Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

10-01-2009 to 12-31-2009

Missouri I-70 Situation

Question

State: MO
Date: 10-02-2009

We have an old bridge on I-70 which, at one point in its life, was rehabilitated and new safety barrier curbs added. In order to attach the bridge anchor section of guardrail without spanning the expansion joint, a stand-alone end post was cast and the rail attached. (See attached Figure 1)
We believe the post was insufficiently anchored because subsequent hits on the rail have caused it to move out significantly. Obviously, we're concerned about future pocketing and know we have to fix it. I don't want to repair this setup because I don't believe it functions as well as it could, and, as these photos show, there isn't a lot of substrate to attach to. (See Figures 2, 3, and 4)
Do you have any ideas on how we could properly mount the bridge anchor section in this situation? I was considering one left-out post (nearest the parapet) with a double nested, or otherwise stiffened rail to compensate
We have no intention of rebuilding the buttress because it is cost-prohibitive. We want to tear it out and find a hardware solution.

Attachment: http://mwrsf-qa.unl.edu/attachments/11daf700d111c2a8234f7e01c481dd32.gif

Attachment: http://mwrsf-qa.unl.edu/attachments/799a804cf12c42d395541ac889137a4a.gif

Attachment: http://mwrsf-qa.unl.edu/attachments/b056f4603afbbaa18e0002119bb145b2.gif

Attachment: http://mwrsf-qa.unl.edu/attachments/2f7deb30638b52981e0a0a25e2bba3b4.gif

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Response

Date: 10-02-2009
I am attaching a copy of the report that contained the approach guardrail transition system with a post missing and a special simulated post. You will need to download the report from the UNL dropbox using the directions contained below.
NDOR may be able to provide you with the CAD details for this transition system.
The file 'TTI_NCHRP_350_Testing_404211-F.pdf' (21.0 MB) is available for download at
http://dropbox.unl.edu/uploads/20091009/8bcd9dbc1ec19ad8/TTI_NCHRP_350_Testing_404211-F.pdf
for the next 7 days.
It will be removed after Friday, October 9, 2009.

TCB to Bridge Rail Connection

Question

State: IA
Date: 09-02-2009

As we spoke on the phone, I have a question regarding the roadside TCB to bridge connection utilizing thrie-beam panels. Do you have any rules of thumb regarding how the length of thrie-beam sections should be distributed onto the bridge and the first TCB section? For example, I would like to connect the thrie-beam end shoe to the bridge through existing bolt holes. This would place a significant majority of the thrie-beam length onto the first TCB section (see attached file). Do you see any issues with this approach?

Attachment: http://mwrsf-qa.unl.edu/attachments/f8267cbeceb5b854e171d91784dd9dbf.pdf

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Response

Date: 09-28-2009
The thrie beam connection shown in your schematic should be acceptable and I don't see any problems with the attachment as you have it shown.
The schematic you sent me is acceptable even with a gap as large as 12" between the final PCB and the bridge transition piece as long as the there is only one way traffic on the roadway. We would recommend gaps smaller than 12" if at all possible. If traffic is moving from the PCB's towards the bridge in your schematic, then the chance for snagging on the end of the bride is minimal, and the thrie beam sections should possess sufficient capacity to hold the joint between the bridge transition and the PCB together even with the larger gap.
If the traffic is moving the other direction, we would recommend filling the overlap area with concrete to reduce the snag potential. This is necessary and quite critical. It may be your best option to fill that area for now, but you may want to think about a redesigned transition section in the future to reduce the snag potential. In general, we would recommend not running two way traffic in this type of installation unless the snag issues can be sufficiently eliminated.

Response

Date: 09-29-2009
Upon further inspection, I realized that my scaling was off on the thrie-beam section I showed in my drawing. Having corrected that, and using the dimensions to the existing bolt holes in the bridge rail and a 12-inch gap between the bridge and the first PCB, I am now showing that the thrie-beam piece will extend entirely beyond the first PCB section. I assume this is an issue (maybe not?).

Minimizing the gap between the bridge and the first PCB would allow bolting the rail into both the first and second PCBs. Would this be acceptable? Are there any other spacings or modifications you might recommend, such as using a shorter thrie-beam section?

Attachment: http://mwrsf-qa.unl.edu/attachments/11279963928d24f4a50463b8ac08f448.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/42d5d3439f7c94018536a3ff5ab55434.pdf

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Response

Date: 10-01-2009
I don't like the idea of extending the thrie beam past the end barrier in either of those schematics. If we do so, the thrie beam is no longer tied to the end barrier. In addition, the 12" gap detail would create a snag hazard on the thrie beam end shoe.

I have attached details of the connection that Florida uses. They have many different variations of the thrie beam connection to permanent railing that they worked out with us when the transition was first developed. Will these details work for you?

Attachment: http://mwrsf-qa.unl.edu/attachments/0212002b5af7d05d30ace2ee731c81bf.pdf

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Response

Date: 10-01-2009
I've been using Florida's details as a guide. Unfortunately, they don't have anything that would solve this particular issue. Since my main goal is to try and utilize the existing bolt holes in the bridge end post, the only other option to make it work would be to use a thrie-beam section SHORTER than 12'-6". What are your thoughts on that?

Response

Date: 10-02-2009
We do not have a problem with using shorter thrie beam sections, such as 6'-3" sections, rather than the 12.5' sections we tested with. We went with the longer section because it was more common.

MGS Bridge Approach Transition

Question

State: NE
Date: 10-05-2009

We are attempting to switch to MGS/ 31" w-beam guardrail height.
Re. In light of the MGS 31" guardrail using 12" blockouts & changing post spacing away from the joints...
Should the bridge approach section use 12" Blockouts?
If it was tested with 8" blockouts & these should stay 8", where should the transition to 12" blockouts take place?
The original design tested did not include posts 10 & 12.
With the post spacing changing away from the joint, are Posts 10 & 12 required?
Or should 10 & 12 be placed to start the transition to stiffen the area?
What end treatments are available for 31" guardrail?

Attachment: http://mwrsf-qa.unl.edu/attachments/dbc99f3859a938aa21930bb9af439745.pdf

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Response

Date: 10-06-2009
We have successfully crash tested two steel post, thrie beam approach guardrail transition alternatives for the Midwest States Pooled Fund Program. These two transition designs were based on starting with a thrie beam system that was supported by W6x15 steel posts near the bridge end and spaced on 3 ft " 1.5 in. centers. See the attached for the two approach transitions successfully crash tested by MwRSF.

File "MWT FD R0" shows the design which utilized 3 W6x15's (7 ft long), 7 W6x12's (7 ft long), and 2 W6x9's (6 ft long). The transition length of this system " as defined in your drawing as the distance from the bridge rail to the DS end of standard MGS rail segment (additional post at this location) " is 37.5 ft.

File "MWT-SP R4" shows the design which utilized 3 W6x15's (7 ft long) and 7 W6x9's (6 ft long). The transition length of this system " as defined in your drawing as the distance from the bridge rail to the DS end of standard MGS rail segment (additional post at this location) " is 25 ft.

The system that you have sent (file "7400e00") have the same rail elements and post spacings / locations as MwRSF's first approach transition design (file "MWT FD R0"). However, the posts used in the 2 designs are very different. The transition you are working with was designed with long heavy posts near the bridge rail (designed to accommodate the circumstance of Post 1 being omitted) and did not consider the upstream end " or the approach transition. Thus, NDOT's transition utilizes three 8.5 ft long W6x25's followed by seven 7.5 ft long W6x20's. (In comparison were the MwRSF transition has three 7 ft long W6x15's followed by seven 7.5 ft long W6x12's.) Although we see no problem with the transition from 8.5 ft long W6x25's to 7.5 ft long W6x20's, major problems are likely at the beginning of the transition where the 7.5 ft W6x20's are adjacent to 6 ft W6x9's. This large difference in stiffness would likely create critical pocketing and snagging. Therefore, I recommend softening the front end of the approach transition.

In order to soften the approach transition but still keep the large W6x25's, the transition length will have to be increased. As stated previously, I like the transition between the W6x25's and the W6x20's, so nothing will change on the DS end of the transition. To attach either MwRSF designed approach transition, the W6x20's will be used to represent the W6x15's of the tested MwRSF system. Both W6x20's and W6x15's have 6 in. wide flanges and the embedment depth of the two posts is only changed by 0.5 ft. The additional stiffness provided to the W6x15's by rail cap used in the MwRSF system should make the total stiffness similar to the W6x20's with a slightly deeper embedment.

Therefore, using the relation described in the previous paragraph, the 2 approach transitions designed by MwRSF can be attached to the DS end of NDOR's current transition. These attachments will result in the transition being extended for an additional 6 ft " 3 in. (the length of the segment supported by W6x25's), as shown in the attached sketches titled "Nebraska Transition Design Adaptations". What do you think of these 2 designs?

One note: MwRSF has previously adapted the approach transition to Iowa's transition. Although the posts are very different, the result was the same in that the total length of the transition was extended 6 ft " 3 in. see attached PDF.

To answer your blockout question: I would keep the blockout depth at 12 in. for the W6x25's and W6x20's (I'm assuming that was the blockout depths of the system as tested). Thus, the DS end remains the same as tested and accepted. Additionally, the increased blockout depth of 4 in. (12 in. deep blockouts on the W6x20's are replacing the 8 in. deep blockouts on the W6x15's used in the MwRSF system) can be thought of as an extra safeguard to prevent snagging between the untested transition between the W6x20's and the approach guardrail transitions.

Attachment: http://mwrsf-qa.unl.edu/attachments/f47996081c3ee586c12fa3c3cf264d59.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/677004b749dd1fa9f48d380815206bad.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/48f3c6607281b96ca53acd16b5166c8e.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/af54387a4b5ae045b421360db63767da.pdf

Cable Anchor and Foundation Design

Question

State: KS
Date: 10-27-2009

We are gearing up for our first cable barrier installation in March of next year. We are trying to meet a minimum anchor and post foundation design. Do you have some structural information and assumptions (soils, loads, etc) for your designs?

Response

Date: 10-27-2009
We actually put a TRB paper together on these topics. "Development of Guidelines for Anchor Design for High-Tension Cable Guardrails". It will be presented at the 2010 winter meeting. We can send it to you upon request.

Hopefully it will address some of your problems.

We have not come up with recommendations for the post foundations yet. We have run some preliminary tests, but have not finished with the work. I will update you when we get more information.

Alternative Connections for the F-shape PCB

Question

State: WI
Date: 11-16-2009

Several states have been approached regarding the use of alternative connection designs with the F-shape PCB developed by the Midwest States Pooled Fund.

Response

Date: 11-16-2009
I am writing in response to some questions you raised regarding the use of alternative connection designs with the F-shape PCB developed by the Midwest States Pooled Fund. When looking at this issue, one has to consider the use of the barrier in both its free-standing and tie-down configurations. Currently, the F-shape PCB has been tested to NCHRP 350 and MASH in its free-standing configuration and has also been tested in several different tie-down configurations including a steel strap tie-down, and asphalt pin tie-down, and a concrete bolt tie-down. These tie-down systems have been applied to develop approach transitions between the F-shape PCB and rigid hazards on both the roadside and the median.
When we consider the use of alternative connections and the free-standing PCB design, it is likely that many different connections will perform acceptably. The main function of the connection in free-standing PCB's is to develop tension and moment at the joint during impact with the barrier. To a lesser degree, the joint needs to have the ability to resist torsional loads along the barrier axis and shear loads at the joint. When the free-standing barrier is impacted, the barrier segments deflect and are held together based on the tension in the connection. When the barriers have deflected sufficiently, the corners of the barrier segments come into contact creating a compressive load that is combined with the tensile load in the joint connections to create a moment. This is the main load on the free-standing barrier connection, and it is the main force providing the continuity of the PCB system. Because the critical loading of the joint is a tensile load, there are several connection designs that may work adequately for a given barrier section. I believe that the FHWA has generally approved alternative, free-standing barrier connections to be used on previously tested PCB designs as long as the reinforcement of the barrier is equal to or greater than the tested barrier and that the development of the connection reinforcement is sufficient. I think that this is a rational approach for free-standing barrier given the loading conditions. However, it should be noted that it is difficult to infer the performance of alternative barrier connections without more analysis and full-scale testing. I would recommend using a connection with shear, tensile, moment, and torsional capacities equal or greater to the connection you are replacing.

When consider the use of alternative barrier segment connections with the tie-down and approach transition applications, the loading of the barrier connection is significantly different, and the use of alternative connection designs with the F-shape PCB becomes more hazardous. Tie-down barriers have some form of constraint on the barrier. In the case of the tie-downs used in the F-shape PCB, the tie-downs consist of anchors that pass through the toes of the barrier and constrain lateral an longitudinal movement. This greatly affects the joint loading. When a tie-down barrier segment is impacted, the lateral and longitudinal translation of the barrier is limited by the anchors. Thus, the barrier edges do not generally contact and develop high tensile and moment loads at the joint. The majority of the tensile loads are developed by the tie-down. Because of the constraint and the lack of tension developed between the barrier segments, the behavior of the barrier system is such that the impacted barrier tends to deflect laterally and rotate back along its longitudinal axis. The barriers downstream of the impact have that motion transferred to them based on the shear and torsional loading of the barrier connection. Thus, as the first impacted barrier segment deflects laterally and rotates, the constrained, downstream barrier segments do not move until the shear and torsional loads are transferred through the joint. This creates a potential for vehicle snag on the end of the downstream barrier segment as it is exposed by the deflection and rotation of the impacted barrier unless the barrier connection can effectively transfer the shear and torsional loads to cause the downstream barrier to begin moving as well. This can be seen schematically in the attached Figure 1.

Based on the different behavior and loading of the barrier connections in the tie-down configuration, we would not recommend using alternative barrier connection designs unless the barrier connection was shown to provide greater shear and torsional capacity along the longitudinal axis of the barrier than the pin and loop connection used in the tested design. In addition, the connection would need to develop those loads relatively quickly (i.e., the barrier connection would have to develop loads before excessive rotation of the impacted barrier segment caused potential snag). Failure to meet these conditions could potentially result in vehicle snag on an exposed barrier end and corresponding excessive vehicle decelerations and instability. We have observed some degree of vehicle snag in the tie-down and approach transition testing conducted on the F-shape PCB. The difference in the loading of the connection between free-standing and tie-down barrier systems makes it very difficult to infer the performance of alternative connections in tie-down applications without full-scale crash testing. Thus, we would not recommend alternative barrier connections for the tie-down F-shape barrier or its associated transition designs without detailed analysis of the load capacity and behavior of the alternative joint or full-scale crash testing.

Hopefully this provides some insight on our concerns with the use of alternative connections with the F-shape PCB. Please contact me with any further questions or concerns.

Attachment: http://mwrsf-qa.unl.edu/attachments/b149cb408c14e0592e244cb477e811a1.jpg

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Trailing End Terminal

Question

Date: 11-18-2009

A representative from the IL Tollway called requesting guidance on the use of a trailing end terminal to protect the upstream end of the rigid concrete parapet.

Response

Date: 11-18-2009

Recently, you inquired about the implementation of the MGS with trailing end terminal for use in protecting the upstream end of a rigid concrete parapet. For your special situation, the concrete parapet was positioned approximately 4 ft behind the back side of the steel line posts. It should also be noted that you were referencing the ILDOT Standard 631011-06, Traffic Barrier Terminal, Type 2.

It was stated that there are special situations where the parapet is farther away from the traveled way than desired for the guardrail offset. In addition, there may be other circumstances that do not allow for the guardrail to be flared back toward the upstream end of the parapet and anchored to it. Therefore, preliminary guidance was requested for safely positioning the downstream region of the MGS and trailing end terminal to longitudinally overlap the upstream end of the parapet so that the MGS would shield blunt end impacts on the parapet end. Based on engineering judgment and in the absence of crash test results, we believe that a reasonable positioning would be to align post no. 5 with the upstream end of the parapet. This configuration would place approximately 22 ft of guardrail past the upstream end of the parapet.

It should be noted that future research should be directed toward determining the length of need where downstream guardrail systems with trailing end terminals are effective in safely containing and redirecting high-energy impacts with pickup trucks as well as small cars under the MASH safety guidelines.

Response

Date: 11-25-2009

Post #12 (the middle post in the asymmetrical transition piece) was omitted to avoid conflict with a drainage structure. See attachments. Is this allowed or what recommendation would you have for this detail?

Attachment: http://mwrsf-qa.unl.edu/attachments/ef5301db1315c399933de4a312cd04e2.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/ef5301db1315c399933de4a312cd04e2.JPG

Response

Date: 12-07-2009

Over the last year or so, I have fielded a few inquiries regarding the inability to place a post in a transition region due to a pipe culvert flowing out of a curb inlet. In past inquiries and depending on the transition used, I have suggested using a simulated post at such a location. I believe you are depicting the transition that used W6x12s by 90 in. long in the noted region. If we wanted to replace the midspan capacity, it would seem reasonable to use two W6x9s by 72 in. long posts " one on each side of the lateral pipe and behind the rail. Then, a WF beam would be placed between the two posts. This beam would be used to support a deep blockout and allow the two smaller and shorter posts to serve a one post where it could not be placed. Of course, this surrogate system has not been actually designed for your case but would be a concept that likely would work in this scenario. It would give back the resistive capacity at a location in the transition that may be prone to vehicle pocketing and snag. Let me know what you think.

Concrete Barrier Protrusion

Question

Date: 11-20-2009

What is the maximum protrusion along a concrete median barrier that would be acceptable? We have some sections of median barrier wall that no longer line up with the light pole foundations. There is now a snag point where a vehicle sliding along the wall would hit. For some reason, I thought that 1" was allowable. Does it matter if the edge is beveled?

Response

Date: 11-20-2009

Previously, MwRSF provided guidance noting that it was preferred to have the maximum lateral barrier misalignment limited to 1" or less. In addition, I believe that it would be preferred to utilize chamfered corners or edges to assist in mitigating vehicle snag on any exposed sharp edges. I have attached a pdf copy of a prior Pooled Fund Progress Report and Consulting Summary which contained this general guidance.

Attachment: http://mwrsf-qa.unl.edu/attachments/5f35bd97d958ec978079a07a84f23d82.pdf

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Pipe Runners (Safety End Treatment) For Skewed Culverts

Question

Date: 11-20-2009

I am looking for guidance on the use of pipe runners at skewed culverts. The current IL Tollway standards show pipe runners perpendicular to the roadway when the pipe is perpendicular to the roadway. Based on a departure angle of 25 degrees, a vehicle leaving the road would hit the pipe runners 25 degrees from perpendicular. For our skewed pipes and headwalls, the pipe runners are shown parallel to the pipe. Therefore for a culvert that is on a 30 degree skew (right hand forward) with pipe runners parallel to the pipe, that same vehicle departing at 25 degrees would now hit the pipe runners 55 degrees from perpendicular. This seems like too much of an angle. I was under the impression that the pipe runners should ideally be perpendicular to the path of the departing vehicle. Is there guidance for usage of pipe runners on skewed pipes?

Attachment: http://mwrsf-qa.unl.edu/attachments/f32140286d96602cef8061b70d212f0f.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/8181b63c3a368b10ed861678616fe527.PDF

Response

Date: 01-04-2010

I have discussed your prior emails on the noted subject with my colleagues. Following this discussion, I must report that we are unaware of any design guidance for placing the culvert grates or pipe runners at angles other than at 90 degrees with respect to the traveled way when used with transverse drainage structures.

In recent years, MwRSF successfully performed full-scale crash testing on a culvert safety grate system that was used to protect a large culvert opening on a 3:1 fill slope according to the Test Level 3 (TL-3) safety performance criteria found in NCHRP Report No. 350. This testing involved both small car sedan and full-size pickup truck vehicles leaving the roadway and slope break point at 20 and 25 degrees, respectively, and at a target departure speed of 100 kph. For this test installation, the center-to-center pipe spacing was 30 in. From this testing, MwRSF researchers observed that the test vehicles could safely traverse the culvert grate system at high speeds and when the approach path was not orthogonal to the pipe runners.

Under oblique angles with respect to the pipes, the clear opening distance between pipes is increased from that found when the vehicle path is perpendicular to the pipes. As the approach angle is further increased, there exists a point when the vehicle could no longer traverse the pipes but instead would snag within the pipe system or contact the concrete culvert edge. For vehicles launched off of a fill slope and subsequently landing on the grate system, there would be increased safety risks as the effective clear opening width were increased, such as for higher approach angles or under situations where the pipes were skewed away from traffic.

In your email, you noted that there are situations where the culvert system is skewed with respect to the roadway, thus causing the pipe runners to be installed in the same skewed orientation on the fill slope. As noted above, skewed pipes could increase the potential for vehicles to drop between the pipe runners, thus resulting in front end or wheel snag on the pipes or at the culvert edges. As mentioned previously, we are not aware of any research nor guidance pertaining to the placement of skewed pipe runners. In the absence of testing and/or computer simulation modeling, we offer the following opinions and recommendations based on our best engineering judgment and available information.

1. In general, the culvert grate system was designed with the pipe runners to be installed perpendicular to the roadway. As such, the grate system or pipes should be installed orthogonal to the roadway when placed in combination with skewed culvert systems.
2. When reverse direction impacts cannot occur, pipes may be skewed when the bottom end of the pipes are located upstream from the top end of the pipes.
3. If it is absolutely necessary to skew the bottom end of the pipe runners downstream from the top end of the pipes, then it is our opinion that skew angles ranging between 0 and 10 degrees can be safely accommodated. In addition, there exists the potential that skew angles between 10 and 20 degrees may possibly be accommodated, although the safety risks are believed to be greatly increased.
4. If skewed pipe runners are deemed necessary, it may be possible to utilize a narrower pipe spacing in order to decrease the clear opening width, thus reducing the potential for vehicle snag and instabilities. However, a reduced pipe spacing may also lead to an increased potential for debris to clog the drainage system.
5. Further research and testing is necessary to accurately determine the safety performance of culvert safety grates when installed with skewed pipe runners.

54â€• Concrete Barrier

Question

State: IA
Date: 11-30-2009

IADOT needs the following:

-54-in. tall, single-face, reinforced concrete parapet with foundation system for use in shielding bridge piers according to AASHTO 3.?.?

-reinforcement design for the interior and end locations of wall and foundation

-design based on WsDOT report and other more recent TL-5 barriers with reinforced footings/grade beams/slabs

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Response

Date: 11-30-2009
See the attached PDF file for a simplified drawing for a 54" tall, F-shape, TL-5 barrier. A few notes:

1. The only difference between the interior and ends sections for the barrier is the reduction of stirrup spacing from 9" to 6".
2. All longitudinal steel should be evenly spaced
3. 10 of 12 longitudinal steel bars in the interior footing can continue through the end section footing as well. The remaining two bars should be extended at least 2 feet into the end footing.
4. The end section shows the barrier positioned on the front of the footing, but it could be placed on the backside of the footing as well.
5. Other footing dimensions can be created to provide adequate strength, however the steel reinforcement may need to be reconfigured.

Attachment: http://mwrsf-qa.unl.edu/attachments/6b056c85523dc7c21fc7aa48f145d9f8.pdf

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Response

Date: 12-11-2014
here is a related 54" concrete barrier, near vertical shape with head ejection considerations
Attachment: http://mwrsf-qa.unl.edu/attachments/f9593b049273d48b9b9700edeeb06476.pdf

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TL-5 Concrete Barrier

Question

State: IL
Date: 11-30-2009

Kevin Riechers (Bureau of Bridges and Structures, Structural Standards Engineer) and I were talking about barrier requirements in the AASHTO LRFD Design Specifications at 3.6.5. The required barriers are either 42 inch tall or 54 inch tall meeting TL-5, depending on whether the barrier is located more than 10.0 feet from a pier or abutment or 10.0 feet or less.

I brought up some development and testing done at MwRSF for the Pooled Fund. These reports are TRP-03-149-04 (April 30, 2004) for development of TL-5 F-Shape barrier at 42 inch or 51 inch height; and TRP-03-194-07 (Dec 10, 2007) for the TL-5 vertical wall at 42 inch height.

These designs include footings developed to meet the objectives of the research.
In both cases the footings are 24 inch in depth and about the width of the base of the barrier.

It appears that either of the 42 inch tall designs could be adopted here, perhaps with footing design adjustments for any frost heave consideration or for our weaker soils.

From the Roadside Design Manual we note that the F-Shape may be extended to a taller configuration by "following the slope of the upper face if the barrier is thick enough or adequately reinforced at the top, or the extension may be vertical" (6.4.1.7). However, this would not be adopted directly from the 51 inch barrier as provided in the 2004 work, but would appear to require some modification to the barrier and footing design.

For the vertical face barrier there is no taller version yet, and the head ejection envelope does not allow for providing a taller barrier without apparent thickening and redesign of the overall structure. We speculate that crash testing might be needed for such a taller version.

Based on the AASHTO requirements in the LRFD Specs, the Roadside Design Guide, and the MwRSF work, we think the following might be possible:

> Adaptation of the footing for either of the 42 inch tall barriers to meet Illinois requirements (weaker soils, freeze thaw considerations).

Modification of the 51 inch F-shape to a 54 inch height and with modifications to the footing design to accommodate this, plus the Illinois footing conditions.

Do these appear to be realistic goals?

If so, we would like to confer with MwRSF before proceeding into this. Would you, or someone at MwRSF be available to help get us on the right track?

Response

Date: 02-02-2010
MwRSF prepared a similar detail for IaDOT.
See the attached PDF file for a simplified drawing for a 54" tall, F-shape,
TL-5 barrier. A few notes:

1. The only difference between the interior and ends sections for the barrier is the reduction of stirrup spacing from 9" to 6".
2. All longitudinal steel should be evenly spaced
3. 10 of 12 longitudinal steel bars in the interior footing can continue through the end section footing as well. The remaining two bars should be extended at least 2 feet into the end footing.
4. The end section shows the barrier positioned on the front of the footing, but it could be placed on the backside of the footing as well.
5. Other footing dimensions can be created to provide adequate strength, however the steel reinforcement may need to be reconfigured.

Attachment: http://mwrsf-qa.unl.edu/attachments/f666e1a81fdb5f712a5d337758b00554.pdf

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Low Profile Barrier End Reinforcement

Question
State: KS
Date: 12-01-2009

Quick question, we were detailing the low profile barrier and noticed about 5 extra H1 bars in the transition section with a total of 10 H1 bars. Can you explain the extra H1 bars in your table and the footnote a little more? The detail came from Iowa but originally came from MWRSF.

Attachment: http://mwrsf-qa.unl.edu/attachments/db9051f615f93a3023b7f0e4498069d0.png

Attachment: http://mwrsf-qa.unl.edu/attachments/a08b3c07ba29c1173d0557f79b4ac59f.png

Attachment: http://mwrsf-qa.unl.edu/attachments/ad10b13768aeda2746b5f5738ba94275.png

Attachment: http://mwrsf-qa.unl.edu/attachments/3cb6f1fdbe5d97f7742f945d6e42de10.png

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Response
Date: 12-01-2009
Based on the detail you sent you have more H1 bars than you need in the transition section.
It appears that the report shows only 5 H1 bars in the transition section, but the table states there are 10. We believe that the table is in error and that the CAD is correct.
Sorry for the confusion.

Low Tension Cable End Terminal Details

Question

State: NE
Date: 12-10-2009

We are drawing up the cable guardrail end treatment from reports TRP-03-155-05 & TRP-03-192-08 & need a little help with the details. Please see standard sheet attached.
For A (below) " will you agree to allow the exterior gussets be brought in, to 3/8" from the outside edges?
This is where we believe the 3/8" weld will work.
I assume all gusset plates are welded inside & out, or is a weld on the inside only.
For B (below) - will the welds be allowed for the 1 ¼" across the release lever plate connecting to the base plate?
The bracket plate or cable plate is the only 3/8" plate with the rest being ½". What is the reason for this?
Can it be changed to ½" so all plates in this assembly are the same?
Is there a reason for the holes in the triangle release lever plate gussets?
We found a bolt in these holes of report TRP-03-192-08 Figure A-2 Cable Terminal Detail, sheet 2 of 12.
Should there be a light weight tie down cable to keep the cable release lever from flying somewhere it shouldn't go?
This was used in the testing on the high tension cable.

Attachment: http://mwrsf-qa.unl.edu/attachments/e5b55505794da6c632e4e9a36b9ce02e.jpeg

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Response

Date: 12-15-2009
I have responded to your questions below in red.

For A (below) " will you agree to allow the exterior gussets be brought in, to 3/8" from the outside edges?

-We believe there is no problem with moving the gussets in 1/8" to allow for the 3/8" weld. This is an error in our CAD.

This is where we believe the 3/8" weld will work.

I assume all gusset plates are welded inside & out, or is a weld on the inside only.

-Gussets are to be welded inside and out as well as the ends (all the way around).

For B (below) - will the welds be allowed for the 1 ¼" across the release lever plate connecting to the base plate?

-This is another error in our CAD. The weld callout should read " 1" @ 4 ¼" C-C". This will center the weld correctly on the release lever plate.

The bracket plate or cable plate is the only 3/8" plate with the rest being ½". What is the reason for this?
Can it be changed to ½" so all plates in this assembly are the same?

-We have no problem with using ½" plate for the cable plate. It should not adversely affect the design.

Is there a reason for the holes in the triangle release lever plate gussets?

-The holes in the plate are for a bolt to connect a retention cable for the release lever. The rationale behind this is detailed in TRP-03-131-08.

-The design calls for a 5/8" Grade 5 Hex Bolt.

We found a bolt in these holes of report TRP-03-192-08 Figure A-2 Cable Terminal Detail, sheet 2 of 12.
Should there be a light weight tie down cable to keep the cable release lever from flying somewhere it shouldn't go?

This was used in the testing on the high tension cable.

- -Yes, there should be a retention cable. It should be ¼" diameter 7x19 aircraft cable. The cable should be 36" long and formed into a loop with 1" cable clips. See attached photos.
- -One other note. The CAD details show the use of 2 washers on the end fittings that connect to the cable plate. This is incorrect. We used a minimum of three washers on the end fittings. As an alternative, we designed 3"x2.125"x0.5" plate washers for the high tension design. These would be acceptable as well.

What is the length of need for the new cable end treatment system?

** Test no. CT-1 was conducted on a low-tension, three-cable barrier system according to test designation no. 3-35 of NCHRP Report No. 350. The test was successfully performed as a Length-of-Need (LON) evaluation at the target conditions of 100 kph and 20 deg. The test occurred at post no. 3 or approximately 15 ft from the upstream end of the barrier system. Based on the results obtained from the 2000P test (test no. CT-1), the length of need for the low-tension, three-cable barrier has been determined to be 15 ft.

Attachment: http://mwrsf-qa.unl.edu/attachments/60002ae7d26d7099d67b17a6a11b2e9c.JPG

Attachment: http://mwrsf-qa.unl.edu/attachments/59c21bb7613e33217f4d28a98fa602fa.JPG

Cable End Terminal Anchor Bracket

Question

State: NE
Date: 12-10-2009

We are drawing up the cable guardrail end treatment from reports TRP-03-155-05 & TRP-03-192-08 & need a little help with the details. Please review the E-mail below & see standard sheet attached.
For A (below) " will you agree to allow the exterior gussets be brought in, to 3/8" from the outside edges?
This is where we believe the 3/8" weld will work. I assume all gusset plates are welded inside & out, or is a weld on the inside only.

B-Shows the weld from the base plate to the release lever plate. Shown on the Front view, I drew out in blue, the weld that will work (11/16"), and in green the weld that won't work (3/16") according to what they specified. The upper 3 3/8" dimension shows the spacing between the welds, and the bottom 1 ¼" and the 3" dimensions are for the release lever plate. For B (below) - will the welds be allowed for the 1 ¼" across the release lever plate connecting to the base plate?

The bracket plate or cable plate is the only 3/8" plate with the rest being ½". What is the reason for this? Can it be changed to ½" so all plates in this assembly are the same?

Is there a reason for the holes in the triangle release lever plate gussets?
We found a bolt in these holes of report TRP-03-192-08 Figure A-2 Cable Terminal Detail, sheet 2 of 12.

Should there be a light weight tie down cable to keep the cable release lever from flying somewhere it shouldn't go? This was used in the testing on the high tension cable.

What is the length of need for the new cable end treatment system?

Attachment: http://mwrsf-qa.unl.edu/attachments/4ee8ef0ea0c3e4b95e060074dfdfaf5b.jpg

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Response

Date: 12-17-2009
I have responded to your questions below in red.

We are drawing up the cable guardrail end treatment from reports TRP-03-155-05 & TRP-03-192-08 & need a little help with the details. Please review the E-mail below & see standard sheet attached.

For A (below) " will you agree to allow the exterior gussets be brought in, to 3/8" from the outside edges?
This is where we believe the 3/8" weld will work.
I assume all gusset plates are welded inside & out, or is a weld on the inside only.

- -We believe there is no problem with moving the gussets in 1/8" to allow for the 3/8" weld. This is an error in our CAD.
- -Gussets are to be welded inside and out as well as the ends (all the way around)

B-Shows the weld from the base plate to the release lever plate. Shown on the Front view, I drew out in blue, the weld that will work (11/16"), and in green the weld that won't work (3/16") according to what they specified. The upper 3 3/8" dimension shows the spacing between the welds, and the bottom 1 ¼" and the 3" dimensions are for the release lever plate. For B (below) - will the welds be allowed for the 1 ¼" across the release lever plate connecting to the base plate?

- -This is another error in our CAD. The weld callout should read " 1" @ 4 ¼" C-C". This will center the weld correctly on the release lever plate.
- -I have reviewed the original CAD for the bracket welds and you CAD which appears to be based on it. The weld labeling is confusing and not entirely consistent with what was built. As such, I have drawn a revised detail that should help clear the confusion. Note that some of the dimensions have changed as well. I put in the space for the 3/8" welds that was lacking as well.

The bracket plate or cable plate is the only 3/8" plate with the rest being ½". What is the reason for this?
Can it be changed to ½" so all plates in this assembly are the same?

- -We have no problem with using ½" plate for the cable plate. It should not adversely affect the design.

Is there a reason for the holes in the triangle release lever plate gussets?
We found a bolt in these holes of report TRP-03-192-08 Figure A-2 Cable Terminal Detail, sheet 2 of 12.

- -The holes in the plate are for a bolt to connect a retention cable for the release lever. The rationale behind this is detailed in TRP-03-131-08.
- -The design calls for a 5/8" Grade 5 Hex Bolt.

Should there be a light weight tie down cable to keep the cable release lever from flying somewhere it shouldn't go? This was used in the testing on the high tension cable.

- -Yes, there should be a retention cable. It should be ¼" diameter 7x19 aircraft cable. The cable should be 36" long and formed into a loop with 1" cable clips.

What is the length of need for the new cable end treatment system?

- -Test no. CT-1 was conducted on a low-tension, three-cable barrier system according to test designation no. 3-35 of NCHRP Report No. 350. The test was successfully performed as a Length-of-Need (LON) evaluation at the target conditions of 100 kph and 20 deg. The test occurred at post no. 3 or approximately 15 ft from the upstream end of the barrier system. Based on the results obtained from the 2000P test (test no. CT-1), the length of need for the low-tension, three-cable barrier has been determined to be 15 ft.

Attachment: http://mwrsf-qa.unl.edu/attachments/97aa9872af45bb73838c0e01c9b121a1.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/994cbe1db12a471b7c13f08def013c02.pdf

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MGS with 1/4-Post Spacing to Shield Hazard and Placement Guidelines

Question

Date: 12-18-2009

I have another MGS question for you. We have a situation where a sign truss foundation is located 13" from the back of guardrail posts. MGS with standard post spacing was installed which would deflect into the concrete foundation. The minimum deflection distance we are using for the 1/4-post spacing installation is 14" measured from the back of post to the near edge of the hazard. The designer is proposing to add posts and to stiffen the rail by doubling up on the rail element thereby further reducing the deflection.
Do you have any comments/objections to this approach seeing that we only need to reduce the deflection by 1"?
Do you have any data on the anticipated deflection distance for this proposed installation?
See attached drawing of the proposed modifications. Note that the rail is gradually stiffened by using 1/2-post spacing and then the 1/4-post spacing. On the departure end of the 1/4-post spacing is 1/2-post spacing needed before getting back to standard spacing?

Attachment: http://mwrsf-qa.unl.edu/attachments/c219f254ce2b506a645508839593 43a0.pdf

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Response

Date: 01-06-2010
I have attached a pdf copy of our prior TRB journal paper for the Midwest Guardrail System (MGS). Within the paper, guardrail placement guidelines are provided for treating hazards. These guidelines pertain to the distance between the front face of the rail to the front face of the hazard. As noted, the minimum recommended distances are 1.25, 1.12, and 0.90 m (49, 44, and 35 in.) for the standard, half-, and quarter-post spacing designs, respectively. It should be noted that the width of the steel-post MGS is 0.54 m (21.25 in.). Using this information, one would need to consider using a clear distance of approximately 0.35 m (13¾ in.) between the back of the steel posts and the front face of the vertical hazard when utilizing the quarter-post spacing system.
Below, you noted that the available clear distance between the back of the steel posts and the front face of the rigid, vertical concrete foundation is 0.33 m (13 in.). Based on the guidelines noted above, your noted solution to use the MGS quarter-post spacing design and noted placement would result in 20 mm (¾ in.) less clear distance than that recommended in the paper (as noted above). However, I do not have significant concerns with using the basic ¼-post spacing MGS configuration to shield the noted hazard nor deem it necessary to use nested W-beam rail to cover the 20 mm (¾ in.) deficit in provided clear distance.
Finally, the use of the ½-post spacing in advance of the ¼-post spacing MGS seems reasonable and an appropriate transition in stiffness. In addition, I see no reason to utilize a ½-post spacing MGS on the departure end if reverse-direction impacts cannot occur.
Attachment: http://mwrsf-qa.unl.edu/attachments/3db3e4edc03cdf94152d1084e7415673.pdf

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Longer Posts for Guardrail Adjacent to Steep Slopes

Question

State: IL
Date: 12-29-2009

We have information about how to build line posts for MGS when the steep slope begins at the back of post. However, a designer in one of our Districts is dealing with a location where the slope continues in a similar manner all the way to a bridge. Are there any recommendations about adding to the length of posts for the transition from MGS to a bridge parapet?
Our particular transition to the parapet, given adequate support behind the posts is given in our Standard 630001.
http://www.dot.il.gov/desenv/hwystds/rev211/Web%20PDFs/

Response

Date: 01-04-2010
At this time, we do not have any available design information for modifying the length of the transition posts when located on a steep slope near the bridge end. However, MwRSF does have a research project with the Wisconsin DOT to provide recommendations for addressing various transition issues. One of the noted issues will be to provide design guidance for situations when steep slopes are found behind the posts. It is expected that this effort will be initiated within 2 months.
