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# Seismic shear strengthening of R/C columns with ferrocement jacket

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**Abstract**

This paper presents results of an experimental study to evaluate a retrofit technique for strengthening shear deficient short concrete columns. In this technique a ferrocement jacket reinforced with expanded steel meshes is used for retrofitting. Six short concrete columns, including four strengthened specimens, were tested. Specimens were under a constant compressive axial force of 15% of column axial load capacity based on original concrete gross, $A_g$, and the concrete compressive strength, $f'_c$. Main variables were the spacing of ties in original specimens and the volume fraction of expanded metal in jackets. Original specimens failed before reaching their nominal calculated flexural strength, $M_n$, and had very poor ductility. Strengthened specimens reached nominal flexural strength and had a ductility capacity factor of up to 5.5. Based on the test results, it can be concluded that ferrocement jackets reinforced with expanded steel meshes can be used effectively to strengthen shear deficient concrete columns.


*Keywords:* Ferrocement; Expanded steel mesh; Concrete; Short column; Ductility; Shear strength; Retrofit; Strengthening

## 1. Introduction

A large number of existing reinforced concrete structures, which were designed in accordance with old design provisions, could be considered as having poor performance during earthquakes. Many of the existing short columns have poor seismic detailing. Due to short dowels and little transverse reinforcement, risk of brittle shear failure in such members is very high. Premature shear failure prevents formation of flexural plastic hinges and decreases ductility capacity. In some structures such as bridges, in order to have a good amount of ductility capacity, the plastic hinges must form at the ends of columns. It is very important to develop efficient techniques to retrofit shear critical columns and increase their ductility capacity. Wrapping concrete columns with a proper strengthening material can be an effective solution. Reinforced concrete, steel plates, steel straps and fiber-reinforced polymer, FRP, composites are common retrofit techniques that have been investigated by many researchers [1–9]. Although each technique may have some disadvantages that limit its use for practical applications, all above techniques could improve ductility capacity and shear strength of short columns in laboratory tests.

In this investigation, a ferrocement jacket reinforced with expanded steel meshes was used to retrofit shear deficient concrete column specimens. Although ferrocement has been used as a structural material for more than 50 years [10], but its application in shear strengthening of concrete columns is more recent. Takiguchi and Abdullah performed an experimental research program on the use of ferrocement jackets reinforced with wire meshes for strengthening small square reinforced concrete columns (120 × 120 mm cross-section) with inadequate shear strength [11,12]. Based on the test results, both circular and square ferrocement jacket could enhance the shear strength and ductility of small-scale

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**Nomenclature**

| | |
|---|---|
| $A_g$ | gross cross-sectional area |
| $A_{st}$ | overall area of longitudinal reinforcement |
| $a$ | shear span |
| $b, h$ | dimensions of concrete column section |
| $d$ | effective depth |
| $f'_c$ | compressive strength of concrete |
| $f_y$ | yield strength of longitudinal reinforcement |
| $f_{yf}$ | yield strength of ferrocement reinforcement |
| $f_{yv}$ | yield strength of transverse reinforcement |
| $g$ | ratio of distance between the centers of longitudinal reinforcement in tension and compression to the column thickness |
| $N$ | axial compression force |
| $t_f$ | thickness of ferrocement jacket |
| $V_f$ | volume fraction of ferrocement reinforcement |
| $V_m$ | the shear force associated with the ultimate flexural capacity |
| $V_{sf}$ | shear strength of ferrocement jacket |
| $V_n$ | ultimate shear strength |
| $W_{ca}$ | actual dissipated energy per cycle (enclosed area in each cycle of load–displacement curve) |
| $W_{cp}$ | full plastic dissipated energy per cycle (Four times the product of member ultimate strength and the maximum displacement for cycle) |
| $\Delta_u$ | postpeak load point displacement corresponding to 80% of peak load |
| $\Delta_y$ | yield load point displacement calculated based on bilinear approximation of load–displacement curve |
| $\mu_\Delta$ | displacement ductility factor (defined in Fig. 3) |
| $\rho$ | tension reinforcement ratio |
| $\rho_v$ | web shear reinforcement ratio |
| $\eta$ | global efficiency factor for ferrocement reinforcement |

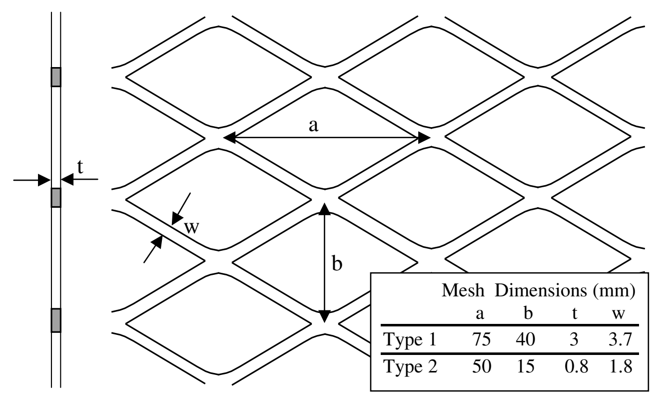
square column specimens. Ferrocement can be an ideal retrofitting material especially in developing countries. Material cost is relatively low and it is durable and does not need any special fire or corrosion protection. Its manufacture is easy and needs no advanced technique.

## 2. Research significance

The main purpose of this study was to evaluate the effectiveness of expanded steel mesh reinforced ferrocement jacket in retrofitting shear critical columns. This was achieved by comparing the behavior of retrofitted columns with that of shear deficient columns under cyclic lateral loading. The proposed jacketing technique could be considered as a competitive alternative to enhance seismic performance of shear deficient concrete columns especially in developing countries.

## 3. Ferrocement and expanded steel meshes

Ferrocement is a thin-wall composite material normally with isotropic behavior in two principal directions in its plane. In some cases, it is more desirable to have only one strong direction. This type of ferrocement, that has an orthotropic behavior, is achieved by using expanded steel meshes. Slitting steel sheets and expanding them in a direction perpendicular to the slits form expanded steel sheets. Expanded steel sheets have a diamond-shaped mesh pattern (Fig. 1). Rolling could




| Mesh | Dimensions (mm) | | | |
|---|---|---|---|---|
| | a | b | t | w |
| Type 1 | 75 | 40 | 3 | 3.7 |
| Type 2 | 50 | 15 | 0.8 | 1.8 |

Fig. 1. Details of flattened expanded steel meshes.

flatten these sheets and enhances their performance as reinforcement in concrete or mortar [13]. This type of ferrocement is stronger and relatively stiffer in the long diagonal direction of diamonds and has lower strength and stiffness in the perpendicular direction [10,14]. By wrapping the concrete column with layers of expanded metal and mortar so that the short diagonal direction is in the longitudinal direction of member, the effect of ferrocement jacket on flexural strength of the column will be minimized. This technique is, particularly, similar to the strengthening of columns using FRP composite straps.

Two types of expanded meshes were used in the tests (Fig. 1). Although the expanded metal sheets were formed from mild steel plates, tensile test of steel specimens, cut from the expanded mesh, did not show any indication of yield plateau. They had an average ultimate strength of 470 MPa and nominal offset yield strength of 400 MPa. Plastic deformations developed in manufacturing process of expanded metal could be the reason. As recommended by Refs. [10,14], for better understanding of mechanical properties of strengthening jacket, three ferrocement specimens reinforced with expanded meshes were tested under tension in the long and short diagonal direction of expanded mesh (Fig. 2). These specimens were reinforced with one layer of Type I expanded mesh. The volume fraction of expanded mesh for all of the specimens was 0.022. Fig. 3 shows experimental load–displacement relations for tensile ferrocement specimens. Ductility factors of 3.5 and 5, as defined in Fig. 4, were observed in the long diagonal direction of expanded mesh that proves a ductile behavior for composite jacket. Concrete panels with well distributed reinforcement, behave more ductile and are less size dependent. A low strength with brittle failure was observed when the tensile force applied in the short diagonal direction of expanded mesh (Fig. 3).

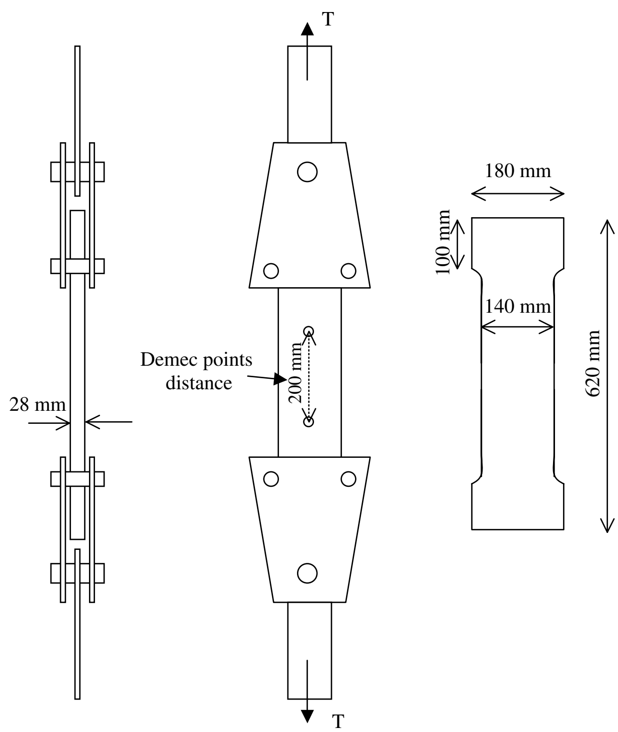



Fig. 2. Details of tensile ferrocement specimens.

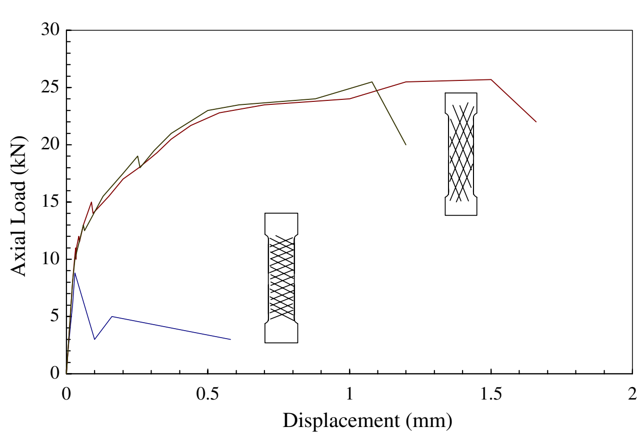



Fig. 3. Load–displacement curves of tensile ferrocement specimens.

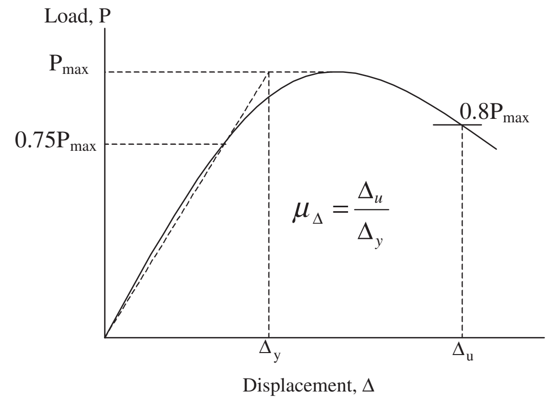



Fig. 4. Definition of displacement ductility capacity factor, $\mu_\Delta$.

## 4. Experimental program

Six shear-critical short concrete columns (three two-sided specimens) were built. This special shape was designed for economy and ease of comparison (Fig. 5). Each two-sided specimen consists of a thicker and stronger middle part and two cantilever parts at the sides. The middle part was fixed to the testing machine frame and each cantilever part, while was under a constant axial compression force, was tested separately under cyclic lateral loading. Original specimens had square

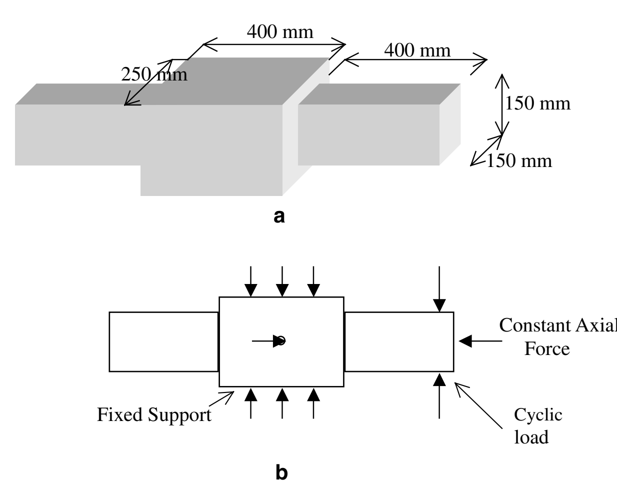



Fig. 5. (a) Dimensions of test specimens and (b) schematic test setup.

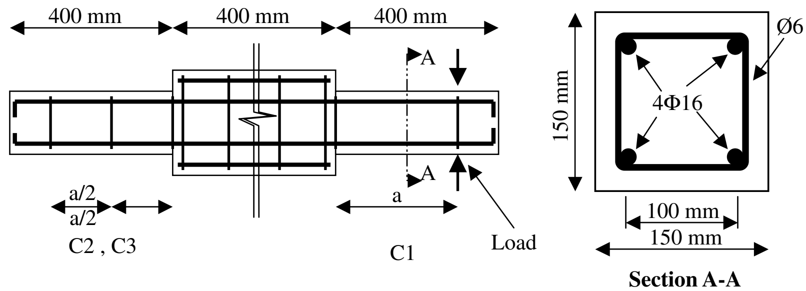



Fig. 6. Original test specimens' reinforcement, $a = 260$ mm.

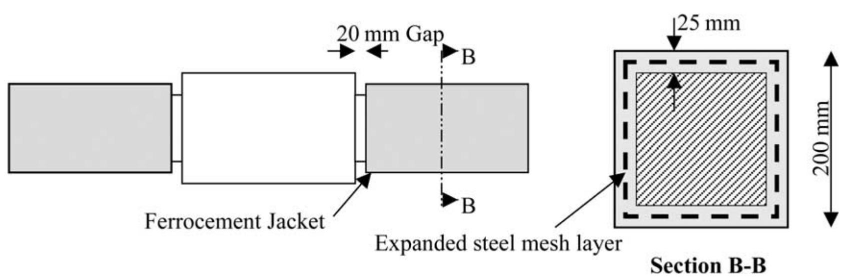



Fig. 7. Details of retrofitted specimens.

cross-sections with 150 mm sides and were reinforced uniformly (Fig. 6). Tie spacing was the only variable before strengthening. Four specimens were strengthened with 25 mm thick ferrocement jackets (Fig. 7): three jackets reinforced with expanded steel meshes and one with ties. There was a 20 mm gap between strengthening jacket and the base, the point of maximum bending moment, to minimize the increase in flexural capacity (Fig. 7). The variables were volume fraction and size of expanded meshes. Specimens were tested as cantilever columns under constant axial compression force of 0.15 $f'_c A_g$. The axial compression force was not very high because most of shear critical concrete columns, such as bridge columns, have low axial force. Shear span, the distance between lateral load and column base ($a = 260$ mm), was about two times the effective depth of column's section. Details of specimens are presented in Table 1 and Figs. 5–7.

A concrete mix with Type I Portland cement, river sand and crushed limestone with maximum size of 19 mm and water to cement ratio equal to 0.5 was used. The slump of the fresh concrete was about 70 mm. Test specimens and their respective concrete cylindrical samples were cured in a moist room at a temperature of about 23 °C to the test time. Average compressive strength of concrete cylindrical samples, $f'_c$, at the test time was 35 MPa.

Longitudinal reinforcement in all samples consist of four $\phi$16 deformed bars with the yield strength of 500 MPa (Fig. 6). Transverse reinforcement was $\phi$ 6 mm steel tie with the yield strength of 300 MPa.

Table 1
Details of test specimens

Two-sided specimens before strengthening

| Specimen | Main rebars | Transverse reinforcement (mm) | $f'_c$ (MPa) |
|---|---|---|---|
| C1 | 4$\Phi$16 | $\phi$6@260 | 35 |
| C2 and C3 | 4$\Phi$16 | $\phi$6@130 | 35 |

Strengthened specimens

| Specimen | Jacket reinforcement | Mortar strength (MPa) |
|---|---|---|
| C1-SC | One Layer Exp. mesh, Type 1, $V_f^a = 0.024$ | 30 |
| C2-SF | Two Layer Exp. mesh, Type 2, $V_f = 0.016$ | 30 |
| C3-SF | One Layer Exp. mesh, Type 2, $V_f = 0.008$ | 30 |
| C3-ST | Tie$\Phi$8 @ 90 mm, $f_y = 300$ MPa, $V_f = 0.024$ | 30 |

[a] Volume fraction of ferrocement reinforcement.

A shrinkage compensated mortar was prepared for jacketing by mixing fine sand and Type I Portland cement in equal amounts and expansive grouting admixture at a ratio of 0.01 cement weight. The average compressive strength of 50 mm mortar cube samples was 30 MPa. Two types of flattened expanded steel meshes were used for the jacket reinforcement, Type 1 with coarse mesh (specimen C1-SC) and type 2 with fine mesh (specimens C2-SF and C3-SF) (Fig. 1). Surface unit masses of these expanded steel meshes were 4.8 kg/m$^2$ and 1.6 kg/m$^2$, respectively. For comparison, one of the specimens was retrofitted with a mortar jacket reinforced with 8 mm deformed bar ties (Table 1).

## 5. Testing procedure and results

Reversed lateral load was applied by a Dartec 1000 kN universal testing machine with specially designed attachments and setup. After fixing the specimen to the testing machine frame, axial load was applied at its predetermined level, $0.15\,f'_cA_g$, by a manual hydraulic jack (Fig. 8). The incremental lateral displacement reversals were applied, with 2 mm increments in the first 5 displacement steps and 5 mm increments in subsequent displacement steps. Two cycles were performed at each displacement step. Hysteresis curves of lateral load versus lateral displacement are shown in Fig. 9. Some pinching is seen in hysteretic behavior that may be due to concrete cracking, bond failure and interface slip.

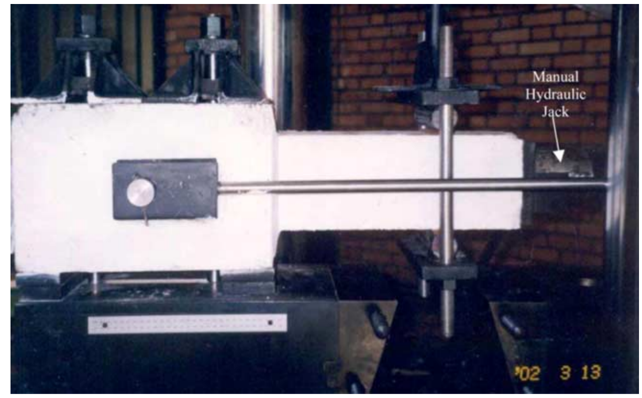



Fig. 8. Column specimens setup.

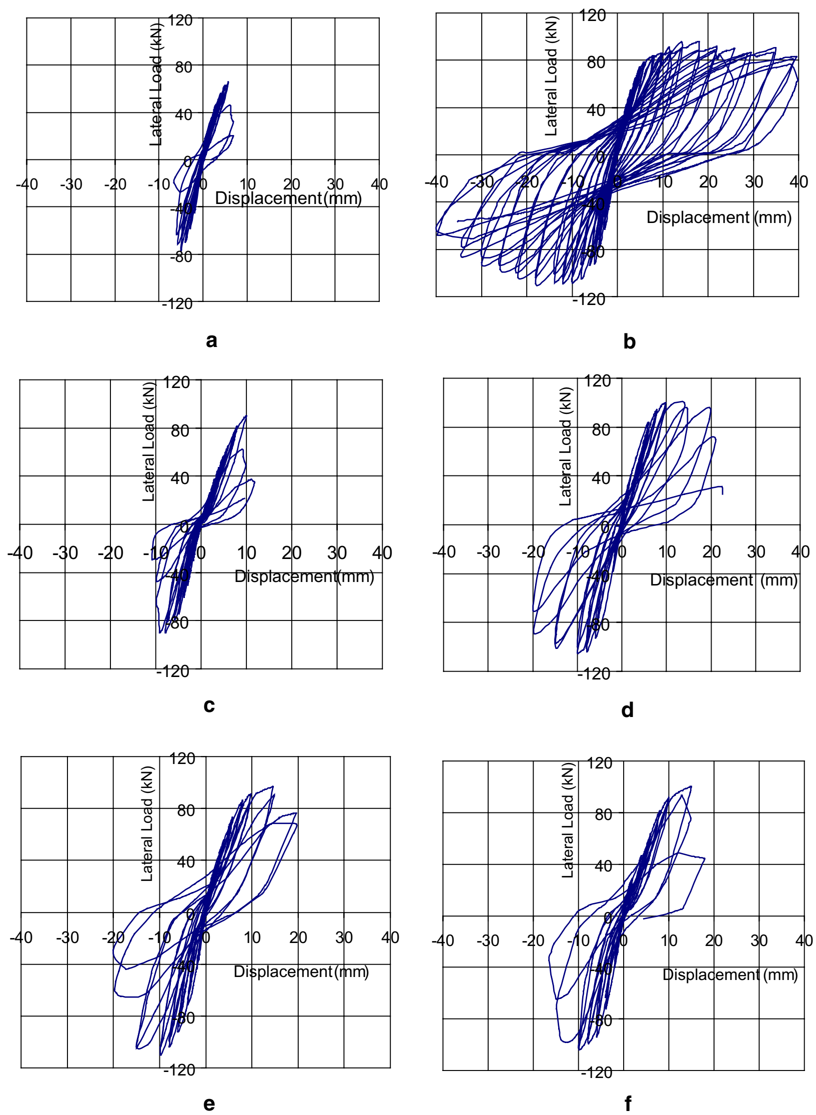



Fig. 9. Hysteresis curves for column specimens: (a) specimen C1, (b) specimen C1-SC, (c) specimen C2, (d) specimen C2-SF, (e) specimen C3-ST, (f) specimen C3-SF.

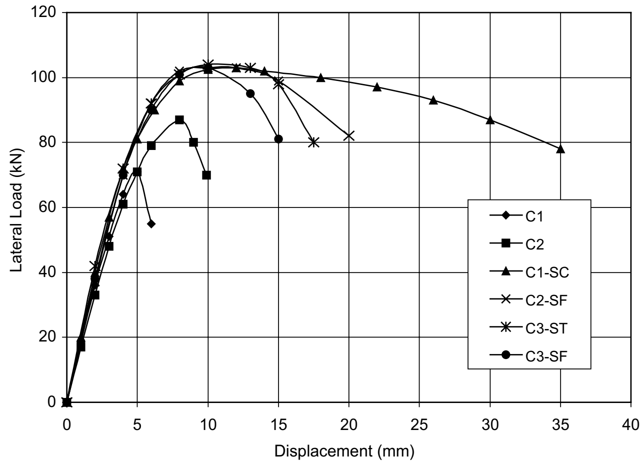



Fig. 10. Envelopes of hysteresis curves for columns.

There is also some decrease in lateral stiffness in successive loops. In Fig. 10, envelopes of cyclic experimental results are compared.

### 5.1. Displacement ductility

Table 2, summarizes the ductility factor and limit lateral displacements for column specimens. The displacement ductility factor is defined as the ratio of the displacement at a point corresponding to 80% of the maximum lateral load on the descending branch to the yield displacement (Fig. 4). The yield displacement is calculated based on bilinear approximation of load–displacement envelope. Both of the original specimens (C1 and C2) had a brittle shear failure before reaching their nominal flexural strength capacity and showed poor ductility. All of the retrofitted specimens reached their calculated nominal flexural strength, but those retrofitted with lower volume fraction of expanded mesh could not keep their capacity at higher displacement ductility levels (specimens C2-SF and C3-SF). Test results show the interaction between shear strength and ductility. At higher ductility levels the shear strength of specimen decreases and more shear reinforcement is needed to keep the required shear strength [15,16]. Specimen C1-SC showed a ductility factor about twice that of the specimen C3-ST, which was retrofitted with ties

Table 2
Limit displacements and ductility factors

| Specimen | $\Delta_y$ (mm) | $\Delta_u$[a] (mm) | $\mu_\Delta$ |
|---|---|---|---|
| C1 | 4.5 | 5.5 | 1.2 |
| C2 | 7 | 10 | 1.4 |
| C1-SC | 6.2 | 34 | 5.5 |
| C2-SF | 6 | 20 | 3.5 |
| C3-ST | 6 | 17 | 2.8 |
| C3-SF | 6 | 15 | 2.5 |

[a] Postpeak displacement corresponding to 80% of peak load.

with the same steel volume and approximately similar yield strength. Observed ultimate lateral displacements were between 6% and 13% of 260 mm shear span that are very noticeable for short columns.

### 5.2. Energy dissipation

Energy dissipation ability of structural members plays an important role in the behavior of structures against earthquakes. Members with the perfect plastic behavior dissipate some energy per each cycle of loading, $W_{cp}$, which is equal to four times the product of member plastic strength and the maximum displacement for the cycle. The dimensionless ratio of actual dissipated energy per cycle, $W_{ca}$, to $W_{cp}$ is selected for the comparison study of test results. This ratio is compared for the tested columns in Fig. 11. Strengthened specimens have much more energy dissipation ability and this ability increases with the increase of strengthening steel volume. Ferrocement strengthened specimen C1-SC dissipated more energy than tie strengthened specimen C3-ST with the same steel volume.

### 5.3. Shear strength

Shear deficiency estimation of column specimens is an important issue. Specially, decrease of shear strength with the increase of displacement ductility must be considered in design. Our test specimens had small size, which resulted a higher shear strength than predicted by codes [17]. Although size effect is considered in some design procedures concerning the shear strength of concrete columns, large-scale tests are needed to complete this research. The shear strength of concrete columns can be estimated as the sum of concrete, transverse reinforcement and axial load components [15]. The concrete component reduces with increasing ductility [15] and size [17]. ACI approach does not consider the influences of ductility and size [18]. The shear strength of original

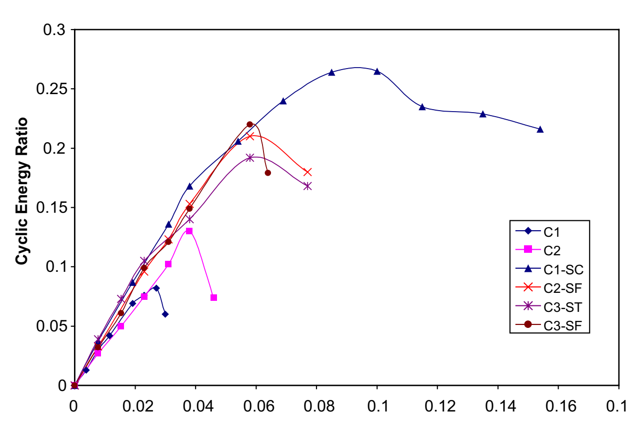



Fig. 11. Comparison of cyclic energy ratio for column specimens.

columns may be evaluated based on modified Ohno–Arakawa's equation, which considers the size effect [19]:

$$V_n = \left[0.115 K_s K_\rho \frac{17.6 + f'_c}{\frac{a}{d} + 0.12} + 0.85\sqrt{\rho_v f_{yv}} + 0.1\frac{N}{bh}\right] bjd \tag{1}$$

where

$$K_s = \begin{cases} 1.0, & d \leqslant 160 \text{ mm} \\ 1.19 - 0.0012d, & 160 \text{ mm} < d \leqslant 400 \text{ mm} \\ 0.72, & d > 400 \text{ mm} \end{cases} \tag{2}$$

$$K_\rho = 0.82(100\rho)^{0.23} \tag{3}$$

$$\rho_v = \frac{A_v}{bs} \tag{4}$$

$f'_c$ and $f_{yv}$ are concrete compressive strength and transverse reinforcement yield strength, respectively (in MPa); $b$ and $h$ are column dimensions (in mm); $d$ is the effective depth and $jd$ is distance between the tensile and compressive force resultants; $\rho$ and $\rho_v$ are tension and shear reinforcement ratios, respectively; a is the shear span; $N$ is the compressive axial force (in $N$); $A_v$ and $s$ are the cross-sectional area and spacing of shear reinforcements. Based on above equations, the ultimate shear strength of original columns is estimated about 60 kN for specimen C1 and 74 kN for specimens C2 and C3.

Interaction between displacement ductility and shear strength is considered in the following equation [16]:

$$\mu_\Delta = 10\left[\frac{V_n}{V_m} - 0.9\right] \tag{5}$$

where

$$V_m = \frac{M_n}{a} \tag{6}$$

is the shear strength associated to the ultimate flexural capacity, $M_n$.

$M_n$ for low axial load can be computed as follows [16]:

$$M_n = 0.5 A_{st} f_y gh + 0.5Nh\left(1 - \frac{N}{bhf'_c}\right) \tag{7}$$

where $A_{st}$ is the total longitudinal reinforcement ratio and $g$ is the ratio of distance between the centers of longitudinal reinforcement in tension and compression to the column thickness.

In the original column specimens, the shear strength, $V_n$, was less than $V_m$, that was calculated about 105 kN for all specimens, and shear brittle failures were observed. After strengthening, the specimens had enough shear strength capacity to show a ductile behavior. This shear strength can be considered equal to the sum of the shear strength of original column, $V_{no}$, and the shear strength of ferrocement jacket, $V_{sf}$. If we assume a diagonal crack between the support and lateral load point at failure, as seen in the tests, the shear strength of ferrocement jacket can be estimated by the following simple equation:

$$V_{sf} = 2\eta V_f t_f a_f f_{yf} \tag{8}$$

where $V_f$ = volume fraction of ferrocement reinforcement, $t_f$ = thickness of ferrocement jacket, $a_f$ = distance between load point and edge of the jacket (a gap distance less than shear span), $\eta$ = global efficiency factor for ferrocement reinforcement [14] (0.65 for long diagonal direction of expanded mesh), $f_{yf}$ = the yield strength of ferrocement reinforcement.

The above equation can also be used for ties reinforced jacket with $\eta = 1$. Total shear strength of strengthened column is

$$V_n = V_{no} + V_{sf} \tag{9}$$

where $V_{no}$ is calculated from Eq. (1).

In Table 3, the estimated and observed displacement ductility factors are compared for strengthened column specimens. As seen in Table 3 all ferrocement-retrofitted specimens had better performance than that estimated. The tie-strengthened specimen had a lower ductility than predicted and had a poor performance.

### 5.4. Servicability and cracking

Crack patterns for strengthened and original specimens were different. For the original specimens large dominant diagonal shear cracks were observed in relatively small amount of lateral displacements. In ferrocement-retrofitted specimens, fine distributed shear cracks were observed in jacket at relatively large deflections. These shear cracks were not observed at small deflections even after reaching the ultimate flexural capacity of columns. At larger deflections, the lateral load capacities of retrofitted columns were reduced and shear cracks in jackets were appeared. In tie-retrofitted specimen, C3-ST, concrete and mortar had been completely collapsed at the end of the test (with a lateral displacement of about 20 mm), but the ferrocement layer could prevent from spilling of core concrete in specimen C1-SC even at higher ductility levels (Fig. 12). These

Table 3
Comparison of observed and estimated ductility factors for strengthened columns

| Specimen | Shear strength[a] (kN) | $\mu_\Delta$, estimated[b] | $\mu_\Delta$, observed |
|---|---|---|---|
| C1-SC | 134.5 | 3.8 | 5.5 |
| C2-SF | 124 | 2.8 | 3.5 |
| C3-ST | 160 | 6 | 2.8 |
| C3-SF | 99[c] | – | 2.5 |

[a] Estimated from Eq. (9).
[b] Estimated from Eq. (5).
[c] Observed value was more than 105 kN.

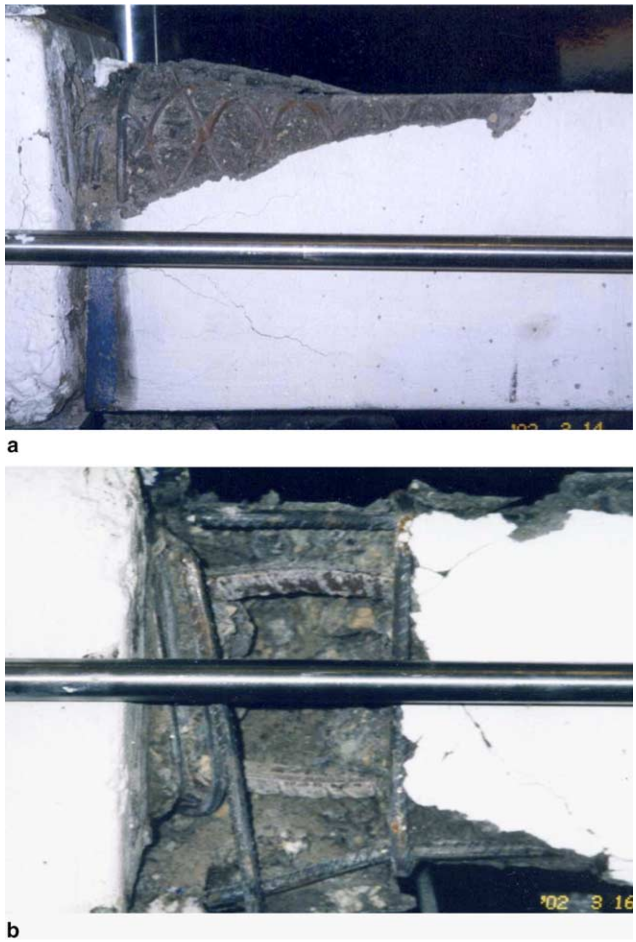



Fig. 12. (a) Specimen C1-SC and (b) specimen C3-ST after completion of tests.

observations indicate the good performance of expanded steel meshes in retrofitting concrete columns.

## 6. Summary and conclusions

Results from an experimental research are presented in which six short concrete column specimens were tested under constant axial compression force of 0.15 $f'_c A_g$ and cyclic lateral load. Three specimens were retrofitted with ferrocement jackets reinforced with expanded steel meshes and one specimen was retrofitted with ties before testing. The following conclusions can be drawn from this study.

1. Wrapping short shear critical concrete column specimens, with an expanded steel mesh reinforced ferrocement layer, significantly increased their shear strength and ductility capacity.
2. Expanded meshes were more effective than ties in shear strengthening of concrete columns.
3. Even a small amount of expanded steel meshes ($V_f = 0.008$) increased shear strength, considerably, but a larger steel volume ($V_f = 0.024$) was needed to attain a good amount of ductility capacity.
4. Ferrocement jacketing decreased shear cracking. Concrete specimens that were strengthened with expanded meshes showed distributed fine shear cracking even at the large amounts of displacement ductility capacity.
5. The specimens were of small size, which affected the results and made the columns less brittle. Columns with larger dimensions are more brittle and shear critical. For the size effect factor, Eq. (2) or any other suitable relation can be used.

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