Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

01-01-2007 to 04-01-2007

Retaining Pin Bolt with F-shape Steel Strap Tie-Down

Question

State: KS
Date: 01-10-2007

In regards to the tie down strap with the retainer bolt it seems overly conservative to use the retainer bolt since we got approval to remove the retainer for freestanding and bolted down options. It seems inappropriate/overly expensive to have to do a full scale crash test. Is the concern that the strap will pull the pin up? If I recall correctly when you did the freestanding crash test on the KS Type F3 TCSB (maybe with the new 350 update vehicle) that there was little, if any deformation of the connection pin. Was this do to the fact that with the addition of more loop bars and the resulting double shear on the connection pin?

Response

Date: 01-29-2007

We agree with you that in the free-standing and vertical bolted-down applications, the pin's retainer bolt is no longer needed to hold down the pin when the double shear loops are incorporated. In the strap toe-down concept, the strap will be loading the pin between the loops with the potential for pin deformations, even with the double shear loops present instead of the original single shear loops. Now, of course we do not have proof that this will occur nor will be a problem. However, we are cautiously guiding you on this matter until we have evidence to support a change in our opinion. If you asked others in our group, you may obtain a slightly different response to that which was provided by me.

F-shape Bolt Through Tie-Down Anchor Size

Question

State: WI
Date: 01-18-2007

Can we reduce the size of the bolt through tie-down anchor from its current 1.125" diameter size if we increase the grade of the anchor?

Response

Date: 01-18-2007
The short answer is yes. Ron and I calculated the shear, tensile, and bending capacities of the 1.125" diameter A307 anchor we used to test the system. We then compared those capacities with a 1" diameter Grade 5 (A325) rod. The 1" diameter Grade 5 rod possesses sufficient capacity and ductility to be substituted for the larger anchor if desired. So as long as the anchor grade is increased to a Grade 5 equivalent, the diameter can be reduced to 1"

We would suggest that you approach FHWA for approval on the use of the smaller anchor in order to alleviate any potential liability issues raised by switching to an anchor other than what was full-scale tested.
Attachment: http://mwrsf-qa.unl.edu/attachments/0a359886c2d79e6e05a9b80ce4e80a51.xls

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Coil Bolt Anchors for the F-shape Steel Strap Tie-Down

Question

State: MO
Date: 01-19-2007

Attached is the information regarding the bolts for the tie-down system on temporary concrete traffic barrier. APAC has requested to use coil bolts rather than the drop-in anchor system. A colleague said he spoke with you and asked that I send this to you for review.
Attachment: http://mwrsf-qa.unl.edu/attachments/b4225f4c45e9e7f818ed1aaca89d9dce.pdf

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Response

Date: 01-26-2007
We reviewed the anchor data from our testing and the Hilti published data. Our ¾-in. drop-in provided 18.9 k in tension and 25.6 k in shear. Any replacement anchor needs to provide equivalent capacity to be considered as an alternate. The Hilti coil anchor provides: 2.940 k tension allowable or 11.760 k tension ultimate (4X) and 5.540 k shear allowable and 22.160 k shear ultimate (4X). As such, we do not believe that this anchor option is acceptable in the absence of crash testing.

Report TRP-03-162-07 Traffic/Bicycle Bridge Railing

Question

State: IA
Date: 03-08-2007

Q1. The concrete barrier in the report is called a "standard single slope concrete barrier". Is this the barrier found in "Figure B7.65 Single Slope Concrete Bridge Rail (94, 36, 95, 96)" at the FHWA site, http://safety.fhwa.dot.gov/roadway_dept/docs/appendixb7g.pdf, ... or is it something else? I'm curious because there are several standards out there (California,Florida,Missouri, etc.), and they have different widths at the top of the barrier. This is important because at no point in the report is the setback dimension given for the steel railing from the top traffic face corner, and it isn't drawn up in either Figure 4 or Figure 37. Increasing the setback is mentioned as one strategy for improving the barrier's performance (Part 10 Recommendations).
Q2. See the photo labeled "0.060 sec" in Figure 21. Did your team make an effort to estimate the total intrusion of the test vehicle beyond/through the railing system? Did the steel railing actually prevent some intrusion that would have occurred with a plain SS barrier w/o attachments?
Q3. See photos "0.086 sec" in Figure 22, and "0.058 sec" and "0.096 sec" in Figure 46. Would you conclude that it was primarily the middle rail of Test MOBR-1 and the third rail up (from top of barrier) of Test MOBR-2 that were preventing vehicle ride-up on the barrier? Those rails exhibit significant upward deflection. It appears from the photos that the rails below those that I mentioned were actually deflected downward, and so probably did not apply a downward force to the vehicle. So, it seems that a steel rail with its bottom surface as low as 1136 MM (middle rail of first test) will potentially apply a downward force on the pickup hood, but a rail with its bottom surface as high as 1045 MM (2nd rail up on second test) may not have that potential.
This is important to us here in Iowa because we want to develop a new standard separation barrier that features a steel bicycle railing attachment up to the AASHTO minimum 1070 MM (3'-6) height for such an application (see attachment "SepBarrierStudy1.pdf"). I think our 1070 MM steel railing, with its bottom surface at 1020 MM high maximum, may keep the rail BELOW the critical zone where it would negatively affect vehicle trajectory if contacted. This is, of course, predicated on the assumption that the trail is not raised relative to roadway gutter elevation.
The separator we develop may use exclusively vertical-face concrete barrier, making less critical some of the issues of vehicle climb and potential rollover. We may want to retain an option to use a safety shape, however. We also don't build pedestrian/bicycle facilities on vehicular bridges that have design speeds over 45 mph, which we assume means we wouldn't see quite as significant zones of intrusion beyond the barrier face as your 62 mph tests showed. Thanks for your help with these questions. We're also wondering if you ever considered any testing of our 44-inch vertical face with setback (see "SetbackBarr1.pdf")? Any input on that design would be appreciated

Attachment: http://mwrsf-qa.unl.edu/attachments/a4b862c80497e8dc08add50172cc047e.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/c20d5500b0634560f2bc2ec937340b06.pdf

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Response

Date: 03-08-2007

See my comments inred.

Q1. The concrete barrier in the report is called a "standard single

slope concrete barrier". Is this the barrier found in "Figure B7.65

Single Slope Concrete Bridge Rail (94, 36, 95, 96)" at the FHWA site,

http://safety.fhwa.dot.gov/roadway_dept/docs/appendixb7g.pdf,

... or is it something else? I'm curious because there are several

standards out there (California, Florida, Missouri, etc.), and they have

different widths at the top of the barrier. This is important because at

no point in the report is the setback dimension given for the steel

railing from the top traffic face corner, and it isn't drawn up in

either Figure 4 or Figure 37. Increasing the setback is mentioned as one

strategy for improving the barrier's performance (Part 10

Recommendations).

**I must apologize for the oversight of omitting the concrete barrier details. The single slope concrete barrier used in this study was also used in two other MwRSF studies. For all of this work, we selected the use of the same single slope barrier that was used in the Washington State DOT bridge railing study performed by TTI and published in Transportation Research Record No. 1468. For this barrier, a 32-in. top barrier height was used, in conjunction with top and bottom base widths of 9.5 in. and 15.625 in., respectively. This geometry resulted in a front slope of 10.84 degrees off of the vertical.

**In terms of the relative distance between the front of the tubes and the top corner of the barrier, I will acquire a detail from the existing CAD that will show those dimensions. I am glad you caught this error now so that we can send out a PDF correction, but I am surprised that no one caught it when the draft was submitted to the States for review and comment. Even we missed this detail in our internal reviews!

Q2. See the photo labeled "0.060 sec" in Figure 21. Did your team make

an effort to estimate the total intrusion of the test vehicle

beyond/through the railing system? Did the steel railing actually

prevent some intrusion that would have occurred with a plain SS barrier

w/o attachments?

**Yes, we track the lateral extent of the vehicle over the barrier's top surface. With regard to question 2, TTI researchers did not determine working width in their testing years ago. In addition, a TTI simulated median barrier, using temporary barrier segments anchored down, was used to represent the bridge railing system. As such, direct comparisons are difficult to make. At this time, I am trying to obtain working width data from the CALTRANS testing programs on a single slope shape with a 9.1 degree slope off of vertical.

**As denoted in Figure 17, the working width was reported as 507 mm, as measured from the front toe of the barrier to the maximum lateral extent from the toe to either the vehicle components or the total barrier width. The total barrier width was 511 mm (20.125 in.). From film analysis, the front corner of the engine hood provided a working width of 507 mm, while the barrier geometry resulted in a working width of 511 mm. As such, we should have reported a 511 mm working width in lieu of the 507 mm value that was reported for the first test.

**As denoted in Figure 42, the working width was reported as 333 mm, as measured from the front toe of the barrier to the maximum lateral extent from the toe to either the vehicle components or the total barrier width. The total barrier width was 511 mm (20.125 in.). From film analysis, the front corner of the engine hood provided a working width of 333 mm, while the barrier geometry resulted in a working width of 511 mm. As such, we should have reported a 511 mm working width in lieu of the 333 mm value that was reported for the first test.

Q3. See photos "0.086 sec" in Figure 22, and "0.058 sec" and "0.096 sec" in Figure 46. Would you conclude that it was primarily the middle rail of Test MOBR-1 and the third rail up (from top of barrier) of Test MOBR-2 that were preventing vehicle ride-up on the barrier? Those rails exhibit significant upward deflection. It appears from the photos that the rails below those that I mentioned were actually deflected downward, and so probably did not apply a downward force to the vehicle. So, it seems that a steel rail with its bottom surface as low as 1136 MM (middle rail of first test) will potentially apply a downward force on the pickup hood, but a rail with its bottom surface as high as 1045 MM (2nd rail up on second test) may not have that potential.

**From my re-review of the crash testing videos, I have the same opinion as from years ago following the tests whereby the middle rail of the three rail system (test no. 1) and the third rail of the four rail system (test no. 2) provided the greatest vertical restraint for the right-front corner of the engine hood and vehicle. As such, other railing systems with these general geometries and offsets would provide similar behaviors when attached to sloped face concrete barriers. However, it is believed that similar railing systems attached to vertical shaped barriers would provide improved safety performance.

**Consequently, a review of the crash tests performed on a smooth, steel single slope barrier without upper railings resulted in similar propensities for vehicle roll but with slightly lower roll angles and prevention of rolling onto the vehicle's side. This truck behavior demonstrated that vertical restraint near the corner is believed to be similar to that observed with pickup truck test climb inhibited by a smooth, low-friction, steel front face. In the future, bicycle railing heights may be reduced to the pedestrian height of 42 in. (1,067 mm). Using this lower height for future system may alleviate some of the propensity to get under the rails and restrain the truck corner region.

This is important to us here in Iowa because we want to develop a new standard separation barrier that features a steel bicycle railing attachment up to the AASHTO minimum 1070 MM (3'-6) height for such an application (see attachment "SepBarrierStudy1.pdf"). I think our 1070 MM steel railing, with its bottom surface at 1020 MM high maximum, may keep the rail BELOW the critical zone where it would negatively affect vehicle trajectory if contacted. This is, of course, predicated on the assumption that the trail is not raised relative to roadway gutter elevation.

**Please note that the AASHTO bicycle railing height requirements may be changing from 54 in. to 42 in. in the near future. Second, we developed a different bicycle/pedestrian railing system for MnDOT several years ago and which uses two longitudinal rails with vertical spindles tightly spaced. This system was attached to a NJ shape bridge railing and tested to TL-4.

The separator we develop may use exclusively vertical-face concrete

barrier, making less critical some of the issues of vehicle climb and

potential rollover. We may want to retain an option to use a safety

shape, however. We also don't build pedestrian/bicycle facilities on

vehicular bridges that have design speeds over 45 mph, which we assume

means we wouldn't see quite as significant zones of intrusion beyond the

barrier face as your 62 mph tests showed.

Thanks for your help with these questions. We're also wondering if you

ever considered any testing of our 44-inch vertical face with setback

(see "SetbackBarr1.pdf")? Any input on that design would be appreciated

- thanks.

**MwRSF has previously developed a 42-in. high, TL-5 aesthetic, open concrete bridge railing, with other design variations, and one that incorporates a setback to account for head ejection of passengers out the side windows and against the face of taller, rigid concrete parapets. In addition, we have developed and are constructing another near-vertical (small slope), concrete median barrier with a similar setback near the upper region for the same reasons. Vertical barriers reduce vehicle climb and rollover propensity. However, we need to also begin to consider head ejection and contact with barrier hardware. Please email me if you have further questions and comments regarding the material contained herein. Also, let me know if you what details for any of the barrier systems noted above.

Response

Date: 03-13-2007

As denoted in my email last week, we prepared CAD details that depict the bicycle/pedestrian railing system attached to the actual single-slope concrete bridge railing system. Those new CAD details are attached.
As shown therein and as you requested, dimensions are given which indicate the offset from the upper front-face corner of the parapet and the face of the tubes and posts.

Attachment: http://mwrsf-qa.unl.edu/attachments/c5b1ceb6515f386f59e89a107bb51266.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/e4d93eeef0c5a1721a5b9d05b26130db.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/a7f9ce8e8db87b097022397dc4b4875c.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/c9665c81c1e8fc52540b4de5452bc58c.jpg

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MGS Long Span Anchorage

Question

State: IA
Date: 03-27-2007

I have a question regarding proper anchorage for the long span system. We are working on a project that will use the long span design along an interstate (traffic approaching from one end only). We will be using the FLEAT-MGS at the approach end, but I'm unsure how to lay out the trailing end anchorage. Can we start our anchorage immediately following the third CRT post after the culvert? And if we use our standard design (attached), it appears that we will end up with an extra post as part of the transition from MGS to our end anchor. Is this what we'll have to do, or do you have any other recommendations?

Attachment: http://mwrsf-qa.unl.edu/attachments/102cc1d0a8a8bfe71bea84b7b76db8e4.pdf

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Response

Date: 03-28-2007
I have some comments regarding you MGS Long Span questions. The question you raise is a good one. Because you have traffic from only one side, you won't need a terminal on the downstream end as you have suggested, but the length of the downstream end and the anchorage are critical to proper performance of the system.
The first question address should be is there a minimum length of guardrail that is required to ensure that the guardrail system adequately contains and redirects the impacting vehicles?
Most of the strong-post, W-beam guardrail systems have been crash tested using a system length of approximately 175 ft. For these lengths, it has been demonstrated that the barrier system will meet impact safety standards and allow the designer/researcher to gain knowledge on dynamic barrier performance. Whether or not the system's performance or deflection is adversely affected by an installed length shorter than the tested length is unknown. For an impact closer to the barrier system ends, dynamic barrier deflection may actually increase when impacted at the same 25-degree angle. However, the LON test on the terminal is currently conducted at 20 degrees instead of 25 degrees. In the Update to NCHRP Report No. 350, this LON test will become 25 degrees, potentially requiring modifications to be made to existing terminal anchors.
Flared systems or systems such as the long span system can actually further increase the loading of the barrier system and create higher anchor loads and affect the length of the system and the anchorage. Although it is likely that guardrail lengths shorter than 175-ft can redirect 2270P vehicles impacting at the TL-3 conditions, there is no crash test data to support or recommend the use of shorter lengths at this time.
In addition, trailing-end guardrail treatments are typically used to anchor the downstream end of strong-post, W-beam guardrail systems when vehicular impacts are not expected on the system end. These trailing-end designs consist of varying configurations using blunt ends or spoons, turned-down terminals, tension rods with concrete anchors, etc. In addition, these downstream anchorage devices are often located longitudinally near to the hazard that is shielded by the roadside barrier system. To date, no trailing-end (downstream) terminals have been evaluated according to the NCHRP Report No. 350 guidelines. There are concerns that vehicle impacts slightly upstream of the trailing-end terminals may induce rollover or severe snagging on the anchor system. Further, if the downstream anchor proves to release too quickly, vehicles impacting a short distance upstream of the terminal may be allowed to penetrate through the guardrail and strike the shielded hazard.
There exists a need to standardize the trailing-end, guardrail anchorage systems that are capable of meeting current impact safety standards.
There will be proposals to better address these issues at the upcoming Pooled Fund Meeting.

That said, we do not believe that you can install the downstream anchorage immediately following the third CRT post on the downstream end. Due to the concerns listed above, we would recommend that the downstream length of the installation including the end anchorage be no less than 62.5 ft beginning at the third CRT post. This length is based on the 175 ft system length that was tested. We believe that we may be able to reduce this distance based on the proposed pooled fund studies mentioned above.

I have a couple of additional notes regarding you end anchorage that was attached. The end anchorage shown is not appropriate for use with the MGS Long Span system due to the high anchor loads developed in this type of system. I have attached details of the anchorage used in our testing, and I would recommend that you use this anchorage on the downstream end. As for the end spacing issues raised by the MGS, you are free to add another post if you like. You could also hang the additional guardrail length off the end of the anchorage along with your end shoe if that is easier.

Attachment: http://mwrsf-qa.unl.edu/attachments/53a12669696abfaa315e972c71cbecbc.pdf

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MGS Long Span Length

Question

State: IA
Date: 03-27-2007

I have a question regarding proper anchorage for the long span system. We are working on a project that will use the long span design along an interstate (traffic approaching from one end only). We will be using the FLEAT-MGS at the approach end, but I'm unsure how to lay out the trailing end anchorage. Can we start our anchorage immediately following the third CRT post after the culvert? And if we use our standard design (attached), it appears that we will end up with an extra post as part of the transition from MGS to our end anchor. Is this what we'll have to do, or do you have any other recommendations?

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