

SEISMIC STRENGTHENING OF RIGID FRAME BY FERROCEMENT WITH EXPANDED METAL MESH AND EXTERNAL FLEXURAL REINFORCEMENT

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Abstract

Typical low rise commercial buildings are often damaged due to the soft-story effect under earthquake loading. Failure modes include flexural and shear failure at the ends of columns. To improve the flexural strength of the rigid frame, an analytical model to predict the lateral resistance of the strengthened frame was proposed. The study focused on the strengthening of the RC column with expanded metal and external longitudinal reinforcement and the strengthening of the beam-column joint. The specimens were subjected to the axial vertical load and the lateral load. A single-story, single-bay frame was selected as the prototype. Two sets of specimens were prepared to the full scale. The experimental program consisted of two types of test specimens: (1) a bare reinforced concrete (BF) frame as the control and (2) a reinforced concrete frame strengthened with ferrocement (BFS). The experimental results showed that the BFS specimen exhibited significant improvements in the lateral load capacity, stiffness, and ductility with 119%, 110%, 47%, respectively greater than the original frame. The structural performance enhancements are attributed to the respective contributions of ferrocement layer strengthening (22%), external reinforcement (45%), and the original column's inherent moment resistance (33%), as derived from the total moment capacity. The lateral resistance resulting from the laboratory test and the analytical model showed a difference of 2.48%, indicating an acceptable prediction. The test results were also compared with the results of numerical analysis using the nonlinear structural analysis by the RUAUMOKO software. The results were found to be in good agreement.

Keywords: Seismic Strengthening; Ferrocement; Reinforced Concrete Frame; Expanded Metal; External Reinforcement

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Introduction

The 2014 earthquake, which occurred in the Mae Lao District of Chiang Rai Province, in the northern part of Thailand, showed that typical low-rise buildings were severely damaged due to seismic loads. Among these, several commercial buildings were damaged due to the soft-story behavior of the columns. The front elevation of the ground floor of these buildings is slender, bare frame, which is much taller and possesses more open space than the upper floors. In contrast, the upper floors contain masonry-infilled frames. Therefore, the stiffness of the ground floor columns is lower than those of the upper floors, resulting in a soft-story failure due to the bending stress at both ends of the ground floor columns. Strengthening the ground floor columns to prevent soft-story failure is essential in improving the earthquake resistance of these buildings.

In the past years, many researchers have focused on improving and strengthening existing building structures that are at risk from earthquakes, which is a major concern for structural engineers. Consequently, various methods have been studied to enhance lateral load capacity in vulnerable building structures. Previous research has shown that strengthening by using ferrocement is an interesting method due to its various advantages. These include low cost, minimal skill requirements for construction, lightweight properties, good impact resistance, crack prevention, enhanced confinement, adaptability to desired shapes, and the availability of local materials. Hebru and Philip (2018) investigated the effectiveness of various types of wire mesh for strengthening reinforced concrete columns using ferrocement techniques, including these mesh types: diagonal, hexagonal, square welding, and square woven. The experimental tests under axial loads revealed that among these wire meshes, diagonal mesh was able to support higher axial forces and also provided better confinement within the reinforced concrete columns. In the following years, additional research was conducted in the same direction. Erfan *et al.* (2019) evaluated the effectiveness of various types of steel wire mesh for reinforced concrete columns using ferrocement technique. Three mesh types were examined: expanded metal, welded wire and galvanized welded. It was found that the number of layers and the volume fraction of reinforcement significantly improved strength, crack resistance, ductility, and energy absorption. Among these mesh types, the expanded metal offers the best ductility and performance. Further comparative studies were conducted to reinforce different RC columns by Tariq *et al.* (2020). These methods included CFRP,

steel jacketing, ferrocement, steel fiber, and silica fume under constant vertical loads and cyclic lateral loads. They found that all methods could effectively enhance the load bearing capacity, stiffness, ductility, and energy absorption of the columns. However, CFRP was found to be the most expensive technique. In addition, RC columns confined with the ferrocement technique were examined by Elsibaey *et al.* (2020). It was found that the expanded metal mesh was superior to the square mesh. The contribution of layers, the number of layers, and the volume fraction improved the confinement, resulting in the enhancement of strength, ductility, and lateral shear resistance. Mabrouk *et al.* (2022) investigated the effects of retrofitting short square columns, employing wire mesh of various types with varying numbers of layers and the ferrocement jacketing technique. Among these, the 2-layer diagonal mesh provided the best improvement. The addition of mesh in the transverse direction was more effective in overcoming the stress concentration at the corners compared to adding mesh in the vertical direction. Similar findings were also investigated by Aiswarya and Mohan (2019).

The development of the strengthening columns with ferrocement has been performed by several researchers in the past. Soman and Mohan (2018) found that the failure in ferrocement-jacketed specimens was due to the separation of the concrete columns at the corners owing to tensile stresses at the corners. This finding is consistent with research by Kaish *et al.* (2012). They overcame the problem of corner stress concentration using various techniques, such as rounding the corners of columns, adding shear keys, and incorporating additional double-layer mesh at the corners. The latter technique provided better results under axial loading, while the former was more suitable for eccentric loading. In addition, Abd-ELhamed and Ezz-Eldeen, (2014) found that installing the steel angles at column corners significantly enhanced the load-carrying capacity under different eccentric loads and contributed to improving the efficiency and toughness of ferrocement jackets. Furthermore, Komathi and Sengupta (2020) investigated the effects of three different interface types: rough surfaces, shear keys, and deformed steel to prevent delamination. They found that the connections using deformed steel were more effective and had higher strength compared to other types. Similarly, Li and Lam (2018) revealed that the roughness of the concrete surface and the shear key significantly affect the shear bond between the concrete surface

and ferrocement. Among the expansive shear key and the adhesive shear key, the latter was more reliable. Furthermore, it was found that L-shaped or end-extension adhesive shear keys were the most effective. Finally, it is recommended to prepare the concrete surface to be as rough as possible and use end-extension adhesive shear keys to anchor to the reinforcing material. Panyamul *et al.* (2019) investigated the use of retrofitted ferrocement with expanded metal lath to enhance the shear resistance of short columns. Two sets of specimens were reinforced with two types of expanded metal lath having different specific surface areas and tested under constant axial load and cyclic lateral loading. Both sets of specimens enhance shear strength, stiffness, ductility, and energy dissipation compared to the control specimens. Furthermore, it was found that the effect of expanded metal lath with high specific surface area was more effective in controlling concrete surface cracking. In the past, there have been studies on the effect of masonry walls on the efficiency of building RC frame structures. Similar retrofitting techniques have been investigated (Balamura likrishnan *et al.*, 2019; Kumar *et al.*, 2020; Villar-Salinas *et al.*, 2021; Theint *et al.*, 2020). Leeanansaksiri *et al.* (2018) further investigated the behavior of masonry walls strengthened using a retrofitted ferrocement technique with expanded metal mesh. It was found that the strengthened walls exhibited higher strength, ductility, and energy dissipation compared to the original walls. However, the strengthening technique led to increased stress concentration at the corners of the masonry walls, which led to shear failure of the columns. A few years later, a follow-up study was conducted by Longthong *et al.* (2020). The study aimed to further improve the corner damage prevention of masonry walls and RC columns, using the ferrocement technique with expanded metal mesh. Two specimens were tested: a bare frame and an infilled frame. The results showed that the frame strength was significantly enhanced. The beams and columns were well-protected from shear damage, and corner compression damage was also prevented. Furthermore, researchers have continued their study with additional investigations (Shi *et al.*, 2021; Ruiz-Pinilla *et al.*, 2022), to enhance the strength of beam-column joints using external steel reinforcement techniques. It was found that the external bars increased flexural strength and prevented failure in the column, and beam joints.

In addition, the external longitudinal and diagonal steel rods also increased the bending resistance of column. Similar study was also conducted by Cho *et al.* (2018). Two sets of reinforced concrete columns were encased with high-performance fiber-reinforced cement (HPFRC), using transverse and longitudinal reinforcements. The results showed an improvement in load-carrying capacity. Several failure modes were also diminished including shear damage, diagonal cracking, and flexural cracking, in the plastic hinge region. However, external reinforcement with steel bars still has limitations, such as the bond between the reinforcement and the existing concrete, the type of reinforcement steel used, installation complexity, and shape constraints. Therefore, further investigation is necessary.

In this research, an innovative retrofitting technique was proposed for existing reinforced concrete RC frames to enhance their seismic performance. Two full-scale test specimens were prepared: (1) a bare frame as a control specimen (BF), and (2) a bare frame as a strengthened specimen (BFS). In the retrofitting process, the expanded metal mesh was wrapped around the columns and welded to the corner angle steels to reduce the mesh sharpness and stress concentrations at the corners. Special external longitudinal reinforcement was devised to enhance flexural strength at the lower column end and the beam-column joints.

Model of Strengthened Bare Frame

The model of strengthened RC bare frame includes two main components: the strengthening of reinforced concrete columns using a ferrocement technique with expanded metal mesh confinement along the full height of the columns. This reinforcement aims to prevent premature failure due to insufficient shear resistance in the window openings of the columns, as shown in Figure 1.

For the strengthening of the beam, ferrocement reinforcement is applied over a length of 2.5 times the beam depth. Since the lateral forces induce moments that are transferred from the column-beam connection to the beam, therefore, ferrocement reinforcement is applied over a length equivalent to the plastic hinge length that may develop at the beam ends.

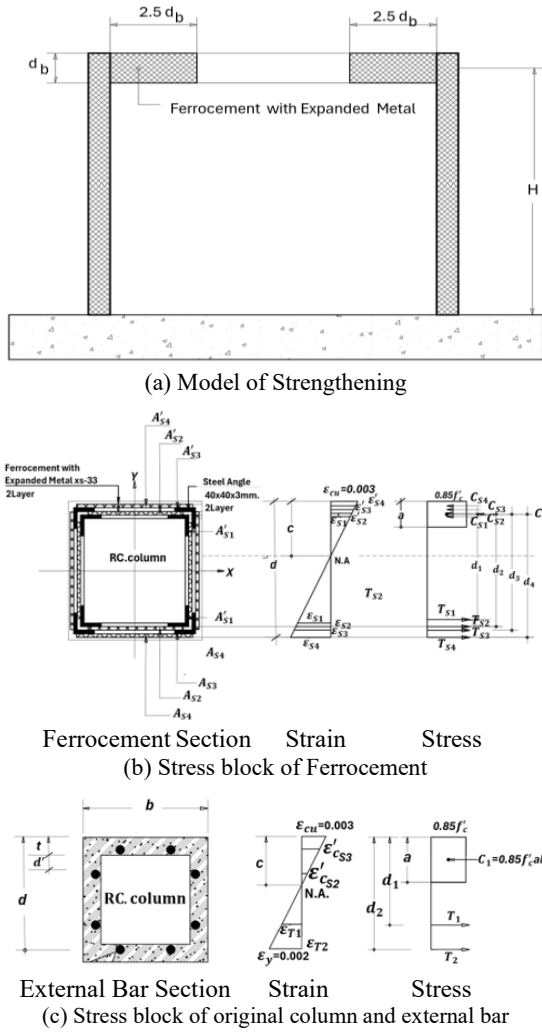


Figure 1. Strengthened bare frame

The moment resistance of the strengthened RC column (M_{sc}) can be calculated by the summation of the moment resistance of the ferro-cement confinement (M_n^F) and the moment resistance of the original column (M_{exc}) strengthened with external longitudinal rebar (M_{ext}), as shown in equation (1).

$$M_{sc} = M_n^F + M_{ext} + M_{exc} \quad (1)$$

From the strain and stress distribution of the ferro-cement confined column as shown in Figure 1b, the moment capacity of ferro-cement confinement under bending can be calculated by taking moments of the tensile forces around point C_c , which is the centroid of the concrete compressive stress block. This analysis is based on the assumption that the primary moment is resulting from the tensile forces as suggested by ACI549

(1993), since the moment resulting from the compressive force is very small, it can be neglected.

$$M_n^F = \sum_{i=1}^n T_{si} \left(d_i - \frac{\beta_1 c}{2} \right) \quad (2)$$

The tensile resistance capacity of ferro-cement (T_s) as shown in equation (2) can be calculated using the formula $T_{si} = f_{si} A_{si}$, where f_{si} is the stress corresponding to the strain of the reinforcement, and (A_{si}) is the effective cross-sectional area of the reinforcement in the direction being considered.

The moment resistance capacity of the external rebar, as shown in Figure 1(c), can be calculated by taking the moment of the centroid of the compressive stress block (C_1), as presented in Equation 3.

$$M_{ext} = T_1 \left(d_1 - \frac{a}{2} \right) + T_2 \left(d_2 - \frac{a}{2} \right) \quad (3)$$

As presented in Equation 3, b and d are the width and the effective depth of the strengthened column, respectively; t is the effective thickness of the cement mortar; a is the width of the stress block and it can be calculated from $a = \beta c$; d' is the covering of concrete; c is the distance from neutral axis to the concrete surface in compression; T_1 and T_2 are the tensile force of the reinforcement of the external rebar.

The moment resistance of the original reinforced concrete column subjected to low axial load can be calculated by the expression proposed by Tumialan *et al.* (2001).

$$M_{exc} = A_{st} f_y g d + 0.5 N d \left(1 - \frac{N}{b d f_c} \right) \quad (4)$$

In this equation, b , d are the width and depth of the original reinforced concrete (RC) column, respectively; A_{st} is the cross-sectional area of the reinforcement in the existing (RC) column; f_y is the tensile strength at the yield point of reinforcement; f_c' is the compressive strength of concrete; g is the ratio of the spacing of the tension and compression reinforcement, proportional to the depth of the original column; and N is the axial load on the reinforced concrete column.

The lateral resistance of the strengthened bare frame (R_{BFS}) can be calculated as follows:

$$R_{BFS} = \frac{2(M_{sc} + M_{PJ})}{H} \quad (5)$$

The lateral resistance of the original bare frame (R_{BF}) can be calculated using Equation (6):

$$R_{BF} = \frac{2(M_{exc} + M_{pj})}{H} \quad (6)$$

where M_{pj} is the minimum moment between M_{pc} , M_{pb} , M_j .

M_{pc} is the plastic moment in the column,

M_{pb} is the plastic moment in the beam and

M_j is the moment of the column-beam joint.

Experiment of Specimens

Specification of Materials

The cylindrical compressive strength of concrete, used in the rigid frame specimen, is 21 MPa at 28 days according to ASTM C39/39M (2024). Meanwhile, the mixed design of plaster mortar for ferrocement had a cement to sand ratio of 1:2 and a water to cement ratio of 0.45, resulting to the compressive strength of 22.25 MPa at 28 days according to ASTM C349-97 (1997).

The longitudinal reinforcement with a diameter of 16 mm was employed for beams and columns with a yield strength of 400 MPa according to the TIS 24-2016 standard (2016). The transverse reinforcement with a diameter of 6 mm was employed for beams and columns with a yield strength of 240 MPa grade steel according to the TIS 20-2000 standard (2000). In addition, the external bar reinforcement with a diameter of 12 mm and the yield strength of 400 MPa as per the TIS 24-2016 (2016) standard was employed for improving the bending resistance.

In this study, the expanded metal was employed for the reinforcement of ferrocement. It consists of metal sheets perforated into a diamond-shaped mesh, as shown in Figure 2. The expanded metal mesh has basic physical properties according to the JIS G3351-1987 standard (1987), as presented in Table 1. The expanded metal mesh has a tensile strength of 337 MPa and an ultimate strength of 400 MPa.

Table 1. Physical properties of expanded metal

Type of mesh	SW (mm)	LW (mm)	T (mm)	w (mm)	Weight (kg/sqm)
XS-33	12.0	30.5	2.3	3.0	6.42

Test Specimens

In this test, a single-story, single-bay rigid frame was modelled as the bare frame. The test specimen dimension was 4,000 mm in span length and 3,000 mm in height. The columns had a square cross-section of 250 mm x 250 mm, the base width

1,500 mm and base length 5,500 mm, and the beams had a cross-section of 200 mm width by 400 mm depth. The bare frame of the original building was chosen as the control specimen (BF) (Figure 3a), while the second bare frame was reinforced using ferrocement with expanded metal (BFS), as shown in Figure 3b.

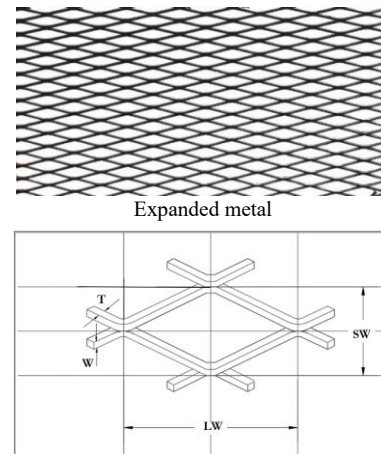


Figure 2. Details of Expanded Metal Mesh (Japanese Industrial Standard, 1987)

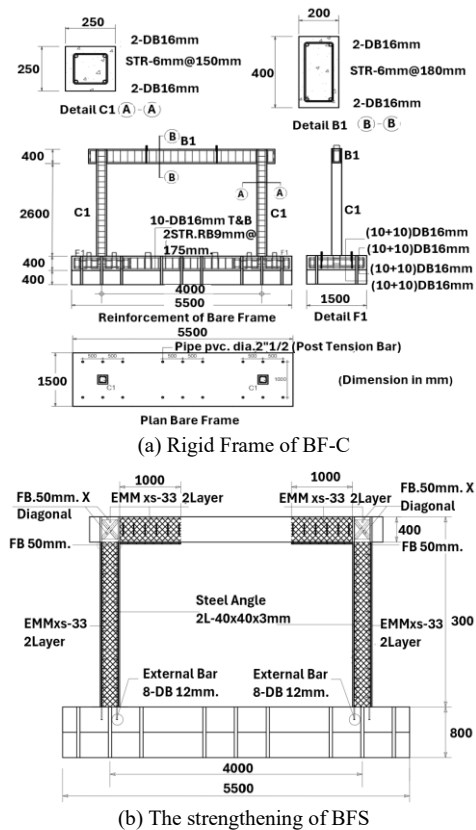
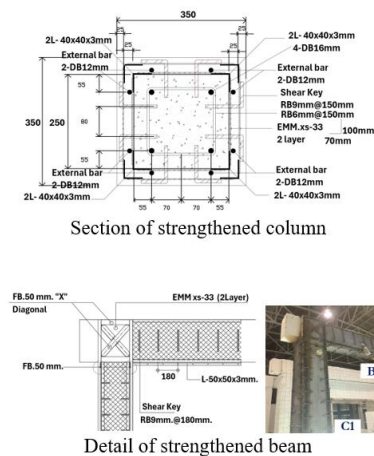


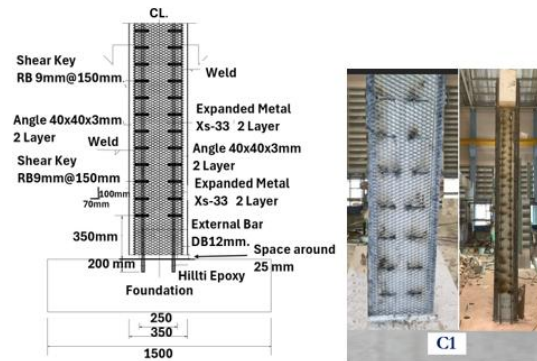
Figure 3. Selected reinforced concrete bare frames

Strengthening Process for Bare Frame

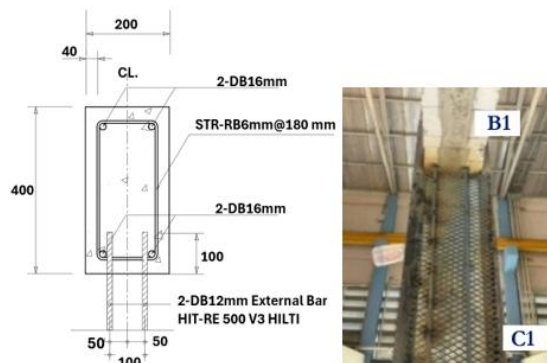
The strengthening process as presented in Figure 4 is divided into three steps. Firstly, the concrete surface was prepared by creating grooves at 50 mm intervals and 6 mm deep throughout the surface area of the column, which increased the bonding surface for better adhesion with the reinforcement materials. Next, the steel angles were installed at all four corners of the column to reduce the sharpness of the mesh, and steel angles were installed at both ends of the beam with the length of 2.5 times the depth of the beam (Figure 4a). Then, the expanded metal was wrapped around as the first layer and secured to the steel angles. In the second step, the external steel bar with a diameter of 12 mm was embedded into the base of the column with 200



(a) Strengthened beam and column



(b) Detail of strengthened column



(c) Section Rebar in beam

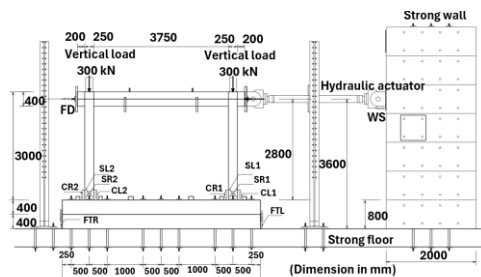


(d) Strengthened bare frame

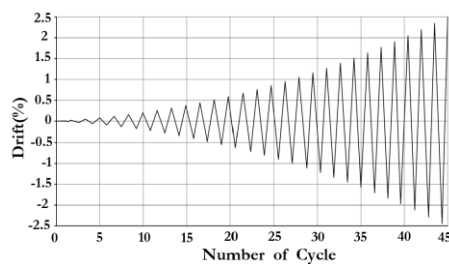
Figure 4. Details of strengthened bare frame (BFS)

Experimental Programs

The procedure for installing the specimens begins with the foundation being anchored to a strong floor using high-tensile steel bolts with a diameter of 32 mm. A constant compressive force is applied to the column head using a hydraulic jack with a capacity of 300 kN. A lateral horizontal force is applied using a hydraulic actuator with a capacity of 1500 kN at the middle of the beam's end. The horizontal hydraulic actuator was installed at a height of 3.60 meters above the ground and anchored to a strong wall, as shown in Figure 5a. When lateral force was applied to push the frame forward, the frame was pulled back in the opposite direction using a couple of high-strength steel rods with a diameter of 32 mm. Strain measurement sensors (LVDT KYOWA) are then installed on the specimen, as illustrated in Figure 5a. Displacement measurements (pull) are attached to the end of the upper beam opposite the force application side. The installation also includes measuring shear deformation at the front column. In addition, the curvature of the columns, wall sliding, and foundation movement are measured. All data are recorded via a data logger for processing. The specimens were tested under cyclic loading with the loading protocol according to the displacement control (Federal Emergency Management Agency, 2007), present in Figure 5b. Displacement is increased from 0.10 % to 0.50 %, with increments of 0.25 %, and the applied force continues until the specimen fails, which is approximately 80 % of the maximum force.



(a) Setup lab test of the specimens



(b) Lateral loading in laboratory test

Figure 5. Setup laboratory test and lateral loading of the specimens

Discussion of Experimental Results

Failure Mode of the Specimens

For the specimen BF, the lateral load of the displacement started at 0.5-1.0% drift and the minor crack occurred at the joint of the beam-column due to the flexural stress. Following the further increase of load up to the displacement of 1.0-1.5% drift, as shown in Figure 6a, the cracks at the beam-column joint were widened due to flexural failure. During the addition of load up to the displacement of 2.0-3.0% drift, as shown in Figure 6b, the large flexural cracks occurred at the beam-column joint. When the displacement reached 3.5% drift, as shown in Figure 6c, the cracks at the column base were enlarged resulting in the decrease of resistance. After that, the lateral load carrying capacity was reduced until the test ended at 4.0% drift, as shown in Figure 6d.



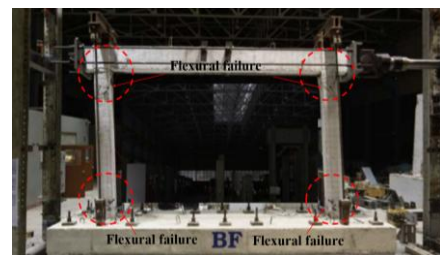
(a) story drift at 1.5%



(b) Cracks at the upper of beam-column joint



(c) Cracks at the upper of beam-column joint



(d) story drift at 4 %

Figure 6. The cracks failure of the bare frame, BF

As for the strengthened rigid frame BFS, when the lateral load at a displacement reached 0.10-0.50% drift, no damage was detected in the frame. However, during lateral loading, when the displacement reached 1.0% drift, the first crack appeared at the right side of the beam-column joint at the top of the building frame. A similar crack was also observed on the left side. As the lateral force increased, leading to a displacement of 2.0% drift, flexural stress caused concrete cracking near the lower base of the columns, approximately 0.60 m above the foundation, as shown in Figure 7a. With further lateral load and displacement between 2.0% and 3.0% drift, the crack at the upper beam-column joint widened. However, the crack did not propagate further, while the external rebar yielded at the lower end of the column, resulting in concrete fragments falling off, as shown in Figure 7b and Figure 7c. The test continued until the displacement

reached 4.0% drift. At this stage, the external bar was broken at the lower end of the right column, approximately 0.10 m above the base, as depicted in Figure 7c. The external rebar was ruptured due to the strain of the external bar exceeding the yield point of 0.002, as shown in Figure 7d. The test was ultimately terminated at a displacement of 5.0% drift when the lateral resistance decreased to 60% of the peak load. It was observed that the column failed in a weak joint failure pattern. Despite this, the overall structure remained in good condition (Figure 7e). This indicates that strengthening the beam and column ends with expanded metal mesh, along with the use of external longitudinal rebar, significantly increased the strength of the columns and beams. This reinforcement reduced the severity of bending failure that occurred in the original bare frame.



(a) Concrete cracks at the lower base of the column



(b) Cracks of the beam-column joint



(c) The external bar failure at column base



(d) Relationship of lateral load and strain in external bars of the left and right columns



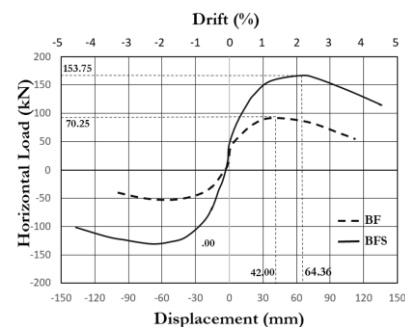
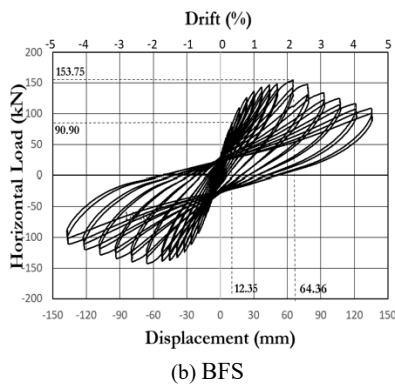
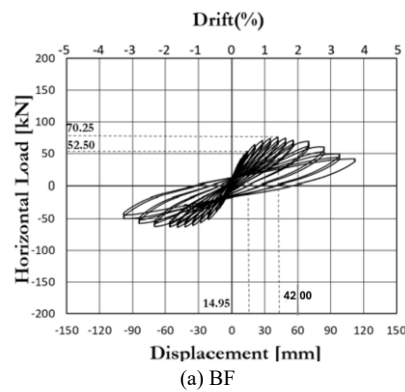
(e) story drift at 5.0%

Figure 7. Failure of the strengthened specimen BFS

Hysteretic Behavior

The hysteretic behavior of the control specimen (BF) and the strengthened specimen (BFS) is compared in Figures 8a and 8b, respectively. The enveloped curves for both specimens are shown in Figure 8c.

For the control specimen (BF), the frame exhibited elastic behavior up to a displacement of 0.50% drift. Beyond this point, the behavior gradually started to a non-linear. The structure maintained the moment resistance up to a displacement of 1.35% drift, reaching a peak load of 70.25 kN and a corresponding lateral displacement of 42.00 mm. After reaching this peak, the resistance gradually decreased until the end of the test at 4.0% drift.



For the strengthened specimen (BFS), the frame maintained linear elastic behavior up to a displacement of 0.40% drift. At this drift level, the strength reached 90.90 kN with a corresponding lateral displacement of 12.35 mm. Beyond this drift level, the frame's behavior started to show plastic deformation. The specimen maintained its lateral strength until a displacement of 2.20% drift, achieving an ultimate strength of 153.75 kN and a corresponding lateral displacement of 64.36 mm. The frame's strength then gradually decreased until it reached the target drift of approximately 4.50%. The hysteresis loop for the BFS specimen demonstrates significantly higher displacement values compared to the original frame. This indicates that the strengthening of the frame effectively enhances its lateral strength and ductility.

Lateral Strength

The lateral strength of the specimens at the yield point was calculated using the slope of the force-displacement graph. This point was determined by the intersection of the initial stiffness and the post-yield stiffness on a bilinear representation curve, following the method described by Panyakapo (2014). The experimental results are summarized in Table 2. The findings revealed that the load capacity, stiffness, and ductility at the yield point of the strengthened specimen (BFS) were 73%, 110%, and 47% higher, respectively, compared to the control specimen (BF). The maximum strength levels are shown in Table 3, where the results indicate that the maximum load capacity and displacement of the BFS specimen were 119% and 53% higher, respectively, than those of the BF specimen. These results clearly demonstrate that ferrocement strengthening combined with external rebar reinforcement significantly enhance the lateral strength and ductility of the frame.

Table 2. Strength, stiffness, ductility, at the yield point

Type of frame	BF	BF-S	Difference (%)
$V_y (kN)$	52.50	90.90	73
$\Delta_y (mm)$	14.95	12.35	21
$\Delta_u (mm)$	112.00	135.61	21
$k_o (kN/m)$	3.51	7.36	110
$\alpha k_o (kN/mm)$	0.66	1.21	83
Ductility	7.49	10.98	47

Figure 8. Hysteresis of the samples test

Table 3. Maximum strength and displacement, at the ultimate point

Type of specimens	(BF)	(BF-S)	Difference (%)
$V(kN)_{max}$	70.25	153.75	119
$\Delta(mm)_{max}$	42.00	64.36	53
$Drift_m(\%)$	1.50	2.50	67
$V_u(kN)$	37.20	101.55	173
$\Delta_u(mm)$	112.00	135.61	21
$Drift_u(\%)$	3.50	5.00	43

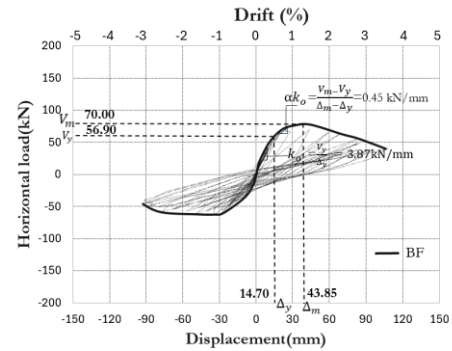
The parameters for moment capacity and design capacity of the BF and BFS specimens are presented in Table 4. These parameters were used to simulate the behavior of the rigid bare frame, with calculations based on equations (5) and (6) as described above. For the control specimen (BF), the maximum moment (M_{exc}), the plastic moment capacity of the beam-column joint (M_{pj}), and the lateral resistance capacity (R_{BF}) were calculated as 57.69 kN-m, 46.04 kN-m, and 70 kN, respectively. For the strengthened specimen (BFS), the calculated parameters include the moment resistance of the ferrocement with expanded metal mesh ($M_n^F = 38.50$ kN-m), by equation (2), the moment resistance of the external bar reinforcement ($M_{ext} = 78.40$ kN-m), by equation (3), the moment resistance of the strengthened column ($M_{SC} = 174.59$ kN-m), by equation (1), and the lateral resistance capacity ($R_{BFS} = 157.56$ kN), by equation (5). These calculations were performed using equations (1) - (6), as outlined in the study. In addition, the moment resistance contributed by the ferrocement ($M_n^F = 38.50$ kN-m), the external reinforcement ($M_{ext} = 78.40$ kN-m), and the original column ($M_{exc} = 57.69$ kN-m) were evaluated based on the moment capacity of the strengthened frame ($M_{SC} = 174.59$ kN-m). The proportions of the ferrocement, the external reinforcement, and the original column were weighted as follows: $M_n^F = (38.50/174.59) \times 100 = 22\%$, $M_{ext} = (78.40/174.59) \times 100 = 45\%$, and $M_{exc} = (57.69/174.59) \times 100 = 33\%$, respectively.

Table 4. Moment capacity and design capacity of BF and BF-S

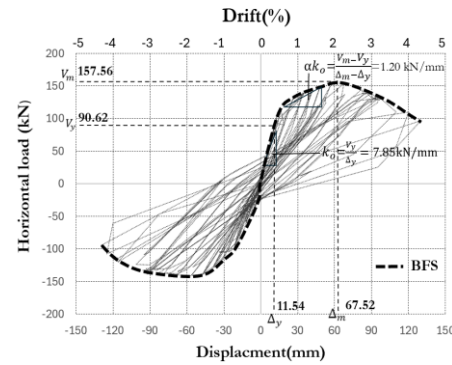
Specimens	(BF)	(BF-S)	Difference (%)
$M_{exc}(kN - m)$	57.69	57.69	-
$M_{pj}(kN - m)$	46.06	46.06	-
$M_n^F(kN - m)$	-	38.50	-
$M_{ext}(kN - m)$	-	78.40	-
$M_{sc}(kN - m)$	-	174.59	-
Design capacity (kN)	70.00	157.56	125

Verification of Analytical Modelling of Bare Frame

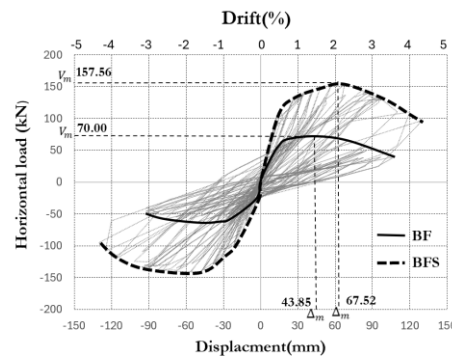
The analysis of the bare frame model was conducted to verify the proposed analytical model. In this study, the moment capacity parameters of BF and BFS, as shown in Table 4, were employed for the simulation of the hysteresis behaviors using the RUAUMOKO program (Carr, 2006). In the modelling of column and beam, the SINA degrading model (Saiedi and Sozen, 1979) was used to represent the flexural and shear hysteretic behavior of the column and beam.



(a) Behavior of enveloped curves (BF)



(b) Behavior of enveloped curves (BFS)



(c) Behavior of enveloped curves (BF, BFS)

Figure 9. Simulation results of RC frame by RUAUMOKO Program

The results of the analysis by RUAUMOKO for BF and BFS are presented in Figures 9a, 9b, and 9c. The parameters of the hysteresis behavior of the BF and BFS resulting from the enveloped curves were also presented in Figures 9a, and 9b. These parameters were summarized in Table 5.

Table 5. Parameters for the hysteresis behavior of the BF and BFS resulting from the enveloped curves

Model	(BF)	(BF-S)	Difference (%)
$k_o(kN/m)$	3.87	7.85	103
$V_y(kN)$	56.90	90.62	59
$\Delta_y(mm)$	14.70	11.54	27
$V_m(kN)$	70.00	157.56	125
$\Delta_m(mm)$	43.85	67.52	54
$\alpha k_o(kN/mm)$	0.45	1.20	167

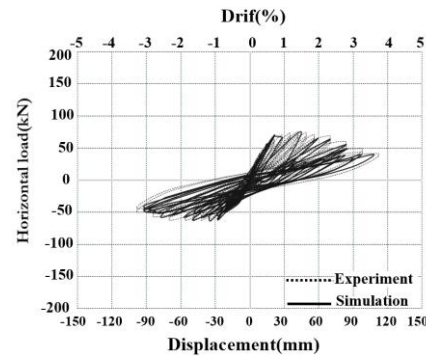
A comparison between the hysteresis behavior parameters of the test results and the analysis results of BF and BFS are summarized in Table 6. It can be observed that the hysteresis behavior parameters of the analysis results are close to those of the test results. Particularly, the maximum lateral resistance resulting from the laboratory test and the analysis showed a difference of 2.48%.

The comparison of these parameters indicates that the proposed model of strengthening the rigid frame can be effectively used to evaluate the enveloped curves of the lateral strength and stiffness.

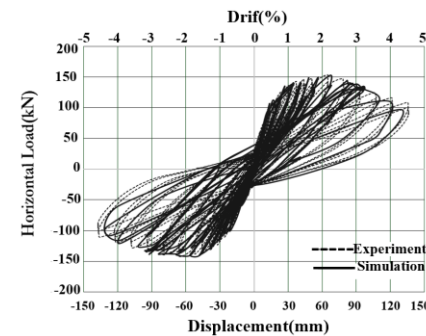
Table 6. A comparison between the hysteresis behavior parameters of the test results and the analysis results of BF and BFS

Parameter	Test results		Analysis		Difference (%)	
	BF	BFS	BF	BFS	BF	BFS
$k_o(kN/m)$	3.51	7.36	3.87	7.85	10.26	6.66
$V_y(kN)$	52.50	90.90	56.90	90.62	8.38	0.31
$\Delta_y(mm)$	14.95	12.35	14.70	11.54	1.70	7.02
$V_m(kN)$	70.25	153.75	70.00	157.56	0.36	2.48
$\Delta_m(mm)$	42.00	64.36	43.85	67.52	4.40	4.91
$\alpha k_o(kN/mm)$	0.66	1.21	0.45	1.20	46.67	0.83

The hysteresis loops of the simulation and the experimental results are compared for BF and BFS in Figures 10a and 10b, respectively. Notably, the two samples demonstrated a very good agreement in the hysteresis loop shape. This indicates a strong correlation between the predicted behavior of the model and the actual response, suggesting that the proposed model is both highly accurate and reliable.



(a) Experiment and Simulation of BF



(b) Experiment and Simulation of BFS

Figure 10. Simulation and experimental results of RC frame

Conclusions

This research focuses on studying the seismic strengthening of the existing reinforced concrete (RC) rigid frame. The model was strengthened using ferrocement with expanded metal mesh and external longitudinal reinforcement to enhance its resistance to flexural and shear failure at the column ends. The model was tested under axial compression and cyclic loading. The conclusions are as follows:

a) The strengthened RC frame (BFS) exhibited significant improvements in yield strength, stiffness, and ductility compared to the original bare frame (BF). These were enhanced by 73%, 110%, and 47%, respectively. The strengthening by the ferrocement method using expanded metal mesh and external longitudinal reinforcement notably improved the frame's bending moment resistance.

b) The maximum strength of the strengthened bare frame was significantly higher than the original bare frame. Specifically, the frame's maximum strength and lateral displacement improved by up to 119% and 53%, respectively. This improvement is primarily attributed to the confinement of the

columns using expanded metal mesh combined with the ferrocement method, as well as the increased flexural resistance provided by external longitudinal reinforcement. The test results indicate that the structure remains well-maintained after these enhancements, as the strengthened columns and beam ends effectively prevent shear and flexural failures.

c) The analytical modeling of the frames, conducted using the RUAUMOKO software, effectively verified the proposed model. Validation was achieved by comparing the analytical results with the experimental results. This demonstrates that the proposed model is both reasonable and accurate in predicting the structural behavior.

d) The increased moment resistance observed in the strengthened column is primarily attributed to the combined effects of ferrocement, external reinforcement, and the inherent capacity of the original column, which contributed approximately 22%, 45%, and 33% of the total moment capacity, respectively. Notably, the external reinforcement exhibited a more substantial influence on the structural enhancement compared to the ferrocement layer, indicating its effectiveness as a primary strengthening mechanism.

Recommendation

The present study is limited by the inclusion of only two specimens, which constrains the generalization of the findings. To address this limitation, future research is encouraged to extend the experimental program by incorporating a larger number of specimens. It is further recommended to investigate alternative structural configurations, such as circular column cross-sections and frame geometries with increased span lengths and story heights. Moreover, the use of larger column and beam sections should be considered to more accurately capture structural behavior under varying conditions. These enhancements would contribute to a more comprehensive validation of the current findings and improve the overall robustness and applicability of the conclusions.

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