = 0.7$ ,  $S = 0.45$ ) raises the average F1 score from 0.281 to 0.589. The main improvement is seen in precision from 0.170 to 0.451, while recall only slightly decreases from 0.980 to 0.930. Because the P and S thresholds are paired in the grid design, they share the same average values shown in Table 3. The minimum pick fraction and the matching tolerance window have only secondary effects on the results.

Table 3. Average performance at various levels of the main parameters.

| Parameter        | Level | Mean  | Mean   | Mean      |
|------------------|-------|-------|--------|-----------|
|                  |       | F1    | Recall | Precision |
| p_prob_threshold | 0.1   | 0.281 | 0.980  | 0.170     |
|                  | 0.7   | 0.589 | 0.930  | 0.451     |
| s_prob_threshold | 0.1   | 0.281 | 0.980  | 0.170     |
|                  | 0.45  | 0.589 | 0.930  | 0.451     |

## Discussion

Threshold dominance reflects the waveform features of VT events under low SNR. In the low-probability range (below about 0.3), the model tends to output many weak picks; only a small portion are real VT events, while the rest are background noise, non-event interference, or local high-frequency scattering. A modest increase in the threshold therefore results in a slight loss in recall but a significant gain in precision. This pattern is consistent with the findings of reported in the original Volpick paper reports and suggests that VT picking can be adjusted according to general principles.

Under the loosest setting ( $P = 0.1$ ,  $S = 0.1$ ,  $\text{frac} = 0.6$ ,  $\text{tol} = 0.2$ ), the number of additional events reach 16,454, far exceeding the 2,294 reference events. These additional events should not be assumed to be false detections: many are likely real VT microearthquakes that the original catalog missed. We recommend a two-step verification: first, randomly sample 50 to 100 additional events and manually check for the high-frequency P and S phases characteristic of VT events; second, create space-time maps and compare them with established VT zones such as the southern flank of Mt. Chihshin and the area around DaYouKeng. If most events are validated, the

workflow can yield a more complete catalog and facilitate subsequent b-value and stress-state analyses.

The three operating points map directly to practical needs. Point A (max F1) fits balanced research and offline catalog work, where both metrics matter. Point B (recall  $\geq 0.95$ ) fits real-time monitoring of swarms, where missing an event cost more than a false alarm; under this point, the workflow keeps almost all true events and lets a downstream filter or analyst remove the extras. Point C (precision  $\geq 0.50$ ) fits catalog building with manual review, where a smaller and cleaner candidate list lowers the review time. The observatory can switch between points based on the current goal.

Several limitations are worth noting. The reference catalog is based on previous selections and is not a perfect ground truth, so both recall and precision depend on its accuracy. The current parameter grid includes four variables; additional settings, such as minimum stations per event or maximum residual, could be incorporated in future analyses. Finally, the workflow has been optimized and tested for TVG only; applying it to other volcanoes will likely require some adjustment to local waveform characteristics.

## Conclusions

We developed an automated VT catalog workflow for the TVG that integrates SeisBench, a fine-tuned Volpick, and PyOcto. We verified its feasibility through a 320-set parameter scan. The optimal balanced point achieves an F1 score of 0.786 (recall 0.864, precision 0.720) with thresholds of 0.7 / 0.45, fraction 0.75, and a tolerance of 0.125 s. With the current data, the workflow demonstrates potential for near real-time operation. Future work will focus on manual validation of additional events, location accuracy assessment, development of a VT activity index based on space-time changes in b-value, and cross-year stability tests with a long-term maintenance plan. The workflow is also a candidate for transfer to other active volcanic fields such as Turtle Island.

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