

PERFORMANCE EVALUATION AND COST ASSESSMENT OF WEAK-STORY RETROFITS IN RC BUILDINGS SURVEYED AFTER THE 2024 HUALIEN EARTHQUAKE

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ABSTRACT

To provide evidence on the performance of retrofitting systems for informing retrofit practice in New Zealand, 18 buildings with retrofits including reinforced concrete (RC) column jacketing, RC wing walls, RC shear walls, and steel frames with braces were surveyed after the 2024 Hualien Earthquake in Taiwan. The primary goal of this study was to quantify amounts of retrofitting installed in vulnerable existing buildings prior to the earthquake observed to be sufficient to prevent severe structural damage and disruption to building functionality. The idea would be to use these quantified amounts as a rough threshold for comparing between competing retrofitting systems in preliminary design. In general, the retrofits observed in Taiwan were low-cost and designed to target mitigation of soft and weak first stories, a well-known structural vulnerability within the Taiwanese building stock. Nevertheless, the scope of retrofits ranged from being installed in first story only to along the full height of the building.

Simple indices estimated as ratios of cross-sectional areas of columns and walls to total floor area used as a proxy for base shear strength showed that on average, retrofitting nearly doubled the base shear capacity compared with the original state. To observe the effect of retrofitting on observed earthquake damage, case studies consisting of sets of building pairs, one building without retrofitting that experienced severe or moderate damage and one building with low-cost retrofitting that had minor or no damage, were investigated. Building pairs were selected to have similar size, structural layout, and seismic demand. Based on three case studies, if the amount of retrofitting increased column and wall indices to a certain threshold, no severe damage was observed.

A cost assessment of typical retrofitting systems was performed based on estimated cost schedules provided by Taiwanese structural consulting firms. For a 5-story mixed-use building consisting of both commercial and residential units, and assuming the cost to build a new structure is approximately USD \$1000 per square meter, the cost of installing a retrofitting system comprising RC column jacketing and additional shear walls to double base shear strength was estimated to be between 15-20% of the rebuilding cost of the original building (USD \$165/m²) based on current construction practice in Taiwan. The average cost of retrofitting installed in 23 private residential buildings was less expensive at approximately USD \$130/m² indicating that increasing seismic capacity of vulnerable buildings can be achieved at a reasonable cost.

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INTRODUCTION

Background

The 2024 Taiwanese earthquakes including the M_w 7.4 mainshock (3 April 2024) and subsequent M_w 5.6 aftershock (23 April 2024) occurred near the eastern coast of Central Taiwan, caused damage to many buildings in Hualien City [1-2], and produced several partial and full collapses (Figure 1). Despite the magnitude of both earthquakes, the low numbers of casualties were achieved by a national effort to mitigate vulnerabilities in the existing building stock over the last 25 years, including two retrofitting programs that targeted specific structural weaknesses and reduced building damage [1-9]. The

types of retrofits typically used in Taiwanese buildings and an overview of the programs are discussed next.

Types of Retrofitting Systems

The most common types of retrofitting used in Taiwan include reinforced concrete (RC) column jacketing, RC wing walls, RC shear walls, and steel frames with steel braces (Figure 2). RC jacketing involves constructing a steel cage consisting of longitudinal and transverse reinforcing bars along the height of an existing RC column through the depth of adjoining beams framing into the column and casting additional concrete (Figure 2-a) to increase lateral stiffness and lateral strength in both building directions [3-5]. Because increasing base shear

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strength is prioritized over increasing ductility as a general retrofitting policy for buildings in Taiwan, transverse reinforcing bars are not anchored into the existing column, and crossies only (no closed-hoop stirrups) are used in beam-column joints [5]. Nevertheless, the spacing of the transverse reinforcement along the clear height of the jacketed column is tighter (in general) than the spacing of the transverse reinforcement cast along the clear height of the existing column (Figure 2-c) resulting in increased ductility [3].



Figure 1: Demolition of a partially collapsed residential building in Hualien City.

Adding wing walls to existing columns increases lateral stiffness and strength in one direction while not disrupting room functionality (Figure 2-b). Installing RC shear walls in buildings is the most efficient way to increase base shear capacity and reduce drift demand in one direction of a building but may disrupt functionality and natural lighting (Figure 2-d). Steel frames and braces increase lateral stiffness and strength but may decrease aesthetical appeal (Figure 2-e).

Design Considerations Based on Building Use

Separate retrofitting strategies are used for school buildings, private residential dwellings, and structures with commercial entities based on construction costs and disruption to building functionality. For school buildings, and low-rise buildings in general, achieving adequate seismic performance is governed by a minimum required base shear strength. In this case, conventional retrofitting methods including RC column jacketing, shear walls, and wing walls are common as they have been shown to increase strength reliably while minimizing structural damage, design work, and construction costs [5-6].

If a school building is weak in both directions, column jacketing is preferred as it increases strength along both axes. Nevertheless, the typical school tends to be strong in its short direction, where full-height, solid masonry infill walls (without window or door openings) serve as partitions between adjacent classrooms, and weak in the orthogonal (long) direction parallel to the corridor, where windows allowing natural light to pass through reduce the number of full-height walls. Because of this, installing RC shear walls in the direction of the corridor is a common retrofitting strategy for school buildings.

For structures with commercial enterprises, steel frames with braces may be installed to cause minimal interruption to building functionality and to reduce economic downtime. But it is necessary to recoat steel braces to prevent corrosion and strengthening techniques using structural steel members require an ongoing maintenance budget.



(a) RC column jacketing
(Source: Chiou [3])



(b) RC wing wall



(c) Beam-column joint with RC column jacketing.
Figure is taken from Chiou [3].



(d) RC shear wall



(e) Steel frame with buckling-restrained brace

Figure 2: Examples of retrofitting systems used in Taiwanese buildings.

School Building Retrofit Program

After the devastation of the 1999 Chi-Chi earthquake which caused severe damage and collapse of approximately 50% of the primary, secondary, and high school buildings in Taiwan, a government-led initiative was undertaken to increase the seismic capacity of vulnerable school buildings built prior to modern seismic design codes [3-6]. The objectives of the program were to conduct a nationwide survey of educational facilities to identify common seismic vulnerabilities, prioritize the most deficient buildings, and implement retrofitting systems which increase seismic capacity.

Observations of damaged school buildings and results of numerical analysis indicated that most of the collapsed and severely damaged buildings had shear failures in first-story columns followed by axial failures [3-6]. To mitigate this type of failure, many school buildings were retrofitted using reinforced concrete column jacketing techniques to increase lateral stiffness and strength by supplementing the total cross-sectional area of vertical members. Comparisons of laboratory and field testing of vulnerable frames with and without RC column jacketing provide evidence that retrofitted frames increased lateral strength and ductility (Figure 3). To date, approximately 10,000 school buildings in Taiwan have adopted retrofitting as a part of this program and none has suffered column shear failures [5].

The required amount of retrofitting for vulnerable school buildings specified by the Taiwanese building seismic design code is based on minimum lateral strength and maximum drift demand (Figure 4) using a procedure based on the Capacity Spectrum Method [3-6]. The acceleration at the performance point (A_p) must exceed the peak ground acceleration (PGA) of the design spectrum. The story drift ratio corresponding to the performance point (D_p) must not exceed 2%.

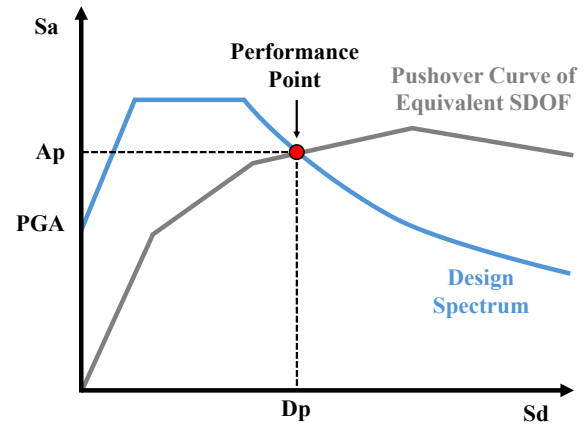
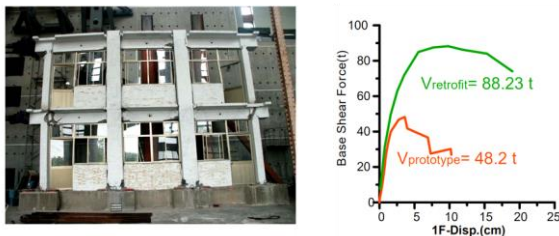


Figure 4: Basic concept of Taiwanese Seismic Design Method.

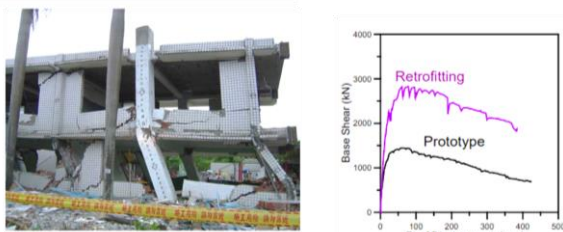
Retrofits for commercial buildings are designed using similar methods as those for school buildings and must meet 100% of the demands outlined in the 2022 Seismic Design Specifications for Buildings in Taiwan [7].

Private Residential Building Retrofit Program

In addition to the school retrofitting program, the Taiwanese government also implemented a strategy to target vulnerable private residential buildings [8-9]. The path to a successful retrofitting policy for private buildings had more roadblocks than it did for school and commercial buildings. One reason was that private buildings often have multiple owners and reaching a unanimous decision was challenging. But the distinguishing feature of the current retrofit program for private buildings is that it allows for a phased approach, allowing short-term emergency retrofitting, or “partial retrofit”, a first step towards a more complete retrofit (Figure 5).



(a) Laboratory testing of a planar frame with and without RC column jacketing. Figure is taken from Chiou [4].
 Note: Envelopes were used to connect the peak response observed during cyclic tests.



(b) Field testing of a school building with and without RC column jacketing. Figure is taken from Chiou [3].
 Note: Monotonic tests were conducted on the referenced building and measured response was reduced based on results from cyclic tests conducted on other existing school buildings.

Figure 3: Results of experiments and field tests used to verify the effectiveness of RC column jacketing.



(a) Before retrofitting (a) After retrofitting

Figure 5: Residential building with RC shear walls applied to first story only (Partial retrofitting using ‘Plan A’).

Note: Figure is taken from Weng [9].

The phased retrofit approach has the advantage of reducing seismic risk by installing partial retrofitting typically mitigating weak first stories while waiting for consensus from all building owners for a full retrofit design. Since the execution of the program in 2019, 80 private residential buildings have been approved for retrofitting, and more than 23 buildings have completed retrofitting [9].

To enable wider adoption of retrofitting for private structures, one option (‘Plan A’) requires that vertical irregularities including soft stories are mitigated, by requiring that the shear

strength of a given story is at least 80% of the shear strength of the story immediately above. Shear strength is evaluated using simple methods based on cross-sectional areas of vertical members and their unit strengths.

For buildings with seismic vulnerabilities other than soft stories, there is an alternative, more rigorous retrofit design procedure ('Plan B'). This plan explicitly accounts for seismic demand by requiring that the base shear capacity shall be at least 80% of the seismic design code requirement (Figure 4).

OVERVIEW OF SURVEYED BUILDINGS

May 2024 Earthquake Reconnaissance Mission

An earthquake reconnaissance team conducted building surveys and collected data during a one-week period in Taiwan. The results of these surveys, effects of retrofitting on earthquake damage, case studies comparing buildings with and without retrofitting, and a cost assessment of retrofitting systems are discussed in this study. Detailed information about the surveyed buildings may be found in the 2024 Hualien Earthquake NCREC report [10].

Surveyed Retrofitted Buildings in Taiwan

Data collected from surveys of 18 buildings with retrofits located in Hualien City, Taipei City, and New Taipei City are discussed next. Each retrofitted building was designed using either the school building or private residential building retrofit program described earlier. All surveyed buildings with retrofits remained operational during inspection. Retrofits consisted of one or more of the following: RC column jacketing, RC shear walls, RC wing walls, and steel frames with buckling-restrained braces (Figure 2). A method for quantifying the increase in seismic performance provided by retrofitting is discussed next.

Estimation of Hassan Index

The Hassan index, or "Priority Index" as coined by Hassan and Sozen [11], was initially proposed to rank low-rise RC buildings by their vulnerability to earthquake damage, where a smaller index indicates a building where strengthening or demolition should be prioritized. The advantage of this method was that it organized buildings into two categories based on their expected seismic performance; specifically, whether they were likely or unlikely to be damaged. The Hassan index was estimated as the sum of column index (Eq. 1) and wall index (Eq. 2) calculated using cross-sectional areas of all lateral-force resisting elements on the ground floor (RC columns, RC shear walls, and masonry infill walls) and normalized to total building floor area. The wall index was calculated separately for orthogonal directions, and the minimum Hassan index was used to represent a single building.

This method was applied to institutional buildings in the aftermath of the 1992 Erzincan Earthquake by Hassan and Sozen [11]. Hassan defined damage levels as follows:

Light: Reinforcement exposed but not buckled near joint faces. Fine flexural cracks in structural and non-structural elements.

Moderate: Reinforcement buckled near joint faces and/or inclined cracks in structural walls.

Severe: Structural failure of individual elements.

Results for Turkish buildings showed that structures with Hassan indices exceeding 0.25%, depicted by the dashed red line in Figure 6, were not likely to have severe damage. This line is referred to as the 'safety boundary' in this investigation and is defined using Eq. 3.

There is criticism against the use of methods (like the Hassan index) which do not account for the ductility of structural members necessary for buildings to survive multiple earthquakes. Nevertheless, the safety boundary defined by $CI + WI = 0.25\%$ has been vetted by hundreds of buildings, some with poor reinforcement detailing and small deformation capacities, and despite the simplicity of the index, this threshold has been shown to separate reliably buildings suffering severe damage from buildings suffering less damage [12].

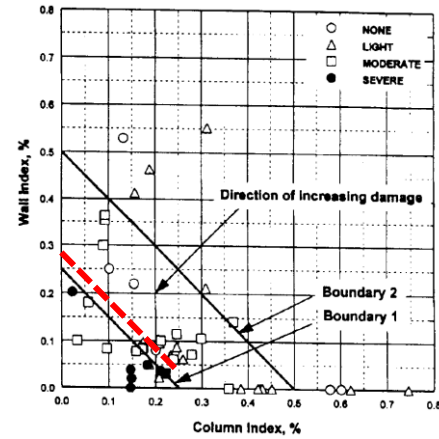


Figure 6: Application of Hassan index to institutional buildings surveyed after 1992 Erzincan Earthquake.
Note: Figure is taken from Hassan [11].

$$CI = \frac{0.5 \sum A_{RC_col}}{\sum A_{floor}} \quad (1)$$

$$WI = \frac{\sum A_{RCw} + 0.1 \sum A_{Masonryw}}{\sum A_{floor}} \quad (2)$$

$$WI + CI = 0.25\% \quad (3)$$

where,

CI = Column index

WI = Wall index

A_{RC_col} = Cross-sectional area of RC columns on ground floor

A_{RCw} = Cross-sectional area of RC shear walls on ground floor

$A_{Masonryw}$ = Cross-sectional area of masonry infill walls on ground floor

A_{floor} = Floor area of typical story

Because the Hassan index is the ratio of cross-sectional areas of vertical members and total floor area, the index cannot be compared directly with base shear strength. Nevertheless, it can be interpreted as a proxy for base shear strength given the following modifications: 1) multiply the numerator, cross-sectional area of all lateral-force resisting elements, by a constant shear strength (expressed as shear stress) and 2) multiply the denominator, total floor area, by the unit weight of a building (expressed as weight per area).

As an example, and based on the stated assumptions, a building at the safety boundary with a Hassan index of 0.25% can be interpreted to have a base shear coefficient of 0.5 given the following: a constant shear stress of 2 MPa for all vertical members and a unit weight of building of 10 kN/m². This approximation for estimating a rough base shear coefficient is not accurate as shear strengths of RC columns, RC shear walls, and masonry infill walls deviate from 2 MPa, but it does serve

as a tool to compare lateral strengths of similar buildings with and without retrofits. This example, which showed a building at the safety boundary to have a base shear coefficient of 0.5, mirrored typical Japanese buildings dominated by shear walls, designed to remain elastic with minimum base shear coefficients of 0.55, and observed to have good performance following major earthquakes since 1981 [13-14].

Some retrofitted buildings were observed to have steel braces, and a procedure for transforming the brace into an equivalent shear wall is explained next.

Transforming Steel Braces into Equivalent RC Shear Walls

To account for the increase in Hassan index provided by steel braces, a procedure for transforming the steel brace into an ‘equivalent’ RC shear wall (Figure 7) is proposed. The fundamental concept is that the lateral strength of the steel brace is equal to the lateral strength of an equivalent RC shear wall (Eq. 4) where both have the same span length. Because steel braces are typically installed inside a steel frame anchored to sides of RC columns, the equivalent wall is assumed to have two boundary columns. The assumed shear strength for an RC shear wall with two boundary columns as given in the Japanese Seismic Capacity evaluation methods is 2 MPa [14]. Because the brace also has two boundary columns, the same value of 2 MPa is assumed for the shear strength of steel brace acting as an equivalent wall (Eq. 5).

By equating lateral strength, the thickness of an equivalent RC shear wall is computed (Eq. 6), and an equivalent wall index is estimated as the product of span length and thickness of wall (Eq. 7). This equivalent wall index is added to the original wall index of the structure before retrofitting (Eq. 8). For x-braces with two diagonal members, only the tension brace is considered here as a conservative estimate. Based on the 5 surveyed buildings with steel braces, thicknesses of equivalent RC shear walls varied between 10-25 cm. In cases where dimensions of steel braces are not available, using the average thickness of 15 cm may be a reasonable estimate.

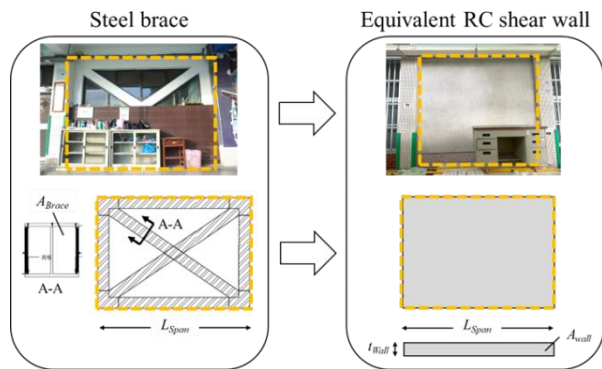


Figure 7: Transforming steel brace into an equivalent RC shear wall.

$$V_{Brace} = V_{Eq Wall} \quad (4)$$

$$\sigma_{Brace} \times A_{Brace} = \sigma_{Eq Wall} \times A_{Eq Wall} \\ = 2 \text{ MPa} \times t_{Eq Wall} \times L_{Span} \quad (5)$$

$$t_{Eq Wall} = \frac{\sigma_{Brace} \times A_{Brace}}{2 \text{ MPa} \times L_{Span}} \quad (6)$$

$$W_{I_{Eq Wall}} = \frac{\sum(t_{Eq wall} \times L_{Span})}{\sum A_{floor}} \quad (7)$$

$$W_{I_{RET}} = W_{I_{Eq Wall}} + W_I \quad (8)$$

where,

V_{Brace}	=	Lateral strength of steel brace
σ_{Brace}	=	Yield stress of steel brace
A_{Brace}	=	Cross-sectional area of steel brace
$V_{Eq Wall}$	=	Lateral strength of equivalent RC shear wall
$\sigma_{Eq Wall}$	=	Shear strength of equivalent RC shear wall
$A_{Eq Wall}$	=	Cross-sectional area of equivalent RC shear wall
$t_{Eq Wall}$	=	Thickness of equivalent RC shear wall
L_{Span}	=	Span length where steel brace is installed
$W_{I_{Eq Wall}}$	=	Wall index of equivalent RC shear wall
$\sum A_{floor}$	=	Total floor area of building
$W_{I_{Ret}}$	=	Wall index after retrofitting
W_I	=	Wall index before retrofitting

An overview of the increase in capacity provided by retrofitting installed in Taiwanese buildings is discussed next.

Increase in Seismic Capacity Provided by Retrofitting

To quantify amounts of retrofitting installed in Taiwanese buildings, Hassan indices are compared before and after retrofitting for the 18 surveyed buildings (Figure 8). For each building, a single Hassan index based on the column index and the minimum wall index of the two orthogonal building directions is discussed here.

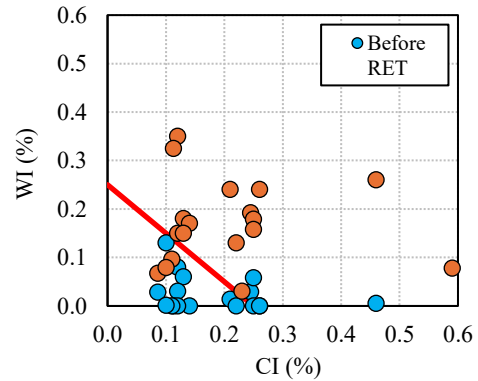


Figure 8: Comparison of Wall Indices (WI) and Column Indices (CI) of 18 buildings before and after retrofitting.
Note: “RET” indicates structural properties after retrofitting.

In general, buildings before retrofitting had Hassan indices below the safety boundary ($CI + WI < 0.25\%$), and buildings after retrofitting had indices near or above the safety boundary. Before retrofitting the Hassan index was 0.20% on average and after retrofitting, it increased to 0.37% suggesting that the base shear strength of the retrofitted buildings nearly doubled (Figure 9). None of the retrofitted buildings had more than minor damage. Each of the mentioned trends suggested that the Hassan index may serve as a valuable tool for deciding preliminary amounts of retrofitting and as a reasonable threshold for minimum base shear strength. This procedure would be similar to methods described in the Japanese Seismic Design Code which uses the “Is index” as a threshold for minimum lateral strength and stiffness [14].

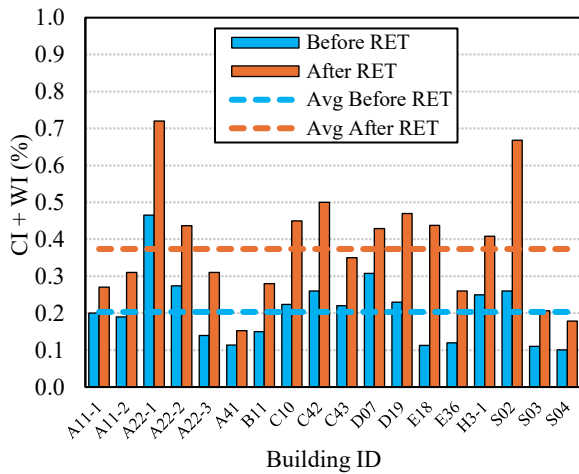


Figure 9: Comparison of Hassan Indices before and after retrofitting. Note: “RET” indicates structural properties after retrofitting.

The increase in Hassan index, defined as the difference between Hassan index after retrofitting and Hassan index before retrofitting, with number of stories is shown in Figure 10. To reduce scatter, the increase in Hassan index of the ‘average’ building was computed as the arithmetic mean for buildings with the same number of stories. In general, buildings with more stories had smaller increases in Hassan index. This negative trend, estimated as a reduction in Hassan index of 0.015% for each additional story, was influenced by the calculation method used for CI and WI indices where buildings with retrofits in the first story only are normalized by total floor area, which increases with number of stories. The retrofitted buildings with the smallest increases in Hassan index were a result of at least one of the following factors: (1) the structure had 6 or more stories (Buildings A11, A41, S03, and S04), or (2) additional shear walls were applied in only one building direction (Building C43).

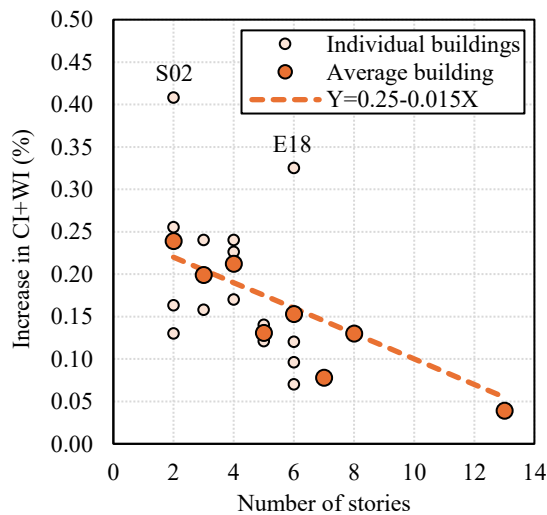


Figure 10: Variation of increase in Hassan index with number of stories.

The buildings which had the largest increases in Hassan index had no more than 4 stories and large amounts of RC shear walls were applied in both directions. Nevertheless, the two buildings with the largest increases in Hassan index (S02 and E18) were exceptions. One 2-story school building (Building S02) had a large amount of RC column jacketing work done producing an increase in CI exceeding 0.4%. One 6-story residential building (Building E18) had many shear walls and wing walls with

thicknesses between 30-50 cm installed in both building directions producing an increase in minimum WI exceeding 0.3%.

To investigate the effects of retrofitting applied to buildings on earthquake damage, comparisons of surveyed buildings with and without retrofitting with similar typology and characteristics are investigated next.

CASE STUDIES

This section summarizes case studies of pairs of surveyed buildings with similar characteristics including structural framing, geographical locations, earthquake demands, and dynamic properties [10]. Each case study consisted of one building with no retrofitting observed to have severe or moderate damage and one building with retrofitting observed to have minor or no damage. The objective was to highlight examples of low-cost retrofitting systems observed to be robust enough to 1) prevent severe structural damage and 2) reduce disruption to functionality and normal operations for each strengthened building. ‘Low-cost retrofitting’ used here implied a minimal amount of strengthening which increased seismic capacity (as interpreted by Hassan index) such that the sum of column and minimum of wall indices in orthogonal directions reached near or exceeded the safety boundary. Two case studies of buildings in Hualien city and one in New Taipei City are discussed next as detailed in Table 1.

Table 1: Overview of case studies.

Case Study	1	2	3
Location	Hualien City	Hualien City	New Taipei City
Building type	School buildings	Residential building, hotel	Mixed-use building
Building with no retrofit	S01	C05	E35
Building with retrofit	C43	A11	E36
Retrofit program	School building	Commercial building	Private residential building: Plan A
Retrofitting system	RC shear walls	Steel frames with steel braces	RC shear walls, RC column jacketing

Case Study 1: School Buildings in Hualien City

Description of Buildings

General information of the school buildings compared in Case Study 1 is given in Table 2 and additional details are included in the Appendix. The two school buildings surveyed in Hualien City were low-rise structures with regular plans and prior to retrofitting each had no full-height walls oriented in the x-direction. Building S01 had no retrofitting and was demolished after the earthquake due to severe damage (Figure 11). Building C43 had retrofits applied in 2010 (before the 2024 Hualien Earthquake) and suffered minor damage after the earthquake (Figure 12).

Table 2: Summary of buildings in Case Study 1.

Building ID	S01	C43
Year of construction	1990	1984
Number of stories	4	2
Structural type ¹	RC frame, RC shear wall, URM infill wall	RC frame, URM infill wall
Building use	Girls' high school	Elementary school
Profile (vertical) irregularity ²	First-story height is 1.3 times taller than upper story height	None
Plan (horizontal) irregularity ²	Building has no full-height walls in x-direction	Building has no full-height walls in x-direction
Seismic vulnerability ²	<ul style="list-style-type: none"> ◦ Soft story at ground floor ◦ Frame along x-axis is flexible and weak relative to frame with walls along y-axis 	Frame along x-axis is flexible and weak relative to frame with walls along y-axis
Retrofit type and issue addressed ²	Building C43 had RC shear walls added in x-direction to: <ol style="list-style-type: none"> 1) increase lateral strength and stiffness 2) reduce variation between initial periods in orthogonal building directions 	

¹ Prior to retrofitting

² Refer to Figure 11-Figure 15 for elevation views, plan views, orientation of x- and y-directions for buildings in Case Study 1



Figure 11: Building S01 without retrofitting.
Note: Figure is taken from Shegay [10].



Figure 12: Building C43 with retrofitting.

Geographic Location and Earthquake Demand

Buildings S01 and C43 were located within 2 km of one another (Figure 13) and each was no farther than 1.5 km from the nearest strong ground motion recording station, HWA019 [1]. Because of their proximity to the station, both buildings likely experienced earthquake demands similar to those obtained from Station HWA019 [16]. Buildings and station were within 15 km of the epicentral location of the mainshock. Measured ground motion parameters indicated intense shaking based on approximate values of PGA = 0.45 g, PGV = 50 cm/s, and PGD = 10 cm. Refer to the Appendix for additional information about stations and earthquake demands.

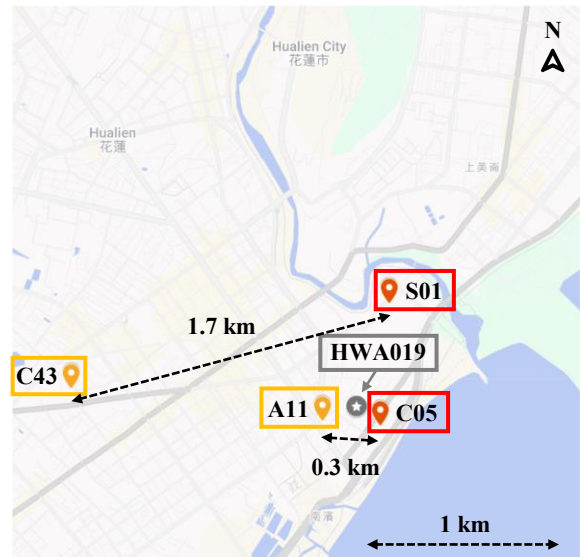


Figure 13: Map showing locations of building pairs and strong ground motion recording station in Hualien City.
Note: Figure is adapted from Google Maps [16].

Structural Layout

Building S01 was a 4-story structure with 14 spans in the x-direction and 3 spans in the y-direction (Figure 14). Typical column sizes were 45×60 cm (x, y). Height of first story was 1.3 times taller than upper floors. Pairs of RC shear walls oriented along the y-direction bounded stairwells and divided them from classrooms at each end. Masonry infill walls also were oriented along the y-direction in multiple locations serving as partitions between classrooms. No full-height walls spanned along the x-direction.

Building C43 was a 2-story structure with a basement, had 27 spans in the x-direction and 1 span in the y-direction in addition to a balcony corridor used to access classrooms (Figure 15). Typical column sizes were 30×40 cm and masonry infill walls oriented along the y-direction served as partitions between classrooms. Prior to retrofitting, there were no RC shear walls in either direction and no full-height masonry infill walls in the x-direction.

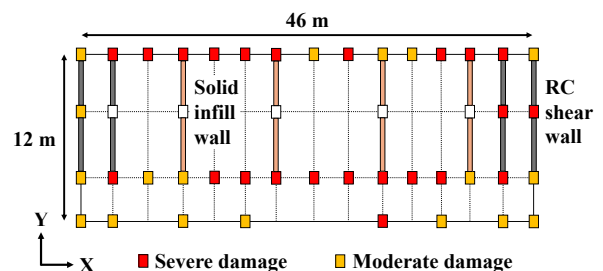


Figure 14: Building S01 structural plan.

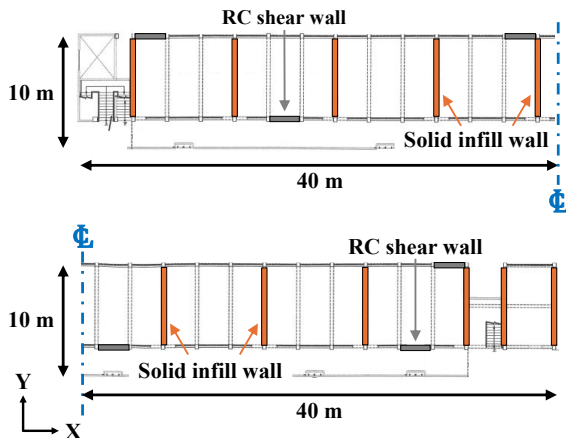


Figure 15: Building C43 structural plan.

Seismic Vulnerability

Both buildings had seismic vulnerabilities consisting of stiff frames with walls in one direction (y-axis) and flexible frames with no walls in the orthogonal direction (x-axis). Building S01 had a soft story and weak, flexible frames in the x-direction relative to frames oriented in the y-direction which had reinforced concrete shear walls. Building C43 also had weak and flexible frames in the x-direction relative to frames with full-height masonry infill walls in the y-direction.

Retrofitting

Retrofitting applied to Building C43 consisted of six 25 cm-thick, 3-m long RC shear walls added to bays running along the x-axis in first and second stories and eight RC shear walls with the same dimensions added to bays along x-axis in basement (Figure 16). Walls increased lateral stiffness and lateral strength in x-direction to become more comparable to y-direction.



Figure 16: RC shear walls added to Building C43.

Initial First-Mode Period

Initial first-mode periods for each building in both directions were calculated assuming that the building response was dominated by either frames or walls. To determine which

system governed the response of the building, periods were estimated for both systems and the minimum period was selected to represent building response. Periods for frame systems were estimated using Rayleigh's method. Periods for wall systems were estimated assuming an idealized cantilever with uniformly distributed mass and stiffness along its height. The total mass of a building was estimated as the product of total floor area (given in Appendix) and unit weight building of 150 psf (7.2 kN/m²) divided by gravity, assumed to represent typical low-rise reinforced concrete buildings.

Prior to retrofitting, both buildings were flexible in their x-direction and stiff in their y-direction producing large differences between periods in orthogonal directions (Table 3). Periods of frames without walls (nearly 0.50 sec) were approximately 4 times longer than periods of frames with walls (nearly 0.12 sec). After adding shear walls in the x-direction of Building C43, the initial period reduced from 0.48 sec to 0.13 sec and was within 30% of the period in the y-direction effectively reducing seismic vulnerability.

Table 3: Dynamic properties and spectral demands for buildings in Case Study 1.

ID	H ¹ , m	M ² , kg (×10 ⁵)	T ³ , sec	Sa ⁴ , g	Sd ⁵ , cm	MDR ⁶ , %
S01-x	15.0	16.1	0.55	1.0	7.6	0.6
S01-y	15.0	16.1	0.14	0.54	<1	<0.1
C43-x	6.9	11.7	0.48	1.2	7.0	1.3
RET ⁷	6.9	11.7	0.13	0.53	<1	<0.1
C43-y	6.9	11.7	0.10	0.57	<1	<0.1

¹ Total height of building

² Total mass of building

³ Initial first-mode period

⁴ Spectral acceleration at a given period

⁵ Spectral displacement at a given period

⁶ Mean drift ratio (equivalent to roof drift ratio)

⁷ "RET" indicates dynamic properties after retrofitting

Response Spectra and Seismic Demand

Response spectra using the strong ground motion records obtained from Station HWA019 in Hualien, Taiwan were generated using a constant damping ratio of 5%. The acceleration and displacement spectra (Figure 17-Figure 18) suggest that demands in the EW direction are smaller than those in the NS direction by approximately 40% on average between periods of 0 and 1.5 seconds. To estimate seismic demands including roof drift ratio of the buildings, the initial first-mode period (Table 3) and the direction of the building (given in the Appendix) were used to interpolate between spectral values in NS and EW directions. Roof drift was calculated as the product of spectral displacement (Sd), participation factor (1.25 for buildings with frame-dominated response, 1.5 for buildings with wall-dominated response), and a mode shape ordinate at roof normalized to 1.0. Mean drift ratio (MDR) was calculated as roof drift divided by total building height.

For Building S01, the roof drift ratio in the direction of the frame with no walls (x-axis) was estimated to be 0.6%, but in the orthogonal direction which had RC shear walls and full-height infill walls, MDR was smaller than 0.1% indicating a large reduction in drift demand because of the presence of walls (Table 3). Building C43 had retrofits consisting of RC shear walls added along the x-axis which reduced MDR from 1.3% prior to retrofitting to smaller than 0.1% after retrofitting and similar to the estimated drift demand in the orthogonal direction.

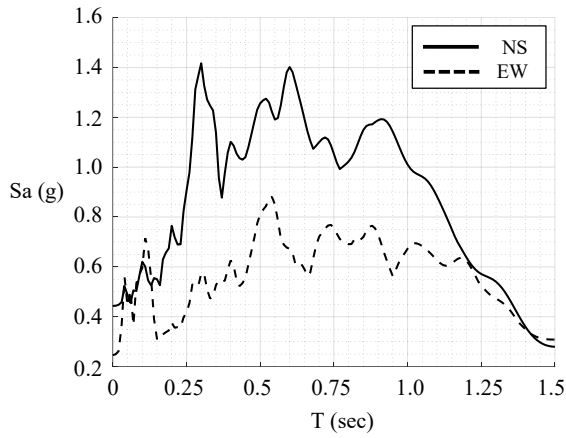


Figure 17: Acceleration spectra in Hualien City.

Note: Response spectra were generated from record obtained from Station HWA019 using a constant damping factor of 5%.

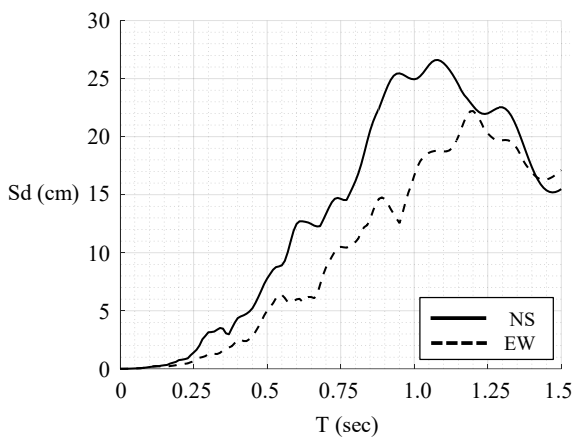


Figure 18: Displacement spectra in Hualien City.

Note: Response spectra were generated from record obtained from Station HWA019 using a constant damping factor of 5%.

Column and Wall Indices

Hassan indices plotted in figures discussed in the following sections are based on minimum wall indices only. Despite a relatively large column index of 0.23%, Building S01 had no shear walls oriented along the x-direction of the structure. This resulted in a minimum wall index of zero and a Hassan index located just below the safety boundary (Figure 19). Building C43 had a column index of 0.22% and prior to retrofitting had no walls oriented along the x-direction of the structure resulting in a Hassan index approximately equal to that of Building S01. After retrofitting, the minimum wall index increased to 0.13% and the Hassan index exceeded the safety boundary by 0.1% indicating a moderate increase in seismic capacity accomplished through simple retrofits. Refer to the Appendix for additional information about column and wall indices.

Damage

Building S01 suffered severe earthquake damage while Building C43 suffered only minor damage. Building S01 had shear failures with subsequent axial failures in most columns located next to windows (captive columns) as seen in Figure 20. Building C43 was observed to have much less damage consisting of small, inclined cracking in RC shear walls (Figure 21).

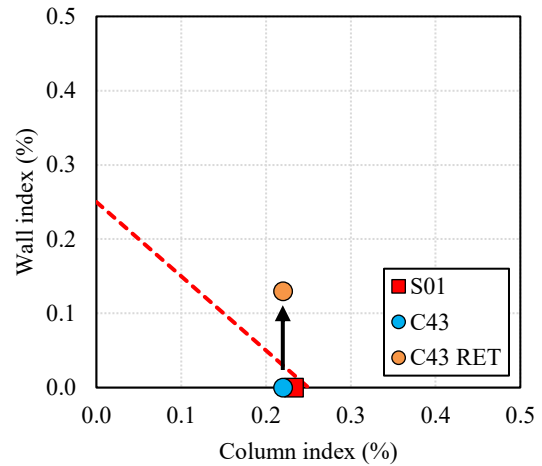


Figure 19: Case Study 1 - Hassan indices.



Figure 20: Shear and axial failure of typical column in Building S01. (Source: Shegay [10])

Figure 21: Small inclined cracks in RC shear wall in Building C43.

Structures with either RC shear walls or full-height masonry infill walls oriented in two orthogonal directions have been observed to have lower seismic vulnerability than similar buildings without walls oriented in two orthogonal directions [15]. And in this case study, the building with walls spanning both directions prevented severe damage compared with the building with a large column index but no walls in one direction.

Case Study 2: Residential and Hotel Buildings in Hualien City

Description of Buildings

Parameters of the residential and hotel buildings compared in Case Study 2 are summarized in Table 4. These two buildings surveyed in Hualien city were low-rise structures with regular plans and prior to retrofitting had few full-height walls oriented in the x-direction. Building C05 had no retrofitting, suffered moderate structural damage, and building functionality was disrupted after the earthquake including temporary displacement of tenants (Figure 22). Building A11 consisted of two similar buildings side by side (A11-1 and A11-2) separated by a seismic expansion joint. Both A11 Buildings had retrofits applied in 2013 and each suffered minor structural damage (Figure 23).

Table 4: Summary of buildings in Case Study 2.

Building ID	C05	A11-1, A11-2
Year of construction	1992	1967
Number of stories	5	6
Structural type ¹	RC frame, RC shear wall, URM infill wall	RC frame, RC shear wall
Building use	Residential apartments	Hotel
Profile (vertical) irregularity ²	First-story height is 1.25 times taller than upper story height	First-story height is 1.2 times taller than upper story height
Plan (horizontal) irregularity ²	Building has no full-height walls bounded between two columns in x-direction	Walls were concentrated along one building edge
Seismic vulnerability ²	<ul style="list-style-type: none"> ◦ Soft story at ground floor ◦ Frame along x-axis is flexible and weak relative to frame with walls along y-axis 	<ul style="list-style-type: none"> ◦ Soft story at ground floor ◦ Asymmetric orientations of walls producing torsional response
Retrofit type and issue addressed ²	A11 Buildings had steel frames with buckling-restrained braces added in x- and y- directions to: <ol style="list-style-type: none"> 1) reduce torsional response 2) increase lateral strength and stiffness 3) reduce variation between initial periods in orthogonal building directions 	

¹ Prior to retrofitting

² Refer to Figure 22-Figure 25 for elevation views, plan views, orientation of x- and y-directions for buildings in Case Study 2



Figure 22: Building C05 without retrofitting.

Geographic Location and Earthquake Demand

Buildings C05 and A11 were located within 0.3 km of one another (Figure 13) and each was no farther than 0.2 km from the nearest strong ground motion recording station, HWA019 [1]. Because of their proximity to the station, buildings likely experienced earthquake demands similar to those obtained from Station HWA019 [16]. Buildings and station were within 15 km

of the epicentral location of the mainshock. Measured ground motion parameters indicated intense shaking based on approximate values of PGA = 0.45 g, PGV = 50 cm/s, and PGD = 10 cm. Refer to the Appendix for additional information about stations and earthquake demands.



Figure 23: Building A11 with retrofitting.

Structural Layout

Building C05 was a 5-story structure with 5 spans in the x-direction and 3 spans in the y-direction (Figure 24). Typical column sizes were 35×60 cm. RC shear walls and masonry infill walls were oriented along the y-direction. One RC shear wall oriented in the x-direction was bounded on one side by stairwell wall and free at the other end. There were no full-height masonry walls oriented in the x-direction.

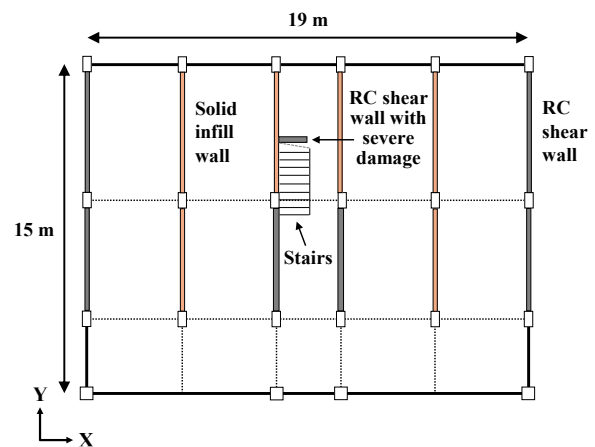


Figure 24: Building C05 structural plan.

Building A11 was a 6-story structure. The structure on the left side of the building (A11-1) had 3 spans in the x-direction and 3 to 4 spans in the y-direction. The structure on the right side of the building (A11-2) had 2 spans in the x-direction and 3 spans in the y-direction. Typical columns sizes were 45×45 cm (Figure 25). Prior to retrofitting RC shear walls were oriented along both x- and y-directions. There were no full-height masonry infill walls in either building.

Seismic Vulnerability

Both buildings had seismic vulnerabilities consisting of soft stories at the ground floor. Building C05 had weak, flexible frames in the x-direction relative to stiffer frames oriented in the y-direction consisting of reinforced concrete shear walls and masonry infill walls. Building A11 had asymmetric orientations of RC shear walls located along one building edge increasing its torsional response.

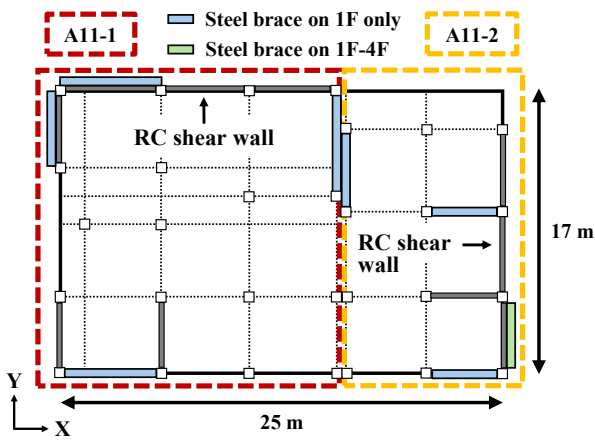


Figure 25: Building A11 structural plan.

Retrofitting

Retrofitting applied to Building A11 consisted of installing steel frames with buckling-restrained braces along both building directions to first story only (Figure 26). One bay had braces added to stories 1-4. Steel frames with braces mitigated seismic vulnerability by reducing torsional response and increasing lateral strength and stiffness.



Figure 26: Steel braces added to Buildings A11-1 and A11-2.

Initial First-Mode Period

Building C05 was flexible in its x-direction and stiff in its y-direction producing an initial period in the x-direction (0.72 sec) approximately 3 times longer than the initial period in the y-direction (0.25 sec) as shown in Table 5. Prior to retrofitting, Building A11 had initial periods estimated to be between 0.6 and 1.1 seconds with relative differences in periods in orthogonal directions between 30-60%. After adding steel frames with braces in both directions, periods reduced to values between 0.50 and 0.65 seconds and variations in periods in orthogonal directions were within 30% of each other effectively reducing seismic vulnerability.

Response spectra and seismic demand

Response spectra generated for Case Study 2 are the same spectra discussed for Case Study 1 (Figure 17-Figure 18). To estimate seismic demands including roof drift ratio of the buildings, the initial first-mode period (Table 5) and the direction of the building (given in the Appendix) were used to interpolate between spectral values in NS and EW directions.

For Building C05, in the direction of the frame with no walls (x-axis) the roof drift ratio was estimated to be 0.9%, but in the orthogonal direction which had RC shear walls and full-height infill walls, the MDR was smaller than 0.1%. For Building A11,

there was a smaller difference in demands between orthogonal directions and a less pronounced influence of retrofits on reduction in demand. Nevertheless, roof drift ratios were on average larger than 1% prior to retrofitting and after retrofitting, MDR reduced to between 0.5% and 0.7%.

Table 5: Dynamic properties and spectral demands for buildings in Case Study 2.

ID	H ¹ , m	M ² , kg (×10 ⁵)	T ³ , sec	Sa ⁴ , g	Sd ⁵ , cm	MDR ⁶ , %
C05-x	16.8	10.2	0.72	0.93	11.5	0.9
C05-y	16.8	10.2	0.25	0.61	0.87	<0.1
A11-1-x	19.3	10.9	0.64	0.95	9.3	0.7
RET ⁷	19.3	10.9	0.50	1.0	6.0	0.5
A11-1-y	19.3	10.9	0.98	0.85	19.8	1.3
RET ⁷	19.3	10.9	0.65	0.93	9.4	0.7
A11-2-x	19.3	10.9	1.1	0.77	22.6	1.5
RET ⁷	19.3	10.9	0.63	0.97	9.2	0.7
A11-2-y	19.3	10.9	0.83	0.89	14.7	1.0
RET ⁷	19.3	10.9	0.60	1.0	8.7	0.7

- ¹ Total height of building
- ² Total mass of building
- ³ Initial first-mode period
- ⁴ Spectral acceleration at a given period
- ⁵ Spectral displacement at a given period
- ⁶ Mean drift ratio (equivalent to roof drift ratio)
- ⁷ "RET" indicates dynamic properties after retrofitting

Column and Wall Indices

Building C05 had a relatively large column index of nearly 0.2% but the minimum wall index of 0.01% was observed to be small because of a single RC shear wall oriented along the x-axis (Figure 27) producing a Hassan index below the safety boundary indicating seismic vulnerability. Prior to retrofitting, Building A11 had column indices between 0.12-0.13% and wall indices between 0.06-0.15% producing Hassan indices near the safety boundary indicating small to moderate seismic vulnerability. After retrofitting, steel braces transformed into equivalent shear walls (estimated using Eqs. 4-8) increased minimum wall indices to values between 0.15-0.30%. Refer to the Appendix for additional information about column and wall indices.

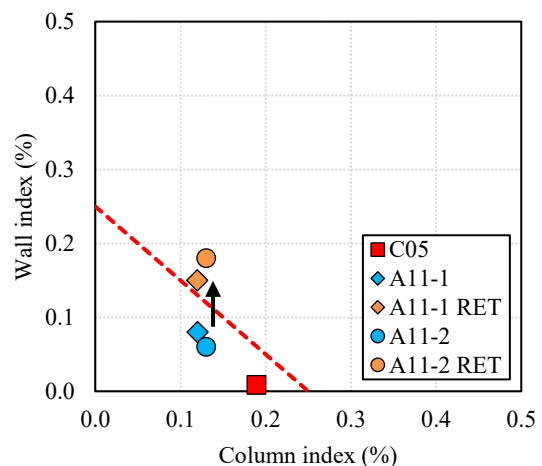


Figure 27: Case Study 2 - Hassan indices.

Damage

In the weaker and more flexible x-direction of Building C05, shear failure occurred in the single RC shear wall (Figure 28). This wall was located adjacent to a door opening increasing its vulnerability. In the stronger y-direction of the building, small, inclined cracks were observed in RC shear walls and minor concrete spalling occurred at joints of stairs. Masonry infill walls in the y-direction had moderate to severe damage but did not collapse (Figure 29).

Building A11 was observed to have minor damage consisting of small, inclined cracking on exterior columns (Figure 30). Additional cracking and damage at the seismic joint may have been caused by pounding between Buildings A11-1 and A11-2 (Figure 31).



Figure 28: Shear failure in RC shear wall of Building C05.

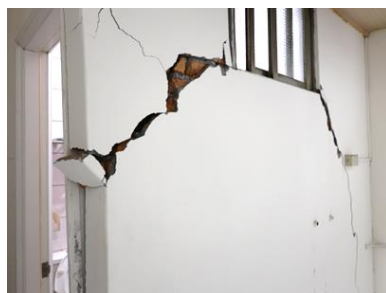


Figure 29: Large inclined cracks in masonry walls of Building C05.

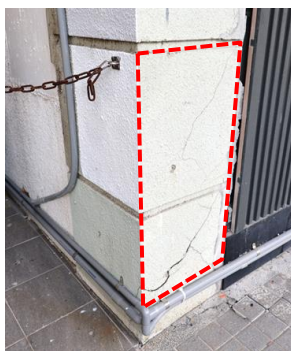


Figure 30: Inclined cracks at base of exterior column of Building A11-1.



Figure 31: Cracking at seismic joint between Buildings A11-1, A11-2.

Case Study 3: Mixed Commercial and Residential Buildings in New Taipei City

Description of Buildings

Parameters of the mixed-use buildings with commercial entities in first stories and residential apartments in upper stories compared in Case Study 3 are summarized in Table 6. These two buildings were low-rise structures surveyed in New Taipei City with regular plans. Prior to retrofitting both buildings had no full-height walls oriented in the x-direction, had nearly identical Hassan indices well below the safety boundary, and longitudinal reinforcement in columns were reported or observed to have corrosion. Building E35 had no retrofitting, suffered severe structural damage, and building functionality was disrupted after the earthquake including reduced service hours provided by the bank on the first floor (Figure 32). Building E36 had retrofits finished in January 2024 [17], three months before the mainshock, and suffered minor structural damage (Figure 33).

Table 6: Summary of Case Study 3.

Building ID	E35	E36
Year of construction	1953	1989
Number of stories	5	5
Structural type ¹	RC frame, RC shear wall, URM infill wall	RC frame, RC shear wall, URM infill wall
Building use	Commercial on first floor, residential units on upper floors	Commercial on first two floors, residential units in upper stories
Profile (vertical) irregularity ²	First-story height is 1.25 times taller than upper story height	First-story height is 1.15 times taller than upper story height
Plan (horizontal) irregularity ²	Building has no full-height walls in x-direction	Building has no full-height walls in x-direction
Seismic vulnerability ²	<ul style="list-style-type: none"> ◦ Soft story at ground floor ◦ Frame along x-axis is flexible and weak relative to frame with walls along y-axis 	<ul style="list-style-type: none"> ◦ Frame along x-axis is flexible and weak relative to frame with walls along y-axis
Retrofit type and issue addressed ²	Building E36 had RC column jacketing and RC shear walls added in x-direction to: <ol style="list-style-type: none"> 1) increase lateral strength and stiffness 2) reduce variation between initial periods in orthogonal building directions 	

¹ Prior to retrofitting

² Refer to Figure 32-Figure 36 for elevation views, plan views, orientation of x- and y-directions for buildings in Case Study 3

Geographic Location and Earthquake Demand

Earthquake demands observed in New Taipei City were between 25-50% of the demands observed in Hualien City, as the epicenter was within 15 km of Hualien but farther than 100

km from Taipei (refer to the Appendix for more detailed information). Despite lower-intensity shaking, Buildings E35 and E36 served as a good comparison study, as they were located within 5 km of one another (Figure 34) and each was no farther than 3 km from the nearest strong ground motion recording station, TAP037 [1]. Because of their proximity to the station, buildings likely experienced earthquake demands similar to those obtained from Station TAP037 [18]. Measured ground motion parameters indicated moderate shaking based on approximate values of $PGA = 0.1\text{ g}$, $PGV = 20\text{ cm/s}$, and $PGD = 5\text{ cm}$.



Figure 32: Building E35 without retrofitting.



Figure 33: Building E36 with retrofitting.

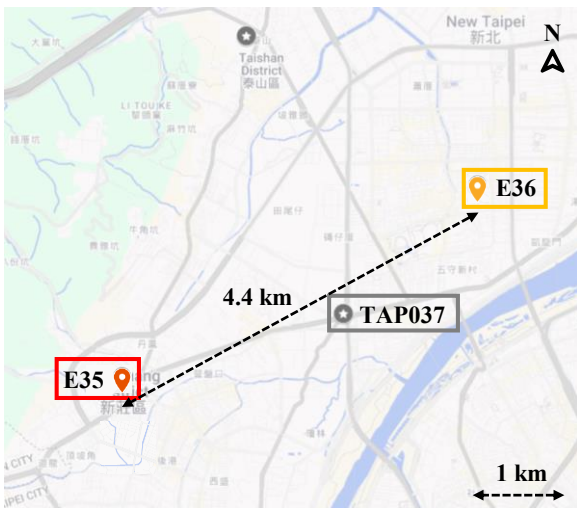


Figure 34: Map showing locations of building pair and strong ground motion recording station in New Taipei City. Note: Figure is adapted from Google Maps [18].

Structural Layout

Building E35 was a 5-story structure with 4 spans in the x-direction and 3 spans in the y-direction (Figure 35). Typical column sizes were 35×60 cm. A single RC shear wall and full-height infill walls were oriented along the y-direction and no walls were oriented along the x-direction.

Building E36 was a 5-story structure with 2 to 4 spans in the x-direction and 4 spans in the y-direction (Figure 36). Typical column sizes were 18×50 cm and 18×60 cm. Full-height masonry infill walls were oriented along the y-direction and no walls were oriented along the x-direction.

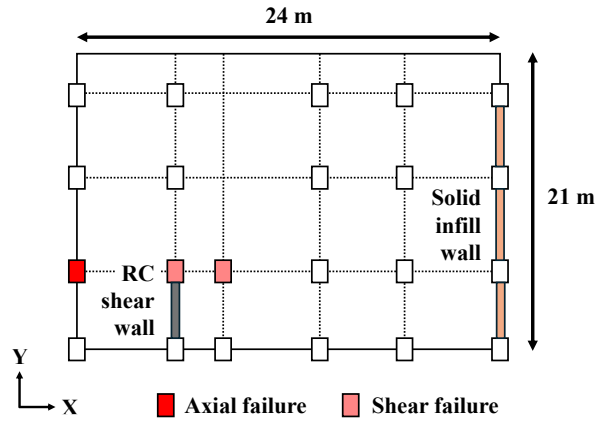


Figure 35: Building E35 structural plan.

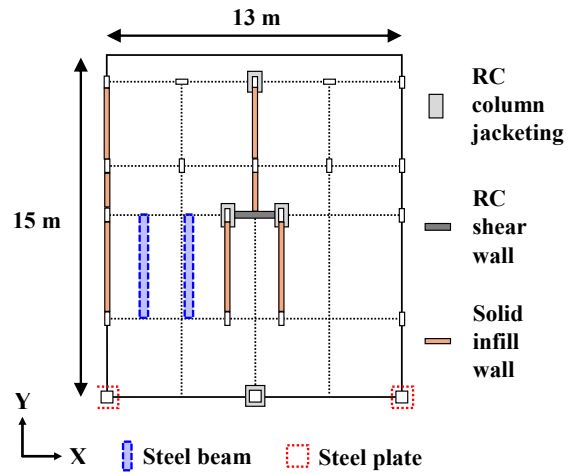


Figure 36: Building E36 structural plan.

Building E36 was located at the end of a row of buildings and is in contact with a single building on its left side. Because the structural properties of the adjacent building were not investigated, the fundamental periods and estimated spectral response of Building E36 were calculated ignoring the effect of the adjacent building.

Seismic Vulnerability

Both buildings had seismic vulnerabilities consisting of soft stories at the ground floor and stiff frames with walls (y-axis) orthogonal to flexible frames with no walls (x-axis).

Retrofitting

Retrofitting applied to Building E36 comprised two components which increased seismic capacity and two components which addressed serviceability issues. The two retrofits which increased seismic capacity by increasing lateral

stiffness and strength consisted of RC shear walls and RC column jacketing. Wall retrofits included a 30 cm-thick, 2.4 m-long RC shear wall added to the first story and 12 cm-thick, 2.4 m-long RC shear walls added to stories 2-4 oriented along the x-axis. Column retrofits included RC column jacketing applied to boundary columns of the additional shear wall added to first story (Figure 37) and exterior columns of stories 1-2.



Figure 37: Exterior column with RC jacketing.

The two retrofits which addressed serviceability issues consisted of encasing exterior columns with steel plates to reduce rate of corrosion affecting column longitudinal bars and adding steel beams to soffit of second level floor slab in y-direction to reduce vertical deformations. Both retrofits were designed to increase the service life of structure.

Initial first-mode period

Prior to retrofitting, both buildings were flexible in their x-direction and stiff in their y-direction producing large differences between periods in orthogonal directions (Table 7). Periods of frames without walls were between approximately 1.5 to 2 times longer than periods of frames with walls. After adding shear walls in the x-direction of Building E36, the initial period reduced from nearly 1 sec to 0.6 sec and was within 15% of the period in the y-direction effectively reducing seismic vulnerability.

Table 7: Dynamic properties and spectral demands for buildings in Case Study 3.

ID	H ¹ , m	M ² , kg (×10 ⁵)	T ³ , sec	Sa ⁴ , g	Sd ⁵ , cm	MDR ⁶ , %
E35-x	18.3	18.5	1.3	0.25	10.5	0.7
E35-y	18.3	18.5	0.66	0.24	2.5	0.2
E36-x	15.5	6.6	0.97	0.34	7.8	0.6
RET ⁷	15.5	6.6	0.62	0.19	1.7	0.1
E36-y	15.5	6.6	0.66	0.13	1.5	0.1
RET ⁷	15.5	6.6	0.53	0.15	1.0	<0.1

¹ Total height of building

² Total mass of building

³ Initial first-mode period

⁴ Spectral acceleration at a given period

⁵ Spectral displacement at a given period

⁶ Mean drift ratio (equivalent to roof drift ratio)

⁷ "RET" indicates dynamic properties after retrofitting

Response Spectra and Seismic Demand

Response spectra using the strong ground motion records obtained from Station TAP037 in New Taipei City, Taiwan were generated using a constant damping ratio of 5% (Figure 38-Figure 39). The displacement spectra suggest that demands in the NS direction are smaller than those in the EW direction by approximately 10% on average between periods of zero and 1.5 seconds (Figure 39). There is more scatter for acceleration spectra but demands in the EW direction tended to be larger than demands in the NS direction (Figure 38). To estimate seismic demands including roof drift ratio of the buildings, the initial first-mode period (Table 7) and the direction of the building (given in the Appendix) were used to interpolate between spectral values in NS and EW directions.

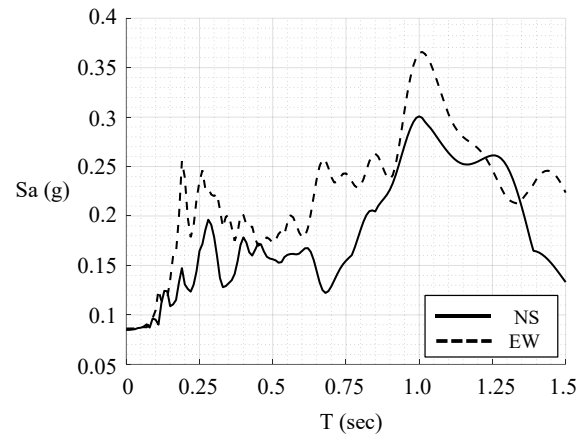


Figure 38: Acceleration spectra in New Taipei City.

Note: Response spectra were generated from record obtained from Station TAP037 using a constant damping factor of 5%.

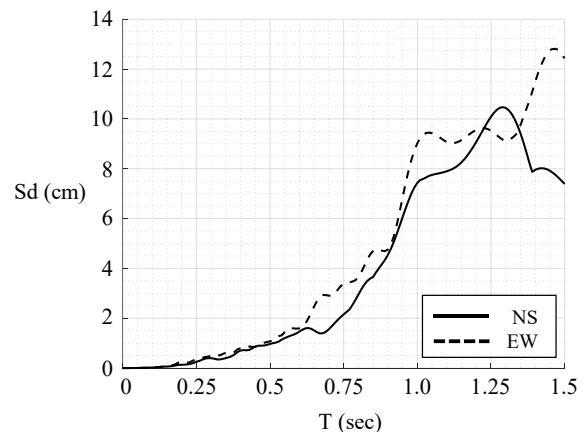


Figure 39: Displacement spectra in New Taipei City.

Note: Response spectra were generated from record obtained from Station TAP037 using a constant damping factor of 5%.

For Building E35, in the direction of the frame with no walls (y-axis) the roof drift ratio (MDR) was estimated to be 0.7%, but in the orthogonal direction which had RC shear walls and full-height infill walls, the MDR was approximately 0.2% indicating a large difference in drift demand because of the presence of walls. Building E36 had retrofits consisting of RC column jacketing and RC shear walls added along the x-axis which reduced MDR from nearly 0.6% prior to retrofitting to no more than 0.1% after retrofitting, again highlighting the effect of additional walls on large reductions in drift demand.

Column and Wall Indices

Prior to retrofitting, Buildings E35 and E36 had column indices of approximately 0.1% and no walls in the y-direction, indicating significant seismic vulnerability (Figure 40). After retrofitting, column indices of Building E36 more than doubled and the minimum wall index increased to 0.04% resulting in a Hassan index just above the safety boundary. Refer to the Appendix for additional information about column and wall indices.

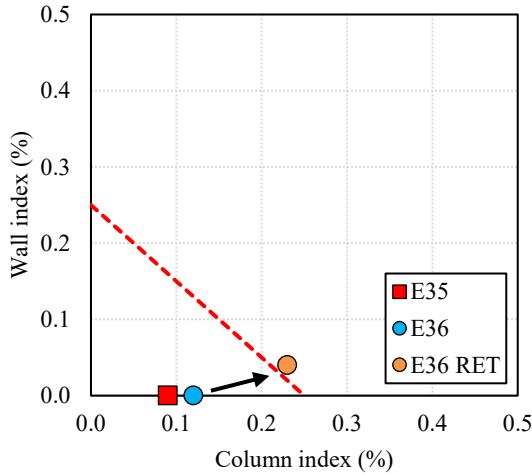


Figure 40: Case Study 3 - Hassan Indices.

Damage

After the earthquake, Building E35 had one column with axial failure and two columns with shear failures (Figure 41). Steel columns were used to shore the soffit of the second level floor slab around the column with axial failure which showed signs of severe corrosion and significant section loss for both longitudinal and transverse reinforcement. Masonry infill walls were observed to have inclined cracks wider than 1 mm. The building was evacuated except for the bank located on the first floor which operated one hour in the morning and one hour in the afternoon.



Figure 41: Axial failure of column consisting of buckling of longitudinal reinforcing bars in Building E35.

Building E36 was observed to have minor damage consisting of 0.5-mm wide inclined cracks observed in columns with RC jacketing serving as boundary columns for the retrofitted shear wall in the first story (Figure 42). No disruption to commercial or residential units was observed or reported.

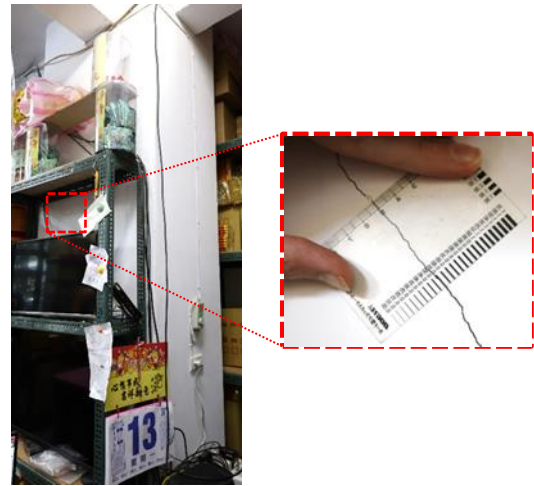


Figure 42: Boundary column of added shear wall with RC jacketing and 0.5-mm wide shear cracks in Building E36.

Summary

18 buildings surveyed in Hualien City, Taipei City, and New Taipei City had retrofitting applied before the 2024 earthquake, suffered no more than minor damage, and remained operational after the earthquake and at the time of the visual assessment surveys. In general, retrofitting increased Hassan indices by a factor approaching two suggesting that the base shear strength of buildings nearly doubled. For low-rise buildings of no more than 6 stories, the amount of retrofitting was enough to increase Hassan indices to be located above the safety boundary. Although the Hassan index method seems to be a reasonable crude approach for deciding preliminary amounts of retrofitting, it may be advantageous, at least from a cost perspective, to consider additional design methodologies for retrofit systems when targeting buildings with more than 6 stories.

COST ESTIMATION

Case Study of Private Building with ‘Plan A’ Retrofit

A cost assessment was performed by analyzing the weak-story retrofitting system installed in Building E36. In general, the construction time required to retrofit a building designed according to Plan A varied between 3-6 months, and the construction time for Building E36 was 4 months [17]. The cost data discussed here was supplied from cost schedules provided by the structural consulting firm that designed the retrofit [19, and from T. Chiou, private correspondence, February 11, 2025].

The total cost of the retrofitting project was estimated to be approximately USD \$165,000. This cost consisted of three primary components: direct project cost, indirect project cost, and structural design fee (Table 8). The direct project costs accounted for nearly two-thirds of the total project cost and included site preparation, structural retrofitting work, structural damage repair, and restoration work. The indirect project costs accounted for no more than 15% of the total project cost and included contractor fee and vendor taxes. The combined direct and indirect costs were approximately 80% of the total project cost. The structural design fees were nearly 20% of the total project cost and included drafting structural drawings, supervision of construction work, and permitting. To compare the retrofitting cost with the rebuilding cost, a series of assumptions were made as explained next.

Based on data from the Ministry of Education of Taiwan [5], the average cost to construct a new school building is NTD \$28,000 per square meter (approximately USD \$875/m²). According to the Retrofitting Project Office for Residential

Buildings in Taiwan, the cost to build privately owned structures exceeds that of school buildings and vary significantly based on parameters including structural layout and geographical location [9,19].

Table 8: Total project cost schedule.

Cost description	Cost (USD)	Percent of total project cost (%)	Percent of rebuilding cost (%)
Site preparation	24,831	15%	2%
Structural retrofitting	83,950	51%	8%
Structural repair	2,238	1%	<1%
Restoration work	1,094	1%	<1%
Direct project cost	112,113	68%	11%
Contractor fee	11,211	7%	1%
Vendor taxes	6,382	4%	1%
Other	4,313	3%	<1%
Indirect project cost	21,906	13%	2%
Direct + Indirect cost	134,019	81%	13%
Structural design fee + Permitting	31,373	19%	3%
Total project cost	165,392	100%	17%

Note: USD \$1 is assumed to be equivalent to TWD \$32.00.

Table is based on data from Chiou [19, and private correspondence].

To make fair comparisons between retrofitting and rebuilding costs, a value of USD \$1000/m² was assumed to represent the cost to rebuild a privately owned structure (including Building E36). This lower-bound cost was 15% larger than the cost assumed for a typical school building. Because Building E36 had a total floor area of approximately 1000 m² [17], the total cost for rebuilding was estimated to be USD \$1,000,000. Based on this amount, the cost of the retrofitting system applied to Building E36, which consisted of modest amounts of RC column jacketing and additional shear walls, was approximately 17% of the rebuilding cost. Considering that this retrofitting work produced a doubling of the base shear strength, this project highlighted a low-cost and effective strengthening design applied to a typical 5-story building in Taiwan. A more detailed analysis of the structural retrofitting costs is discussed next.

The structural retrofitting cost for Building E36 was nearly 50% of total project cost (Table 9). But the amount which increased stiffness and strength (RC column jacketing, RC shear walls, and larger foundations) was smaller than 20% of total project cost. The remaining structural retrofitting cost was aimed at addressing serviceability issues including encasing exterior columns with steel plates to reduce effects of weathering, installing steel beams under the second-floor slab to prevent excessive deflections, and other fees such as temporary lavatories, temporary openings in walls, and restoration of openings after retrofit completion. Based on these data, it is plausible that installing additional RC shear walls in the weak direction of the building to double the wall index (y-axis of Figure 36) could be financed by increasing the total project cost by no more than 5% (based on the 4% value for RC shear wall retrofits indicated in Table 9).

Table 9: Structural retrofitting cost schedule.

Cost description	Cost (USD)	Percent of direct project cost (%)	Percent of total project cost (%)
RC column jacketing	10,750	10%	7%
RC shear wall	7,125	6%	4%
Foundations	12,063	11%	7%
Column encased with steel plates	5,625	5%	3%
Steel beam reinforcement	17,344	15%	11%
Temporary works, incidental costs	9,625	9%	6%
Other	21,418	19%	13%
Total Structural Retrofitting	83,950	75%	51%

Note: USD \$1 is assumed to be equivalent to TWD \$32.00.

Table is based on data from Chiou [19], and private correspondence.

Summary of Cost Data

A summary of the retrofitting costs of the private residential buildings in Taiwan with completed retrofitting systems is discussed in this section. Figure 43 shows the retrofitting costs of the 23 buildings with retrofitting systems installed to date normalized to total floor area [20]. This data included the 18 buildings discussed earlier as well as 5 additional buildings. The buildings retrofitted according to Plan A tended to have lower costs, approximately half as expensive as retrofits designed according to Plan B. Nevertheless, differences between buildings in the same plan were large because of variations in the extent of retrofitting necessary to conform to the Taiwanese seismic design code. Refer to Figure 8-Figure 10 for variations in amounts of retrofitting.

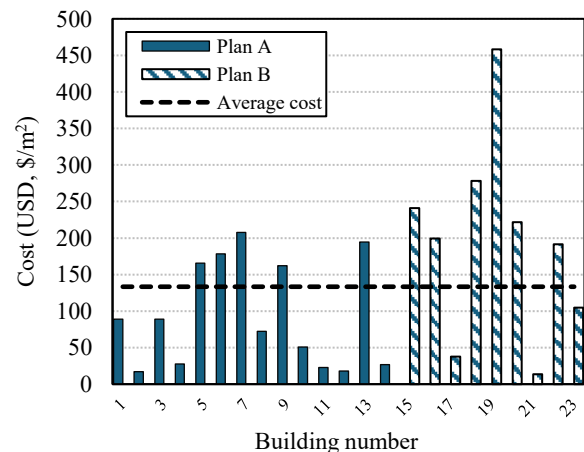


Figure 43: Comparison of retrofitting costs for private residential buildings. (Source: Chiou [20])

For both Plan A and Plan B buildings, the average retrofitting cost is approximately USD \$130/m² indicating a slight reduction compared with the retrofitting costs estimated for Building E36 (USD \$165/m²). The average retrofitting cost compared with the cost of rebuilding (USD \$1000/m²) suggests that a building can be retrofitted (at a minimum) to mitigate the weak first-story vulnerability at a cost between 10-20% of the rebuilding cost of the structure.

The cost of retrofitting may be reduced by a larger margin if there were more consistency in the retrofit approach, for example, by implementing a program-type initiative rather than dealing with individual buildings on a case-by-case basis. This was the case for the Taiwanese school retrofit program, which strengthened over 9500 buildings by focusing on simple retrofits (RC shear walls, column jacketing and wing walls) and reduced the average retrofitting cost per building to USD \$100/m² [5,20].

CONCLUSIONS

The following conclusions were made based on data obtained from surveys of Taiwan buildings with installed retrofitting systems.

1. On average, the retrofits installed in the 18 buildings surveyed in Hualien City, Taipei City, and New Taipei City comprising various methods including RC column jacketing, RC wing walls, RC shear walls, and steel frames with braces, increased the base shear strength by a factor approaching 2.
2. In general, retrofits targeted mitigation of soft and weak first stories, a well-known structural vulnerability within the Taiwanese building stock.
3. The use of Hassan indices as a crude method to determine the amount of retrofitting necessary for preventing severe earthquake damage ($CI + WI > 0.25\%$) was observed to be a reasonable preliminary evaluation for low-rise structures.
4. Pairwise case studies consisting of buildings with retrofitting with minor damage only and buildings without retrofitting suffering severe or moderate damage suggested that simple, low-cost retrofits were sufficient in preventing severe structural damage and disruption to building functionality.
5. For a 5-story mixed commercial and residential building located in Taiwan with a total floor area of 1000 square meters, the retrofitting cost was estimated to be approximately 15% of the rebuilding cost.
6. The average retrofitting cost of the 23 private residential buildings with retrofits installed to date in Taiwan was estimated to be approximately USD \$130/m². This corresponded to 10-20% of rebuilding costs indicating the practicality of increasing seismic capacities of vulnerable buildings at reasonable costs.

REFERENCES

- 1 Central Weather Administration (CWA). *Earthquake Information 019 4/3 7:58 ML 7.1 23.88N 121.57E, i.e. 13.6 km SSW of Hualien County*. <https://scweb.cwa.gov.tw/zhtw/earthquake/Imgs/EE2024040307580971019> (Accessed: 2 June 2025)
- 2 Central Weather Administration (CWA). *Earthquake Information 267 4/23 2:32 ML 6.2 23.85N 121.52E, i.e. 18.8 km SW of Hualien County*. <https://scweb.cwa.gov.tw/zhtw/earthquake/Imgs/EE2024042302324862267> (Accessed 2 June 2025)
- 3 Chiou TC *et al.* (2008). “*Field Test and Analysis for School Building Retrofitted by RC Jacketing System*”. National Center for Research on Earthquake Engineering, Taiwan, NCREE-08-033. (in *Chinese*).
- 4 Chiou TC *et al.* (2008). “*Seismic Retrofitting and Experimental Verification for Typical School Buildings*”. National Center for Research on Earthquake Engineering, Taiwan, NCREE-08-038. (in *Chinese*).
- 5 Hwang SJ *et al.* (2022). “*Seismic Evaluation and Retrofitting of School Buildings in Taiwan*”. National Center for Research on Earthquake Engineering, Taiwan, NCREE-22-017. (in *Chinese*).
- 6 Hsiao FP, Hwang SJ, Chung LL, Chien WY, Shen WC, Yeh YK (2012). “Seismic upgrading for public school buildings in Taiwan”. *The 8th International Symposium on Social Management Systems SSMS2012 – Disaster Prevention and Reconstruction Management*, Kaohsiung, Taiwan. https://ssms.jp/img/files/2019/04/sms12_0116.pdf
- 7 Chou CC *et al.* (2023). “*Seismic Design Specifications and Commentary of Buildings*”. National Center for Research on Earthquake Engineering, Taiwan, NCREE-23-022.
- 8 Ou YC *et al.* (2024). “*Reconnaissance Report of Seismic Damages Caused by the 3 April 2024 Earthquake, Hualien, Taiwan*”. National Center for Research on Earthquake Engineering, Taiwan, NCREE-24-006. (in *Chinese*).
- 9 Weng YT, Lee TY, Lin JL and Chiou TC (2024). “Promoting seismic rehabilitation strategies and legal development for private buildings”. *The Seventeenth National Conference on Structural Engineering / The Seventh National Conference on Earthquake Engineering*, Taichung City, Taiwan.
- 10 Shegay AV *et al.* (2025). “*2024 Hualien Earthquake Reconnaissance of Reinforced Concrete Buildings*”. National Center for Research on Earthquake Engineering, Taiwan, NCREE-25-004.
- 11 Hassan A and Sozen M (1997). “Seismic vulnerability assessment of low-rise buildings in regions with infrequent earthquakes”. *ACI Structural Journal*, **94**(4): 31-39.
- 12 Puranam A, Irfanoglu A, Pujol S, Chiou TC and Hwang SJ (2019). “Evaluation of seismic vulnerability screening indices using data from the Taiwan Earthquake of 6 February 2016”. *Bulletin of Earthquake Engineering*, **17**: 1963-1981. <https://doi.org/10.1007/s10518-018-0519-1>
- 13 Kusunoki K, Maida Y, Shiohara H, Pujol S, Hogan LS, Shegay AV and Kuwabara R (2025). “Preliminary report on the performance of reinforced concrete buildings during the 2024 Noto Earthquake”. *Structural Engineering International*, **35**(1): 37-45. <https://doi.org/10.1080/10168664.2024.2412581>
- 14 JBDPA (2001). “*Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings*”. The Japan Building Disaster Prevention Association, Tokyo, Japan. (in *English* and *Japanese*).
- 15 Monical J and Pujol J (2024). “A study of the response of reinforced concrete frames with and without masonry infill walls to earthquake motions”. *Structures*, **63**: (106345). <https://doi.org/10.1016/j.istruc.2024.106345>
- 16 Google Map of Hualien City. <https://www.google.com/maps/place/Hualien+City> (Accessed 24 February 2026).
- 17 Information Network for Weak-Story Seismic Retrofits for Private Buildings. *Detailed Explanation of Weak-Story Seismic Retrofits - Case 10*. <https://privatebuilding.ncree.org.tw/reinforcement-detail/#cases> (Accessed 20 March 2026)
- 18 Google Map of New Taipei City. <https://www.google.com/maps/place/New+Taipei+City> (Accessed 24 February 2026)
- 19 Chiou TC (2023). “*Weak-Story Seismic Retrofit Project for Private Buildings in 2023*”. Ministry of the Interior National Land Management Agency / National Center for Research on Earthquake Engineering, Vol. 2 (in *Chinese*). <https://privatebuilding.ncree.org.tw/download/report/>
- 20 Chiou TC (2025). “Seismic retrofitting project of private buildings in Taiwan - Policy, regulations, and progress”. *Japan-New Zealand Retrofit Project Workshop #1*, Osaka, Japan.

APPENDIX

Supplementary Data for Case Study 1

Refer to Figure 11-Figure 15 for elevation views, plan views, orientation of x- and y-directions, and geographical locations of Buildings S01 and C43.

Building Location and Directionality

ID	Distance between buildings, km	Distance from station HWA019, km	Building directionality
S01	1.7	0.5	X: N 43° E Y: N 47° W
C43			X: N 3° E Y: N 87° W

Floor Area, Column/Wall Indices, Hassan Indices

ID	Total floor area, m ²	CI, %	WI, %	CI + WI, %
S01-x	2200	0.23	0.00	0.23 ²
S01-y	2200	0.23	0.29	0.52
C43-x	1900	0.22	0.00	0.22 ²
RET ¹	1900	0.22	0.28	0.50
C43-y	1900	0.22	0.13	0.35 ³

¹ "RET" indicates structural properties after retrofitting

² Minimum Hassan index prior to retrofitting

³ Minimum Hassan index after retrofitting

Supplementary Data for Case Study 2

Refer to Figure 13 and Figure 22-Figure 25 for elevation views, plan views, orientation of x- and y-directions, and geographical locations of Buildings C05 and A11.

Building Location and Directionality

ID	Distance between buildings, km	Distance from station HWA019, km	Building directionality
C05	0.30	0.16	X: N 54° W Y: N 36° E
A11		0.19	X: N 55° E Y: N 35° W

Floor Area, Column/Wall Indices, Hassan Indices

ID	Total floor area, m ²	CI, %	WI, %	CI + WI, %
C05-x	1400	0.19	0.01	0.20 ²
C05-y	1400	0.19	0.40	0.59
A11-1-x	1500	0.12	0.11	0.23
RET ¹	1500	0.12	0.18	0.30
A11-1-y	1500	0.12	0.08	0.20 ²
RET ¹	1500	0.12	0.15	0.27 ³
A11-2-x	950	0.13	0.06	0.19 ²
RET ¹	950	0.13	0.18	0.31 ³
A11-2-y	950	0.13	0.15	0.28
RET ¹	950	0.13	0.27	0.40

¹ "RET" indicates structural properties after retrofitting

² Minimum Hassan index prior to retrofitting

³ Minimum Hassan index after retrofitting

Supplementary Data for Case Study 3

Refer to Figure 32-Figure 36 for elevation views, plan views, orientation of x- and y-directions, and geographical locations of Buildings E35 and E36.

Building Location and Directionality

ID	Distance between buildings, km	Distance from station TAP037, km	Building directionality
E35	4.4	2.6	X: N 20° E Y: N 70° W
E36		1.9	X: EW axis Y: NS axis

Building Overview

ID	Year built	Number of stories	Structural Type	Building Use
E35	1953	5	RC frame, RC shear wall, URM infill	Commercial on ground floor, residential units in upper stories
E36	1989	5	RC frame, RC shear wall, URM infill	Commercial on first two floors, residential units in upper stories

Floor Area, Column/Wall Indices, Hassan Indices

ID	Total floor area, m ²	CI, %	WI, %	CI + WI, %
E35-x	2500	0.09	0.00	0.09 ²
E35-y	2500	0.09	0.06	0.15
E36-x	900	0.12	0.00	0.12 ²
RET ¹	900	0.23	0.08	0.31
E36-y	900	0.12	0.04	0.16
RET ¹	900	0.23	0.04	0.27 ³

¹ “RET” indicates structural properties after retrofitting

² Minimum Hassan index prior to retrofitting

³ Minimum Hassan index after retrofitting

Station HWA019 in Hualien City

General Information of Recording Station

Station ID	GPS Coordinates	Distance to epicenter, km	Direction of records
HWA019	23°58'30" N 121°36'47" E	11	Vertical, NS, EW

Strong Ground Motion Parameters

Direction	PGA, g	PGV, cm/sec	PGD, cm
NS	0.44	49	10
EW	0.25	35	13

Station TAP037 in New Taipei City

General Information of Recording Station

Station ID	GPS Coordinates	Distance to epicenter, km	Direction of records
TAP037	25°02'03" N 121°26'30" E	130	Vertical, NS, EW

Strong Ground Motion Parameters

Direction	PGA, g	PGV, cm/sec	PGD, cm
NS	0.08	20	4.7
EW	0.09	16	4.7