RETROFIT OF SPLIT BRIDGE COLUMNS
Supporting Appendices

WA-RD 482.2

October 2001
Research Report – Supporting Appendices

Research Project 9902-26
Split Columns

RETROFIT OF SPLIT BRIDGE COLUMNS

APPENDICES

RETROFIT DESIGN CALCULATIONS

by

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Washington State Department of Transportation
Technical Monitor
Hongzhi Zhang
Bridge and Structures Branch Engineer

Prepared for

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Department of Transportation
and in cooperation with
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Federal Highway Administration

October, 2001
APPENDIX A

FYFE COMPANY RETROFIT DESIGN CALCULATIONS
WSU COLUMN RETROFIT

Equations utilized in following spreadsheet:

Effective column height based on provided moment values and corresponding diagrams.
Plastic hinge length = (0.8)(Effective column height) + 0.15(20ksi)(bar diameter).
Ideal yield curvature is taken from the moment-curvature analysis.
Yield displacement = (ideal yield curvature)(effective column height)^2/3.
Neutral axis depth is taken from the moment-curvature analysis.
Mu is taken from the moment-curvature analysis.
Required ultimate curvature = (curvature ductility)(ideal yield curvature).
Required compressive strain = (required ultimate curvature)(neutral axis depth)
Required volumetric ratio = 0.8(required ultimate compressive strain - 0.004)(1.5f_c)(fu' / eu')
   fu = ultimate composite stress = 60ksi
   eu = ultimate composite strain = 0.02
Required jacket thickness = 0.5(required volumetric ratio)((depth x width)/(depth x width))
Required number of layers for required curvature = required thickness/0.051
Max. Shear is from a 50% overstrength value.

### IN PHASE TRANSVERSE LOADING

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### OUT OF PHASE LOADING

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REFERENCES:
OUT-OF-PHASE

SECTION PROPERTIES:

- Section Depth = 16.0 in.
- Section Width = 13.0 in.
- Cover to Main Steel = 0.7 in.
- Concrete Strength = 5.00 ksi
- Concrete Model = Mander
- Steel Strength = 40.0 ksi
- Young's Modulus = 29000.0 ksi
- Steel Model = Mild Strength Steel
- Tension Side Reinforcement = 4 Bars (#3 Bars)
- Compression Side Reinforcement = 4 Bars (#3 Bars)
- Side Reinforcement = 4 Bars (#3 Bars) each side
- Hoop Size = #3 Bars
- Hoop Spacing = 12.0 in.
- Average Number of Legs = 2.0
- Hoop Strength = 40.0 ksi

SECTION ANALYSIS RESULTS:

- Applied Axial Load = 0.0 Kips
- Moment Capacity = 56.4 Kips ft (677.4 Kips in.)
- Section N.A. Depth = 1.0 in.
- Section Curvature = 3.95e-03 1/in.
- Maximum Concrete Strain = 4.0000e-03
- Extreme Steel Strain = 5.5818e-02
SECTION PROPERTIES:

Section Depth = 16.0 in.
Section Width = 13.0 in.
Cover to Main Steel = 0.7 in.

Concrete Strength = 5.00 ksi
Concrete Model = Mander

Steel Strength = 40.0 ksi
Young's Modulus = 29000.0 ksi
Steel Model = Mild Strength Steel

Tension Side Reinforcement = 4 Bars (#3 Bars)
Compression Side Reinforcement = 4 Bars (#3 Bars)
Side Reinforcement = 4 Bars (#3 Bars) each side

Hoop Size = #3 Bars
Hoop Spacing = 12.0 in.
Average Number of Legs = 2.0
Hoop Strength = 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:
The Ideal Moment Capacity is based on the concrete strain of 0.005.

Applied Axial Load = 0.0 Kips
Eeff = 2.32e+04 Kips sq.ft
Curvature Ductility = 23.1 1/in.

Moment Curvature Plot

1. Theoretical Yield 31.1 Kips ft 1.12e-04 1/in.
2. Ideal Yield 58.1 Kips ft 2.09e-04 1/in.
<table>
<thead>
<tr>
<th>Conc. Strain</th>
<th>N.A. Depth</th>
<th>Steel Strain</th>
<th>Moment Cap.</th>
<th>Curvature</th>
</tr>
</thead>
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<td>0.0001</td>
<td>3.1 in.</td>
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<td>3.27e-05 1/in.</td>
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<td>0.0002</td>
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<td>-0.00080</td>
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<td>6.55e-05 1/in.</td>
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<td>0.0003</td>
<td>3.1 in.</td>
<td>-0.00120</td>
<td>28.7 Kips ft</td>
<td>9.81e-05 1/in.</td>
</tr>
<tr>
<td>0.0004</td>
<td>2.9 in.</td>
<td>-0.00171</td>
<td>35.7 Kips ft</td>
<td>1.38e-04 1/in.</td>
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<td>0.0005</td>
<td>2.6 in.</td>
<td>-0.00244</td>
<td>38.6 Kips ft</td>
<td>1.92e-04 1/in.</td>
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<td>0.0006</td>
<td>2.3 in.</td>
<td>-0.00333</td>
<td>40.3 Kips ft</td>
<td>2.57e-04 1/in.</td>
</tr>
<tr>
<td>0.0007</td>
<td>2.1 in.</td>
<td>-0.00439</td>
<td>41.3 Kips ft</td>
<td>3.32e-04 1/in.</td>
</tr>
<tr>
<td>0.0008</td>
<td>1.9 in.</td>
<td>-0.00559</td>
<td>42.0 Kips ft</td>
<td>4.17e-04 1/in.</td>
</tr>
<tr>
<td>0.0009</td>
<td>1.8 in.</td>
<td>-0.00693</td>
<td>42.4 Kips ft</td>
<td>5.11e-04 1/in.</td>
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<tr>
<td>0.0010</td>
<td>1.6 in.</td>
<td>-0.00839</td>
<td>42.8 Kips ft</td>
<td>6.13e-04 1/in.</td>
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<tr>
<td>0.0011</td>
<td>1.5 in.</td>
<td>-0.00998</td>
<td>43.0 Kips ft</td>
<td>7.22e-04 1/in.</td>
</tr>
<tr>
<td>0.0012</td>
<td>1.4 in.</td>
<td>-0.01165</td>
<td>43.2 Kips ft</td>
<td>8.38e-04 1/in.</td>
</tr>
<tr>
<td>0.0013</td>
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<td>9.60e-04 1/in.</td>
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<tr>
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<td>43.5 Kips ft</td>
<td>1.08e-03 1/in.</td>
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<tr>
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<td>-0.01713</td>
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<td>1.22e-03 1/in.</td>
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<tr>
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<td>1.2 in.</td>
<td>-0.01906</td>
<td>43.7 Kips ft</td>
<td>1.35e-03 1/in.</td>
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<tr>
<td>0.0017</td>
<td>1.2 in.</td>
<td>-0.02082</td>
<td>44.4 Kips ft</td>
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</tr>
<tr>
<td>0.0018</td>
<td>1.1 in.</td>
<td>-0.02251</td>
<td>45.2 Kips ft</td>
<td>1.59e-03 1/in.</td>
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<tr>
<td>0.0019</td>
<td>1.1 in.</td>
<td>-0.02422</td>
<td>46.1 Kips ft</td>
<td>1.70e-03 1/in.</td>
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<td>0.0020</td>
<td>1.1 in.</td>
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<td>1.82e-03 1/in.</td>
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<tr>
<td>0.0025</td>
<td>1.0 in.</td>
<td>-0.03415</td>
<td>50.4 Kips ft</td>
<td>2.39e-03 1/in.</td>
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<td>0.0030</td>
<td>1.0 in.</td>
<td>-0.04191</td>
<td>53.0 Kips ft</td>
<td>2.93e-03 1/in.</td>
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<td>3.45e-03 1/in.</td>
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<td>-0.06874</td>
<td>58.1 Kips ft</td>
<td>4.81e-03 1/in.</td>
</tr>
</tbody>
</table>

*Ultimate concrete strain was exceeded.*
SECTION PROPERTIES:

Section Depth = 7.0 in.
Section Width = 13.0 in.
Cover to Main Steel = 0.7 in.

Concrete Strength
Concrete Model = 5.00 ksi = Mander

Steel Strength
Young's Modulus = 40.0 ksi
Steel Model = 29000.0 ksi = Mild Strength Steel

Tension Side Reinforcement = 4 Bars (#3 Bars)
Compression Side Reinforcement = 4 Bars (#3 Bars)
Side Reinforcement = 1 Bars (#3 Bars) each side

Hoop Size = #3 Bars
Hoop Spacing = 12.0 in.
Average Number of Legs = 2.0
Hoop Strength = 40.0 ksi

SECTION ANALYSIS RESULTS:

Applied Axial Load = 0.0 Kips
Moment Capacity = 13.0 Kips ft (156.6 Kips in.)

Section N.A. Depth = 0.7 in.
Section Curvature = 5.74e-03 1/in.
Maximum Concrete Strain = 4.0000e-03
Extreme Steel Strain = 3.1231e-02
SECTION PROPERTIES:

Section Depth = 7.0 in.
Section Width = 13.0 in.
Cover to Main Steel = 0.7 in.

Concrete Strength Concrete Model = 5.00 ksi = Mander

Steel Strength Young's Modulus Steel Model = 40.0 ksi = 29000.0 ksi = Mid Strength Steel

Tension Side Reinforcement = 4 Bars (#3 Bars)
Compression Side Reinforcement = 4 Bars (#3 Bars)
Side Reinforcement = 1 Bars (#3 Bars) each side

Hoop Size = #3 Bars
Hoop Spacing = 12.0 in.
Average Number of Legs = 2.0
Hoop Strength = 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:
The Ideal Moment Capacity is based on the concrete strain of 0.005.

Applied Axial Load = 0.0 Kips
Eeff = 2.60e+03 Kips sq.ft
Curvature Ductility = 15.9 1/in.

Moment Curvature Plot

1. Theoretical Yield 8.9 Kips ft 2.85e-04 1/in.
2. Ideal Yield 13.3 Kips ft 4.26e-04 1/in.
3. Ultimate 13.3 Kips ft 6.77e-03 1/in.
<table>
<thead>
<tr>
<th>Conc. Strain</th>
<th>N.A. Depth</th>
<th>Steel Strain</th>
<th>Moment Cap.</th>
<th>Curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0001</td>
<td>1.5 in.</td>
<td>-0.00031</td>
<td>2.1 Kips ft</td>
<td>6.55e-05 1/in.</td>
</tr>
<tr>
<td>0.0002</td>
<td>1.5 in.</td>
<td>-0.00063</td>
<td>4.3 Kips ft</td>
<td>1.31e-04 1/in.</td>
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<tr>
<td>0.0003</td>
<td>1.5 in.</td>
<td>-0.00094</td>
<td>6.4 Kips ft</td>
<td>1.96e-04 1/in.</td>
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<td>0.0004</td>
<td>1.5 in.</td>
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<td>8.5 Kips ft</td>
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<tr>
<td>0.0005</td>
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<td>-0.00171</td>
<td>9.9 Kips ft</td>
<td>3.49e-04 1/in.</td>
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<tr>
<td>0.0006</td>
<td>1.3 in.</td>
<td>-0.00238</td>
<td>10.3 Kips ft</td>
<td>4.71e-04 1/in.</td>
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<td>0.0007</td>
<td>1.1 in.</td>
<td>-0.00316</td>
<td>10.5 Kips ft</td>
<td>6.10e-04 1/in.</td>
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<td>0.0008</td>
<td>1.0 in.</td>
<td>-0.00403</td>
<td>10.7 Kips ft</td>
<td>7.63e-04 1/in.</td>
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<td>9.27e-04 1/in.</td>
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<td>11.0 Kips ft</td>
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<td>11.1 Kips ft</td>
<td>1.64e-03 1/in.</td>
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<td>11.1 Kips ft</td>
<td>1.83e-03 1/in.</td>
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<td>0.0015</td>
<td>0.7 in.</td>
<td>-0.01125</td>
<td>11.2 Kips ft</td>
<td>2.01e-03 1/in.</td>
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<td>0.7 in.</td>
<td>-0.01232</td>
<td>11.2 Kips ft</td>
<td>2.20e-03 1/in.</td>
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<tr>
<td>0.0017</td>
<td>0.7 in.</td>
<td>-0.01339</td>
<td>11.2 Kips ft</td>
<td>2.38e-03 1/in.</td>
</tr>
<tr>
<td>0.0018</td>
<td>0.7 in.</td>
<td>-0.01444</td>
<td>11.3 Kips ft</td>
<td>2.57e-03 1/in.</td>
</tr>
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<td>0.0019</td>
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<td>-0.01548</td>
<td>11.3 Kips ft</td>
<td>2.75e-03 1/in.</td>
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<td>0.0020</td>
<td>0.7 in.</td>
<td>-0.01650</td>
<td>11.3 Kips ft</td>
<td>2.92e-03 1/in.</td>
</tr>
<tr>
<td>0.0025</td>
<td>0.7 in.</td>
<td>-0.02120</td>
<td>11.6 Kips ft</td>
<td>3.74e-03 1/in.</td>
</tr>
<tr>
<td>0.0030</td>
<td>0.7 in.</td>
<td>-0.02517</td>
<td>12.3 Kips ft</td>
<td>4.45e-03 1/in.</td>
</tr>
<tr>
<td>0.0035</td>
<td>0.7 in.</td>
<td>-0.02885</td>
<td>12.7 Kips ft</td>
<td>5.11e-03 1/in.</td>
</tr>
<tr>
<td>0.0040</td>
<td>0.7 in.</td>
<td>-0.03231</td>
<td>13.0 Kips ft</td>
<td>5.74e-03 1/in.</td>
</tr>
<tr>
<td>0.0045</td>
<td>0.7 in.</td>
<td>-0.03582</td>
<td>13.3 Kips ft</td>
<td>6.34e-03 1/in.</td>
</tr>
<tr>
<td>0.0050</td>
<td>0.7 in.</td>
<td>-0.03785</td>
<td>13.3 Kips ft</td>
<td>6.77e-03 1/in.</td>
</tr>
</tbody>
</table>

*Ultimate concrete strain was exceeded.*
SECTION PROPERTIES:

Section Depth = 13.0 in.
Section Width = 7.0 in.
Cover to Main Steel = 0.7 in.

Concrete Strength
Concrete Model
Steel Strength
Young’s Modulus
Steel Model

= 5.00 ksi
= Mander

= 40.0 ksi
= 29000.0 ksi
= Mild Strength Steel

Tension Side Reinforcement
Compression Side Reinforcement
Side Reinforcement

= 4 Bars (#3 Bars)
= 4 Bars (#3 Bars)
= 3 Bars (#3 Bars) each side

Hoop Size
Hoop Spacing

= #3 Bars
= 12.0 in.

Average Number of Legs
Hoop Strength

= 2.0
= 40.0 ksi

SECTION ANALYSIS RESULTS:

Applied Axial Load = 0.0 Kips
Moment Capacity = 35.5 Kips ft (426.6 Kips in.)

Section N.A. Depth = 1.2 in.
Section Curvature = 3.37e-03 1/in.
Maximum Concrete Strain = 4.0000e-03
Extreme Steel Strain = 3.6907e-02
SECTION PROPERTIES:

Section Depth = 13.0 in.
Section Width = 7.0 in.
Cover to Main Steel = 0.7 in.
Concrete Strength = 5.00 ksi
Concrete Model = Mander
Steel Strength = 40.0 ksi
Young's Modulus = 29000.0 ksi
Steel Model = Mild Strength Steel
Tension Side Reinforcement = 4 Bars (#3 Bars)
Compression Side Reinforcement = 4 Bars (#3 Bars)
Side Reinforcement = 3 Bars (#3 Bars) each side
Hoop Size = #3 Bars
Hoop Spacing = 12.0 in.
Average Number of Legs = 2.0
Hoop Strength = 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:
The Ideal Moment Capacity is based on the concrete strain of 0.005.

Applied Axial Load = 0.0 Kips
Eieff = 1.25e+04 Kips sq ft
Curvature Ductility = 15.5 1/in.

![Moment Curvature Plot](image)
<table>
<thead>
<tr>
<th>Conc. Strain</th>
<th>N.A. Depth</th>
<th>Steel Strain</th>
<th>Moment Cap.</th>
<th>Curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0001</td>
<td>3.2 in.</td>
<td>-0.00028</td>
<td>4.8 Kips ft</td>
<td>3.12e-05 1/in.</td>
</tr>
<tr>
<td>0.0002</td>
<td>3.2 in.</td>
<td>-0.00057</td>
<td>9.5 Kips ft</td>
<td>6.24e-05 1/in.</td>
</tr>
<tr>
<td>0.0003</td>
<td>3.2 in.</td>
<td>-0.00085</td>
<td>14.3 Kips ft</td>
<td>9.36e-05 1/in.</td>
</tr>
<tr>
<td>0.0004</td>
<td>3.2 in.</td>
<td>-0.00114</td>
<td>19.0 Kips ft</td>
<td>1.25e-04 1/in.</td>
</tr>
<tr>
<td>0.0005</td>
<td>3.2 in.</td>
<td>-0.00143</td>
<td>23.5 Kips ft</td>
<td>1.57e-04 1/in.</td>
</tr>
<tr>
<td>0.0006</td>
<td>2.9 in.</td>
<td>-0.00191</td>
<td>25.5 Kips ft</td>
<td>2.04e-04 1/in.</td>
</tr>
<tr>
<td>0.0007</td>
<td>2.7 in.</td>
<td>-0.00249</td>
<td>26.7 Kips ft</td>
<td>2.58e-04 1/in.</td>
</tr>
<tr>
<td>0.0008</td>
<td>2.5 in.</td>
<td>-0.00315</td>
<td>27.6 Kips ft</td>
<td>3.20e-04 1/in.</td>
</tr>
<tr>
<td>0.0009</td>
<td>2.3 in.</td>
<td>-0.00390</td>
<td>28.2 Kips ft</td>
<td>3.89e-04 1/in.</td>
</tr>
<tr>
<td>0.0010</td>
<td>2.2 in.</td>
<td>-0.00473</td>
<td>28.6 Kips ft</td>
<td>4.65e-04 1/in.</td>
</tr>
<tr>
<td>0.0011</td>
<td>2.0 in.</td>
<td>-0.00564</td>
<td>29.0 Kips ft</td>
<td>5.46e-04 1/in.</td>
</tr>
<tr>
<td>0.0012</td>
<td>1.9 in.</td>
<td>-0.00651</td>
<td>29.2 Kips ft</td>
<td>6.33e-04 1/in.</td>
</tr>
<tr>
<td>0.0013</td>
<td>1.8 in.</td>
<td>-0.00763</td>
<td>29.4 Kips ft</td>
<td>7.24e-04 1/in.</td>
</tr>
<tr>
<td>0.0014</td>
<td>1.7 in.</td>
<td>-0.00871</td>
<td>29.6 Kips ft</td>
<td>8.20e-04 1/in.</td>
</tr>
<tr>
<td>0.0015</td>
<td>1.6 in.</td>
<td>-0.00984</td>
<td>29.7 Kips ft</td>
<td>9.20e-04 1/in.</td>
</tr>
<tr>
<td>0.0016</td>
<td>1.6 in.</td>
<td>-0.01101</td>
<td>29.8 Kips ft</td>
<td>1.02e-03 1/in.</td>
</tr>
<tr>
<td>0.0017</td>
<td>1.5 in.</td>
<td>-0.01219</td>
<td>29.9 Kips ft</td>
<td>1.13e-03 1/in.</td>
</tr>
<tr>
<td>0.0018</td>
<td>1.5 in.</td>
<td>-0.01343</td>
<td>29.9 Kips ft</td>
<td>1.24e-03 1/in.</td>
</tr>
<tr>
<td>0.0019</td>
<td>1.4 in.</td>
<td>-0.01467</td>
<td>30.0 Kips ft</td>
<td>1.34e-03 1/in.</td>
</tr>
<tr>
<td>0.0020</td>
<td>1.4 in.</td>
<td>-0.01592</td>
<td>30.0 Kips ft</td>
<td>1.45e-03 1/in.</td>
</tr>
<tr>
<td>0.0025</td>
<td>1.3 in.</td>
<td>-0.02197</td>
<td>31.0 Kips ft</td>
<td>1.98e-03 1/in.</td>
</tr>
<tr>
<td>0.0030</td>
<td>1.2 in.</td>
<td>-0.02734</td>
<td>32.8 Kips ft</td>
<td>2.46e-03 1/in.</td>
</tr>
<tr>
<td>0.0035</td>
<td>1.2 in.</td>
<td>-0.03266</td>
<td>34.4 Kips ft</td>
<td>2.93e-03 1/in.</td>
</tr>
<tr>
<td>0.0040</td>
<td>1.2 in.</td>
<td>-0.03754</td>
<td>35.5 Kips ft</td>
<td>3.37e-03 1/in.</td>
</tr>
<tr>
<td>0.0045</td>
<td>1.3 in.</td>
<td>-0.03981</td>
<td>35.9 Kips ft</td>
<td>3.59e-03 1/in.</td>
</tr>
<tr>
<td>0.0050</td>
<td>1.4 in.</td>
<td>-0.04041</td>
<td>35.7 Kips ft</td>
<td>3.68e-03 1/in.</td>
</tr>
</tbody>
</table>

*Ultimate concrete strain was exceeded.*
SECTION PROPERTIES:

Section Depth = 13.0 in.
Section Width = 16.0 in.
Cover to Main Steel = 0.7 in.
Concrete Strength = 5.00 ksi
Concrete Model = Mander
Steel Strength = 40.0 ksi
Young’s Modulus = 29000.0 ksi
Steel Model = Mild Strength Steel
Tension Side Reinforcement = 6 Bars (#3 Bars)
Compression Side Reinforcement = 6 Bars (#3 Bars)
Side Reinforcement = 2 Bars (#3 Bars) each side
Hoop Size = #3 Bars
Hoop Spacing = 12.0 in.
Average Number of Legs = 2.0
Hoop Strength = 40.0 ksi

SECTION ANALYSIS RESULTS:

Applied Axial Load = 0.0 Kips
Moment Capacity = 46.6 Kips ft (558.6 Kips in.)
Section N A. Depth = 0.8 in.
Section Curvature = 4.86e-03 1/in.
Maximum Concrete Strain = 4.0000e-03
Extreme Steel Strain = 5.4985e-02
SECTION PROPERTIES:

- Section Depth = 13.0 in.
- Section Width = 16.0 in.
- Cover to Main Steel = 0.7 in.
- Concrete Strength = 5.00 ksi
- Concrete Model = Mander
- Steel Strength = 40.0 ksi
- Young's Modulus = 29,000.0 ksi
- Steel Model = Mild Strength Steel
- Tension Side Reinforcement = 6 Bars (#3 Bars)
- Compression Side Reinforcement = 6 Bars (#3 Bars)
- Side Reinforcement = 2 Bars (#3 Bars) each side
- Hoop Size = #3 Bars
- Hoop Spacing = 12.0 in.
- Average Number of Legs
- Hoop Strength = 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:
The Ideal Moment Capacity is based on the concrete strain of 0.005.

- Applied Axial Load = 0.0 Kips
- Eeff = 1.65e+04 Kips sq.ft
- Curvature Ductility = 24.4 1/in.

Moment Curvature Plot

1. Theoretical Yield 27.6 Kips 1.39e-04 1/in.
2. Ideal Yield 47.6 Kips 2.40e-04 1/in.
3. Ultimate 47.6 Kips 5.86e-03 1/in.
<table>
<thead>
<tr>
<th>Conc. Strain</th>
<th>N.A. Depth</th>
<th>Steel Strain</th>
<th>Moment Cap.</th>
<th>Curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0001</td>
<td>2.5 in.</td>
<td>-0.00040</td>
<td>8.6 Kips ft</td>
<td>4.02e-05 1/in.</td>
</tr>
<tr>
<td>0.0002</td>
<td>2.5 in.</td>
<td>-0.00079</td>
<td>17.1 Kips ft</td>
<td>8.04e-05 1/in.</td>
</tr>
<tr>
<td>0.0003</td>
<td>2.5 in.</td>
<td>-0.00119</td>
<td>25.6 Kips ft</td>
<td>1.20e-04 1/in.</td>
</tr>
<tr>
<td>0.0004</td>
<td>2.3 in.</td>
<td>-0.00174</td>
<td>31.3 Kips ft</td>
<td>1.74e-04 1/in.</td>
</tr>
<tr>
<td>0.0005</td>
<td>2.0 in.</td>
<td>-0.00263</td>
<td>32.8 Kips ft</td>
<td></td>
</tr>
<tr>
<td>0.0006</td>
<td>1.7 in.</td>
<td>-0.00372</td>
<td>33.6 Kips ft</td>
<td>2.54e-04 1/in.</td>
</tr>
<tr>
<td>0.0007</td>
<td>1.5 in.</td>
<td>-0.00500</td>
<td>34.0 Kips ft</td>
<td>3.51e-04 1/in.</td>
</tr>
<tr>
<td>0.0008</td>
<td>1.4 in.</td>
<td>-0.00643</td>
<td>34.4 Kips ft</td>
<td>4.62e-04 1/in.</td>
</tr>
<tr>
<td>0.0009</td>
<td>1.2 in.</td>
<td>-0.00801</td>
<td>34.6 Kips ft</td>
<td>5.87e-04 1/in.</td>
</tr>
<tr>
<td>0.0010</td>
<td>1.2 in.</td>
<td>-0.00970</td>
<td>34.7 Kips ft</td>
<td>7.23e-04 1/in.</td>
</tr>
<tr>
<td>0.0011</td>
<td>1.1 in.</td>
<td>-0.01149</td>
<td>34.8 Kips ft</td>
<td>8.68e-04 1/in.</td>
</tr>
<tr>
<td>0.0012</td>
<td>1.0 in.</td>
<td>-0.01333</td>
<td>34.9 Kips ft</td>
<td>1.02e-03 1/in.</td>
</tr>
<tr>
<td>0.0013</td>
<td>1.0 in.</td>
<td>-0.01523</td>
<td>35.0 Kips ft</td>
<td>1.18e-03 1/in.</td>
</tr>
<tr>
<td>0.0014</td>
<td>0.9 in.</td>
<td>-0.01715</td>
<td>35.1 Kips ft</td>
<td>1.34e-03 1/in.</td>
</tr>
<tr>
<td>0.0015</td>
<td>0.9 in.</td>
<td>-0.01908</td>
<td>35.2 Kips ft</td>
<td>1.50e-03 1/in.</td>
</tr>
<tr>
<td>0.0016</td>
<td>0.9 in.</td>
<td>-0.02079</td>
<td>35.9 Kips ft</td>
<td>1.67e-03 1/in.</td>
</tr>
<tr>
<td>0.0017</td>
<td>0.9 in.</td>
<td>-0.02242</td>
<td>36.8 Kips ft</td>
<td>1.82e-03 1/in.</td>
</tr>
<tr>
<td>0.0018</td>
<td>0.9 in.</td>
<td>-0.02404</td>
<td>37.6 Kips ft</td>
<td>1.96e-03 1/in.</td>
</tr>
<tr>
<td>0.0019</td>
<td>0.9 in.</td>
<td>-0.02564</td>
<td>38.4 Kips ft</td>
<td>2.10e-03 1/in.</td>
</tr>
<tr>
<td>0.0020</td>
<td>0.8 in.</td>
<td>-0.02722</td>
<td>39.1 Kips ft</td>
<td>2.23e-03 1/in.</td>
</tr>
<tr>
<td>0.0025</td>
<td>0.8 in.</td>
<td>-0.03489</td>
<td>42.0 Kips ft</td>
<td>2.37e-03 1/in.</td>
</tr>
<tr>
<td>0.0030</td>
<td>0.8 in.</td>
<td>-0.04220</td>
<td>44.0 Kips ft</td>
<td>3.03e-03 1/in.</td>
</tr>
<tr>
<td>0.0035</td>
<td>0.8 in.</td>
<td>-0.04917</td>
<td>45.5 Kips ft</td>
<td>3.67e-03 1/in.</td>
</tr>
<tr>
<td>0.0040</td>
<td>0.8 in.</td>
<td>-0.05590</td>
<td>46.6 Kips ft</td>
<td>4.27e-03 1/in.</td>
</tr>
<tr>
<td>0.0045</td>
<td>0.8 in.</td>
<td>-0.06227</td>
<td>47.3 Kips ft</td>
<td>4.86e-03 1/in.</td>
</tr>
<tr>
<td>0.0050</td>
<td>0.9 in.</td>
<td>-0.06721</td>
<td>47.6 Kips ft</td>
<td>5.42e-03 1/in.</td>
</tr>
</tbody>
</table>

*Ultimate concrete strain was exceeded.*
APPENDIX B

SUMITOMO RETROFIT DESIGN CALCULATIONS
OUT-OF-PHASE

SPECIMEN DETAILS

& TEST SETUP
Test setup for out-of-phase Loading
f',c = 4.0 ksi  
fy = 44.0 ksi  
Confinement: Other 
cir cover = 0.67 in 
spacing = 2.27 in 
t10-83 at 1.21 in 
= 1 in^2 
Ix = 1282 in^4 
Iy = 372 in^4 
Xo = 0.00 in 
Yo = 0.00 in  

© 1993 PCA  
Licensed To: Licensee name not yet specified.  
File name: O:\1997\A97106\DOT-SP-1\GAO\OUTPHASE.COL  
Project: DOT Split Column Retrofit  
Column Id: out-of-phase  
Engineer: YOG  
Date: 12/16/98 Time: 10:24:59  
Code: ACI 318-89  
...its: in-lb  

Material Properties:  
Ec = 383 ksi  
fc = 3.4 ksi  
Ea = 29000 ksi  
Betal = 0.85  
Stress Profile: Block  
phi(c) = 1.00, phi(b) = 1.00  

Y-axis slenderness is not considered.
Computer program for the Strength Design of Reinforced Concrete Sections

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General Information:

File Name: D:\1997\A87106\DOT-SP-1\GAO\OUTPFILE.COL
Project: DOT Split Column Retrofit
Column: out-of-phase
Engineer: YOG
Project Code: ACI 318-89
Units: US in-lbs
Date: 12/16/98 Time: 10:24:59

Run Option: Investigation
Run Axis: Y-axis

Material Properties:

f'c = 4 ksi
Ec = 3834.25 ksi
fc = 3.4 ksi
eu = 0.003 in/in
Stress Profile: Block
fy = 44 ksi
Ee = 30000 ksi
erup = 0 in/in
Betaf = 0.85

Geometry:

Rectangular: Width = 7 in
Depth = 13 in

Gross section area: A_g = 91 in^2
Ix = 1281.56 in^4
Iy = 371.583 in^4
Xo = 0 in
Yo = 0 in

Reinforcement:

Rebar Database: ASTM

<table>
<thead>
<tr>
<th>Size</th>
<th>Dia (in)</th>
<th>Area (in^2)</th>
<th>Diam (in)</th>
<th>Area (in^2)</th>
<th>Diam (in)</th>
<th>Area (in^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.38</td>
<td>0.11</td>
<td>4</td>
<td>0.50</td>
<td>5</td>
<td>0.63</td>
</tr>
<tr>
<td>6</td>
<td>0.75</td>
<td>0.44</td>
<td>7</td>
<td>0.88</td>
<td>8</td>
<td>1.00</td>
</tr>
<tr>
<td>9</td>
<td>1.13</td>
<td>1.00</td>
<td>10</td>
<td>1.27</td>
<td>11</td>
<td>1.41</td>
</tr>
<tr>
<td>14</td>
<td>1.69</td>
<td>2.25</td>
<td>18</td>
<td>2.26</td>
<td>4.00</td>
<td></td>
</tr>
</tbody>
</table>

Confinement: User-defined: phi(c) = 1, phi(b) = 1, a = 1
#3 ties with #10 bars, #4 with larger bars.

Layout: Rectangular
Pattern: Sides Different [Cover to longitudinal reinforcement]

Total steel area. As = 1.10 in^2 at 1.21%

<table>
<thead>
<tr>
<th>Top</th>
<th>Bottom</th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bars</td>
<td>3 - #3</td>
<td>3 - #3</td>
<td>2 - #3</td>
</tr>
<tr>
<td>Cover (in)</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
</tr>
<tr>
<td>Bending about</td>
<td>Load, P (kips)</td>
<td>X-Mom. (ft-k)</td>
<td>Y-Mom. (ft-k)</td>
</tr>
<tr>
<td>---------------</td>
<td>----------------</td>
<td>---------------</td>
<td>--------------</td>
</tr>
<tr>
<td>Pure Comp.</td>
<td>354</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Balanced</td>
<td>154</td>
<td>0</td>
<td>31</td>
</tr>
<tr>
<td>Pure Bend.</td>
<td>-0</td>
<td>0</td>
<td>12</td>
</tr>
</tbody>
</table>

Program completed as requested!
Rotational Flexibility / Fixed or Not?

\[ K_{\text{eff}} = \frac{V}{\lambda} = \frac{EI}{L^3} \left[ 12 - \frac{9}{1 + \frac{k_r L}{4EI}} \right] \]

\[
\begin{align*}
K_{\text{eff}} & = \frac{15EI}{L^3} \\
& = \frac{10EI}{L^3} \\
& = \frac{5EI}{L^3} \\
& = \frac{3EI}{L^3} \\
& = \frac{k_r}{L} \\
& = \frac{4EI}{L}
\end{align*}
\]

For \( k_r \leq \frac{4EI}{L} \) the system is fixed, for \( k_r \geq \frac{4EI}{L} \) the system is pinned.

---

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UMD-ITV
Seismic Bridge Design Applications
25 July 1996, NHI Course Code No. 13063
f'c = 4.0ksi
fy = 44.0 ksi
Confinement: Other
clr cover = 0.67 in
spacing = 2.27 in
10-#3 at 1.21 in
I = 1 in^2
Ix = 1282 in^4
Iy = 372 in^4
Xo = 0.00 in
Yx = 0.00 in

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Licensed To: Licensee name not yet specified.

File name: O:\1997\A97106\DOT-SP-1\GAC\TRANSY.COL
Project: DOT Split Column Retrofit
Column Id: Transverse
Engineer: YOG
Date: 12/16/98, Time: 10:24:59
Code: ACI 318-89
-Jits: in-lb
X-axis slenderness is not considered.

Material Properties:
Ec = 3834 ksi
Es = 29000 ksi
Ec = 0.003 in/in
f = 3.40 ksi

Stress Profile: Block

phi(c) = 1.00, phi(b) = 1.00
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General Information:

File Name: 0:\1997\A97106\DOT-SP-1\GAO\TRANSV.COL
Project: DOT Split Column Retrofit Code: ACT 318-89
Column: Transverse Units: US in-lbs
Engineer: YOC Date: 12/14/98 Time: 10:24.59
Run Option: Investigation Short (nonslender) column
Run Axis: Z-axis Column Type: Structural

Material Properties:

\( f'c = 4 \text{ ksi} \)  \( \sigma_y = 44 \text{ ksi} \)
\( f_c = 3834.25 \text{ ksi} \)  \( E_s = 29000 \text{ ksi} \)
\( f_{cu} = 0.003 \text{ in/in} \)  \( e_{rup} = 0 \text{ in/in} \)
Stress Profile: Block  \( \beta_{zal} = 0.85 \)

Geometry:

Rectangular: Width = 7 in  \( \text{Depth} = 13 \text{ in} \)
Gross section area, \( A_g = 91 \text{ in}^2 \)
\( I_x = 1281.58 \text{ in}^4 \)
\( I_y = 371.583 \text{ in}^4 \)
\( x_0 = 0 \text{ in} \)
\( y_0 = 0 \text{ in} \)

Reinforcement:

Rebar Database: ASTM

<table>
<thead>
<tr>
<th>Size</th>
<th>Diam</th>
<th>Area Size</th>
<th>Diam</th>
<th>Area Size</th>
<th>Diam</th>
<th>Area</th>
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<tbody>
<tr>
<td>3</td>
<td>0.38</td>
<td>0.11</td>
<td>4</td>
<td>0.50</td>
<td>0.20</td>
<td>5</td>
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<tr>
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<td>0.60</td>
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<td>1.00</td>
<td>10</td>
<td>1.27</td>
<td>1.27</td>
<td>11</td>
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<td>2.25</td>
<td>18</td>
<td>2.26</td>
<td>4.00</td>
<td>18</td>
</tr>
</tbody>
</table>

Confined: User-defined; \( \phi(r) = 1 \), \( \phi(b) = 1 \), \( a = 1 \)
#3 ties with #10 bars, #4 with larger bars.

Layout: Rectangular
Pattern: Sides Different [Cover to longitudinal reinforcement]

Total steel area, \( A_s = 1.10 \text{ in}^2 \text{ at } 1.21\% \)

<table>
<thead>
<tr>
<th>Bars</th>
<th>Top</th>
<th>Bottom</th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 - #3</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2 - #3</td>
</tr>
<tr>
<td>Cover (in)</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
</tr>
<tr>
<td>Bending about</td>
<td>Load, P (kips)</td>
<td>X-Mom. (ft-k)</td>
<td>Y-Mom. (ft-k)</td>
<td>N.A. depth (in)</td>
</tr>
<tr>
<td>---------------</td>
<td>----------------</td>
<td>---------------</td>
<td>---------------</td>
<td>----------------</td>
</tr>
<tr>
<td>X Pure Comp.</td>
<td>354</td>
<td>-0</td>
<td>-0</td>
<td>24.57</td>
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<tr>
<td>Balanced</td>
<td>169</td>
<td>56</td>
<td>-0</td>
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<td>Pure Bend.</td>
<td>0</td>
<td>23</td>
<td>-0</td>
<td>1.27</td>
</tr>
</tbody>
</table>

Program completed as requested.
FIG 5. Carbon Jacket Regions for Bridge Column Retrofit, Single Bending

FIG 6. Carbon Jacket Regions for Bridge Column Retrofit, Double Bending
FIG 5. Carbon Jacket Regions for Bridge Column Retrofit, Single Bending

FIG 6. Carbon Jacket Regions for Bridge Column Retrofit, Double Bending
List of Reference Materials


Facsimile Cover Sheet

TO: David McLean
Firm: WSU CEE
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Fax Number: 509-335-7632
cc: Michael W. LaNier

FROM: Lee Marsh
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Fax ID Number 99-5-12-1

Total Number of Pages Transmitted (Including Cover Sheet): 7
Date Transmitted: 5/13/99
Time Transmitted: 2:10
Transmitted by: [Signature]

BERGER/ABAM Job Number: PH810H

RE: WSU / WSDOT Split Column Tests

David,

In a recent conversation with one of your researchers, Paul Rognnes, it came to my attention that the calculations and design drawings for the transverse loading specimen of the above referenced tests did not properly include the splice confinement. The methodology that we followed, that developed by the University of California at San Diego, explicitly calculates the confinement for reinforcement lap splice zones. However, this methodology also recognizes that confinement of rectangular splice zones with rectangular jackets is not entirely possible. Since your tests were conducted on such a configuration, and since this effect was one of the parameters that you were investigating, I calculated the numbers of Replark 30 wraps that would have been recommended for the tests of the splice zone. The attached calculations indicate that 11 or so wraps would have been required. We understand that the performance of the specimens with the 6 wraps that were provided was fairly good. We would expect that if 11 wraps had been used, the performance may have been somewhat better. Therefore, the results probably form a lower bound to the performance expected if the calculated number of wraps are used. I hope that this additional information can be of help to you. If you have any questions, please give me a call.

Regards,

Lee

BERGER/ABAM Engineers Inc., 33301 Ninth Avenue South • Federal Way, WA 98003-6370
Phone 206/431-2300 • Fax 206/431-2250

B-62
Jacket Thickness

for Engr. Column

\[ f_j = \frac{500 D (f_y - f_c)}{E_j} \]

\[ D = 16.33' \quad f_y = 300 \text{ psi} \]

\[ f_j = \frac{500 (16.33)(300)}{53426 (1000)} \quad \text{ksi} \]

\[ f_j = 0.0733 \quad E_j = 53426 \text{ ksi} \]

Ref: ACI-95/06

Earthquake Earthquake of Bridge Columns with
Continuous Carbon Fiber Jackets, et al.

\[ \# \text{ wraps} = \frac{0.0733}{0.0066} = 11 \text{ wraps read} \]

\[ f_y = \frac{d}{1.1} \left[ \frac{15}{200} + 2(0.375 + 0.67) / 7.5 \right] \quad 0.254 \text{ ksi use 300 psi} \]

\[ p = \text{mode crack parameter} \]

\[ d_b = \text{bar dia.} \]

\[ n = \# \text{ of bars} \]

\[ c_c = \text{concrete cover} \]

\[ L_5 = \text{splice length} \]

\[ p \approx 16.33 - 0.67(2) \quad H = 15' \]

\[ n = 10 \text{ bars} \]

\[ d_b = 0.375'' \]

\[ c_c = 0.67'' \]

\[ L_5 = 7.5'' (2d_b) \]

**ACI-95/06** Does not recommend using rectangular jackets on rectangular columns to confine splice zones. If some debonding is permissible in the splice zone, then rect. jackets may be effective. Also typically the cal. jacket thickness would be doubled, producing 0.14 in. or 22 wraps.
Alternate method of determining jacket thickness

\[ p_o = 2 \left( \frac{A_{bf}}{A_{pl}} \right) \left( \frac{P_l}{P_{pl}} \right) \]

where:
- \( A_{bf} \) is the sectional area of jacket
- \( A_{pl} \) is the sectional area of pipe
- \( P_l \) is the load in pipe
- \( P_{pl} \) is the load in jacket

Calculate based on:
- 0.001 E
- 0.001

so this is less conservative

\[ p_o = 2 \left( \frac{0.47}{0.0015} \right) = 0.0169 \]

\[ f_j = \frac{P_o}{2} \left[ \frac{b+h}{2} \right] \]

for rectangle

\[ f_j = 0.043 \]

Note: If use \( f_j = 0.047 \)

**ACIT method may be a better predictor since \( f_o \)**

\[ f_j = \frac{500}{3428} \left( \frac{0.001}{0.0015} \right) = 0.077 \]

\[ \# \text{ wraps} = \frac{0.077}{0.006} = 11.6 \text{ wraps} \]

- or about the same as before.
APPENDIX C

XXSYS RETROFIT DESIGN CALCULATIONS
**XXSYS "CHOPSTICK DESIGN"**

**Column Specifications:**

- Column Height, \( L := 60 \) (in)
- Shear Span, \( L_e := 30 \) (in)
- Column Width, \( B := 13 \) (in)
- Column Depth, \( D := 7 \) (in)
- Concrete Cover, \( c_c := 0.67 \) (in)
- Concrete Compressive Strength, \( f_c := 5000 \) (ksi)

**Longitudinal Reinforcement 10 #3**

- \( f_{yl} := 40 \) (ksi)
- Number of Bars \( n := 10 \)
- Bar Diameter \( d_b := 0.375 \) (in)
- Bar Area \( A_b := 0.11 \) (in²)
- Steel Modulus \( E_s := 29000 \) (ksi)

**Transverse Reinforcement 0.12 @ 4 in.**

- \( f_{yt} := 40 \) (ksi)
- Bar Diameter \( d_s := 0.12 \) (in)
- Bar Area \( A_s := 0.011 \) (in²)
- Spacing \( s := 4 \) (in)

**Lap Splice Length,** \( L_s := 7.5 \) (in)

**Section Properties:**

- Axial Load, \( P := 0 \) (kips)
- Moment Capacity, \( M_{yi} := 152.4 \) (kip in)
- Yield Curvature, \( \phi_y := 0.000426 \) (1/in)
- Neutral Axis Depth, \( c_u := 0.7 \) (in)

**Jacket Material Properties:**

- Jacket Modulus, \( E_j := 12000 \) (ksi)
- Ultimate Jacket Strength, \( f_{ju} := 120 \) (ksi)
- Ultimate Strain, \( \varepsilon_{ju} := 0.01 \) (in/in)

**Required Displacement Ductility:** \( \mu_\Delta := 8 \)

C-2
**Shear Strength Retrofit:**

Plastic shear including overstrength =

\[ V_o := 1.5 \left( \frac{M_{yi}}{L_e} \right) \]

\[ V_o = 7.6 \text{ (kips)} \]

Concrete shear contribution =

\[ k := \text{if} \left( \mu_\Delta < 2.3, \text{if} \left( \mu_\Delta < 4.5 - \mu_\Delta, \text{if} \left( \mu_\Delta < 8, 1.5 - \frac{\mu_\Delta}{8}, \text{if} \left( \mu_\Delta \geq 8, 0.5, 0.5 \right) \right) \right) \right) \]

\[ k = 0.5 \]

\[ V_{ci} := k \cdot f_c \cdot 0.8 \cdot (D \cdot B) \cdot 10^{-3} \]

\[ V_{ci} = 2.6 \text{ (kips)} \]

\[ V_{co} := 3 \cdot f_c \cdot 0.8 \cdot (D \cdot B) \cdot 10^{-3} \]

\[ V_{co} = 15.4 \text{ (kips)} \]

Hoop reinforcement shear contribution =

\[ D_c := D - 2 \cdot cc + d_b \]

\[ \theta := 45 \]

\[ \alpha := \theta \cdot \frac{\pi}{180} \text{ (change to radians)} \]

\[ n_{bar} := 2 \text{ (# transverse bars)} \]

\[ V_s := \frac{n_{bar} \cdot A_s \cdot f_y \cdot D_c}{s} \cdot \cot(\alpha) \]

\[ V_s = 1.3 \text{ (kips)} \]

Axial load shear contribution =

\[ V_p := P \cdot \frac{D - c_u}{2 \cdot L_e} \]

\[ V_p = 0 \text{ (kips)} \]

Jacket thickness inside the plastic hinge region \( t_{vi} = \)

Strength reduction factor \( \phi \)

\[ \phi = 0.85 \]

\[ t_{vi} := \frac{125}{(E_j \cdot D)} \left[ \frac{V_o}{\phi} - (V_{ci} + V_s + V_p) \right] \]

\[ t_{vi} = 0.008 \text{ (in)} \]

Jacket thickness outside the plastic hinge region \( t_{vo} = \)

\[ t_{vo} := \frac{125}{(E_j \cdot D)} \left[ \frac{V_o}{\phi} - (V_{co} + V_s + V_p) \right] \]

\[ t_{vo} = -0.012 \text{ (in)} \]
**Flexural Plastic Hinge Confinement:**

Equivalent column diameter \( D_e = \)

\[
A := D \\
k := \frac{2}{(2 \cdot k)} \\
b := \sqrt[4]{\frac{A}{(2 \cdot k)}^2 + \frac{B}{2}^2} \\
a := k \cdot b \\
R_1 := \frac{b^2}{a} \\
R_2 := \frac{a^2}{b} \\
D_e := R_1 + R_2 \\
\]

\[D_e = 16.331 \text{ (in)}\]

Plastic hinge length \( L_p = L_e \cdot 0.08 + 0.15 \cdot f_y \cdot d \cdot b \)

\[L_p = 4.7 \text{ (in)}\]

Curvature ductility demand =

\[\mu_\phi := 1 + \frac{\mu_\Delta - 1}{3 \cdot \left( \frac{L_p}{L_e} \right) \left[ 1 - 0.5 \cdot \frac{L_p}{L_e} \right]} \]

\[\mu_\phi = 17\]

Required ultimate compression strain in the concrete =

\[\varepsilon_{cu} := \mu_\phi \cdot \phi_y \cdot c_u \]

\[\varepsilon_{cu} = 0.0052\]

Jacket thickness \( t_{c1} \) and \( t_{c2} = \)

\[t_{c1} := \left[ \frac{D_e \cdot (\varepsilon_{cu} - 0.004) \cdot 1.5 \cdot f_c}{0.1 \cdot f_{ju} \cdot \varepsilon_{ju}} \cdot 10^{-3} \right]^{0.5} \]

\[t_{c1} = 0.024 \text{ (in)}\]

\[t_{c2} = \frac{t_{c1}}{2} \]

\[t_{c2} = 0.012 \text{ (in)}\]

Thickness to prevent bar buckling =

\[t_b = \frac{n \cdot D}{E_j} \]

\[t_b = 0.006 \text{ (in)}\]
Jacket Specifications:

Shear Strength Component =

\[ L_{vi} := 1.5 \cdot D \]
\[ L_{vo} := L - 2 \cdot L_{vi} \]

\[ L_{vi} = 10.5 \text{ (in)} \quad t_{vi} = 0.008 \text{ (in)} \]
\[ L_{vo} = 39 \text{ (in)} \quad t_{vo} = -0.012 \text{ (in)} \]

Confinement Component =

\[ L_{c1} := \text{if} \left( 0.5 \cdot D \geq 0.125 \cdot L_{e} \right) \] \[ L_{c2} := \text{if} \left( 0.5 \cdot D \geq 0.125 \cdot L_{e} \right) \]
\[ L_{c1} = 3.75 \text{ (in)} \quad t_{c1} = 0.024 \text{ (in)} \]
\[ L_{c2} = 3.75 \text{ (in)} \quad t_{c2} = 0.012 \text{ (in)} \]
\[ t_{b} = 0.006 \text{ (in)} \]
SECTION PROPERTIES:

Section Depth = 7.0 in.
Section Width = 13.0 in.
Cover to Main Steel = 0.7 in.
Concrete Model = Mander (no tensile strength)
Concrete Strength = 5.00 ksi
Steel Model = Mild Strength Steel
Steel Strength = 40.0 ksi
Young's Modulus = 29000.0 ksi
Tension Side Reinforcement = 4 Bars (#3 Bars)
Compression Side Reinforcement = 4 Bars (#3 Bars)
Side Reinforcement = 1 Bars (#3 Bars) each side
Hoop Size = 0.12 Bars
Hoop Spacing = 4.0 in.
Average Number of Legs = 2.0
Hoop Strength = 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:
The Ideal Moment Capacity is based on the concrete strain of 0.004.

Applied Axial Load = 0.0 Kips
Eleft = 2.49e+03 Kips sq.ft
Curvature Ductility = 13.9

Moment Curvature Plot

1. Theoretical Yield 8.9 Kips ft
2. Ideal Yield 12.7 Kips ft
3. Ultimate 13.0 Kips ft
<table>
<thead>
<tr>
<th>Conc. Strain</th>
<th>N.A. Depth</th>
<th>Steel Strain</th>
<th>Moment Cap.</th>
<th>Curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0001</td>
<td>1.5 in.</td>
<td>-0.00030</td>
<td>2.1 Kips ft</td>
<td>6.55e-05 1/in.</td>
</tr>
<tr>
<td>0.0002</td>
<td>1.5 in.</td>
<td>-0.00060</td>
<td>4.1 Kips ft</td>
<td>1.31e-04 1/in.</td>
</tr>
<tr>
<td>0.0003</td>
<td>1.5 in.</td>
<td>-0.00091</td>
<td>6.2 Kips ft</td>
<td>1.96e-04 1/in.</td>
</tr>
<tr>
<td>0.0004</td>
<td>1.5 in.</td>
<td>-0.00121</td>
<td>8.3 Kips ft</td>
<td>2.62e-04 1/in.</td>
</tr>
<tr>
<td>0.0005</td>
<td>1.5 in.</td>
<td>-0.00161</td>
<td>9.7 Kips ft</td>
<td>3.44e-04 1/in.</td>
</tr>
<tr>
<td>0.0006</td>
<td>1.3 in.</td>
<td>-0.00224</td>
<td>10.1 Kips ft</td>
<td>4.63e-04 1/in.</td>
</tr>
<tr>
<td>0.0007</td>
<td>1.2 in.</td>
<td>-0.00297</td>
<td>10.3 Kips ft</td>
<td>5.97e-04 1/in.</td>
</tr>
<tr>
<td>0.0008</td>
<td>1.1 in.</td>
<td>-0.00377</td>
<td>10.5 Kips ft</td>
<td>7.44e-04 1/in.</td>
</tr>
<tr>
<td>0.0009</td>
<td>1.0 in.</td>
<td>-0.00463</td>
<td>10.6 Kips ft</td>
<td>9.00e-04 1/in.</td>
</tr>
<tr>
<td>0.0010</td>
<td>0.9 in.</td>
<td>-0.00553</td>
<td>10.7 Kips ft</td>
<td>1.06e-03 1/in.</td>
</tr>
<tr>
<td>0.0011</td>
<td>0.9 in.</td>
<td>-0.00646</td>
<td>10.8 Kips ft</td>
<td>1.23e-03 1/in.</td>
</tr>
<tr>
<td>0.0012</td>
<td>0.9 in.</td>
<td>-0.00741</td>
<td>10.9 Kips ft</td>
<td>1.40e-03 1/in.</td>
</tr>
<tr>
<td>0.0013</td>
<td>0.8 in.</td>
<td>-0.00838</td>
<td>11.0 Kips ft</td>
<td>1.58e-03 1/in.</td>
</tr>
<tr>
<td>0.0014</td>
<td>0.8 in.</td>
<td>-0.00935</td>
<td>11.0 Kips ft</td>
<td>1.75e-03 1/in.</td>
</tr>
<tr>
<td>0.0015</td>
<td>0.8 in.</td>
<td>-0.01032</td>
<td>11.1 Kips ft</td>
<td>1.92e-03 1/in.</td>
</tr>
<tr>
<td>0.0016</td>
<td>0.8 in.</td>
<td>-0.01128</td>
<td>11.1 Kips ft</td>
<td>2.10e-03 1/in.</td>
</tr>
<tr>
<td>0.0017</td>
<td>0.7 in.</td>
<td>-0.01224</td>
<td>11.2 Kips ft</td>
<td>2.27e-03 1/in.</td>
</tr>
<tr>
<td>0.0018</td>
<td>0.7 in.</td>
<td>-0.01319</td>
<td>11.2 Kips ft</td>
<td>2.44e-03 1/in.</td>
</tr>
<tr>
<td>0.0019</td>
<td>0.7 in.</td>
<td>-0.01412</td>
<td>11.2 Kips ft</td>
<td>2.61e-03 1/in.</td>
</tr>
<tr>
<td>0.0020</td>
<td>0.7 in.</td>
<td>-0.01503</td>
<td>11.2 Kips ft</td>
<td>2.77e-03 1/in.</td>
</tr>
<tr>
<td>0.0025</td>
<td>0.7 in.</td>
<td>-0.01936</td>
<td>11.3 Kips ft</td>
<td>3.56e-03 1/in.</td>
</tr>
<tr>
<td>0.0030</td>
<td>0.7 in.</td>
<td>-0.02290</td>
<td>12.0 Kips ft</td>
<td>4.22e-03 1/in.</td>
</tr>
<tr>
<td>0.0035</td>
<td>0.7 in.</td>
<td>-0.02613</td>
<td>12.4 Kips ft</td>
<td>4.82e-03 1/in.</td>
</tr>
<tr>
<td>0.0040</td>
<td>0.7 in.</td>
<td>-0.02913</td>
<td>12.7 Kips ft</td>
<td>5.39e-03 1/in.</td>
</tr>
<tr>
<td>0.0045</td>
<td>0.8 in.</td>
<td>-0.03192</td>
<td>13.0 Kips ft</td>
<td>5.93e-03 1/in.</td>
</tr>
</tbody>
</table>

Ultimate concrete strain was exceeded.
“Chopsticks” Design – Part 2

Goal: limit crack width to 1 mm in the 16” x 13” wide section to assure aggregate interlock in the concrete.

1)  
1 mm x 1 in/25.4 mm = .03937” total crack width in concrete section

2)  
This crack width spread over the 16” wide section would be:
.03937”/16” = .00246 in/in of width = .246% strain = the strain limit in composite

3)  
Now, using: \( \sigma_{\text{applied}} = \varepsilon E \)
    then:  \( \sigma_{\text{applied}} = (.00246)(12,000,000) \)
    \( = 29,520 \text{ lbs/in}^2 \) would be developed in the composite at that strain level.

4)  
Using a material with a strength of 120,000 lb/in², we would need 29,520/120,000 or .246 in² of material to restrain the concrete.

5)  
If this area was spread out over 6”, it would be .041” thick (.246in²/6” = .041”)  
Over 12”, it would be .0205” thick.

6)  
Therefore, as a safe design that both restrains the crack and allows for a smooth transition of load resistance, we will apply (2) layers of .0266” thick GA180 fabric to the column: (1) 12” wide section overlaid by a (1) 6”-wide section applied to the top 6”. The bottom 6” would have a thickness of .0266” and the top 6” would have a thickness of .0532”.

C-8
Column Specifications:

- Column Height, \( L := 104 \) (in)
- Shear Span, \( L_e := 104 \) (in)
- Column Width, \( B := 16 \) (in)
- Column Depth, \( D := 13 \) (in)
- Concrete Cover, \( c_c := 0.67 \) (in)
- Concrete Compressive Strength, \( f_c := 5000 \) (psi)

Longitudinal Reinforcement 20 #3  
- \( f_{yl} := 40 \) (ksi)
- Number of Bars, \( n := 16 \)
- Bar Diameter, \( d_b := 0.375 \) (in)
- Bar Area, \( A_b := 0.11 \) (in²)
- Steel Modulus, \( E_s := 29000 \) (ksi)

Transverse Reinforcement 0.12 @ 4 in.  
- \( f_{yt} := 40 \) (ksi)
- Bar Diameter, \( d_s := 0.12 \) (in)
- Bar Area, \( A_s := 0.011 \) (in²)
- Spacing, \( s := 4 \) (in)

Lap Splice Length, \( L_s := 7.5 \) (in)

Section Properties:

- Axial Load, \( P := 0 \) (kips)
- Moment Capacity, \( M_{yi} := 547.2 \) (kip in)
- Yield Curvature, \( \phi_y := 0.000235 \) (1/in)
- Neutral Axis Depth, \( c_u := 0.9 \) (in)

Jacket Material Properties:

- Jacket Modulus, \( E_j := 12000 \) (ksi)
- Ultimate Jacket Strength, \( f_{ju} := 120 \) (ksi)
- Ultimate Strain, \( \varepsilon_{ju} := 0.01 \) (in/in)

Required Displacement Ductility: \( \mu_A := 8 \)
Shear Strength Retrofit:

Plastic shear including overstrength =

\[ V_o := 1.5 \left( \frac{M_{yi}}{L_e} \right) \]

\[ V_o = 7.9 \text{ (kips)} \]

Concrete shear contribution =

\[ k := \text{if } \left( \mu \Delta < 2.3, \text{if } \mu \Delta < 4.5, \mu \Delta \cdot \text{if } \mu \Delta < 8, 1.5 - \frac{\mu \Delta}{8}, \text{if } \mu \Delta \geq 8, 0.5, 0.5 \right) \]

\[ k = 0.5 \]

\[ V_{ci} := k \cdot \sqrt{f_c} \cdot 0.8 \cdot (D \cdot B) \cdot 10^{-3} \]

\[ V_{ci} = 5.9 \text{ (kips)} \]

\[ V_{co} := 3 \cdot \sqrt{f_c} \cdot 0.8 \cdot (D \cdot B) \cdot 10^{-3} \]

\[ V_{co} = 35.3 \text{ (kips)} \]

Hoop reinforcement shear contribution =

\[ D_c := D - 2 \cdot cc + d_b \]

\[ \theta := 45 \]

\[ \alpha := \theta \cdot \frac{\pi}{180} \text{ (change to radians)} \]

\[ \text{nbar} := 2 \text{ (# transverse bars)} \]

\[ V_s := \frac{\text{nbar} \cdot A_s \cdot f_{yt} \cdot D_c}{s} \cdot \cot(\alpha) \]

\[ V_s = 2.6 \text{ (kips)} \]

Axial load shear contribution =

\[ V_p := P \cdot \frac{D - c_u}{2 \cdot L_e} \]

\[ V_p = 0 \text{ (kips)} \]

Jacket thickness inside the plastic hinge region \( t_{vi} = \)

Strength reduction factor \( \phi \quad \phi := 0.85 \)

\[ t_{vi} := \frac{125}{(E_j \cdot D)} \left[ \frac{V_o}{\phi} - (V_{ci} + V_s + V_p) \right] \]

\[ t_{vi} = \text{6.044} \cdot 10^{-4} \text{ (in)} \]

Jacket thickness outside the plastic hinge region \( t_{vo} = \)

\[ t_{vo} := \frac{125}{(E_j \cdot D)} \left[ \frac{V_o}{\phi} - (V_{co} + V_s + V_p) \right] \]

\[ t_{vo} = \text{-0.023} \text{ (in)} \]
Flexural Plastic Hinge Confinement:

Equivalent column diameter $D_e =$

$$A := D^2$$

$$k := \left(\frac{A}{B}\right)^3$$

$$b := \sqrt{\frac{A}{(2 \cdot k) \cdot \left(\frac{A}{(2 \cdot k)} + \frac{B}{2}\right)^2}}$$

$$a := k \cdot b$$

$$R_1 := \frac{b^2}{a}$$

$$R_2 := \frac{a^2}{b}$$

$$D_e := R_1 + R_2$$

$D_e = 20.862$ (in)

Plastic hinge length $L_p := L_e \cdot 0.08 + 0.15 \cdot f_y \cdot d_b$ $L_p = 10.6$ (in)

Curvature ductility demand $\mu \phi$

$$\mu \phi := 1 + \frac{\mu_\Delta - 1}{3 \cdot \left(\frac{L_p}{L_e}\right) \cdot \left(1 - 0.5 \cdot \frac{L_p}{L_e}\right)}$$

$\mu_\phi = 25$

Required ultimate compression strain in the concrete $\varepsilon_{cu}$

$$\varepsilon_{cu} := \mu \phi \cdot \varepsilon_{cu}$$

$\varepsilon_{cu} = 0.0053$

Jacket thickness $t_{c1}$ and $t_{c2}$

$$t_{c1} := \left[\frac{D_e \cdot (\varepsilon_{cu} - 0.004) \cdot 1.5 \cdot f_c}{f_{ju} \cdot \varepsilon_{ju}} \cdot 10^3\right] \cdot 2$$

$t_{c1} = 0.035$ (in)

$t_{c2} = \frac{t_{c1}}{2}$

$t_{c2} = 0.017$ (in)

Thickness to prevent bar buckling $t_b := \frac{n \cdot D}{E_j}$

$t_b = 0.017$ (in)
Lap Splice Clamping:

Available lateral clamping pressure =

\[ f_h := 0 \quad \text{f}_h = 0 \quad \text{(ksi)} \]

Required clamping pressure =

\[ dd := \left[ D - 2 \cdot (d_b + cc) \right] \]
\[ bb := \left[ B - 2 \cdot (d_b + cc) \right] \]
\[ p := 2 \cdot (dd + bb) \]

\[ f_1 := \frac{A_b \cdot f_{yl}}{\left( \frac{p}{2 \cdot n} + 2 \cdot (d_b + cc) \right) \cdot L_s} \quad f_1 = 0.161 \quad \text{(ksi)} \]

Required jacket thickness =

\[ t_s := \left[ 500 \cdot \left( \frac{D}{E_j} \right) \cdot (f_1 - f_h) \right] \cdot 2 \quad t_s = 0.28 \quad \text{(in)} \]
Jacket Specifications:

Shear Strength Component =

\[ L_{vi} = 1.5 \cdot D \]
\[ L_{vo} = L - 2 \cdot L_{vi} \]

\[ L_{vi} = 19.5 \text{ (in)} \quad t_{vi} = 6.044 \cdot 10^6 \text{ (in)} \]
\[ L_{vo} = 65 \text{ (in)} \quad t_{vo} = -0.023 \text{ (in)} \]

Confinement Component =

\[ L_{c1} = \frac{0.5 \cdot D \geq 0.125 \cdot L_e, 0.5 \cdot D, 0.125 \cdot L_e}{L_{c1} = 13 \text{ (in)} \quad t_{c1} = 0.035 \text{ (in)}}
\]
\[ L_{c2} = \frac{0.5 \cdot D \geq 0.125 \cdot L_e, 0.5 \cdot D, 0.125 \cdot L_e}{L_{c2} = 13 \text{ (in)} \quad t_{c2} = 0.017 \text{ (in)}}
\]
\[ t_b = 0.017 \text{ (in)} \]
SECTION PROPERTIES:

- **Section Depth** = 13.0 in.
- **Section Width** = 16.0 in.
- **Cover to Main Steel** = 0.7 in.

**Concrete Model**
- **Concrete Strength** = Mander (no tensile strength) = 5.00 ksi

**Steel Model**
- **Steel Strength** = Mild Strength Steel = 40.0 ksi
- **Young's Modulus** = 29000.0 ksi

**Tension Side Reinforcement** = 6 Bars (#3 Bars)
- **Compression Side Reinforcement** = 6 Bars (#3 Bars)
- **Side Reinforcement** = 2 Bars (#3 Bars) each side

**Hoop Size** = 0.12 Bars
- **Average Number of Legs** = 2.0
- **Hoop Spacing** = 4.0 in.
- **Hoop Strength** = 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:
The Ideal Moment Capacity is based on the concrete strain of 0.004.

- **Applied Axial Load** = 0.0 Kips
- **Eff** = 1.62e+04 Kips sq.ft
- **Curvature Ductility** = 21.6
- **Ag** = 208.0 sq.in
- **lg** = 2.93e+03 in^4
- **leff** = 5.48e+02 in^4

Moment Curvature Plot

- **Moment**
  1. Theoretical Yield 27.5 Kips ft Curvature 1.42e-04 1/in.
  2. Ideal Yield 45.6 Kips ft Curvature 2.35e-04 1/in.
  3. Ultimate 46.4 Kips ft Curvature 5.07e-03 1/in.
<table>
<thead>
<tr>
<th>Conc. Strain</th>
<th>N.A. Depth</th>
<th>Steel Strain</th>
<th>Moment Cap.</th>
<th>Curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0001</td>
<td>2.5 in.</td>
<td>-0.00039</td>
<td>8.4 Kips ft</td>
<td>4.02e-05 1/in.</td>
</tr>
<tr>
<td>0.0002</td>
<td>2.5 in.</td>
<td>-0.00078</td>
<td>16.8 Kips ft</td>
<td>8.05e-05 1/in.</td>
</tr>
<tr>
<td>0.0003</td>
<td>2.5 in.</td>
<td>-0.00116</td>
<td>25.2 Kips ft</td>
<td>1.21e-04 1/in.</td>
</tr>
<tr>
<td>0.0004</td>
<td>2.3 in.</td>
<td>-0.00170</td>
<td>31.0 Kips ft</td>
<td>1.73e-04 1/in. _Yield</td>
</tr>
<tr>
<td>0.0005</td>
<td>2.0 in.</td>
<td>-0.00256</td>
<td>32.5 Kips ft</td>
<td>2.52e-04 1/in.</td>
</tr>
<tr>
<td>0.0006</td>
<td>1.7 in.</td>
<td>-0.00360</td>
<td>33.3 Kips ft</td>
<td>3.46e-04 1/in.</td>
</tr>
<tr>
<td>0.0007</td>
<td>1.5 in.</td>
<td>-0.00482</td>
<td>33.8 Kips ft</td>
<td>4.55e-04 1/in.</td>
</tr>
<tr>
<td>0.0008</td>
<td>1.4 in.</td>
<td>-0.00618</td>
<td>34.1 Kips ft</td>
<td>5.75e-04 1/in.</td>
</tr>
<tr>
<td>0.0009</td>
<td>1.3 in.</td>
<td>-0.00767</td>
<td>34.3 Kips ft</td>
<td>7.06e-04 1/in.</td>
</tr>
<tr>
<td>0.0010</td>
<td>1.2 in.</td>
<td>-0.00926</td>
<td>34.5 Kips ft</td>
<td>8.45e-04 1/in.</td>
</tr>
<tr>
<td>0.0011</td>
<td>1.1 in.</td>
<td>-0.01093</td>
<td>34.6 Kips ft</td>
<td>9.91e-04 1/in.</td>
</tr>
<tr>
<td>0.0012</td>
<td>1.1 in.</td>
<td>-0.01264</td>
<td>34.7 Kips ft</td>
<td>1.14e-03 1/in.</td>
</tr>
<tr>
<td>0.0013</td>
<td>1.0 in.</td>
<td>-0.01440</td>
<td>34.8 Kips ft</td>
<td>1.29e-03 1/in.</td>
</tr>
<tr>
<td>0.0014</td>
<td>1.0 in.</td>
<td>-0.01619</td>
<td>34.9 Kips ft</td>
<td>1.45e-03 1/in.</td>
</tr>
<tr>
<td>0.0015</td>
<td>0.9 in.</td>
<td>-0.01798</td>
<td>35.0 Kips ft</td>
<td>1.60e-03 1/in.</td>
</tr>
<tr>
<td>0.0016</td>
<td>0.9 in.</td>
<td>-0.01968</td>
<td>35.2 Kips ft</td>
<td>1.75e-03 1/in.</td>
</tr>
<tr>
<td>0.0017</td>
<td>0.9 in.</td>
<td>-0.02128</td>
<td>36.2 Kips ft</td>
<td>1.89e-03 1/in.</td>
</tr>
<tr>
<td>0.0018</td>
<td>0.9 in.</td>
<td>-0.02275</td>
<td>36.9 Kips ft</td>
<td>2.02e-03 1/in.</td>
</tr>
<tr>
<td>0.0019</td>
<td>0.9 in.</td>
<td>-0.02422</td>
<td>37.7 Kips ft</td>
<td>2.15e-03 1/in.</td>
</tr>
<tr>
<td>0.0020</td>
<td>0.9 in.</td>
<td>-0.02567</td>
<td>38.4 Kips ft</td>
<td>2.28e-03 1/in.</td>
</tr>
<tr>
<td>0.0025</td>
<td>0.9 in.</td>
<td>-0.03266</td>
<td>41.2 Kips ft</td>
<td>2.90e-03 1/in.</td>
</tr>
<tr>
<td>0.0030</td>
<td>0.9 in.</td>
<td>-0.03927</td>
<td>43.2 Kips ft</td>
<td>3.48e-03 1/in.</td>
</tr>
<tr>
<td>0.0035</td>
<td>0.9 in.</td>
<td>-0.04548</td>
<td>44.6 Kips ft</td>
<td>4.03e-03 1/in.</td>
</tr>
<tr>
<td>0.0040</td>
<td>0.9 in.</td>
<td>-0.05141</td>
<td>45.6 Kips ft</td>
<td>4.56e-03 1/in.</td>
</tr>
<tr>
<td>0.0045</td>
<td>0.9 in.</td>
<td>-0.05708</td>
<td>46.4 Kips ft</td>
<td>5.07e-03 1/in.</td>
</tr>
</tbody>
</table>

Ultimate concrete strain was exceeded.
Composite Lav-up for WSU Split Column Tests

Out-of-Phase, Longitudinal Loading, Double Bending
("Chopsticks")

- 0.0268" thick
- 0.0134" thick

Plastic Hinge Region
Standard 12" wide GA090 fabric (0.0134" thick) was applied wet to the resin-coated concrete surfaces of the four plastic hinge regions; another layer of GA090 was then applied wet to the top and bottom 6" of each PH.

- 0.0134" thick
- 0.0268" thick

Crack Propagation Region
Standard 12" wide GA180 fabric (0.0266" thick) was applied dry to the adhesive-coated concrete after the adhesive set up. Resin was applied to wet out the fabric, another layer of GA180 (pre-cut to 6" wide) was then applied wet to the top 6" to increase the thickness to 0.052".

Transverse Loading, Single Bending
("Grouted")

2" split was grouted 24" above the footing.

- 0.0199" thick
- 0.0266" + 0.0134" + 0.0400" thick

Plastic Hinge Region
Standard 12" wide GA130 fabric (0.0199" thick) and GA180 fabric (0.0266" thick) were applied dry to the adhesive-coated concrete of the secondary and primary plastic hinge regions respectively; after the adhesive set up, resin was applied to wet out both sections. A standard 12" wide layer of GA090 fabric (0.0134" thick) was then applied wet to the primary plastic hinge region to increase the thickness to 0.040".

Loading Condition 1

Loading Condition 2
# Quantities of products used to construct the lay-ups

<table>
<thead>
<tr>
<th>Carbon Fiber Fabric</th>
<th>&quot;Chopsticks&quot;</th>
<th>&quot;Grouted&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Hinges (4 places)</td>
<td>Crack</td>
</tr>
<tr>
<td>GA090  8.9 oz/yd² .0134&quot; thick</td>
<td>(1) 47¼&quot; x 12&quot;</td>
<td>–</td>
</tr>
<tr>
<td>GA130 13.2 oz/yd² .0199&quot; thick</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>GA180 17.7 oz/yd² .0266&quot; thick</td>
<td>–</td>
<td>(1) 65&quot; x 12&quot;</td>
</tr>
</tbody>
</table>

(All fabrics are 12K. All come 12" wide standard. 6" wide sections had to be cut. Thickness assumes 50% Vf.)

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>&quot;Chopsticks&quot;</th>
<th>&quot;Grouted&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bondtite R37</td>
<td>Plastic Hinges (4 places)</td>
<td>Crack</td>
</tr>
<tr>
<td></td>
<td>not used</td>
<td>1.210 lbs.</td>
</tr>
<tr>
<td>Bondtite H37</td>
<td>not used</td>
<td>.403 lbs.</td>
</tr>
</tbody>
</table>

(3:1 mix ratio; coverage rate: .33 lbs./ft²)

<table>
<thead>
<tr>
<th>Resin</th>
<th>&quot;Chopsticks&quot;</th>
<th>&quot;Grouted&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Hinges (4 places)</td>
<td>Crack</td>
</tr>
<tr>
<td>826</td>
<td>(2) 2.274 lbs.</td>
<td>2.178 lbs.</td>
</tr>
<tr>
<td>3379</td>
<td>(2) 1.170 lbs.</td>
<td>1.122 lbs.</td>
</tr>
</tbody>
</table>

Note: 2 kits of the size shown were made to wet out the "Chopsticks" PH regions - 1 kit was made for the two "bottom" PHs and another kit was made for the two "top" PHs. Since the "top" PHs were the last to be applied and wetted out, an additional .379 lbs. of 826 and .195 lbs. of 3379 were added to the kit in order to have some resin left over to wet out any "dry" or "drier looking" areas on either test column. As it turned out, this extra resin wasn't really needed, but was applied anyway.