Designing a TL3 Bridge Guardrail System Mounted to Steel Fascia Beams for use on Ohio’s Local System

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Bridges on local road systems are typically steel designs supporting various non-concrete deck types.

Because of the different deck options, the bridge rail is often mounted directly to the steel fascia beams of the bridge superstructure.

These fascia mounted designs have not been crash tested and are not eligible for federal-aid reimbursement on federal aid projects.
Advantages of the Fascia Mount Design

- Provide the option for building steel bridges with lower-cost bridge decks,
- While avoiding having to build excess deck width to accommodate anchorage for a top-mounted bridge railing.
Objectives

The objectives of the project:

- Identify an existing bridge rail system that already has “federal-aid reimbursement eligibility”,
- Modify the mount design to accommodate attachment to steel facia beams,
- Seek “Eligibility” from FHWA for use of the modified mount on the existing bridge rail system.
Illinois 2–Tube BR Crash Performance

- Full-scale crash tested at Texas Transportation Institute (TTI) in 1993 under test conditions corresponding to the AASHTO 1989 GSBR PL2.

**Test 7069-35:**
- 1,800-lb small car
- 59.9 mph
- 20.1 degrees

**Test 7069-36:**
- 5,400-lb pickup
- 60.4 mph
- 20.4 degrees

**Test 7069-37:**
- 18,000-lb single unit truck
- 51.4 mph
- 14.7 degrees
The bridge rail design successfully passed all three tests thus meeting the requirements for AASHTO GSBR PL2.

The system was later given a NCHRP Report 350 TL4 equivalency rating.
Research Approach

- Determine the force-displacement response of the original post-mount,

- Develop a new mount design(s) with equivalent stiffness for attachment to steel bridge fascia beams,

- Perform physical testing to:
  - Verify that the dynamic force-deflection response of the new post-mount is equivalent to the original
  - Validate finite element models of the post-mounts,

- Use FEA to evaluate the response of the bridge superstructure under impact loading to assess damages to the bridge components,

- Perform FEA simulation of NCHRP Report 350 Test 4-12 (SUT test) and compare to the test results on the original design to verify that the modifications do not adversely affect crash performance.
The force–deflection response of the baseline post–and–mount was determined through physical testing.

Impact Conditions
- Pendulum Mass = 2,372 lb
- Impact velocity = 15 mph
- Impact Location = 28.75” above top mounting bolt (i.e., 25.75” above deck surface)

Testing performed at the Turner-Fairbank Highway Research Center’s Federal Outdoor Impact Laboratory (TFHRC - FOIL)
The measured permanent deflection of the post in the full-scale crash test was approximately 2.5”.

* The dynamic deflection was approximated (from the crash test video) to be between 4 - 6 inches.
Basic Fascia Mount Design Concept

Structural Tube
HSS 14 x 6 x ¼” or 12 x 6 x ¼”

3/4” Thick Mounting Plates

15” (max) Tube Length Varies
Post–Stiffener Design

- The increased length of the modified post results in an increased moment.
- To compensate for this, stiffening plates were positioned between the post’s flanges near the mount location and welded in place.
- Two different stiffeners designs were selected, which were optimized for application with the 14” tall tube mount:
In both cases, a 1” thick plate is welded onto the front-side of the post to enable connection to the mount structure.

**Design A**
- Number of stiffeners: 2
- Stiffener size: 8.5” x 5.4” x 0.25”
- Area: 91.8 in²
- Weld Length: 34 in

**Design B**
- Number of Stiffeners: 4
- Stiffener size: 2.88” x 5.4” x 0.25”
- Area = 62.2 in²
- Weld Length = 45 in
Strength Assessment of New Mount Design and Model Validation

- Pendulum tests were performed to:
  - Verify equivalent stiffness response (via comparison to original design).
  - Validate FEA models.
Impact Conditions
- Pendulum Mass = 2,372 lb
- Impact velocity = 15 mph
- Impact Point:
  - 35.75” above top-mounting bolt
  - or 25.75” above theoretical deck surface
Force–Deflection Results

14”x6”x0.25” Mounting Tube

12”x6”x0.25” Mounting Tube
FEA vs Test for Baseline Design

Time = 0.06 seconds
Time = 0.06 seconds
FEA vs Test for Design 4 (12A–15)

Time = 0.06 seconds
FEA vs Test for Design 5 (12B–15)

Time = 0.06 seconds

[Graph showing force vs. time and displacement vs. time for Test 15009K and FEA]
Summary of Tests

- **Strength Verification:**
  - The force–deflection response of the new post–mount was shown to be slightly stronger for the 14” tall mount and essentially equivalent for the 12” tall mount.

- **FE Model Validation:**
  - The force–deflection and force–time history results from the FE model replicated the tests data very well for the given impact conditions.
  - The model was therefore considered **valid** for use in evaluating crash performance of the bridge rail system.
Conduct a more critical evaluation of the resulting loads and stresses imposed on the superstructure.
The table below shows various mounting design options along with the required minimum sizes for critical bridge components as determined from preliminary scoping evaluations (not shown here).

These designs were further evaluated using detailed models of the bridge structure to verify that:

- The components of the bridge structure are sufficiently sized for carrying the resulting loads.

<table>
<thead>
<tr>
<th>Analysis Case</th>
<th>Fascia Beam</th>
<th>Diaphragm</th>
<th>WT Connector</th>
<th>Mounting Tube</th>
<th>Post-Stiffener Design</th>
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</thead>
<tbody>
<tr>
<td>A</td>
<td>W14x30</td>
<td>0.385</td>
<td>0.27</td>
<td>C12x25</td>
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<tr>
<td>D</td>
<td>W16x40</td>
<td>0.385</td>
<td>0.27</td>
<td>C12x20.7</td>
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Bridge FE Model
W16x40 Fascia Beam

ALL SUPERSTRUCTURE MATERIALS ARE MODELED AS ASTM A572
BRIDGE RAIL POST MATERIAL IS MODELED AS ASTM A572
HSS SECTION IS MODELED AS ASTM A500
BOLTS USED ON SUPERSTRUCTURE AND MOUNT ARE MODELED AS A325-1
CORRUGATED DECKING IS ASTM A1011 GR50 (MODELED AS AASHTO M180)

W16x40
WT6x32.5
C12x20.7
L 5”x3.5”x3/8”

Post-Stiffener A
HSS 14” x 6” x ¼” 15” long

Translational constraints on ends of stringers
Translational constraints on back L-brackets
Bridge FE Model
W16x40 Fascia Beam

ALL SUPERSTRUCTURE MATERIALS ARE MODELED AS ASTM A572
BRIDGE RAIL POST MATERIAL IS MODELED AS ASTM A572
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CORRUGATED DECKING IS ASTM A1011 GR50 (MODELED AS AASHTO M180)
Impact Conditions

- Load Applied at 25.75 inches above deck
- Rigid cylinder moving with initial velocity of 21 mph
- Sinusoidal loading with maximum displacement of 12 inches
Analysis Case D
W16x40 Fascia Beam
14” x 6” x ¼” Mounting Tube (15 inches long)
Post Stiffener A

Peak Load = 31 kips

LS-DYNA keyword deck by LS-PrePost
Time = 0

Baseline
W16x40_A_15

25.75”
Effective Plastic Strain Contours
Smallest Fascia Beam Case
(W14x30 Fascia Beam)

4” Post Deflection

Stringer Web = 0.01
WT flange = 0.02
WT web = 0.03
(Diaphragm = 0.04
(at edge of bolt hole)
(At edge of bolt holes)
Effective Plastic Strain Contours
Smallest Fascia Beam Case
(W14x30 Fascia Beam)

12” Post Deflection

Stringer Web = 0.04
WT flange = 0.04
WT web = 0.08
(At edge of bolt holes)

Diaphragm = 0.08
(at edge of bolt hole)

Maximum strain

Fringe Levels
Test 4–12 was simulated for the modified Illinois Two–Tube bridge rail

- to further check for potential issues related to vehicular impact and to ensure that all loading conditions for the post and mount have been addressed.

The detailed model of the bridge superstructure developed in the previous task was used for the analysis.
Impact conditions corresponded to specifications for NCHRP Report 350 Test 4–12 (i.e., 18,000-lb SUT impacting at 50 mph and 15 degrees).

Total length of bridge section was 31.2 feet.

The bottom flange at the ends of the bridge’s stringer beams were modeled with fixed constraints in the x, y, and z direction.

The rail was extended up- and downstream of the bridge to simulate continuation of rail into the transition sections.

The impact point was 2.3 feet downstream of Post 1 (i.e., consistent with full-scale test).
Crash Videos

Simulation of Test 4-12
Time = 0
Crash Videos
Analysis Video

Simulation of Test 4-12
Time = 0
Analysis Video
The maximum dynamic deflection was 4.3 inches and occurred when rear tandem axle impacted rail.

- Measured at top of railing
- Occurred at Post 2 (i.e., first post downstream of impact).

Maximum deflection not measured in full-scale test
The maximum permanent deflection was approximately 1 inch.

Maximum deflection for baseline system in full-scale test was 2.5 inches.
Conclusions

- The new post-mount designs provide equivalent or higher strength values compared to the original design.
  - The **12” tall post-mount design** may be used with W14x30 and larger W14x sections.
  - The **14” tall post-mount design** must be used with W16x40 and larger sections.

- Peak strength in all cases exceeded the LRFD minimum of 24 kips.

- Simulation of R350 Test 4–12:
  - Resulted in a similar response as the full-scale test of the baseline system.
  - The results indicated that damage to the bridge superstructure would be minimal. Repairs (if needed) would be limited to the diaphragm member(s) and possibly the T-connector to the diaphragm.
### Recommended Design Specification

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<td>0.27</td>
<td>C12x25</td>
<td>0.501</td>
</tr>
<tr>
<td>W16x40 and larger section sizes</td>
<td>0.505</td>
<td>0.305</td>
<td>C12x25</td>
<td>0.501</td>
</tr>
</tbody>
</table>

*Same for all cases to avoid confusion*
The system is considered by FHWA as “unmodified” since the bridge rail components above grade are unchanged and the post-response is “equivalent” to the original design.

The system is eligible for use on federal-aid reimbursement projects.

Although the baseline system is rated as TL4, the modified design should only be considered for TL3 applications.
Although it was not discussed here, another aspect of the design involves a “designed failure mechanism” to release the post from the mount under extreme impact cases.

Basic features of this design is that the release of the post occur at between 9–11 inches of deflection, i.e.:

- Should exceed TL4 strength
- But release prior to causing significant damage to the bridge structure.

Physical pendulum testing is being performed to verify performance.
Acknowledgements

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  - Scott Coleman – Logan County Engineer
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