INVESTIGATION OF WELDED REINFORCEMENT GRIDS

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ABSTRACT

The study reports the effectiveness of welded reinforcement grids in providing lateral confinement of core concrete in a column subject to monotonic compressive load. Particular attention was given to the ductile performance of the column, which was compared to ductile behavior of a conventional column with hoop and crosstie reinforcement. The main purpose of improving column ductility is to strengthen the core by distributing the failure along a longer length of the column.

A single column was constructed with welded reinforcement grids. The testing phase subjected the column to a single, low rate, monotonic compressive load. A previous test of a similar conventional column was included in the analysis for comparison.

Experimental data suggested ductility in columns laterally confined by welded reinforcement grids to be significantly lower than in conventionally reinforced columns.
ACKNOWLEDGEMENTS

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**INTRODUCTION**

Concrete is reinforced with steel bars or wires to resist tensile stresses. In structural columns, concrete in compression is typically enclosed by closely spaced hoop or spiral reinforcement. When an axial load is applied to a concrete column, the width of the column increases due to Poisson’s ratio, and transverse hoops or spirals are stressed in tension. The transverse reinforcement provides confinement to the core and has shown to increase both the compressive strain and stress.

Concretes with 28-day strengths in excess of 6000 psi are referred to as high-strength concretes. High strength concrete starts to develop microcracking at a compressive stress of about 0.85$f'_c$, compared to normal strength concrete which has a critical stress of about 0.75$f'_c$. When compared to normal-strength concretes, the stress-strain curves for high-strength concretes tend to have a steeper linear loading branch followed by a sharper decline. High-strength concretes usually dilate less with smaller lateral strains in comparison to normal-strength concretes as the concrete is loaded to longitudinal stain capacity and beyond. These characteristics of high-strength concrete may cause normal-strength spiral and hoop confining reinforcement to be less effective in maintaining the strength and ductility of the concrete core.

Welded reinforcement grids (WRG) [Fig. 3] offer an alternative to conventional ties and hoops. Construction time may be significantly reduced by prefabrication of a layer of transverse reinforcement. Conventional hoop and crosstie reinforcement is fabricated on benders to looser dimensional tolerances which may lead to poor fitting hoops. In addition, layers of closely spaced hoops and crossties may inhibit the flow of concrete or obstruct the movement of the vibrating compactor. Welded reinforcement grids are prefabricated to low dimensional tolerances and have been used in a number of applications.

The Southwark-Emery Universal Testing Machine [Fig. 4] is located at University of California’s neec@berkeley facility. The five stories tall structure is designed to test specimens composed of any kind of material. The machine has the ability to produce loads on a specimen and induce tension, compression, or flexure. The loading capabilities of the machine are matched only by a select number of devices in the world. The design capacity in compression is 4,000,000 pounds.
OBJECTIVE

Objectives of this study are as follows:

- Design an experiment to observe the mechanical behavior of a column laterally confined with welded reinforcement grids.
- Construct and execute the experiment and collect data.
- Compile and analyze the data in a report.

DESIGN AND CONSTRUCTION

The test specimen was designed as 1:2 scale because of the limitations of the Universal Testing Machine. The specimen modeled rectangular structural columns that are used in real world applications. The design width, depth, and height of the column were 15 in, 15 in, 45 in respectively. The as-built dimensions were 15 in, 15 in, 48 in [Fig. 5]. A height-to-width ratio of 3:1 was chosen to allow enough height for development of the failure mechanism.

End confinement was necessary to prevent a localized failure at either end of the column. In experiments such as this, which deal with large compressive loads, it is common practice to increase the lateral confinement near the top and base of the column to prevent the ends from crushing under large stress. In this case two steel rings were placed, one around each end, to provide additional confinement. Each ring had an outside diameter of 24 in, a depth of 6 in, and a thickness of 0.5 in [Fig. 5]. Hydrostone grout was used to affix the rings to the column [Fig. 12].

The steel reinforcement was designed according to the current ACI Building Code requirements in seismic regions. The following material properties were assumed:

\[ f_y = 60 \text{ ksi} \]
\[ f_{yt} = 60 \text{ ksi} \]
\[ f_c' = 10 \text{ ksi} \]

Based on later cylinder tests it was determined that:

\[ f_c' = 7.21 \text{ ksi} \]

The lower than anticipated critical strength, \( f_c' \), was not a concern because the column had a higher total volume ratio of confining reinforcement than what was required by code.

The longitudinal reinforcement consisted of 12 #6 Grade 60 steel rebar spaced as shown in Figure 5. The ACI Code specifies that longitudinal reinforcement in seismic columns is designed the same way as in nonseismic columns. It may range from \( \rho = 0.01 \) to 0.06. In this case \( \rho = 0.024 \).
The transverse reinforcement consisted of welded reinforcement grids. These were essentially four smooth steel bars spaced and laid perpendicular to four additional bars and electric-resistance welded at each contact point. The diameter of each bar was 3/8 inches. See Figure 5 for dimensions. The transverse reinforcement was spaced according to chapter 21 of the ACI Code for seismic design. The following expressions were used:

\[ A_{sh} \geq 0.09 \frac{f'_c}{f_{yt}} \cdot s \cdot b_c \]  

(Eq. 1)

\[ A_{sh} \geq 0.3 \cdot s \cdot b_c \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}} \]  

(Eq. 2)

Longitudinal spacing of transverse reinforcement was calculated as the minimum of the above formulae; s is less than or equal to 3.04 inches. The column was designed and built with a longitudinal spacing of 2 inches [Fig. 5].
TEST SETUP

The desired data was load, longitudinal global displacement, local displacement of the middle 1/3rd so as to avoid end effects, and strain of the transverse WRG steel. Load data was received by the Southwark-Emery Universal Testing Machine. Global displacement was measured using a PT101 cable-extension position transducer anchored to the floor, level with the base of the column with the wire attached to the loading head [Fig. 14]. Local displacement was measured with two DC LVDTs (linear variable differential transformer) on opposite sides mounted directly to the column on 3/8 inch threaded steel rod embedded in the concrete [Fig. 15 and 16]. An average of the displacements on two opposite sides was thought to be a good approximation of the average displacement of the cross-section. Effort was made to prevent the cover concrete from hardening around the threaded rods during the curing of the specimen by cutting a quarter inch of foam material to fit around the rods in the cover region. Three strain gauges were mounted onto the transverse reinforcement before the concrete was poured [Fig. 17].

The specimen was tested in less than an hour. The specimen was moved to position [Fig. 13] and capped at either end with hydrostone. During testing a person operated the computers in the data acquisition room while another operated the Southwark-Emery Universal Testing Machine in the control room and a third person stood on the lab floor at a safe distance from the specimen and operated the video capturing devices. Communication during testing was maintained by a series of two-way radios.

A monotonic compressive loading was applied at a rate of 300 kips/min. The loading was stopped immediately after failure to allow some time for observations and photographs, and then load was applied again until the specimen was on the verge of collapse.
ANALYSIS

To gain an understanding of the mechanical behavior of the welded reinforcement grids used as lateral confinement in this experiment it is important to note what is commonly accepted as the minimum performance for a given amount of reinforcement.

Ductility in a column can be defined in two components. First, concrete itself is ductile in compression. Second, lateral confinement strengthens the core by distributing the failure along a longer length of the column.

Two values are commonly associated with ductility as it relates to compression strength of a column. The spalling load, \( P_0 \), is defined as the load corresponding to the onset of spalling of the cover concrete and usually marks a leveling or drop of the applied load until sufficient expansion takes place in the lateral confinement. \( P_0 \) is commonly expressed as

\[
P_0 = 0.85 f_c (A_g - A_s) + A_s f_y \quad \text{(Eq. 3)}
\]

The post-spalling strength is the ultimate strength achieved by the column and is expressed as

\[
P_{00} = f_{cc} (A_{ch} - A_s) + A_s f_y \quad \text{(Eq. 4)}
\]

Compressive strain is another measure of ductile performance of a column. One method of determining expected strain capacity as proposed by Qi and Moehle (1991) is incorporated in the following expression:

\[
\varepsilon_{cmax} = 0.004 + 0.1 \rho_s \left( \frac{f_y}{f_c} \right) \quad \text{(Eq. 5)}
\]

where \( \rho_s \) is the volume of confining reinforcement divided by volume of confined core.

Table 1 Expected and measured column strength and strain capacities

<table>
<thead>
<tr>
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<th>expected</th>
<th>measured</th>
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<tr>
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<tr>
<td>( P_0 )</td>
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<td></td>
</tr>
<tr>
<td>( P_{00} )</td>
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<td>2069</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>strain</th>
<th>expected</th>
<th>measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>units</td>
<td>in/in</td>
<td>in/in</td>
</tr>
<tr>
<td>( \varepsilon_{cmax} )</td>
<td>0.017</td>
<td>0.010</td>
</tr>
</tbody>
</table>
A discrepancy between the experimental strain capacity and the expected performance strain capacity was noted. The strain capacity of the column was 40% lower than anticipated. This may indicate inadequate ductility in the column.

A careful approach was taken to develop a stress-strain relation [Fig. 33] for the column. The effects of the longitudinal steel were removed so that the stress of the confined concrete remained. Reduction of the cross-sectional area of the column during spalling of the cover must also be considered in calculating the stress. The following expressions were used in developing a stress-strain relation:

\[
\sigma = \frac{P_c}{A_c} \quad \text{where} \quad P_c = P - P_s \quad \text{and} \quad P_s = A_s \cdot \sigma_s
\]  
(Eq. 6)

\[
A_c = A_g - A_s \quad \text{if} \quad \varepsilon \leq 0.002 \quad \text{or} \quad A_c = A_{ch} - A_s \quad \text{if} \quad \varepsilon > 0.004
\]  
(Eq. 7)

if strain is between 0.002 and 0.004 use linear interpolation

The following assumption for Grade 60 steel was used:

**Reinforcing Steel - Stress-Strain Relation**

![Stress vs. strain relation of reinforcing steel](image)

**Figure 1 – Stress vs. strain relation of reinforcing steel**
To make comparisons to conventionally reinforced columns, data from a similar column, reinforced with deformed bar hoops and ties, was made available by the UC Berkeley facility of the Network for Earthquake Engineering Simulation (nees@berkeley). The conventionally reinforced column was made of high-strength concrete of the same dimensions as the specimen tested in this experiment. The only significant difference was the transverse reinforcement in the two columns. Both columns were tested in compression in the Southwark-Emery Universal Testing Machine under similar loading conditions. The two graphical comparisons are similar in shape and show the load as a function of global displacement [Fig.25] and global strain [Fig. 26]. “Global” is taken as the full height of the column, while “local” indicates a height in the middle third of the column.

Both comparisons show that the conventional and the welded reinforced columns were unalike in their behaviors. The conventional column data shows an initial drop in load, presumably at initial spalling of the concrete cover, followed by a second, more gradual, load increase that surpasses the spalling load, eventually peaking at the column’s load capacity. The second peak is caused by lateral expansion, due to the Poisson effect during compression, which yields the transverse reinforcement confining the concrete core. The column utilizing welded reinforcement does not reveal a second peak in load, rather a sudden jump in displacement and a major drop in load bearing.

The video recordings taken from the east and west face are consistent with the data. Video shows a sudden failure, characterized by a dramatic loss in load bearing, after roughly 5% of the column’s cover had spalled.
CONCLUSION

Welded reinforcement grids can reduce construction time by prefabrication of a product that would otherwise require construction site resources. However, as ductility may be the difference between maintaining load bearing strength and total collapse when a column is under compressive loads large enough to induce failure, expected minimum levels of performance must be met.

As shown in this investigation, data indicates significantly lower ductility in columns using welded reinforcement grids than in those with conventional hoop and tie reinforcement. Although construction efficiency may be increased, the use of welded reinforcement grids may ultimately compromise building safety.

Further research is necessary for a comprehensive view of welded reinforcement grids. Poor ductile performance of columns using welded reinforcement grids may be linked to inadequate weld strength, poor concrete-to-steel bonding, the brittle nature of high-strength concrete, or a combination of factors.
Figure 3 – Transverse reinforcement. Conventional (left) and welded reinforcement grids (right)

Figure 4 - Southwark-Emery Universal Testing Machine
Figure 5 – Design schematic
Figure 6 – Side view of steel cage

Figure 7 – Longitudinal reinf.

Figure 8 – Close-up end view
Figure 12 – Close-up steel ring. Space between ring and column is filled with hydrostone.

Figure 13 – Column in place

Figure 14 – PT101 measures global disp.
Figure 15 – LVDT front view

Figure 16 – LVDT profile view

Figure 17 – Strain gage location

Gage1: outer mesh
Gage2: outer mesh
Gage3: inner mesh
APPENDIX: TEST RESULTS

The experiment was documented with high quality photographs, video, and data acquisition. The following is displayed as figures in this report:

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Figure 33 – WRG – Stress vs. strain of the column
REFERENCES

1. "Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)." ACI Committee 318, American Concrete Institute, Farmington Hills, MI, 2005, pp. 305-342.


