The Twelfth East Asia-Pacific Conference on Structural Engineering and Construction

AXIAL LOAD BEHAVIOR OF CONCRETE COLUMNS WITH Welded Wire Fabric AS TRANSVERSE REINFORCEMENT

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Abstract

A new reinforcement system, Welded Wire Fabric (WWF), is proposed to perform the function of transverse steel in reinforced concrete columns. WWF is made from cold-drawn steel wires arranged in two orthogonal directions and is prefabricated in a production line. WWF reinforcement eliminates some of the detailing problems inherent in traditional rebar in the reinforced concrete construction resulting in easier and faster construction, and better economy and quality control. An experimental investigation on the behavior of square concrete columns confined by WWF was carried out. The confinement provided by WWF was investigated by comparing the results from 20 short column tests. The specimens were tested under axially loading. The effects of volumetric ratio, spacing and grid configuration of WWF reinforcement as well as the distribution of longitudinal reinforcement on the behavior of columns were investigated. Based on the observation, the results indicated that the use of WWF as transverse steel has resulted in considerable enhancement both in strength and ductility.

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Keywords: Axial load, Columns, Confinement reinforcement, Ductility, Reinforced concrete, Steel reinforcement, Strength, Welded wire fabric.

1. INTRODUCTION

Welded wire fabric (WWF) generally consists of wires arranged in two orthogonal directions and is prefabricated in a production line. Because of its economy, ease, and faster of construction as well as better quality control, WWF has been widely used in buildings. Recent studies (Saatcioglu and Grira

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1999; Lambert and Tabsh 2001; Tavio et al. 2008; Kusuma et al. 2010) have also shown that WWF can be a good substitute for the conventional reinforcement and yielded excellent results both in strength and ductility. However, there is limited research concerning WWF in Indonesia. This is because of a little is known about the structural behavior of RC columns confined by WWF.

This paper presents an experimental study on the axial load behavior of concrete columns confined by WWF as transverse reinforcement. A total of 20 column specimens were made in this study. Two of the specimens contained closed type conventional steel bars as transverse reinforcement, while the other eighteen contained WWF as transverse reinforcement. The parameters investigated in this research included the volumetric ratio, spacing and grid configuration of WWF reinforcement, and the distribution of longitudinal reinforcement. This paper reported an extensive experimental program, the corresponding results, and the conclusions drawn therefrom. Based on the observation, the results indicated that the use of WWF as transverse steel has resulted in considerable enhancement both in strength and ductility.

2. EXPERIMENTAL PROGRAM

2.1. Test matrix

The test matrix, presented in Table 1, includes a total of twenty compression column specimens. The test matrix was arranged to evaluate the influence of volumetric steel ratio, spacing of WWF reinforcement, WWF grid configuration, and distribution of longitudinal reinforcement and resulting arrangement of WWF grids upon the strength and ductility of reinforced concrete columns. A total of 20 columns of 180 × 180 mm cross section and 720 mm in height, as depicted in Fig. 1, were cast. Each column is described in Table 1. The prefixes “H,” and “W” refer to specimens with hoop and WWF type of confinement, respectively.

Table 1: Description of test specimens

| Specimen | Type | Size, mm | \( s \), mm | \( \rho_s \), % | \( f_{sh} \), MPa | \( \epsilon_{ys} \), % | \( E_s \), GPa | Bars | \( \rho_g \), % | \( f_y \), MPa |
|----------|------|----------|-------------|---------------|----------------|----------------|-------------|------|---------------|-----------|
| W1       | WWF  | 6.93     | 30          | 4.81          | 505            | 0.508          | 173         | 4D12.62 | 1.54          |
| W2       | WWF  | 5.97     | 45          | 3.21          | 502            | 0.522          | 191         | 4D12.62 | 1.54          |
| W3       | WWF  | 30       | 60          | 2.40          | 4D12.62        | 1.54           |            |       |
| W4       | WWF  | 60       | 72          | 2.00          | 4D12.62        | 1.54           |            |       |
| W5       | WWF  | 120      | 90          | 1.60          | 4D12.62        | 1.54           |            |       |
| W6       | WWF  | 120      | 120         | 1.20          | 4D12.62        | 1.54           |            |       |
| W7       | WWF  | 30       | 30          | 4.81          | 2.78           | 199            | 4D12.62    | 1.54    |
| W8       | WWF  | 45       | 45          | 3.21          | 2.78           | 199            | 4D12.62    | 1.54    |
| W9       | WWF  | 72       | 72          | 2.00          | 4D12.62        | 1.54           |            |       |
| W10      | WWF  | 120      | 90          | 1.60          | 4D12.62        | 1.54           |            |       |
| W11      | WWF  | 120      | 120         | 1.20          | 4D12.62        | 1.54           |            |       |
| W12      | WWF  | 30       | 60          | 2.40          | 4D12.62        | 1.54           |            |       |
| W13      | WWF  | 60       | 72          | 2.00          | 4D12.62        | 1.54           |            |       |
| W14      | WWF  | 120      | 90          | 1.60          | 4D12.62        | 1.54           |            |       |
| W15      | WWF  | 120      | 120         | 1.20          | 4D12.62        | 1.54           |            |       |
| W16      | WWF  | 30       | 30          | 4.81          | 2.78           | 199            | 4D12.62    | 1.54    |
| W17      | WWF  | 45       | 45          | 3.21          | 2.78           | 199            | 4D12.62    | 1.54    |
| W18      | WWF  | 72       | 72          | 2.00          | 4D12.62        | 1.54           |            |       |
| H1       | Hoop | 9.77     | 40          | 4.69          | 434            | 0.218          | 199        | 4D12.62 | 1.54          |
| H2       | Hoop | 60       | 60          | 3.13          | 434            | 0.218          | 199        | 4D12.62 | 1.54          |

Table 1 also summarizes the details of each specimen. All specimens had a height-to-square section aspect ratio of 4 to 1. The test specimens had a clear concrete cover between 10 and 12 mm. The design concrete compressive strength was 35 MPa. The two ends of the square columns were heavily reinforced with closely spaced WWF grids (or ties) and confined with external steel collar clamped made from 10
mm thick steel plate of 150 mm in height to avoid premature failure in the end regions of the columns. The casting of the columns was done vertically. Three cylinders of 150 × 300 mm were cast as per the requirements of ASTM C192 for each batch mixer for defining the material properties. The curing conditions for the cylinders and the columns themselves were identical: water bath cured for the twenty-one days and then under ambient laboratory conditions up to the time of testing.

![Column reinforcement details](image)

**Figure 1: Column reinforcement details.**

### 2.2. Test setup and instrumentation

All columns were tested in a universal testing machine with a compressive capacity of 5000 kN with load controlled capabilities in the Structures Laboratory of the Research Center for Human Settlements (RCHS) in Bandung. The columns were prepared in the Laboratory of Concrete and Building Materials of the Sepuluh Nopember Institute of Technology (ITS) in Surabaya, before being shipped to RCHS Laboratory. The columns were tested vertically under concentric loading.

Applied loads were measured by load cells integral to the testing machines. Deformations in the concrete were recorded by linear variable displacement transducers (LVDTs), over a gage length of 320 mm. Electrical resistance strain gages were used to monitor strains in both the tie (or WWF) and longitudinal reinforcement at midheight of all the columns. A total of six or seven strain gages, all placed within the test region, were used on each specimen confined by WWF reinforcement. The strain gages were pasted to the two or three opposite longitudinal steel bars at their middle lengths. In addition, each specimen had four strain gages installed on the WWF reinforcement at approximately middle length of the each specimen. Except, two specimens laterally reinforced with hoop had only four strain gages placed on the one hoops closest to the midheight of the column, and two strain gages mounted at the midheight of two opposite longitudinal bars.
2.3. Loading protocol

Throughout the test setup, care was taken to ensure the load was applied as concentrically as possible. Before the main test, an initial load was applied of about 20 percent of the total ultimate load to each column to check the instrumentation and data acquisition system and then removed. If the readings in the strain gages or the deflections measured by the four longitudinal LVDTs were not approximately equal, this was an indication that the load was not applied concentrically. The column was then unloaded and the ends were shimmed to reduce the load eccentricity. This procedure was repeated until the initial loading was approximately concentric. Although a rigorous procedure was followed for aligning the specimens, some eccentricities were unavoidable.

During testing, the load was applied continuously until the first sign of cover spalling was observed. After the development of initial cracking in the cover concrete, the load was continually applied until the failure of the column. The tests were terminated at the breaking at a welded joint in the corners of the WWF meshes, the crushing of the core concrete, or the buckling of the longitudinal reinforcement. The time taken to complete each test was approximately 40 minutes depending upon the degree of confinement in the specimen.

2.4. Material properties

The compressive strength of each concrete cylinder was slightly different because of the mixing procedure in the Laboratory of Concrete and Building Materials at the Sepuluh Nopember Institute of Technology (ITS), Surabaya, Indonesia. The materials consisted of Ordinary Portland Cement, natural river sand, crushed stone aggregate of maximum size 10 mm, tap water for mixing and curing, superplasticizer admixture to maintain adequate workability of mix. A concrete slump of 210 mm was used to ensure that the concrete could be placed through the dense reinforcement cages. The concrete used in this study was normal weight concrete with an average compressive strength at 28 days of 43.4 MPa. Strength of concrete for all specimens was determined from compression tests of at least twelve 150 × 300 mm standard cylinders at the time of column test. Mean value of the strain corresponding to the maximum stress in concrete, $\varepsilon_c$, was obtained as 0.00258. Poisson’s ratio was found to have an overall mean value of 0.24. The measured secant modulus of elasticity ($E_s$) were 30,122 MPa. Plain concrete compression member specimens were prepared along with the twenty test specimens. The unconfined concrete compressive strength, $f_{uc}$, is taken as 37.6 MPa. The stress-strain curve of the unconfined concrete was determined from the compression test of the plain column. From this test, the axial concrete strain corresponding to the unconfined concrete strength, $\varepsilon_{uc}$, was determined to be 0.00233.

Deformed bars were used to provide longitudinal steel contents of between approximately equal 1.5 and 3.0 percent of the gross cross-sectional area of the column. Deformed steel bars also were used for all transverse reinforcement. Three tension coupons were tested for each type of bar. WWF, used as transverse confinement reinforcement, were manufactured to have a square shape with center-to-center dimension of 150 mm. Two different grid types were used, consisting of: 6.93-mm diameter reinforcement welded to form 4 equal-size square grids and 5.97-mm diameter reinforcement welded to form 9 equal-size square grids. For conventional lateral reinforcement, 10-mm diameter of reinforcing steel was used. The dimension of the wires (or tie) and the young’s modulus and yielding stress and strain values are listed in Table 1. Only the 10 mm-diameter bars exhibited a well-defined yield plateau. For other cases, the yield stress was determined by the 0.2 percent offset method. All conventional ties were anchored with 135-deg hooks and a development length into the concrete core as per the ACI 318-08 code provisions.
3. EXPERIMENTAL RESULTS AND DISCUSSION

3.1. Modes of failure

Columns reinforced by WWF, were observed to fail as a result of breaking at a welded joint of the WWF which supported the maximum lateral pressure from the concrete core. It finally caused lateral buckling in the longitudinal reinforcement. In contrast, columns reinforced by conventional hoops, failed by a mode of extension in the tie hook and longitudinal buckling because of high lateral pressure from the concrete core and axial forces. For both of the conventional hoop columns, it was observed after the tests that the hoops did not open, confirming that they were adequately anchored into the column core. Columns reinforced by WWF, had the same trend as the columns reinforced by conventional hoops. That is the normalized axial force increased slightly when the volumetric reinforcement ratios were increased.

3.2. Strength and ductility enhancement

The load versus average axial shortening (based on the four LVDTs) curves for all of the columns are shown in Fig. 2. The load carried by the concrete $P_{\text{conc}}$ can be obtained by subtracting the load carried by the longitudinal reinforcing bars (derived from the strain data) from the total column load. The concrete load-versus-strain curves for all the columns are then converted to confined concrete stress $f_{cp}$ versus strain curves by dividing their ordinate by $A_c$ (where $A_c = A_g - A_{st}$). $f_{cp}$ is the applied stress on confined concrete in column (where $f_{cp} = P_{\text{conc}}/A_c$). These curves are commonly referred to as confined concrete material curves. To compare directly the effect of confinement on the behavior of the concrete in the columns, it is necessary to normalize the concrete material curves of the columns with respect to their unconfined concrete strengths $f_{c0}$, taken as $0.85f_{c0}$ which commonly used to relate in-place strengths to standard cylinder strengths. These normalized curves are shown in Fig. 3.

A summary of experimental and computed results is shown in Tables 2 and 3. The maximum column load is denoted by $P_{\text{max}}$ and the maximum load carried by the concrete of the column by $P_{c\text{max}}$. The average column strain at which $P_{\text{max}}$ occurs has been denoted by $\varepsilon_{p\text{max}}$ and the average column strain at which $P_{c\text{max}}$ occurs has been denoted by $\varepsilon_{c\text{c}}$. It can be observed from Table 2 that these strains are similar. To provide a means of direct comparison of ductilities in this study, a column is considered to have failed when its load drops to 85 percent of its maximum capacity, a value often used to define the failure of plain concrete. The strain corresponding to this load is called the failure strain and has been denoted by $\varepsilon_{cr}$. Table 2 also lists the strain at which cover spalling was first visually observed. The concrete cover behaved in a similar manner in all twenty specimens. In general, after the maximum axial force, load capacity began to decrease gradually with more visible signs of cracks and spalled concrete covering. Sometimes, there were loud sounds of fractures.

![Figure 2: Total load vs. axial shortening curves: (a) WWF grid 2×2; (b) grid 3×3; (c) hoop.](image-url)
Table 2: Column axial strain and spalling data.

| Specimen | $\varepsilon_{pmax}$ | $\varepsilon_{cc}$ | $\varepsilon_{cf}$ | Strain at start of spalling |
|----------|----------------------|-------------------|-------------------|-----------------------------|
| W1       | 0.01085              | 0.01015           | 0.02362           | 0.00413                     |
| W2       | 0.00414              | 0.00373           | 0.01458           | 0.00405                     |
| W3       | 0.00430              | 0.00430           | 0.01124           | 0.00419                     |
| W4       | 0.00623              | 0.00358           | 0.00966           | 0.00345                     |
| W5       | 0.00545              | 0.00428           | 0.00790           | 0.00428                     |
| W6       | 0.00359              | 0.00359           | 0.00705           | 0.00302                     |
| W7       | 0.01616              | 0.01444           | 0.02330           | 0.00405                     |
| W8       | 0.00851              | 0.00380           | 0.01634           | 0.00371                     |
| W9       | 0.00695              | 0.00407           | 0.01388           | 0.00330                     |
| W10      | 0.01464              | 0.01403           | 0.02185           | 0.00363                     |
| W11      | 0.00975              | 0.00738           | 0.01536           | 0.00340                     |
| W12      | 0.00854              | 0.00808           | 0.01402           | 0.00373                     |
| W13      | 0.00549              | 0.00380           | 0.01311           | 0.00380                     |
| W14      | 0.00506              | 0.00465           | 0.00813           | 0.00301                     |
| W15      | 0.00444              | 0.00415           | 0.00721           | 0.00338                     |
| W16      | 0.01490              | 0.01490           | 0.02363           | 0.00364                     |
| W17      | 0.01133              | 0.00965           | 0.01968           | 0.00390                     |
| W18      | 0.00427              | 0.00427           | 0.01229           | 0.00382                     |
| H1       | 0.00809              | 0.00357           | 0.02499           | 0.00357                     |
| H2       | 0.00556              | 0.00390           | 0.01195           | 0.00345                     |

Figure 2: Continued.

Figure 3: Normalized concrete material curves.
Table 3: Load data and strength enhancement factors.

| Specimen | Experimental capacities | Computed capacities | \( \frac{P_{\text{max}}}{P_o} \) | \( \frac{P_{\text{cmax}}}{P_{oc}} \) | \( \frac{P_{\text{cmax}}}{P_{occ}} \) |
|----------|------------------------|---------------------|----------------|----------------|----------------|
| W1       | 2070                   | 1735               | 1177          | 1.47           | 1.42           | 1.95           |
| W2       | 1820                   | 1602               | 1411          | 1.29           | 1.36           | 1.80           |
| W3       | 1747                   | 1568               | 1177          | 1.24           | 1.33           | 1.76           |
| W4       | 1600                   | 1432               | 1177          | 1.13           | 1.22           | 1.61           |
| W5       | 1537                   | 1257               | 1177          | 1.09           | 1.07           | 1.41           |
| W6       | 1483                   | 1166               | 1177          | 1.05           | 0.99           | 1.31           |
| W7       | 2402                   | 1741               | 1158          | 1.48           | 1.50           | 2.00           |
| W8       | 2148                   | 1611               | 1158          | 1.32           | 1.39           | 1.85           |
| W9       | 1901                   | 1398               | 1158          | 1.17           | 1.21           | 1.60           |
| W10      | 2241                   | 1876               | 1158          | 1.59           | 1.59           | 2.13           |
| W11      | 1855                   | 1538               | 1158          | 1.31           | 1.31           | 1.75           |
| W12      | 1667                   | 1349               | 1158          | 1.18           | 1.15           | 1.53           |
| W13      | 1641                   | 1358               | 1158          | 1.16           | 1.15           | 1.54           |
| W14      | 1588                   | 1319               | 1158          | 1.13           | 1.12           | 1.50           |
| W15      | 1357                   | 1229               | 1158          | 0.96           | 1.04           | 1.40           |
| W16      | 2317                   | 1926               | 1162          | 1.49           | 1.66           | 2.23           |
| W17      | 1999                   | 1594               | 1162          | 1.29           | 1.37           | 1.84           |
| W18      | 1836                   | 1446               | 1162          | 1.28           | 1.24           | 1.67           |
| H1       | 1726                   | 1443               | 1177          | 1.22           | 1.23           | 1.56           |
| H2       | 1651                   | 1380               | 1177          | 1.17           | 1.17           | 1.49           |

In addition to the experimental column capacities, the theoretical capacities of various components of the columns, based on unconfined concrete strengths, are presented in Table 3. \( P_o \) is the theoretical capacity of the column including the contribution of the longitudinal steel, \( P_{oc} \) is the theoretical capacity of the column without the contribution of the steel, and \( P_{occ} \) is the theoretical capacity of the column based on the core of the column without the contribution of the steel (see Eqs. (1)-(3)). For WWF and conventional columns, the region of the column enclosed by the outer of the perimeter WWF or ties is considered to be the core.

\[
P_o = 0.85 f'_c (A_g - A_{st}) + A_{st} f_y
\]

\[
P_{oc} = 0.85 f'_c (A_g - A_{st})
\]

\[
P_{occ} = 0.85 f'_c (A_{co} - A_{st})
\]

The ratios \( \frac{P_{\text{max}}}{P_o} \), \( \frac{P_{\text{cmax}}}{P_{oc}} \), and \( \frac{P_{\text{cmax}}}{P_{occ}} \), corresponding to the peak loads, are also presented in Table 3. The enhancement in the strength of the columns can be seen by studying the ratios \( \frac{P_{\text{max}}}{P_o} \), \( \frac{P_{\text{cmax}}}{P_{oc}} \), and \( \frac{P_{\text{cmax}}}{P_{occ}} \). The ratios \( \frac{P_{\text{max}}}{P_o} \) of the confined columns are all greater than 1.0, which means that the actual capacity of the columns is higher than the theoretical unconfined capacity. This is because of the strength enhancement through confinement. Except for column W15 showed strength enhancement less than 1.0. This is because the characteristic lateral reinforcements have different modes of failure. For the same reason, the ratio \( \frac{P_{\text{cmax}}}{P_{oc}} \) shows that the capacity of the confined concrete in the columns is higher than that of the theoretical unconfined concrete capacity. Table 3 also shows that the ratios \( \frac{P_{\text{max}}}{P_o} \) and \( \frac{P_{\text{cmax}}}{P_{oc}} \) for the unconfined column H1 and H2 are somewhat greater than 1.0. The ratio \( \frac{P_{\text{cmax}}}{P_{occ}} \) is higher than the other two ratios. This ratio is considered to represent the actual strength enhancement factor of the concrete. It can be seen from Tables 1 and 3 that an increase in the level of confinement, as represented by \( \rho_s \), results in a concomitant increase in the strength enhancement factor. In this study, the maximum observed strength enhancement factors due to confinement (Column W16) are 1.66 and 2.23, based on the ratios \( \frac{P_{\text{cmax}}}{P_{oc}} \) and \( \frac{P_{\text{cmax}}}{P_{occ}} \), respectively.
To study the effect of the lateral configuration on the behavior of the confined columns, the results of columns reinforced by WWF can be compared with those of columns reinforced by conventional reinforcement, respectively, as given in Table 3. The mean strength enhancement of the concrete in the columns with welded grid is about 1.20 and 1.25 times that of the columns with conventional hoops based on the ratios $P_{cmax}/P_{oc}$ and $P_{cmax}/P_{occ}$, respectively. The results shown in Table 3 and Figures 2 and 3 indicate that columns reinforced by conventional hoop have strength enhancement factor indices less than columns reinforced by WWF.

Ductility is a very important factor especially for seismic design. The results of all columns, showed ductile failure, although the ductility of columns W5, W6, W14, and W15 was somewhat lower due to the relatively wide spacing of the WWF. The degree of confinement was very low and the column behavior was typical for unconfined concrete. Columns W1, W7, W10, W16, and H1, with WWF and hoop, respectively, exhibited ductile failure because of the closely spaced grids or hoops in the test region. The failure strain was reached at an average strain of 0.0235, which is about ten times the peak strain of unconfined concrete column.

The columns with 4.8 percent volumetric ratio and four supported longitudinal bars, W1, W7, and H1, have a strain ductility of 0.0236, 0.0233, and 0.0250, respectively. This suggests that columns reinforced by conventional hoops have a slightly better ductile efficiency than columns reinforced by WWF. This because conventional hoops can withstand lateral pressure from the concrete core to a larger extent than WWF due to hook stretching, while the welded joint of WWF brakes and therefore cannot restrain the lateral buckling that occurs in longitudinal steel. Therefore, it can be concluded that columns reinforced by conventional hoops have more ductility than columns reinforced by WWF when the volumetric ratios are equal and above 3.5 percent.

4. CONCLUSIONS

A new confinement system by WWF reinforcement has excellent potential application to earthquake-resistant structures as confinement reinforcement through enhancement in both strength and ductility. The WWF columns exhibited a maximum strength enhancement factor of 2.23 (Column W16, strain at peak stress equal to 0.0149), calculated based on the core of the column. By comparison, a conventionally confined column (Column H1) exhibited a strength enhancement factor of 1.56 and a strain at peak stress of 0.0036. For the same volumetric ratio and spacing, 9-cell grids produced higher strength and ductility than 4-cell grids. In general, test results have shown that WWF provides much better concrete confinement than rebar reinforcement system.

ACKNOWLEDGMENTS

This investigation was funded by the Indonesian International Education Foundation (IIEF) under Indonesian Scholar Dissertation Award number 0032-a/IIEF/II/2010. This support is greatly acknowledged. Additional support provided by Union Metal Company Jakarta – Indonesia, who supplied the WWF reinforcement, is deeply appreciated.

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