TEST RESULTS OF STRUCTURAL FUSES MOUNTED ON CANTILEVERED COLUMNS FOR SEISMIC RETROFITTING

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Problem: Soft Story Retrofits

Moment frames inhibited by:

- Congested utilities
- Low ceilings
- Narrow garage doors
- Foundations adjacent to door openings
Difficulties with Steel Frames

• Installation—cannot use overhead cranes
• Special inspection—disproportionate fees for smaller projects
• Fire hazard from welding
• Cost to relocate utilities can exceed cost of frame
Problems with Cantilevered Columns

- Low $R$ factor
- Bad reputation
- Difficult to replace

$R = 1.25$  Ceiling height = 14 ft.  Design reaction = 60 kips (ASD)  Needed W21x111 column
Standard Cantilevered Column

Lateral force

Yielding occurs at point of maximum moment

BMD
Structural Fuse Concept

- Increased ductility
- More predictable failure
- Opportunity for increased $R$ factor
- Higher $R$ allows smaller column and footing (and all other components)
Structural Fuse System

• Flange of top Tee connects to floor framing above
• Stem of Tee slides on column web extension
• Oar-shaped structural fuse yields between center and bottom bolts
• Slots at top and bottom bolts allow free movement
• Ideally, structural fuses can be easily replaced after an earthquake
Structural Fuse Principle

- Max moment at upper support (Point T)
- Linear decrease to zero moment at bottom pin
- For yielding to occur uniformly along fuse, need $S$ to vary linearly between Points T and B
Current Status

• Testing performed at certified laboratory for 12-kip and 2.25-kip ASD capacity assemblies

• Product evaluation report in progress from third-party evaluation agency following ASTM D7989

• $R = 6.5$, $\Omega_0 = 3$, $C_d = 4$

“Earthquake-Resisting Column” (ERC)

“A” size ERC, 8-ft. tall

- ASD load = 2,500 lbs
- W8x21 column tested; will be produced with W8x35
- 3,750 lbs/ft width (compare to plywood shear wall, 870lb/ft max.)
- No welding needed
- Fits into tighter spaces than alternative systems
Before Testing

Top connection to framing above

Bottom of structural fuse
Connection to Wood Framing
Test Results for “A” Size ERC

Peak load before Shear Bolt failure = 5,800 pounds

Intended ASD Load = 2,250 lbs.
(Eval. Report allows 2,500 lbs.)
Shear Bolt Sacrificed

Fig. 6 Cyclic Lateral Load-Drift Hysteresis for SOFT STORY BRACE-Earthquake Resisting Column/Fuse Plate: CF w/fp-6

Shear Bolt fails, load drops to 3,000
Hysteresis Plot; Story Drift > 5%

Fig. 6 Cyclic Lateral Load-Drift Hysteresis for SOFT STORY BRACE-
Earthquake Resisting Column/Fuse Plate: CF w/ fp-6

Two cycles at drift > 4%
Hysteresis Continues; Story Drift > 9%

Fig. 6 Cyclic Lateral Load-Drift Hysteresis for SOFT Steel Earthquake Resisting Column/Fuse Plate: CFD

Two cycles at drift > 6%

Two cycles at drift > 8%
Hysteresis

Fig. 6 Cyclic Lateral Load-Drift Hysteresis for SOFT STORY BRACE-Earthquake Resisting Column/Fuse Plate: CF w/fp-6

AVERAGE PEAK LOAD = 12372 lbs; AVERAGE DRIFT @ PEAK LOAD = 10

Load at Panel Top, (lbs.)

Four cycles at drift > 7%
Two more cycles at drift > 8%
“Failure”
“A” Size Fuse, Near Ultimate Deflection (Specimen A2)
Fatigue Fracture at Column Base
“D” Size Structural Fuse
(12,000 lb. Intended Capacity, W12x65 Column, 1-inch thick fuses both sides) Before and After Testing (Specimen D3)
Phase 2 Tests—June, 2018
PEER Testing Facility (UC Berkeley
Richmond Field Station)
Tests D5, D6, and D7

CUREE Loading Protocol
200-kip Actuator
 +/- 11-inch stroke
Force-Displacement, Test D7

Initial Loading

Inches

kips
Shear Pins Sacrificed

Inches

kips
Story Drift Reaches 2.5% (2.85 in.)
Story Drift Reaches 4%
Story Drift Reaches 6%
Story Drift Reaches 7.5%
Failure; Drift in Excess of 8%
### Summary of Test Results

<table>
<thead>
<tr>
<th>Specimen #</th>
<th>Design Capacity (ASD), kips</th>
<th>Maximum Load, kips</th>
<th>Maximum Deflection, inches</th>
<th>Maximum Story Drift</th>
<th>Number of cycles in drift ranges of:</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4—6%</td>
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<tr>
<td>A1</td>
<td>2.25</td>
<td>11.6</td>
<td>11.0</td>
<td>9.6%</td>
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<td>10.7</td>
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<td>D1</td>
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<td>36.7</td>
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</tr>
<tr>
<td>D3</td>
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</tbody>
</table>

1. Maximum deflection was limited by capacity of the laboratory equipment. Specimens did not fail at the listed deflections; the values are given to show that the system can tolerate drifts matching or exceeding the drift of comparable systems.
2. Story drift is approximate. The base connection for the columns was intended to simulate the worst-case conditions of a column flange treatment used to embed steel columns into reinforced concrete grade beams. A non-structural concrete floor slab would usually be present and would provide some level of fixity above the base of the tested configuration.
3. Test halted due to sudden slippage in loading mechanism
4. Specimen with approximately 60,000 to 80,000 micro-inch edge roughness
Effect of Surface Roughness on Fuses

Per AISC 341 & FEMA, cuts to create reduced beam section profiles must be finished to maximum of 500 micro-inch roughness (1/2000\textsuperscript{th} of an inch). Was greater roughness tested and found to fail prematurely?

Specimen in test D7 had one edge with surface roughness of approximately 50,000 micro-inches (100 times rougher than AISC allows for RBS). Two locations had roughness of approximately 1/16\textsuperscript{th} inch or more (60,000 to 80,000 micro-inches), one of which initiated failure.

Is the 500 micro-inch AISC requirement founded on testing and failure of rougher surfaces, or was that just the starting point of the tests and made a requirement, when much rougher surfaces may perform satisfactorily?
Questions?
References

1. Matteson, T. “Yield Link” Connection Providing Ductility and Hysteretic Energy Dissipation with Easily Replaceable Elements to Reduce Earthquake Damage and Recovery Time, *SEAOC 2016 Convention Proceedings*, Structural Engineers Association of California, Sacramento, CA, 2016. <Note: “Yield Link” is trademarked by the Simpson Strong-Tie Company, which was unknown to the Author until October 2016. No association should be made.>