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

Genda Chen, Ph.D., P.E.
Associate Professor of Civil Engineering
Department of Civil, Architectural, and Environmental Engineering
University of Missouri-Rolla (UMR)
gchen@umr.edu

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***New Madrid Seismic Zone Experience***

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# Participants

- Genda Chen, Ph.D., P.E. (team leader)
- Xiaofei Ying, Ph.D. graduate student
- Xi Huang, Ph.D. graduate student
- Pedro Silva, Ph.D., P.E.
- Roger LaBoube, Ph.D., P.E.

# Background

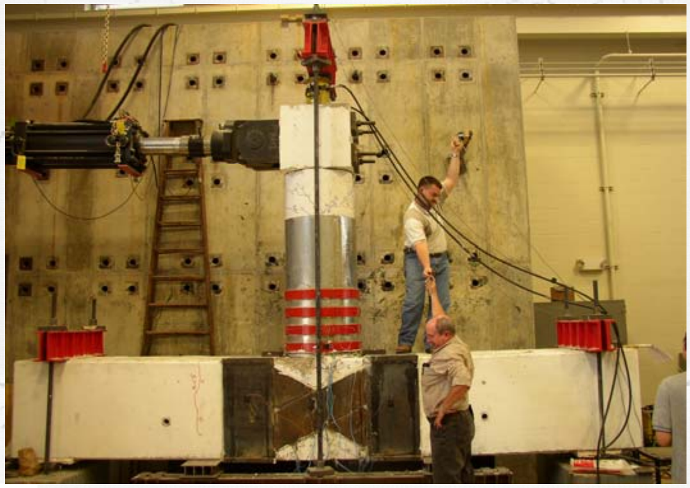
- Both steel and FRP jacketing techniques are available for the seismic retrofitting of RC columns.
- Steel jacketing is ductile and durable. Engineers are confident with the reliable materials.
- FRP jacketing is light and easy to construct in field condition. It has no issue related to steel corrosion.
- 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

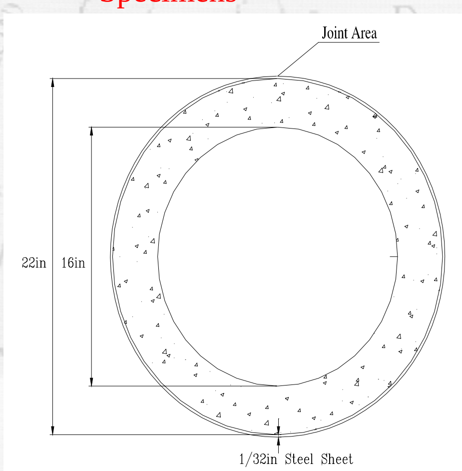
# New Retrofit Scheme



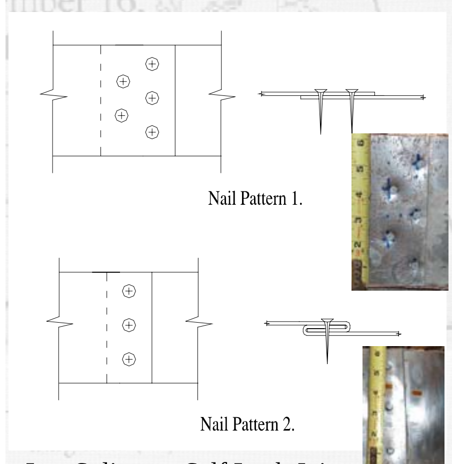


# Nailed Joint Failure Modes

Specimens



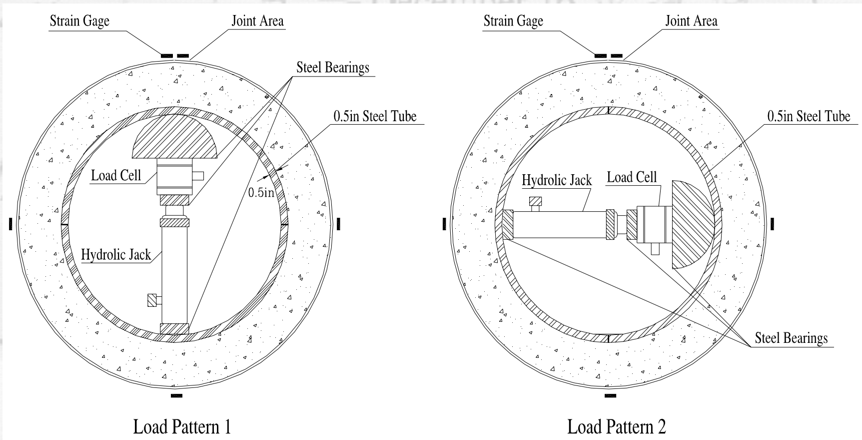

Top View




Lap Splice vs. Self Lock Joint

# Nailed Joint Failure Modes

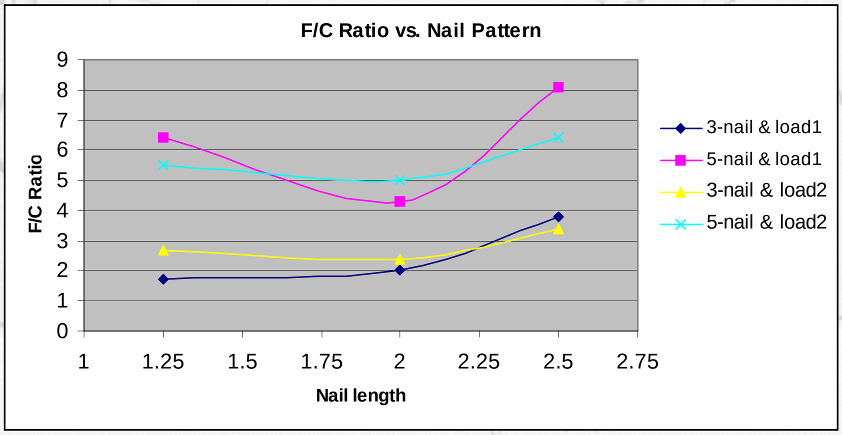
Test Setup





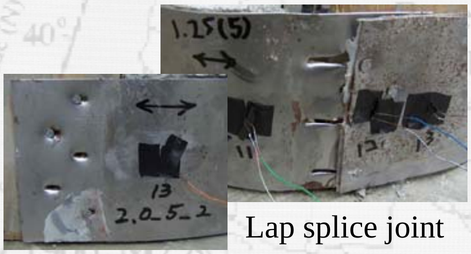

# Nailed Joint Failure Modes

Test Results (12 Specimens)



# Nailed Joint Failure Modes

Failure Modes and Summary




Lap splice joint


Self lock joint

- 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.

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# Test Data of Lap Splice Joints

| Rows of Nails | Number of specimens | Load at Peak (lbf) | Strain at Peak (%) | Strain at Break (%) |
|---|---|---|---|---|
| 2 | 4 | 1990 | 0.39 | 0.59 |
| 3 | 4 | 2360 | 0.68 | 0.88 |
| 4 | 4 | 3370 | 1.94 | 2.66 |
| 5 | 3* | 4100 | 3.23 | 3.36 |

* One specimen damaged before testing

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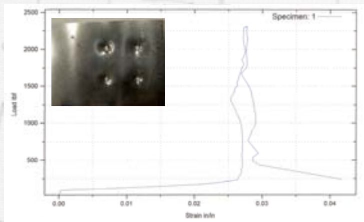
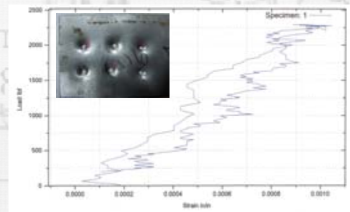
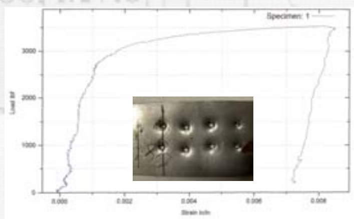
# Typical Load-Strain Relation



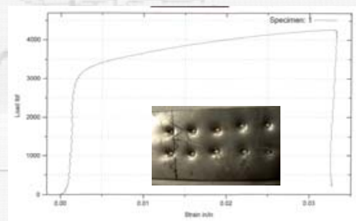
2-Nail Joint



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

$$t_j = \frac{0.1(\varepsilon_{cu} - 0.004)Df'_{cc}}{f_{uj}\varepsilon_{uj}}$$

$\varepsilon_{cu} = c\phi_u$

$c$ -- neutral_axis_length -- 9.2in

$\phi_u = \mu_\phi \phi_y$

$\mu_\phi$ -- objective_curvature_ductility -- 4.57

$\phi_y$ -- curvature_at_yield -- $0.00027 in^{-1}$

$D$ -- column_diameter -- 24in

$f'_{cc}$ -- strength_of_the_confined_concrete -- 6.59ksi

$f_{uj}$ -- ultimate_jacket_stress -- 50ksi



# Retrofit Design

## Column Strengthening for Shear

$$t_j \geq \frac{\frac{V_0}{\phi_s}(V_c + V_s + V_s)}{0.5\pi f_j D \cot\theta}$$

$V_0$ -- shear_demand_on_column = 128.56kips

$\phi_s$ -- factor_of_safety_for_shear = 0.75

$V_c, V_s, V_p$ -- shear_strengths_due_to_the_concrete, stirrups_and_axial_force

$V_c = 29 kips$

$V_s = 41.1 kips$

$V_p = 35 kips$

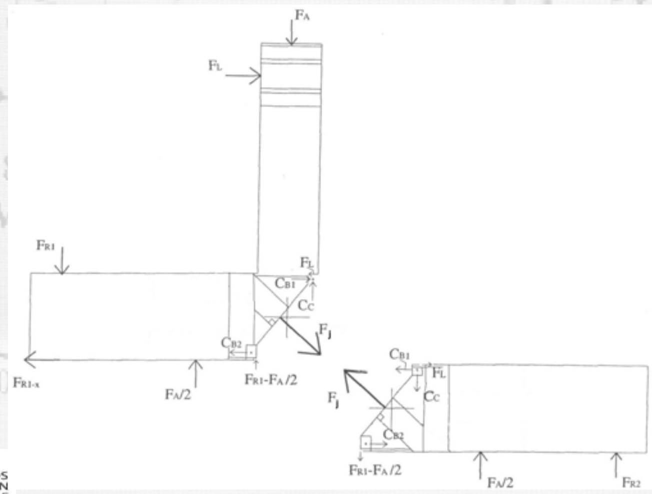
$f_j$ -- design_jacket_strength = 50ksi

$D$ -- column_diameter = 24in

$\theta$ -- the_greater_of_$35^{\circ}$_or_the_column_corner_to_corner_angle = $35^{\circ}$

# Retrofit Design

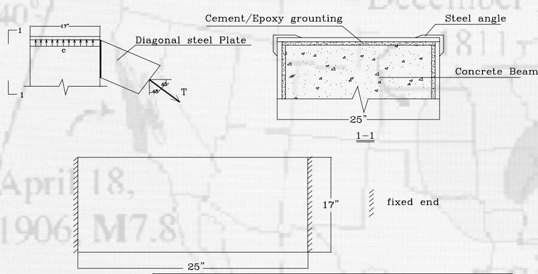
## Statically Determinant (X-Shape Plate)



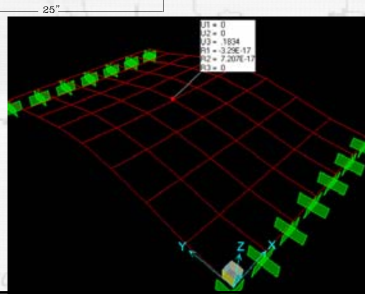



# Retrofit Design

## Thickness of Horizontal Plates




Analytical and computer models




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=2x50ksix12”x0.25”=300kips

- The load on the top plate is equal to $c = \frac{T\cos 45^\circ}{A}$ =0.5ksi
- A=25”x17”=425 in$^2$

U3=0.1834”<L/100=0.25”

The L/100 allowable deflection corresponds to that of the story drift of a steel frame (Table 1617.3.1, IBC2003)

Thickness=0.25”

# Retrofit Design

## Summary

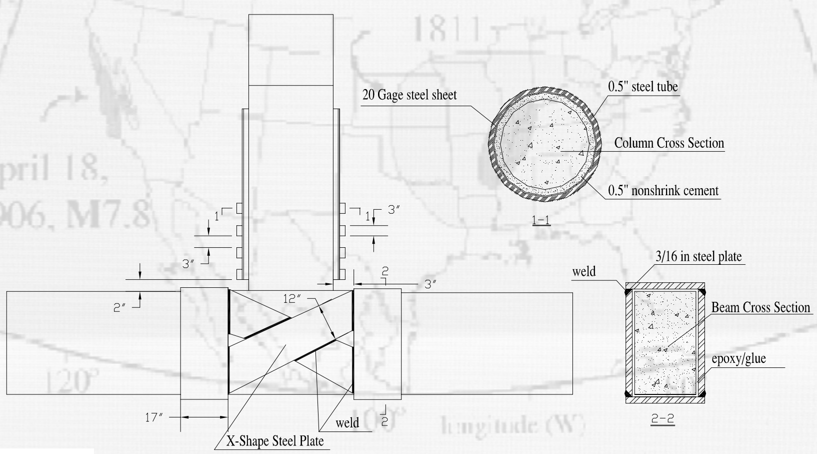
| Retrofit component | Design thickness (in) | Actual thickness (in) |
|---|---|---|
| Steel ring for column ductility | 0.25 | 0.5* |
| Steel sheet for column shear | 0.025 | 0.036(20GA)* |
| Steel plate for beam-column joint shear | 0.25 | 0.25 |
| X-shape steel plate for joint shear | 3/32 | 3/32 |

* Based on availability or ease of fabrication



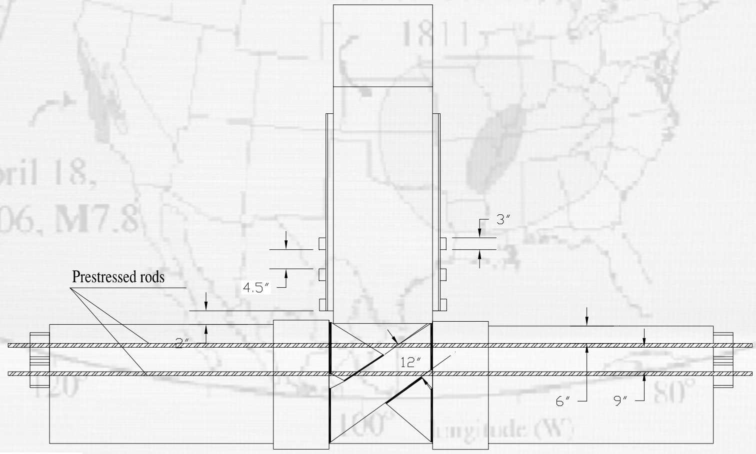
# Retrofit Design

## 3rd Specimen Details



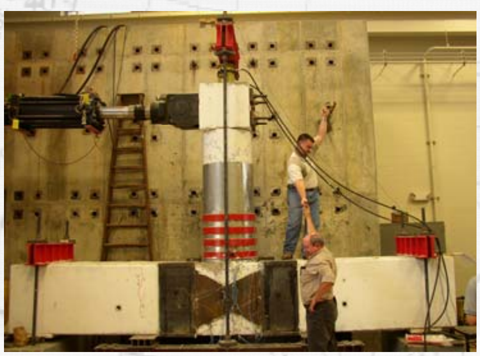
# Retrofit Design

## 4th Specimen Details

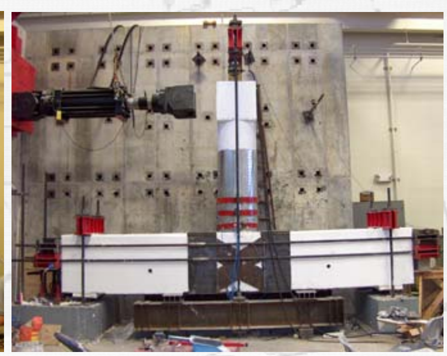





# Test Setup

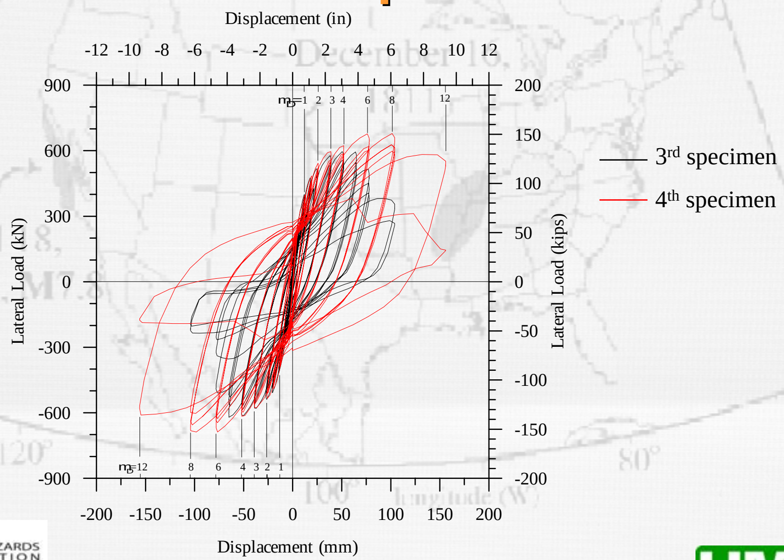


3rd Specimen



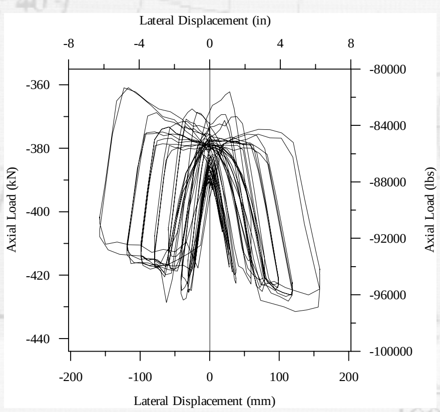
4th Specimen

# Load vs. Displacement

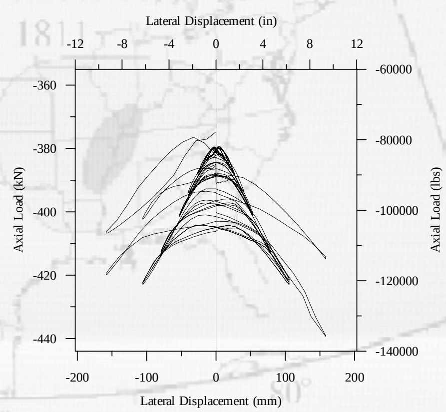



Steel Retrofitting - 21

# Axial Load vs. Displacement



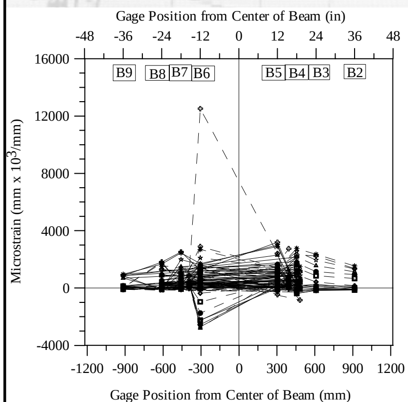

3rd Specimen



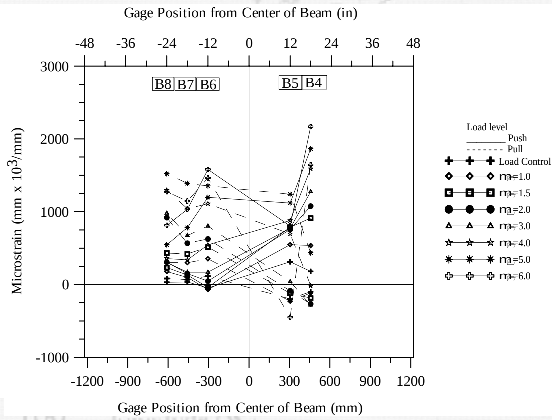

4th Specimen

Steel Retrofitting - 22

# Bent Cap Longitudinal Steel Strain (3rd Specimen)




Top Longitudinal Bar




Bottom Longitudinal Bar

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Steel Retrofitting - 23

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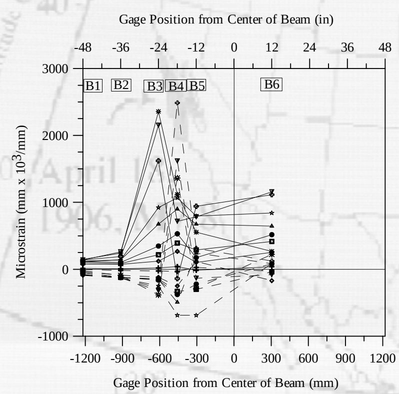
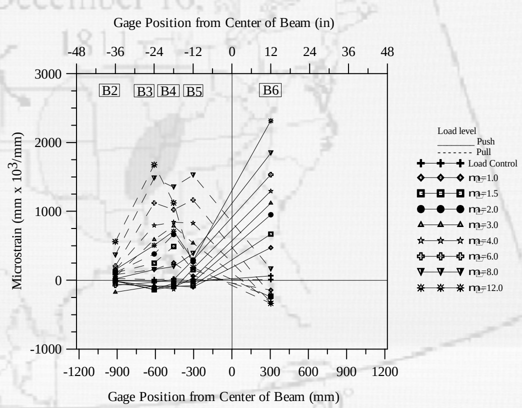
# Bent Cap Longitudinal Steel Strain (4th Specimen)




Top Longitudinal Bar




Bottom Longitudinal Bar

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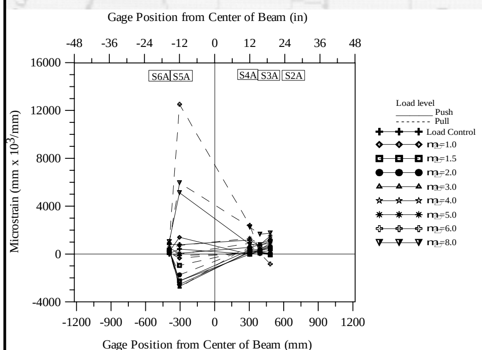
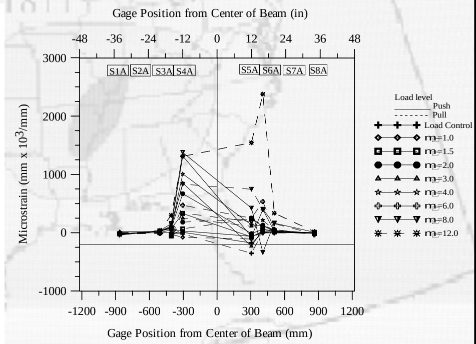
# Bent Cap Stirrup Steel Strain




3rd Specimen




4th Specimen

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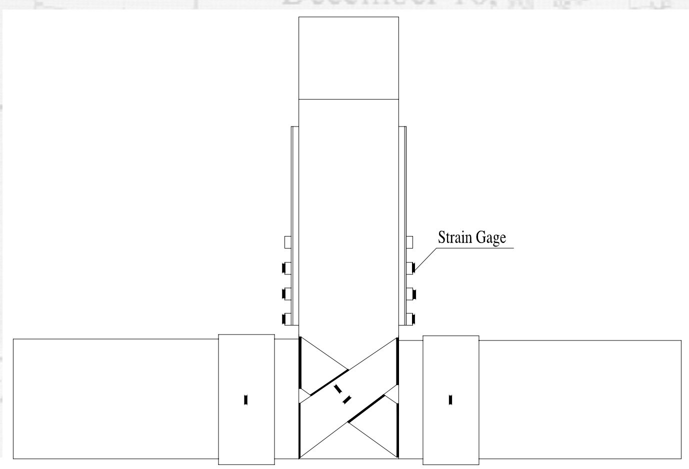
# Strain Gage Location on Retrofit Component

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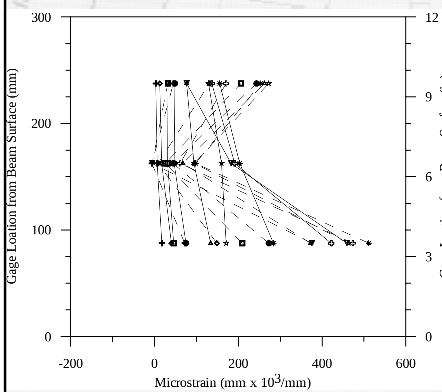
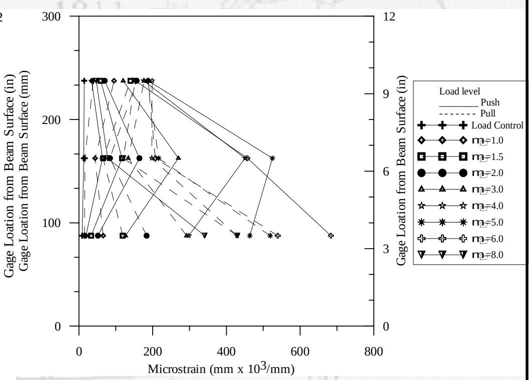
# Strain on Steel Rings (3rd Specimen)



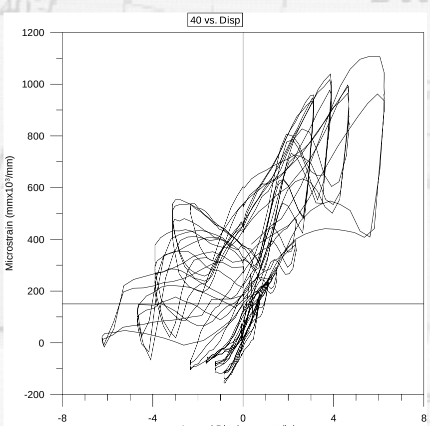

North side

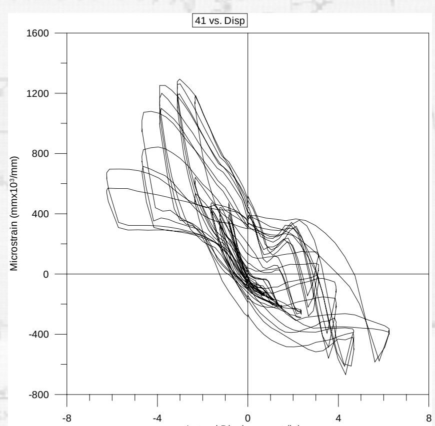



South side

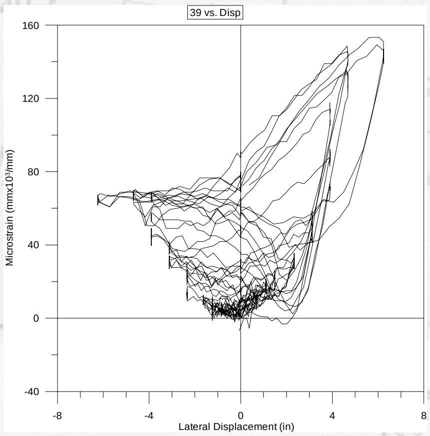


# Strain on X-Shape Steel Plates (3rd Specimen)

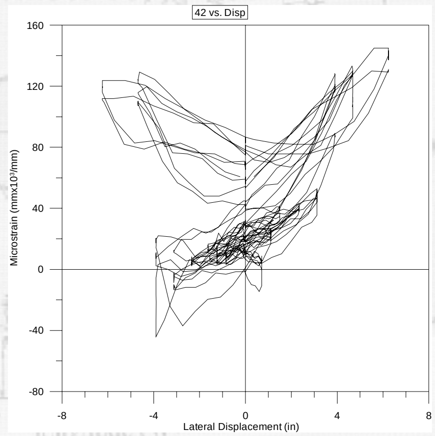


# Strain on Vertical Steel Plates (3rd Specimen)




Left Side




Right Side

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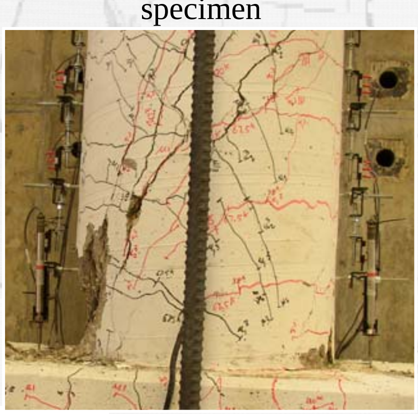
# Unretrofitted vs. Retrofitted Column (3rd Specimen)

Shear failure of unretrofitted specimen



Plastic hinge formed at the beam-column joint of the retrofitted specimen

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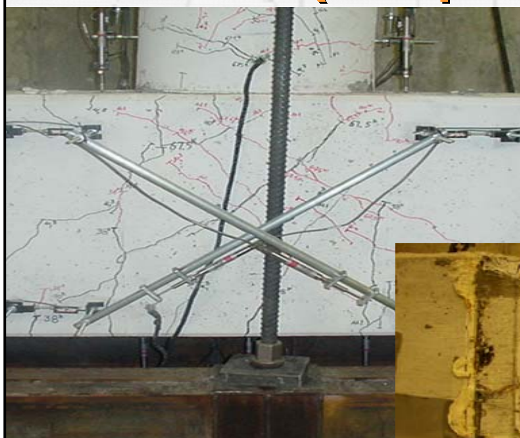
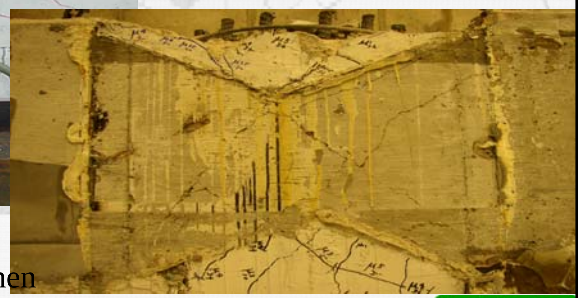
# Unretrofitted vs. Retrofitted Joint (3rd Specimen)



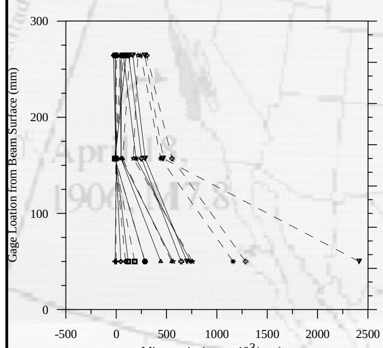
Excessive cracks of unretrofitted specimen



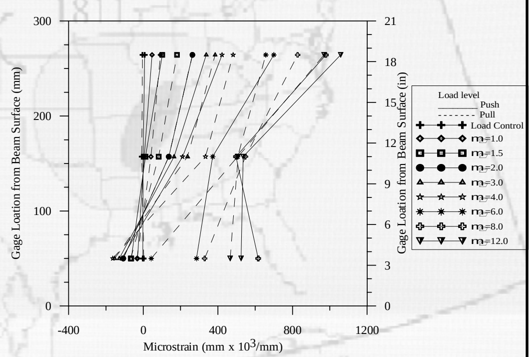
Few cracks of retrofitted specimen



# Strain on Steel Rings (4th Specimen)

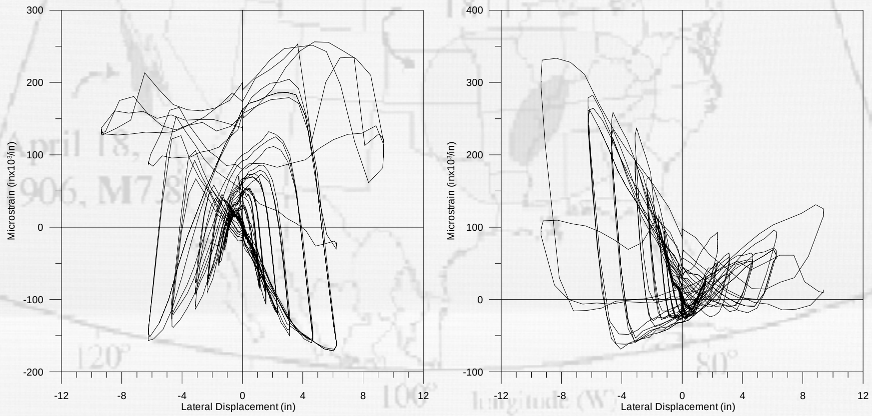



North side




South side

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

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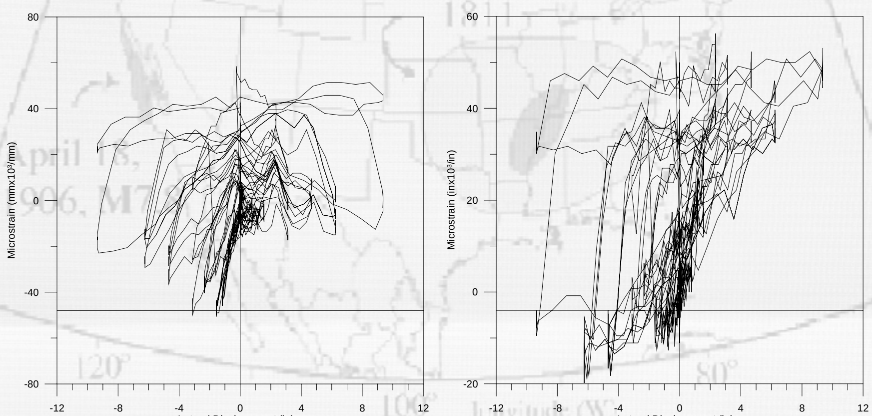
# Strain on Vertical Steel Plates (4th Specimen)




Left Side

Right Side

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## 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.