Seismic Evaluation and Retrofit of Beam-Column Joints of Mid-America Bridges
Part 2: Steel Sheet and Plate Retrofit

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Participants

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• Xi Huang, Ph.D. graduate student
• Pedro Silva, Ph.D., P.E.
• Roger LaBoube, Ph.D., P.E.
**Background**

- Both steel and FRP jacketing techniques are available for the seismic retrofitting of RC columns.
- Steel jacketing is ductile and durable. Engineers are confident with the reliable materials.
- FRP jacketing is light and easy to construct in field condition. It has no issue related to steel corrosion.
- It would be desirable to combine several advantages of the two techniques: ductile, durable, light in weight, and reliable materials. Using stiffened thin steel sheets (galvanized or stainless steel) seems to meet the above requirements.

**Objectives**

- Develop a new seismic retrofit technique with stiffened thin steel sheets for columns and steel plates for beam-column joints
- Test concrete ring specimens wrapped with thin steel sheets to understand the strength and failure modes of nailed joints
- Design the retrofit scheme for an existing bridge in southeast Missouri
- Test two 4/5-scale beam-column specimens to validate the performance of the retrofit scheme
New Retrofit Scheme

Nailed Joint Failure Modes

Specimens

Top View

Lap Splice vs. Self Lock Joint
Nailed Joint Failure Modes

Test Setup

- Strain Gage
- Joint Area
- Steel Bearings
- 0.5 in Steel Tube

Load Pattern 1

Load Pattern 2

Nailed Joint Failure Modes

Test Results (12 Specimens)

F/C Ratio vs. Nail Pattern

Nail length

F/C Ratio

- 3-nail & load 1
- 5-nail & load 1
- 3-nail & load 2
- 5-nail & load 2
Nailed Joint Failure Modes

Failure Modes and Summary

- Self lock joints (3-nail pattern) always fail in pull-out of nails due to potential bending effects on the outer steel sheet while splice joints (5-nail pattern) always fail in bearing of the steel sheets.
- The ratio of failure to crack loads of the 5-nail pattern specimens are always greater than that of the 3-nail pattern specimens. Strength is proportional to the number of nails in joints.
- The strength of joints is independent of the length of nails.

Test Data of Lap Splice Joints

<table>
<thead>
<tr>
<th>Rows of Nails</th>
<th>Number of specimens</th>
<th>Load at Peak (lbf)</th>
<th>Strain at Peak (%)</th>
<th>Strain at Break (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>4</td>
<td>1990</td>
<td>0.39</td>
<td>0.59</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>2360</td>
<td>0.68</td>
<td>0.88</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>3370</td>
<td>1.94</td>
<td>2.66</td>
</tr>
<tr>
<td>5</td>
<td>3*</td>
<td>4100</td>
<td>3.23</td>
<td>3.36</td>
</tr>
</tbody>
</table>

* One specimen damaged before testing
**Typical Load-Strain Relation**

- **2-Nail Joint**
- **3-Nail Joint**
- **4-Nail Joint**
- **5-Nail Joint**

**Retrofit Goals**

- Increase the ductility of the RC column
- Eliminate the potential shear failure of the column
- Increase the shear/flexural capacity of the cap beam
- Eliminate the potential shear failure and reduce the stiffness degradation at the beam-column joint
Retrofit Design

Column Strengthening for Ductility

\[ t_j = \frac{0.1(\varepsilon_{\text{cu}} - 0.004)D_f}{f_u \varepsilon_{\text{cu}}} \]

- \( \varepsilon_{\text{cu}} = \phi \varepsilon \)
- \( c = \) neutral axis length = 9.2 in
- \( \phi = \mu \phi \)
- \( \mu = \) objective curvature ductility = 4.57
- \( \phi = \) curvature at yield = 0.00027 in\(^{-1}\)
- \( D = \) column diameter = 24 in
- \( f_u = \) strength of the confined concrete = 6.59 ksi
- \( f_{\text{uj}} = \) ultimate jacket stress = 50 ksi

Retrofit Design

Column Strengthening for Shear

\[ V_s \geq \frac{\phi}{0.5 \pi f_j D \cot \theta} \]

- \( V_s = \) shear demand on column = 128.56 kips
- \( \phi = \) factor of safety for shear = 0.75
- \( V_s, V_s, V_s, V_s = \) shear strengths due to the concrete, stirrups, and axial force
- \( V_s = 29 \) kips
- \( V_s = 41.1 \) kips
- \( V_s = 35 \) kips
- \( f_j = \) design jacket strength = 50 ksi
- \( D = \) column diameter = 24 in
- \( \theta = \) the greater of 35° or the column corner to corner angle = 35°
Retrofit Design
Statically Determinant (X-Shape Plate)

Assumptions:
1. Tension in vertical plates is significantly smaller (<20%) than that in diagonal plates. It is neglected in calculation.
2. Diagonal steel plates are fully yielded. The total tension force on two diagonal plates is
   \[ T = 2 \times 50 \text{kips} \times 12'' \times 0.25'' = 300 \text{kips} \]
   The load on the top plate is equal to
   \[ c = \frac{T \cos 45^\circ}{A} = 0.5 \text{kpi} \]
   \[ A = 25'' \times 17'' = 425 \text{ in}^2 \]

The L/100 allowable deflection corresponds to that of the story drift of a steel frame (Table 1617.3.1, IBC 2003)

Thickness = 0.25"
Retrofit Design

Summary

<table>
<thead>
<tr>
<th>Retrofit component</th>
<th>Design thickness (in)</th>
<th>Actual thickness (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel ring for column ductility</td>
<td>0.25</td>
<td>0.5*</td>
</tr>
<tr>
<td>Steel sheet for column shear</td>
<td>0.025</td>
<td>0.036(20GA)*</td>
</tr>
<tr>
<td>Steel plate for beam-column joint shear</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>X-shape steel plate for joint shear</td>
<td>3/32</td>
<td>3/32</td>
</tr>
</tbody>
</table>

* Based on availability or ease of fabrication

Retrofit Design

3rd Specimen Details
Retrofit Design

**4th Specimen Details**

Test Setup

**3rd Specimen**

**4th Specimen**
Steel Retrofitting

Load vs. Displacement

Displacement (in)

-12 -10 -8 -6 -4 -2 0 2 4 6 8 10 12

Lateral Load (kips)

-200 -150 -100 -50 0 50 100 150 200

Displacement (mm)

-900 -600 -300 0 300 600 900

Lateral Load (kN)

Axi al Load vs. Displacement

Lateral Displacement (in)

-8 -4 0 4 8

Axial Load (lbs)

-200 -100 0 100 200

Lateral Displacement (mm)

-140000 -120000 -100000 -80000 -60000

Axial Load (kN)

-200 -100 0 100 200

Lateral Displacement (mm)

-12 -8 -4 0 4 8

Axial Load (lbs)

-12 -8 -4 0 4 8

Lateral Displacement (in)

3rd specimen

4th specimen

3rd Specimen

4th Specimen
Bent Cap Stirrup Steel Strain

3rd Specimen

4th Specimen

Strain Gage Location on Retrofit Component
Strain on Steel Rings (3rd Specimen)

North side

South side

Strain on X-Shape Steel Plates (3rd Specimen)
Strain on Vertical Steel Plates (3rd Specimen)

Left Side

Right Side

Unretrofitted vs. Retrofitted Column (3rd Specimen)

Shear failure of unretrofitted specimen

Plastic hinge formed at the beam-column joint of the retrofitted specimen
Unretrofitted vs. Retrofitted Joint (3rd Specimen)

Excessive cracks of unretrofitted specimen

Few cracks of retrofitted specimen

Strain on Steel Rings (4th Specimen)

North side

South side
Strain on X-Shape Steel Plates (4th Specimen)

Strain on Vertical Steel Plates (4th Specimen)
Conclusions

- Lap splice nailed joints of two thin steel sheets are very effective. Their strength is generally proportional to the number of rows of nails. Lap splice joints ultimately fail in bearing of the sheets.
- Self lock nailed joints of two thin steel sheets can be as effective as lap splice joints provided that sufficient space at the end of the sheets, nailed with two or more rows of nails, is available for shear deformation of the joints. Such a well-designed joint did not fail in pull-out of nails that happened to the concrete rings wrapped with a lock joint without space. The number of the rows of nails is significantly smaller than that of the lap splice joints.
- Both lap splice and self lock joints are sufficient in providing strength of nailed steel sheets for column shear retrofitting. Their strength is independent of the length of nails due to concrete cracks.

Conclusions

- Steel rings as stiffeners to thin steel sheets in the plastic hinge zone can enhance the column ductility substantially. A spacing of 7.5 cm seems reasonable to prevent buckling of the thin sheets.
- Retrofitting a beam-column joint with steel plates (one wrap around the cap beam on both sides of the column and x-bracing between two wraps) can effectively reduce the number and width of cracks at the joint. The shear force at the joint is mainly transferred by the x-bracing, not the vertical plates in the two wraps.
- Longitudinal prestress on the cap beam can further control the development of cracks at the beam-column joint so that the longitudinal rebar in column will not be pulled out of the joint and, as a result, the stiffness of the beam-column assemblage will not be degraded significantly.