



RETROFITTING OF ULAK KARANG SHELTER BUILDING IN PADANG CITY, INDONESIA, USING CONCRETE JACKETING

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ABSTRACT: Padang City is highly vulnerable to earthquake and tsunami hazards due to its location along the subduction zone of the Indian and Eurasian plates. To address evacuation challenges, the government constructed the Ulak Karang Shelter as a vertical tsunami evacuation facility. However, this structure was not originally designed to withstand tsunami loads. This study evaluates the structural performance of the shelter and proposes retrofitting using the concrete jacketing method. The research involved field surveys, collection of technical data, and structural modeling using ETABS v22. The analysis was based on Indonesian building codes (SNI 1726:2019, SNI 2847:2019, and SNI 1727:2020) and FEMA P-646 tsunami guidelines. The results of the structural analysis revealed that some columns and beams lacked sufficient capacity to withstand the combined loads of earthquakes and tsunamis, indicating critical structural weaknesses. After retrofitting with concrete jacketing, the moment capacity of the columns improved by 151%, axial capacity by 106%, and shear capacity by 45%, while beam strength increased by 181% in flexure and 68% in shear. Furthermore, structural displacement was reduced by up to 36% in the X direction and 22% in the Y direction. These findings confirm that concrete jacketing is an effective retrofitting method, enabling the Ulak Karang Shelter to function safely as a vertical tsunami evacuation facility.

Keywords: Earthquake, Tsunami, Vertical Evacuation Shelter, Concrete Jacketing, Structural Retrofitting

1. INTRODUCTION

The location of Padang City on the west coast of Sumatra, situated directly adjacent to the Indian Ocean and the active collision zone of the Indian and Eurasian plates, renders it one of the cities most vulnerable to tsunami hazards [1]. Tectonic earthquakes along subduction zones and other active seismic events can trigger extraordinarily powerful waves. Such tsunamis can cause severe damage to facilities and infrastructure, frequently resulting in significant casualties.

During the September 30, 2009, earthquake, Padang faced significant challenges regarding its horizontal evacuation routes. These routes were primarily located along main roads characterized by high population and traffic density. Furthermore, the limited number of roads providing perpendicular access to higher ground hindered effective evacuation. To ensure a successful evacuation process and minimize casualties during a potential tsunami, an alternative rescue strategy is required. One such strategy is vertical evacuation, which involves reaching a floor or structure elevated above the anticipated tsunami wave height.

By using shelters, people in Padang City can reach a safe place from tsunami inundation in a shorter time when horizontal evacuation cannot proceed properly [2]. Therefore, the Padang City government took action by building a temporary evacuation site (TES) known as a "Shelter," located in the Ulak Karang area of North Padang District (Fig. 1). Shelter buildings must be designed and constructed according to earthquake and tsunami-resistant building standards, as they must remain functional during natural disasters [3-5]. However, the Ulak Karang shelter was not designed for tsunami loads by the planning consultant, under the justification that Indonesia currently lacks standard regulations regarding tsunamis. In reality, if a tsunami occurs, it will inevitably impact the shelter, adversely affecting the building's structural integrity and its ability to support existing loads, potentially leading to structural failure.

Researchers have previously analyzed the building's performance against earthquake and tsunami loads using the Indonesian seismic code, SNI 1726:2019, and the FEMA P-646 guidelines. The results indicated that the structure can withstand live loads, dead loads, and earthquake loads;

however, when subjected to additional tsunami loads, the columns are not strong enough to support the structure.



Fig.1 View of Ulak Karang Shelter

Therefore, it is necessary to retrofit the building's structure by strengthening existing components or adding new structural elements. Structural retrofitting is typically performed as a preventive measure to ensure the structure can withstand additional loads and to prevent potential failure [6,7].

One solution to address this is the jacketing method, a common technique used for the repair and retrofitting of reinforced concrete structures [8]. Jacketing involves enlarging the cross-section of existing reinforced concrete columns by adding a new layer of concrete, reinforced with additional steel, which enhances the overall rigidity and load-bearing capacity of the shelter structure.

2. RESEARCH SIGNIFICANCE

The significance of this research lies in its direct impact on public safety in Padang City, an area highly vulnerable to tsunamis. This study addresses a critical gap: the existing Ulak Karang shelter was not designed for tsunami loads due to a lack of formal regulations in Indonesia at the time of construction. By developing a structural retrofitting plan using the jacketing method, this research ensures the shelter can withstand such loads, thereby minimizing potential casualties and providing a reliable vertical evacuation option when horizontal routes are congested or compromised.

3. RESEARCH METHODS

The structural performance evaluation in this study was based on explicit design criteria and acceptance benchmarks from both national and international standards. According to SNI 1726:2019 [9], the maximum inter-story drift is limited to 2% of the story height for buildings classified as Category IV (essential facilities for post-disaster functions),

which includes tsunami vertical evacuation shelters. The strength of structural members was assessed by comparing the applied demands with the design capacities. For beams, flexural and shear capacities were calculated following SNI 2847:2019 [10], and adequacy was determined by checking whether the applied moment and shear forces (M_u , V_u) exceeded the design capacities (ϕM_n , ϕV_n). For columns, combined axial and flexural capacities were verified using P-M interaction diagrams, where demand points falling outside the interaction curve indicated insufficient capacity. Load considerations followed SNI 1727:2020 for dead loads, live loads, and special evacuation live loads [11], while tsunami loads were determined based on FEMA P-646, which includes hydrostatic, hydrodynamic, impulsive, debris impact, and additional gravity effects [12]. Finally, an evacuation live load of 250 kg/m² (FEMA P-646) was applied to represent designated shelter occupancy.

4. EVALUATION OF EXISTING BUILDING

4.1 Existing Site Conditions

The Ulak Karang Shelter is located in Ulak Karang Village, North Padang District. The surrounding site conditions are described below:

4.1.1 Regional location and topography

The Ulak Karang Shelter (Fig. 2) is situated near the coastline, approximately 0.7 km from the shoreline of Padang. The surrounding land-use pattern consists primarily of residential areas.



Fig.2 Location of the Ulak Karang Shelter (Source: Google Earth)

4.1.2 Tsunami vulnerability Level

Based on the tsunami hazard map issued by the Padang City Regional Development Planning Agency, the estimated tsunami inundation depth at the site is 4–5 m. This depth is projected based on the topographic contours of Padang City and predicted tsunami wave characteristics for the region.

Table 1. Column dimensions

Code	Dimension (cm)	Main rebar	Shear Rebar	Concrete Cover (mm)
K1	D 60	16D22	D10-100	25
K2	60 x 40	16D22	D10-100	25

Table 2. Beam dimensions

Code	Dimension (cm)	Main rebar Top Support	Main Rebar Bottom support	Main Rebar Top Midspan	Main Rebar Bottom Midspan	Shear rebar support	Shear rebar midspan	Concrete cover (cm)
BA1	40 x 60	6D22	4D22	3D22	4D22	D10-75	D10-150	3
BA2	40 x 60	7D22	3D22	3D22	3D22	D10-75	D10-150	3
BB1	40 x 50	4D22	3D22	3D22	3D22	D10-75	D10-150	3
BB2	40 x 50	4D22	3D22	3D22	3D22	D10-75	D10-150	3
BB3	40 x 50	4D22	3D22	3D22	3D22	D10-75	D10-150	3
BA3	40 x 50	4D22	3D22	3D22	3D22	D10-75	D10-150	3
BC1	30 x 30	4D22	3D22	3D22	3D22	D10-75	D10-150	3

4.2 Building Structure Data

The building's technical parameters and current condition are as follows:

- Building Name : Temporary Evacuation Shelter (TES)
- Structure Type : Reinforced Concrete Portal
- Concrete Grade : K-350 ($f'_c = 30$ MPa)
- Steel Reinforcement Grade: 400 MPa
- Number of Floors: 5
- Building Function: Vertical Evacuation
- Floor Slab Thickness: 12 cm

The column and beam dimensions are presented in Tables 1 and 2, respectively.

4.3 Structural Modeling

The structural model was developed in three dimensions using ETABS v22 software (Fig. 3).

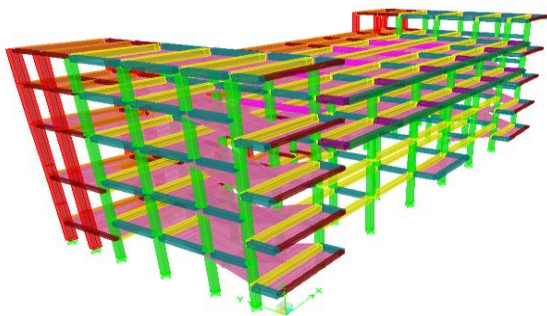


Fig.3 Modelling of existing structures

4.4 Load Analysis

4.4.1 Dead loads

The dead load calculations include the self-weight of the structural members (floor slabs, beams, and columns) and the weight of superimposed dead loads, such as masonry walls and floor finishes. All dead loads are determined in accordance with SNI 1727:2020. The values considered in the analysis are summarized in Table 3.

4.4.2 Live loads

The main live load taken into account is the load due to the occupancy of the room. Live loads are calculated based on PPIUG 1983 and SNI 1727-2020. The list of live loads considered is shown in Table 4.

Table 3. Dead loads

Building Materials/Components	Heavy
Spandrel	21 kg/m ²
Ceiling (including frame and suspension)	20 kg/m ²
MEP (Mechanical, Electrical, and Plumbing) Installation	25 kg/m ²
Waterproofing (Asphalt Layer)	14 kg/m ²
Floor coverings (ceramic) per 1 cm	24 kg/m ²
Wall (masonry)	250 kg/m ²

Table 4. Live loads

Room/Area Function	Live Load	Unit
Corridor	383	kg/m ²
Balcony/Edges	300	kg/m ²
Ramp	479	kg/m ²
Ladder	300	kg/m ²

4.4.3 Refuge live load

In accordance with FEMA P-646, the live load for a refuge area is set at 250 kg/m². This live load is applied to any room or space functioning as an evacuation site.

4.4.4 Earthquake load

Earthquake loads were calculated in accordance with SNI 1726:2019. The design parameters were retrieved from the Indonesian Ministry of Public Works and Housing website. By inputting the site class and project location (city or coordinates), the required design response spectrum data was generated (Fig. 4). The analysis utilizes the response spectrum for Site Class SE (soft soil), with design spectral acceleration parameters of $S_{DS} = 0.805g$ and $S_{D1} = 0.958g$.

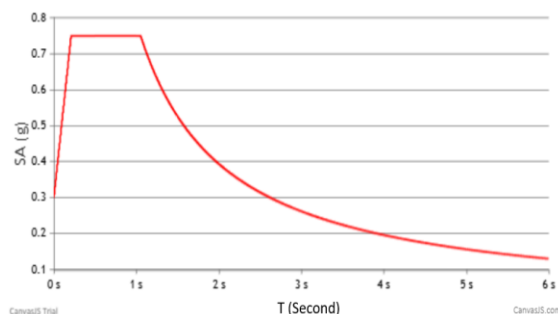


Fig.4 Design response spectrum based on SNI 1726:2019

4.4.5 Tsunami loads

Several types of tsunami loads are taken into account. The magnitude of each load is calculated based on the predicted tsunami wave height, the ground elevation of the site, the distance from the shoreline, and other relevant assumptions. The calculated tsunami loads are presented in Table 5.

4.5 Load Combination

The load combinations used in this analysis are in accordance with SNI 1726:2019 and FEMA P-646.

Table 5. Tsunami loads

Tsunami Load	Force
Fb = Buoyancy force	1815 kg/m ²
Fd = Hydrodynamic force	3251 kg
Fs = Impulsive Force	4876.49 kg
Fi = Impact force (Debris)	47643.17 kg
Fdm = Debris damming force	3250.995 kg
Fu = Uplift force	0.3697 kg/m ²
Fr = Additional gravity force	1815 kg/m ²

In the structural analysis, the following load combinations were considered:

- 1,4D
- 1,2D + 1,6L
- 1,2D + 1,0L + 0,3(ρ EQX + 0,2SDS.D) + 1(ρ EQY + 0,2SDS.D)
- 1,2D + 1,0L + 1(ρ EQX + 0,2SDS.D) + 0,3(ρ EQY + 0,2SDS.D)
- 0,9D + 0,3(ρ EQX - 0,2SDS.D) + 1(ρ EQY - 0,2SDS.D)
- 0,9D + 1(ρ EQX - 0,2SDS.D) + 0,3(ρ EQY - 0,2SDS.D)
- 1,2D + 1Fd + 1Fs + 1Fb + 1Fu + 1 LREF + 0,25 L
- 1,2D + 1Fi + 1Fd + 1Fb + 1Fu + 1 LREF + 0,25 L
- 1,2D + 1Fd + 1Fdm + 1Fb + 1Fu + 1 LREF + 0,25 L
- 1,2D + 1Fr + 1Fb + 1Fu + 1 LREF + 0,25 L
- 0,9D + 1Fd + 1Fs + 1Fb + 1Fu
- 0,9D + 1Fi + 1Fd + 1Fb + 1Fu
- 0,9D + 1Fd + 1Fdm + 1Fb + 1Fu
- 0,9D + 1Fr + 1Fb + 1Fu

5. RESULTS AND DISCUSSION

5.1 Structural Analysis

5.1.1 Inter-story drift

The inter-story drift results were obtained following the structural analysis performed in ETABS v22. The calculated inter-story drifts for both the X and Y directions are presented in Tables 6 and 7, respectively.

The inter-story drift analysis indicates that the shelter experiences displacement in both the X and Y directions under earthquake and tsunami loads. Although the additional tsunami load causes only a slight increase in drift compared to the earthquake load alone, the cumulative effect shows that several upper stories approach critical limits. This finding highlights the structural vulnerability of the shelter, particularly at higher levels, and emphasizes the need for retrofitting measures to improve overall lateral stability.

Table 6. Comparison of inter-story drift in the X-direction with and without the tsunami loads

Floor	Drift Without Tsunami Loads (mm)	Drift With Tsunami Loads (mm)	Increase Ratio (%)
1	0.00	0.00	0.000%
2	4.65	4.65	0.000%
3	10.67	10.71	0.378%
4	15.54	15.65	0.708%
5	19.42	19.68	0.134%
Rooftop	21.42	21.56	0.654%

Table 7. Comparison of inter-story drift in the Y-direction with and without the tsunami loads

Floor	Drift Without Tsunami Loads (mm)	Drift With Tsunami Loads (mm)	Upgrade Ratio (%)
1	0.00	0.00	0.000%
2	10.83	10.83	0.000%
3	20.33	20.34	0.049%
4	26.15	26.23	0.306%
5	29.85	30.04	0.637%
Rooftop	32.29	32.39	0.310%

5.1.2 Internal forces

A summary of the changes in internal forces, consisting of axial force (P), shear force (V), and bending moment (M), resulting from the combined earthquake and tsunami load cases, was obtained following the structural analysis in ETABS v22. The summary of these internal forces for both columns and beams is presented in Tables 8 and 9.

Table 8. Changes in column internal forces due to earthquake and tsunami load combinations

Floor	Column	Normal	Shear	Moment	Information
1	K1	0.28%	30.33%	66.85%	MAX
1	K2	0.03%	0.00%	91.38%	MAX
2	K1	0.29%	0.00%	46.17%	MAX
2	K2	0.40%	0.00%	75.00%	MAX
3	K1	0.36%	0.00%	17.61%	MAX
3	K2	0.00%	0.00%	41.44%	MAX
4	K1	0.00%	0.00%	38.93%	MAX
4	K2	0.00%	0.00%	0.65%	MAX
5	K1	0.00%	0.00%	0.26%	MAX
5	K2	0.00%	0.00%	0.06%	MAX

Table 9. Changes in beam internal forces due to earthquake and tsunami load combinations

Floor	Shear	Moment	Information
1	45.59%	26.13%	MAX
1	49.30%	45.33%	MIN
2	0.00%	0.00%	MAX
2	0.00%	0.00%	MIN
3	0.00%	0.00%	MAX
3	0.00%	0.00%	MIN
4	0.00%	0.00%	MAX
4	2.05%	2.60%	MIN
5	1.00%	0.00%	MAX
5	0.00%	0.00%	MIN

5.1.3 Cross-sectional structural capacity

a. Column

The analysis results for the column cross-sectional capacity are shown in Figs. 5–6. The analysis indicates that the K1 ($\varnothing 60$ cm) and K2 (60×40 cm) columns are unable to resist the earthquake and tsunami load combinations, as their demand points exceed the structural capacity (interaction) curves. The retrofitting is required to enhance the columns' flexural and shear capacity.

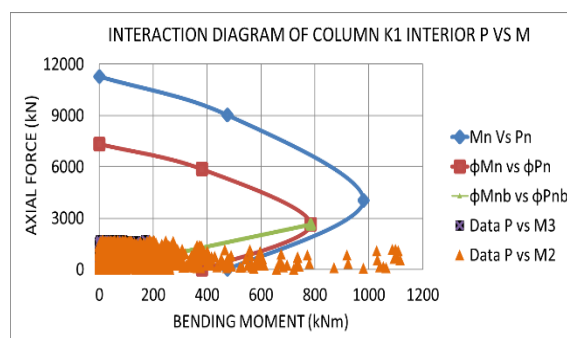
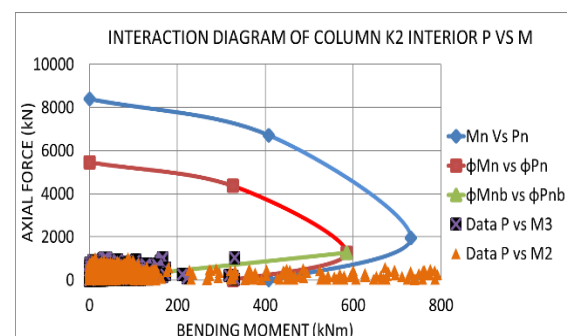
Fig. 5 P-M diagram for column K1 on 1st floorFig. 6 P-M diagram for column K2 on 1st floor

Table 10. Recapitulation of the bending capacity of the beams of shelter structures with tsunami loads

Floor	Beam	Location	Dimension		Rebar		ØMn (kN)	Mu (kN)	Information	
			Width	Height	Pull	Press				
1	BA1 40/60	Edges	400	600	6D22	3D22	395.439	506.805	Support	NOT OK
					4D22	2D22	265.829	347.581	Midspan	NOT OK
		Midle			6D22	3D22	395.439	157.415	Support	OK
					4D22	2D22	265.829	133.074	Midspan	OK
	BB1 40/50	Edges	400	500	6D22	3D22	322.453	282.330	Support	OK
					4D22	2D22	217.172	329.211	Midspan	NOT OK
		Midle			6D22	3D22	322.453	349.684	Support	NOT OK
					4D22	2D22	217.172	313.710	Midspan	NOT OK
	BC1 30/30	Edges	300	300	4D22	4D22	119.297	20.082	Support	OK
					4D22	4D22	119.297	18.150	Midspan	OK
		Midle			4D22	4D22	119.297	174.410	Support	NOT OK
					4D22	4D22	119.297	237.085	Midspan	NOT OK

b. Beam

The results of the bending capacity analysis for the shelter beams are presented in Table 10. The analysis indicates that several first-floor beams, particularly BA1 (40×60 cm) and BB1 (40×50 cm), fail to satisfy flexural requirements under tsunami load combinations. Several midspan and support sections exhibit moment demands exceeding their nominal moment capacity, confirming their vulnerability. Consequently, retrofitting is necessary to enhance their flexural and shear resistance.

5.2 Structural Retrofitting Analysis

Fig. 7 shows the location of the first-floor beam elements requiring retrofitting, while Fig. 8 highlights the second-floor column elements that also require additional retrofitting. Cross-sectional dimensions to be jacketed are as follows:

- K1 = 750 mm, main rebar: 32D22
- K2 = 750 x 550 mm, main rebar : 32D22
- BA1 = 600 x 800 mm, main rebar: 18D22
- BB1 = 600 x 700 mm, main rebar: 16D22
- BC1 = 500 x 500 mm, main rebar : 15D22

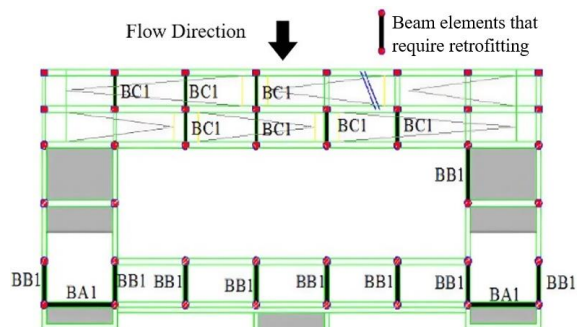
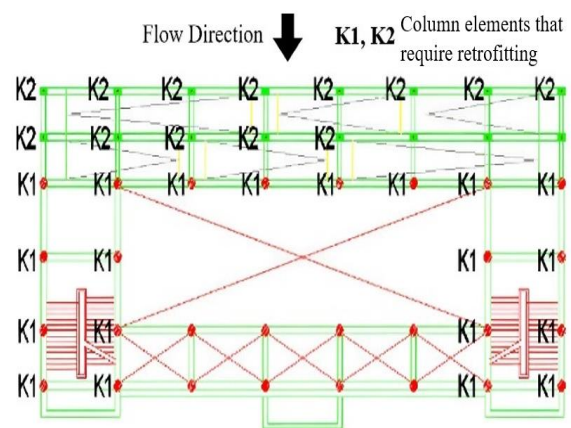
Fig.7 Location of 1st floor beam elements requiring retrofitting

Fig.8 Location of 2nd-floor column elements requiring retrofitting

The modeling of the column retrofitted with concrete jacketing is shown in Fig. 9.

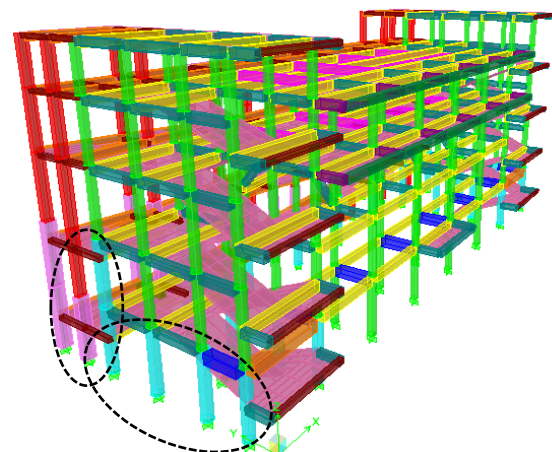


Fig.9 Structural model of the column retrofitting

5.2.1 Inter-story drift

Based on the analysis results of the building retrofitted using the concrete jacketing method, a significant reduction in structural displacement was observed. The maximum inter-story drift decreased by approximately 36% in the X-direction and 22% in the Y-direction compared to the existing condition. This reduction indicates a substantial increase in the global stiffness and lateral load-resisting capacity of the structure. The improved stiffness effectively limits lateral deformation under combined seismic and tsunami loads, thereby enhancing overall structural stability.

5.2.2 Cross-sectional structural capacity

a. Column

The results of the structural capacity analysis for the elements retrofitted using the concrete jacketing method demonstrate significant improvements in both columns and beams. Fig. 10 presents the P-M interaction diagram for the first-floor columns, while Fig. 11 illustrates the P-M interaction diagram for the second-floor columns post-retrofitting.

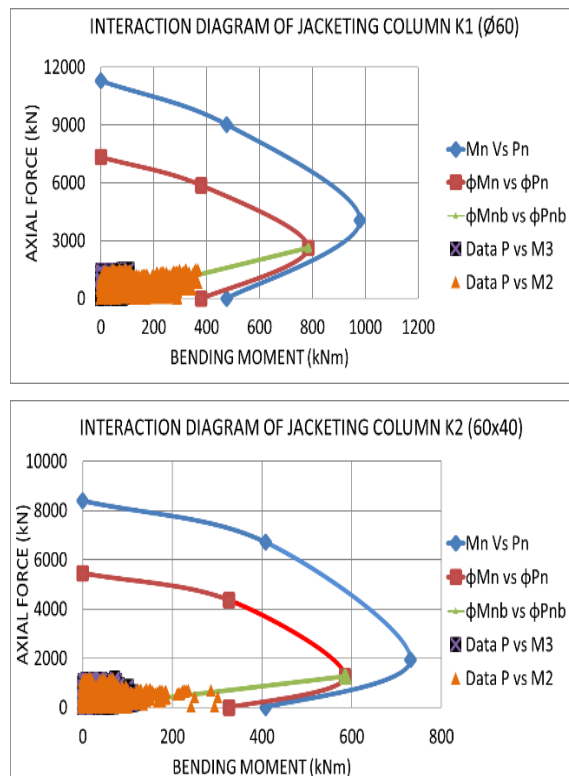


Fig.10 P-M interaction diagram for the retrofitted 1st-floor columns

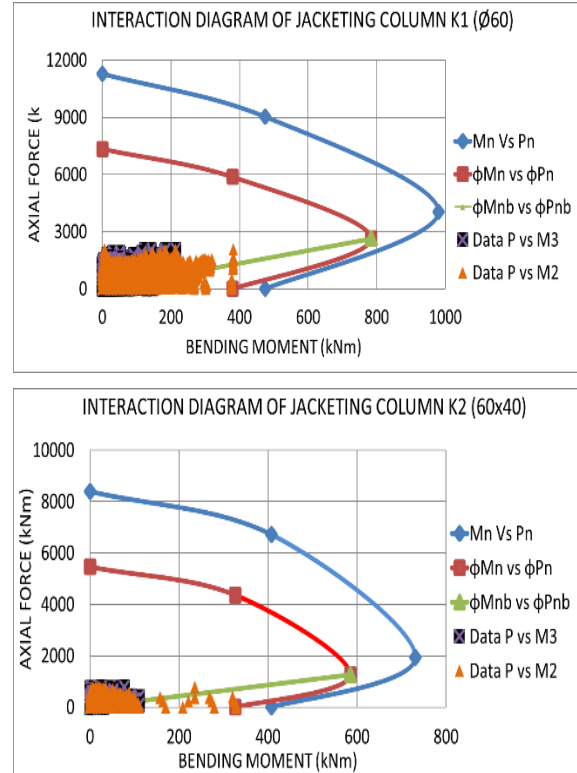


Fig.11 P-M Interaction diagram for the retrofitted 2nd-floor columns

b. Beam

The beam results are summarized in Tables 11 and 12, which compare the flexural and shear capacities between the existing and retrofitted conditions. Overall, the jacketing method effectively enhanced structural performance; column strength improved by 151% in moment capacity, 106% in axial capacity, and 45% in shear capacity, while beam strength increased by 181% in flexure and 68% in shear.

Table 11. Comparison of the bending capacity of existing and retrofitted beams

Beam	Location	Nominal Moment (kNm)		Increased (%)
		Existing	Retrofitting	
BA 1	Support	395.439	782.679	98%
BA 1	Field	265.829	706.645	166%
BB 1	Support	322.453	577.980	79%
BB 1	Field	217.172	505.904	133%
BC	Support	119.297	335.383	181%
BC	Field	119.297	335.383	181%

Table 12. Comparison of beam shear capacity for existing and retrofitted beams

Beam	Location	Shear (kN)		Increased (%)
		Existing	Retrofitting	
BA1	Support	514.242	723.512	40.695%
	Field	335.172	497.317	48.377%
BB1	Support	424.025	623.024	46.931%
	Field	276.370	428.245	54.954%
BC	Support	315.665	531.493	68.372%
	Field	315.665	531.493	68.372%

Beyond analytical improvements, several practical considerations must be addressed when implementing the concrete jacketing retrofit method. Quality control during construction is critical, particularly in ensuring adequate bonding between the existing concrete and the new jacketing layer, the proper placement of additional reinforcement, and sufficient compaction of the new concrete. Without strict quality control, the effectiveness of the retrofit may be compromised. Potential cracking at the interface between old and new concrete must also be anticipated, requiring the use of bonding agents, surface roughening, and adequate curing practices. From a construction logistics perspective, jacketing requires careful planning regarding access to structural elements, formwork installation, and work scheduling, especially since the shelter is located in a densely populated area. These practical issues must be considered to ensure that the retrofitting design is effectively implemented in the field.

6. CONCLUSION

Based on the findings of this research, the following conclusions are drawn:

1. The existing Ulak Karang shelter cannot fully withstand the combined effects of earthquake and tsunami loads, as several column and beam elements lack sufficient capacity. This indicates an urgent requirement for structural retrofitting to ensure the shelter can safely perform its evacuation function.
2. Retrofitting with the concrete jacketing method proves highly effective, resulting in significant increases in column capacity. Moment, axial, and shear capacities increase by 151%, 106%, and 45%, respectively, thereby enabling the columns to effectively sustain combined seismic and tsunami loads.
3. The retrofitting of the structure with concrete jacketing also significantly enhances the

performance of beam elements, with flexural and shear capacities increasing by 181% and 68%, respectively. These improvements ensure that the beams are capable of resisting tsunami-induced forces that previously exceeded their structural capacity.

4. The structural displacement of the retrofitted building is reduced by 36% in the X-direction and 22% in the Y-direction, demonstrating that the structure has become significantly stiffer and more stable against lateral movement during seismic and tsunami events.
5. The implementation of the retrofitting design enhances the structural capacity of the building, thereby validating its suitability for service as an emergency evacuation shelter.

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