Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

10-01-2016 to 12-31-2016

USH 14 Weak Post W-Beam Guardrail System

Question
State: WI
Date: 10-05-2016

I have two potential options.

1. You can drill out the broken anchor and install a Hilti HISN threaded insert with an equivalent bolt. We do this with the SAFER Barrier when we damage an anchor.

2. You could replace the post attachment bracket with a modified one that has a longer lower-left steel tab. The tab could be lengthened by 2" such that a new anchor could be installed without interfering with the original.

Let me know if that helps

Attachment: https://mwrsf-qa.unl.edu/attachments/36494c607124296db05db9bb36b8cdef.jpg

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Response
Date: 10-05-2016

We are putting in the side
mounted weak post MGS system. Contractor snapped off one of the bolts on
the bottom connection.   Got any ideas how to “fix" this.

Attachment: https://mwrsf-qa.unl.edu/attachments/36494c607124296db05db9bb36b8cdef.jpg

W-beam / Bridge with Parapet 9" Tall Safety Curb

Question

State: UT
Date: 10-06-2016

We have an existing bridge that currently has w-beam attached to the side of the parapet. The parapet on the traffic side is 9 inches tall and 1'-6" wide.

Below is a diagram and photograph of this location. I have also attached additional photos of the site location.

Short of replacing the structure, we are looking for the best that can do design understanding that this is a situation were there are no designs that would meet crash testing requirements due to the 9 inch tall curb.

Do you have any suggestions for a design at this location?

Attachment: https://mwrsf-qa.unl.edu/attachments/39b8e39fc5609871df9974f20a8f757c.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/c478a57d02805dbf5fec5b6ceae82b18.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/8e2bc3eb60eb194621e80cefea14b31f.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/c8d9b8d4efe2b530a7aaa1f3b1ca1a45.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/6e5f2319b24e53aeffd3ce3a2ed03b39.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/3fd4b54e5f7f6a946164788f8bc7cfc5.pdf

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Response

Date: 10-06-2016

After reviewing your information, there are a few thoughts that come to mind.

First, there may be potential to remove the W-beam rail and post system. Then, a reinforced concrete parapet could be attached to the top surface of the concrete curb. Concrete buttresses may be needed at the ends with appropriate tapers when adapting approach guardrail transitions. Additional reinforcement would be needed to anchor the buttresses as only one yield line would be available to develop sufficient redirective capacity for impacting vehicles.

Many years ago, we crash tested a retrofit parapet placed on a curb that formerly supported an aluminum beam and post bridge rail. This work was completed for the Iowa DOT. Several links are provided below to guide access to the pertinent reports on our website.

http://mwrsf.unl.edu/reportresult.php?reportId=203&search-textbox=iowa%20bridge

http://mwrsf.unl.edu/reportresult.php?reportId=80&search-textbox=iowa%20bridge

http://mwrsf.unl.edu/reportresult.php?reportId=200&search-textbox=iowa

In recent years, we provided guidance to NDOR regarding the attachment of retrofit parapets to weak deck that have old bridge rails cut off. I am asking that Scott Rosenbaugh forward that information to you.

Second, there may be some potential to attach crashworthy beam and post systems to your curb as long as it has sufficient structural capacity. There is a recent TTI testing effort on the RC 131 bridge rail. The link is provided below.

https://tti.tamu.edu/publications/catalog/record/?id=38552

https://tti.tamu.edu/publications/catalog/record/?id=37169

Steep slopes within transitions

Question

State: IA
Date: 10-17-2016

We are looking at a situation where we have steep slopes in the neighborhood of 2:1 immediately adjacent to a culvert headwall, to which we are attaching thrie-beam across the face. Recent testing (TRP-03-234-10) has focused on longer posts at the 2:1 breakover with the caveat it still needs at least 4 ft of 2:1 beyond the breakover before steeper slopes or a hazard are introduced (Q/A #610).
However, I'm unsure if this longer post guidance can also be used in our barrier transition sections (BA-201)
or if it is strictly limited to standard w-beam situations. I found Q/A #525 from before the 2:1 breakover study that recommended at least 2 ft of 10:1 behind transition posts until further notice but presume there has since been discussion about transition posts in the development of the 2:1 studies. Would you please share the current approach to steeper slopes within transition sections?

Response

Date: 10-17-2016

We addressed something similar for Ohio. Take a look at that response and let me know if that answers your questions.

http://mwrsf-qa.unl.edu/view.php?id=1044

Thanks

Utah Guardrail Bridge Rail Need

Question

State: UT
Date: 10-19-2016

I am in hopes you have seen something similar to this in the past. This is a very old structure. See photos below. I made a site visit and was hoping to be able to remove the railing and railroad tie material and use a 25 foot guardrail span. From the first CRT post location on each side of the structure would require a 28 foot span. The speed limit is 45 mph. Would the additional 3 feet span be an issue?

Other than that I do not see an option because of the width of the structure. With very narrow shoulder and 10 feet lane width there is no room to move the face of barrier towards the edge of travel lane. Mounting hardware to the deck is not an option due to how it is constructed. The steel beam is located just over 2 feet in from edge of deck.

Span it with high tension cable?

Basically a new structure needs to be constructed but money is not available to do so at this time so a best that we can do method is what we are stuck with at this time.

Attachment: https://mwrsf-qa.unl.edu/attachments/0cea72c0421af23d74af09287e61c295.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/d7c16679aca2bdd37e9cd0e1c5a3938f.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/9559d592731b8f2fe2cd5d8d57489c33.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/a859d6e82bbdaaad5484b0eba2545182.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/bd0b3c30cb19673f398caf34bae8c7a3.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/1552a596d24d52e30a944c47ddd8c82f.jpg

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Response

Date: 10-20-2016

What is under the timber curb rail and how thick? Do you have any views from back side or in ravine?

I assume that there is an RC deck with asphalt wearing surface. You note that the timber elements do not rest on either but instead only the corrugated lower formwork?

Response

Date: 10-21-2016
Unfortunately, I do not have additional photos. The timber curb is sitting directly on top of the Corrugated medal sheeting. If needed I could have additional photos taken. I have also put in a request for structure drawings with our Structures folks.

Yes the timber sits on the corrugated lower form work. The outer beam runs just under the white shoulder line.

Response

Date: 10-22-2016

A few years ago, Wyoming had a rural timber bridge that needed a retrofit rail as the bridge was not slated for replacement for a long time. We helped guide them on a few options. I think that they worked out a way to install weak-post MGS bridge rail by attaching sockets to structural tube that replaced a timber curb rail. The tubes supported sockets and attached to outer timber beam. The tubes attached across the road using long HS rods through tubes to bear against thick asphalt roadway. Yes, cuts were made into road to allow for placement of rods. Your challenge is that the outer stringer beam in inset from outer deck. If the weak-post, MGS bridge rail is used, one must find a way to anchor the tube sockets that support posts. The tube sockets must be effectively rigid. The MGS bridge rail would be best option if you can figure out how to anchor sockets.

You mentioned the Long-Span MGS as did a few of my colleagues. That system met TL-3 at 25 ft but had one of two tests fail at 31.25 ft. We all agree that it would be better at TL-2 but have no proof for a 28-ft span for success.

Thus, these two potential directions may be your options for now using best engineering judgement. Let me know if you have any further questions.

Attachment: https://mwrsf-qa.unl.edu/attachments/be145074afa76ecd366192870077184a.pdf

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Response

Date: 10-23-2016

Would there be any negative effects if we stiffen the long span with nested rail on the MGS system?

Response

Date: 10-24-2016

We used nesting in the NCHRP 350 version of the 25-ft long long-span system. The nested rail was 100 ft long and hade some guidelines on position and overall system length. We provided this guidance on our Pooled Fund Consulting website. In terms of a 28-ft long-span system at TL-2, it may be beneficial and conservative to use the 100-ft of rail nesting with MGS. Of course, there is no certainty with this deviation as we did not test with MGS on a 28-ft span.

31" Guardrail Transition to G4(1S)

Question

State: VA
Date: 10-31-2016

As I indicated earlier, we are being directed to accelerate our transition to MASH w-beam guardrail. One of the efforts will be to remove and replace substandard terminals such as the BCT, turn-downs, etc. Our plan calls for this to start in November.

We plan on installing a MASH terminal and then transition back down to the 27 ¾" w-beam for longer runs that still meet the height requirements. We developed our transition detail similar to Washington DOT and Caltrans designs. Can you look over the detail and let me know if you have any concerns?

Thanks for your continued assistance.

Attachment: https://mwrsf-qa.unl.edu/attachments/f833bcdca569d308c1abac0bc9360d00.pdf

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Response

Date: 11-01-2016

I recently provided recommendations in a draft report (out soon hopefully) that were related to this issue. I just finished evaluated of a transition from guardrail to PCBs. The transition uses MGS, but would need to transition back to G4(1S) in many cases.

I have attached that section of the draft report. It appears that your detail matches up with the second option we had listed.

You may want to consider allowing for some additional MGS between the terminal and the transition to the original guardrail. We have seen that many of the terminals have stopping distances longer than their stated length. For example, some designs have stopping distances for test 3-31 of 50', but the terminal length is listed as 37.5'. Conservatively, we have recommended not transitioning the height within known stopping distances. It may not be an issue, but it may be simpler just to account for it.

Take a look at the section I sent and let me know if you need anything else.
Attachment: https://mwrsf-qa.unl.edu/attachments/3036e74af45215901ff8dd9b6d5214cb.pdf

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Curbs with Transitions and Guardrail

Question

State: IA
Date: 08-04-2016

Somewhat of a follow up question to our conversation regarding curb heights (bottom of Q/A 1079):

We have situations where an existing 6 inch standard curb is adjacent to a lane receiving a 3 inch overlay, ultimately making it a 3 inch curb. We have typically treated curb heights less than 4 inches as a non-issue regardless of their offset to face of rail, but your statement that headwalls (and I'm expanding to curbs here) greater than 2 inches affect vehicle stability brings that into question. What aspect of height is causing the issue; the height itself or the vertical face? To express it in another way, which of the following scenarios should be considered an issue?
1. Two inch or less curb in front of guardrail without a height transition or a transition of less than three feet
2. Two inch or less curb in front of guardrail with a height transition of at least three feet
3. Between two and four inch curb front of guardrail without a height transition or a transition of less than three feet
4. Between two and four inch curb front of guardrail with a height transition of at least three feet
5. Two inch or less obstacle behind face of guardrail but within working width, with a vertical face and without a height transition or a transition of less than three feet
6. Two inch or less obstacle behind face of guardrail but within working width, with a vertical face and with a height transition at least three feet
7. Between two and four inch obstacle behind face of guardrail but within working width, with a vertical face and without a height transition or a transition of less than three feet
8. Between two and four inch obstacle behind face of guardrail but within working width, with a vertical face and with a height transition of at least three feet
9. Four inch obstacle or less behind face of guardrail but within working width, with a height transition of at least three feet and with a sloped face (essentially slope the headwall face and ends to mimic a four inch sloped curb – doesn't exist but just a thought of how we could make them non-hazardous)
10. Other combinations we should consider as issues…

I'm wondering if your response in Q/A 1079 necessitates a change in how we deal with curbs, such that our guidance (subject to change based on above response) might read:
1. Curbs less than two inches in height
a. Can be ignored and do not need a transition
2. Curbs between two inches and a typical four inch sloped curb
a. Require a transition of at least three feet and that transition cannot occur within:
i. Fifty feet upstream of the end terminal,
ii. Any point within the end terminal, or
iii. Within the nested w-beam and asymmetrical transition piece of a barrier transition section
3. Standard six inch curbs
a. Require a transition of at least three feet down to a four inch sloped curb and that transition cannot occur within:
i. Fifty feet upstream of the end terminal,
ii. Any point within the end terminal, or
iii. Within the nested w-beam and asymmetrical transition piece of a barrier transition section

b. Cannot exist within:
i. Fifty feet upstream of the end terminal,
ii. Any point within the end terminal (Iowa flares all end terminals), or
iii. Within the nested w-beam and asymmetrical transition piece of a barrier transition section
iv. Any location where the offset from gutter line to face of rail is more than six inches
4. Headwalls less than two inches in height
a. Can be ignored and do not need a transition
5. Headwalls greater than two inches
a. Cannot exist within guardrail working width

Thanks for working through my interpretation of your intent, and as always, thank you for your assistance.

Response

Date: 11-02-2016

Let me try to answer things in a general sense and then we can work down to specifics.

First, the response in Q/A 1079 was specific to the long span system and not intended to be applied to all guardrail in general. The long span system has significantly larger deflections than typical guardrail and the vehicle tends to traverse the headwall and extend over the drop-off. Thus, the height of the head wall become more critical for this application. Additionally, we don't have data for curbs that first impact guardrail and then impact a curb offset that distance behind the rail. Typically curb studies have focused on placement of the curb at or slightly offset behind the face of the rail or with the curb offset at larger offsets in front of the rail. As such , we limited the headwall height for the long span systems to provide a conservative approach.

With respect to the broader aspects of curb and guardrail installations, we believe that the Roadside Design Guide still likely provides the most appropriate guidance based on our current knowledge. Essentially, the RDG states that for high speed facilities guardrail should not be offset from curbs unless crash testing has shown that it is acceptable. If guardrail offset from curbs is needed, it recommends a 1.5" laydown curb that would have minimal effect on the barrier performance. If does note that W-beam can be used with 6" tall curbs if the rail is installed flush with the curb up to 8m km/h and gives guidance for other sloped curbs for at higher speeds. It also notes exception to these guidelines for crash tested systems like the MGS which was tested with a 6" curb and a 6" offset behind the curb. As noted above, the 2" headwall height was specific to the long span and would not supersede the RDG guidance with respect to guardrail systems in general.

You note transitioning or tapering of the curbs. Specifically when to do it with respect to AGTs and end terminals.

Currently there is no set guidance with respect to curbs and end terminals other than it is not generally recommended due to lack of knowledge and testing of the combination. We currently have a limited study funded by WisDOT to investigate this issue, but it is not yet complete. A previous study by CALTRANS with respect to curbs and inertial sand barrels was conducted in the 60's and it does provide some guidance w/r/t placement of curbs in front of the barrels, but it was done using sedans and my not be as relevant as it originally was.

In terms, of transitions, our recent testing of the MGS stiffness transition with curb (http://mwrsf.unl.edu/researchhub/files/Report295/TRP-03-291-14.pdf) found that a 4" tall sloped curb could be used in the region of the AGT. In order to ensure the safety performance of the MGS stiffness, the 4-in. tall curb should be placed through the entire length of the stiffness transition. Thus, the curb should be extended a minimum of 37.5 ft from the bridge parapet before either being terminated or transitioning to a 6-in. high AASHTO Type B curb. Additionally, it was recommended to utilize a minimum length of 3 ft for any curb shape transitions or terminations (e.g. transitioning from 4-in. curb to no curb).

Hopefully this gets us started down the path of answering your questions. Take a look at what is above and see if it addresses the situations you have below. Then if we need to discuss some specific items, we can go over those together.

Thanks

Expansion Gap Joint Width

Question

State: DE
Date: 08-17-2016

MwRSF was contacted through FHWA for the state of Deleware regarding our thoughts on the allowable gap lengths
for unshielded expansion joints in permanent concrete bridge rails.

Response

Date: 11-02-2016

There are two main concerns with expansion joints in concrete bridge rails. The first is the lack of continuity in the rail and the need for increased rail capacity adjacent to the opening to maintain the rail strength. For this discussion, I will assume that the gaps we are discussing are designed with appropriately reinforced rail end sections to address the lack of rail continuity. The second concern is vehicle snag on the downstream end of the gap as the vehicle is redirected. Snag in this area can potentially result in increased vehicle deceleration, vehicle damage, and instability.

Determination of a maximum unshielded gap can be looked at in several ways. First we can look at available test data. MwRSF tested a Nebraska open concrete bridge rail with a 4.5" gap under the PL-2 criteria. We evaluated this barrier at the gap with a 2,449 kg pickup truck at 61 mph and 20 degrees and a 8,165 kg SUT at 51.9 mph and 16.8 degrees. Both of these tests were successful, but snag was evident in both tests. Additionally, the pickup truck test was conducted at a lower angle than NCHRP 350 or MASH TL-3 and TL-4. More recently, TTI tested the Texas T224 bridge rail to MASH TL-4. As part of that evaluation, they successfully tested the 10000S vehicle across a 2" wide expansion gap. The passenger vehicle tests were not conducted across the expansion gap.

http://mwrsf.unl.edu/researchhub/files/Report262/TRP-03-51-95.pdf

I don't have the TTI report. It may not yet be published.

Similarly, temporary barrier designs in free-standing and anchored configurations have had gaps as large as 4" that have been successfully traversed by vehicle in TL-3 testing. This would suggest that the potential for larger gaps may exist. For example, the Midwest States F-shape PCB was tested to NCHRP 350 TL-3 anchored to a bridge deck with a 4" barrier gap. This test did have some snag across the PCB joint, but the vehicle was safely redirected. The snag may have even been exaggerated in this type of system as compared to a rigid bridge rail as the upstream barrier could translate more laterally prior the vehicle traversing the gap, thus exposing the face of the adjacent barrier at the downstream end of the gap even more.

http://mwrsf.unl.edu/researchhub/files/Report133/TRP-03-180-06.pdf

http://mwrsf.unl.edu/researchhub/files/Report223/TRP-03-134-03.pdf

http://mwrsf.unl.edu/researchhub/files/Report54/TRP-03-208-10.pdf

http://mwrsf.unl.edu/researchhub/files/Report151/TRP-03-173-06.pdf

When we have considered this in the past, we have looked at the potential vehicle overlap on the downstream gap edge. We know from previous research that vertical asperities can cause problems with barrier performance. NCHRP 554 concluded that vertical asperities of ¼" or less were recommended to maintain vehicle stability and safe redirection. Previous testing conducted at MwRSF on a portable steel barrier for IaDOT noted similar concerns when a 3/8" thick vertical plate on the face of the portable barrier was sufficient to snag a pickup truck rim and cause the vehicle to roll. However, these two examples may not be completely analogous as NCHRP 554 dealt with aesthetic bridge rail designs and the portable steel barrier had a plate that extended from the face of the barrier

http://mwrsf.unl.edu/researchhub/files/Report245/TRP-03-120-03.pdf

http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_554.pdf

In previous discussions with the state DOT's, the issue of the allowable level of lateral misalignment of the barriers has come up. With regards to permanent concrete barrier, we recommended keeping the lateral offset or alignment offset minimized to eliminate snag. Variations of 1" or less would be preferred. We also recommended that the edges of the gap be chamfered to reduce the severity of any vehicle snag on the gap. The 1" offset was larger than the ¼" or 3/8" noted above based on the fact that these gaps were specific to concrete barrier overlaps where the concrete would be expected to fracture and give when snagged. If we use a similar rational and apply it to the expansion joint problem, we can use a gap length “L" and a 25 degree impact angle or intrusion angle of the impacting vehicle to estimate snag. This would make the snag or overlap on the downstream end of the gap approximately equal to L*sin(25). This results in the following snag for various gap lengths.

Gap Length “L" (in.)	Estimated Snag (in.)
1	0.42
2	0.85
3	1.27

4	1.67

This would seem to suggest that a 2" gap will limit snag to less than 1".

As you can see, there may not be a perfectly defined answer. Previous anchored PCB testing and the PL-2 open concrete rail tests suggest that 4" gaps may be permissible. However, a more conservative approach may be to limit the gaps to 2". In both cases, we recommend chamfering the edges to limit the snag severity.

Thanks

MGS stiffness transition substituation of W6x15 posts

Question

State: IN
Date: 11-07-2016

INDOT is in the process of transitioning to the MGS guardrail. Currently we are working on the stiffness transition. We see in report TRP-03-210-10 the last three downstream post (16-18) are W6x15 at 37.5" spacing. It appears that a couple other states are using six W6x9 posts at 18.75" spacing to substitute for the three W6x15 posts. Is this noted as acceptable in FAQ or report? Thank you

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Response

Date: 11-10-2016
The W-to-thrie beam stiffness transition described within report TRP-03-210-10 was developed as the upstream end of the stiffness transition. Specifically, it addressed the transition from standard W-beam to stiffened thrie beam. The downstream end of the approach transition (characterized in that report as the stiffened thrie beam region) can consist of any of the crashworthy, thrie beam transitions. Guidance on how to connect the upstream stiffness transition to other downstream transitions is discussed in chapter 14 of the report (it shows both 37.5" and 18.75" post spacing systems. Details for connecting the upstream end to one of the more popular 18.75" post spacing systems can be seen on page 167, Figure 96.

If you need further guidance, you can send me the details for the transition you are utilizing or feel free to call.

Underground obstruction near posts

Question

State: WI
Date: 11-09-2016

I
was asked how close can a underground obstruction be to a beam guard post. I search your web site and incorporated some of that guidance into the drawing above.

I
did not know how close a parallel underground obstruction could be from a post. So I tried to use some of the guidance from the mow strip Research to develop this drawing.

What
do you think?

Attachment: https://mwrsf-qa.unl.edu/attachments/1b22c9d157755e5fea81910a177b2f9b.pdf

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Response

Date: 11-09-2016

I think your minimum offsets are justifiable. I have a few comments:

TTI conducted full-scale crash testing on guardrail systems with 18"x18" leave-outs. The posts were offset 3" from the front of the leave-out. Thus, there was 9" of leave-out located between the back of the post and the back of the leave-out. You can reduce your minimum “X" and “Y" distance to reflect 3" and 9" gaps in front and behind the posts, respectively.

We have recommended that MGS posts have a minimum of 6" gap between the side of the post and a rigid hazard.. Accordingly, you could reduce the minimum “Z" distances to 6".

If a hazard is located closer to a post than these limits, you should consider omitting a post. Omitting a single post within an MGS installation has been recently proven crashworthy. Guidelines for often a post can be omitted were included in the project report, TRP-03-326-16.

MGS Median Barrier and Transition

Question

State: MN
Date: 11-11-2016

Do you have information on a design variation of this AGT of a median barrier connection?

Or I guess a more basic question would be; do we have a median barrier version of the MGS?

Response

Date: 11-11-2016

Here is the information that we have to-date for MGS in medians. Years ago, we did a median transition to a Missouri Single-Slope Median Buttress under 350. Using new transition technologies, we could likely do the same on a median concrete buttress.

http://mwrsf-qa.unl.edu/view.php?id=1094

https://tti.tamu.edu/publications/catalog/record/?id=39225

http://mwrsf-qa.unl.edu/view.php?id=779

http://mwrsf-qa.unl.edu/view.php?id=713

http://mwrsf-qa.unl.edu/view.php?id=423

http://mwrsf-qa.unl.edu/view.php?id=714

http://mwrsf-qa.unl.edu/view.php?id=459

http://mwrsf-qa.unl.edu/view.php?id=1108

Attachment: https://mwrsf-qa.unl.edu/attachments/5b472e9fb6d5d481ee0ddceb8b95a21a.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/a27f53c53a4d8192498f88fc4a6195e3.pdf

Web-mounted barrier rail

Question
State: IA
Date: 11-21-2016

We had the
attached image come in from one of our county engineers who is inquiring whether we know of any barrier rail system that the attached may have been modeled after. As you can see, it appears to be welded, though he says he has seen it bolted as well, to the web of an I-beam. Are you, or anyone there at MwRSF, aware of any such installation? I've never seen this before so I'm hesitant to make any sort of design recommendations regarding its applicability.

Attachment: https://mwrsf-qa.unl.edu/attachments/93034eb7bae770a4498bf279cfaa9d72.jpg

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Response
Date: 11-21-2016

We have seen this type of attachment previously on county bridges and other rural installations.

Based on our internal knowledge, we do not believe it has been crash tested. In terms of its applicability, we do not believe that this is a very robust connection. It may have a chance to meet TL-1, but we would have concerns about using it for anything higher than that.

Thanks

MGS Working Width

Question

State: VA
Date: 11-30-2016

We are in the process of finalizing our MASH MGS standard details and would like a quick review for the content.

One of the remaining details is the minimum distance either behind the post or from the face of rail to a hazard. Do you have a detail of the distance and how it is measured for the MASH testing or a description similar to page 225 of the NCHRP 350 report?

Thanks

Response

Date: 02-05-2017

The following text is located in MASH with respect to the deflections and working width. They may help serve as basic definitions for you.

“Test article deflections—Report the permanent and dynamic deflections of the test article plus the working width during impact. These measurements normally apply to longitudinal barriers, terminals, crash cushions, and TMAs. Permanent deflection is the residual lateral displacement of the test article remaining after the impact. Dynamic deflection is the maximum lateral displacement of the test article on the traffic side that occurs during the impact. The working width is the maximum dynamic lateral position of any major part of the system or vehicle. These measurements are all relative to the pre-impact traffic face of the test article. For the working width, the height of the maximum working width should also be documented and reported."

"**working width**—The distance between the traffic face of the test article before the impact and the maximum lateral position of any major part of the system or vehicle after the impact"

Working width would define the distance from the face of the rail to the hazard.

We have fielded questions regarding the variation of the MGS dynamic deflection and working width in the past for several states. Related to that, we have compiled charts of the working width and deflections for the standard system. These can be seen below. A chart with similar values is located in the Roadside Design Guide.

Table 1. Guardrail Testing under Test Designation 3-11.

Testing Agency	Test Number	Testing Criteria	Dynamic Deflection in. (mm)	Working Width in. (mm)
MwRSF	NPG-4	350	43.1 (1,094)	49.6 (1,260)
MwRSF	2214MG-1	MASH	57.0 (1,447)	57.4 (1,457)
MwRSF	2214MG-2	MASH	43.9 (1,114)	48.6 (1,234)
MwRSF	MGSMIN-1	MASH	42.2 (1,072)	48.8 (1,240)

MwRSF	MGSDF-1*	350	60.2 (1,529)	60.3 (1,530)
MwRSF	MGSPP-1*	350	37.6 (956)	48.6 (1,234)
MwRSF	MGSWP-1*	MASH	46.3 (1,176)	58.4 (1,483)
MwRSF	MGSSYP-1*	MASH	40.0 (1,016)	53.8 (1,367)
MwRSF	MGSRF-1*	MASH	55.8 (1,417)	57.4 (1,458)
MwRSF	MGSNB-1**	MASH	34.1 (867)	43.2 (1,097)
TTI	220570-2**	MASH	40.9 (1,040)	44.0 (1,119)
SwRI	GMS-1**	MASH	35.0 (890)	NA
TTI	400001-TGS1**	MASH	38.4 (975)	40.8 (1,036)
Holmes Solutions	057073112**	MASH	41.3 (1,050)	NA

*Guardrail with alternate posts and/or blockouts.

**Guardrail with no blockouts.

Table 2. Guardrail Testing under Test Designation 3-10.

Testing Agency	Test Number	Testing Criteria	Dynamic Deflection in. (mm)	Working Width in. (mm)

MwRSF	NPG-1	350	17.4 (441)	40.3 (1,022)
MwRSF	2214MG-3	MASH	35.9 (913)	48.3 (1,227)
MwRSF	MGSSYP-2*	MASH	22.2 (564)	39.7 (1,008)
MwRSF	MGSRF-3*	MASH	NA	38.4 (975)
MwRSF	MGSNB-2**	MASH	29.1 (740)	34.5 (877)

*Guardrail with alternate posts and/or blockouts.

**Guardrail with no blockouts.

The deflections and working widths listed for the MGS do fluctuate, even for the steel post version with standard 6'-3" post spacing. This fluctuation in the working widths is a reflection of several factors.

1. First, there has been a transition in the soil resistive forces that we use in our full-scale crash tests under MASH. Thus, the original crash testing of the MGS with the 2270P vehicle under 22-14 would have likely used a soil foundation that was less stiff than the soil recommendations that were eventually incorporated into MASH. Thus, there will be some variation of deflection and working width based on the change in the foundation conditions.
2. Second, the table presents tests with both the 2000P and 2270P vehicle types. Again the MGS was developed and tested during the transition between NCHRP 350 and MASH. Thus, the change in pickup truck vehicles represents an approximately 13.5% increase in kinetic energy. This change in impact conditions also accounts for some of the variation you are observing between the working widths and deflections in the full-scale testing.
3. Third, the table here and others in the Roadside Design Guide show deflections for a wide range of MGS systems, including wood and steel post versions as well as several special applications. Thus, the use of different post types, post spacing, slopes, flares, etc… affect the working width numbers.
4. Finally, full-scale crash tests are not an exact science. We have tried over the years to develop test procedures to make crash test results more consistent and repeatable. The current soil standard in MASH

is one part of that effort. However, even with these efforts, there is a certain degree of variation from test-to-test that is difficult to avoid. Thus, full-scale crash tests of two identical MGS systems may result in deflections that vary. This is simply difficult to avoid given all of the potential variation in materials, environmental conditions, soils, and other factors.

While it is clear that deflection and working width data taken from full-scale crash tests can vary for several reasons, we have still not answered the question regarding what values you need to consider for your installations. Our advice here would be to review the available data from the crash tests of most similar systems and error on the side of being conservative. For example, if you have an MGS system installed on a 2:1 slope, then we would recommend using the working width guidance from the full-scale crash test of the 2:1 slope. For standard, steel post installations, we may suggest considering a working width of 60 in. The 60-in. working width corresponds with the upper end of the values observed in the full-scale testing and also allows for some tolerance if the soil for your real world installations in not as stiff as the soil currently specified in MASH. For the wood post versions of the standard MGS system, we would recommend that you refer to the crash tests of the specific wood post system and use those working widths if they are increased over the 60-in. For the ½ post and ¼; post spacing versions of the system, we would recommend using the tested working widths listed in the RDG.

Let me know if this addresses your concerns and if you have further questions.

Concrete Protection Barrier: Useful Life

Question

State: NE
Date: 12-06-2016

Do you know of guidance for the Useful life of a Concrete Protection Barrier?

Is there guidance from any research completred?

Response

Date: 12-06-2016

Historically speaking, we have heard from various states that 7 years if often deemed a useful service life. Of course, this life varies from state to state. Others may use around 10 years. Daily and/or monthly handling of barrier segments often contributes to degraded service life. Barrier drops on concrete pavement, forklift prongs prying on edges, chains wrapped around barrier edges, etc. will result in spalling and cracking over time. Eliminating thin, sharp corners in a section may help to increase service life in PCBs.

If PCB service life could increase by 50% to 100%, would a DOT be willing to accept an alternative design if it had increased upfront acquisition cost but lower lifecycle cost? I suspect this would depend on how much more and what service life currently exists and what may be projected.

Response

Date: 12-07-2016

Thanks so much for your analysis. Are you aware of a state (or you for that matter) that has a description of physical characteristics that would make the PCB unacceptable from a crash test perspective.

It seems to me that the FHWA characterized the acceptability of the NE PCB as “marginal", are there physical deteriorations that would take this characterization from “marginal" to “unacceptable"?

I am looking for something the contractors and state can “see" in order to show why these PCBs are unacceptable.

Response
Date: 12-08-2016

Wisconsin has a document.

http://wisconsindot.gov/rdwy/cmm/cm-01-45.pdf#cm1-45.12>

Section 1-45.12.1 contains general temporary barrier guidance for construction staff.

1-45-12.5 addressed barrier quality

There is one other group that had prepared information. I will need to try to recall which group or state. Maybe Illinois?

Attachment: https://mwrsf-qa.unl.edu/attachments/6b6a750d5a5f411aeb56a93e670be625.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/45d15f5365a8278117a9bf2d84947772.pdf

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VDOT Standard MGS Details

Question

State: VA
Date: 12-07-2016

We are moving forward with the 31" MSG for our MASH compliant w-beam system; however, we wanted to get MwRSF to vet the details. Please review and let me know if you have any concerns / comments. Note that there are slight dimension differences in the trailing end terminal due to the transition from centered holes in the wood posts vs. offset holes in the steel post. In addition, we will be creating the special applications at a later date.

Thanks for your assistance.

Attachment: https://mwrsf-qa.unl.edu/attachments/dd034c00cb1e3a67d55a35b80c3972b7.pdf

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Response

Date: 12-08-2016

I cannot technically “vet" your plans, but we are happy to review them and provide comments.

I have attached an edited version of the pdf with some comments. Please let me know if you have any questions or concerns.

Attachment: https://mwrsf-qa.unl.edu/attachments/fac04157bbd2650575442bb4194c199a.pdf

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Response

Date: 12-09-2016

Bob:

Thanks for reviewing our standard details! I have a few comments on your evaluation:

Standard 506.__ (Tangent End Terminal)

I am not clear on your comment. The transition that we mention is a height transition only (31" to 27 ¾") not a stiffness transition. It will be used to go from a 31" terminal to 27 ¾" w-beam. We will update our bridge rail transition once that project is complete.

We will have a full 50' for the terminal and then beyond that, we will start our height transition.

Standard 506.__ (Transition from MGS 31" to GR-2 27 ¾" height)

This detail was based on the Washington DOT and Caltrans design. The height transition would probably begin at the off-post splice since there is a little flexibility there.

Thanks

Response

Date: 12-10-2016

Hi Chuck.

Replies below in red….

From: Patterson, Charles W., P.E. (VDOT) [mailto:Chuck.Patterson@vdot.virginia.gov]
Sent: Friday, December 09, 2016 9:33 AM
To: Robert Bielenberg <rbielenberg2@unl.edu>
Subject: RE: VDOT Standard Details

Bob:

Thanks for reviewing our standard details! I have a few comments on your evaluation:

Standard 506.__ (Tangent End Terminal)

I am not clear on your comment. The transition that we mention is a height transition only (31" to 27 ¾") not a stiffness transition. It will be used to go from a 31" terminal to 27 ¾" w-beam. We will update our bridge rail transition once that project is complete.

We will have a full 50' for the terminal and then beyond that, we will start our height transition.

If you are referring to the height transition, that should be fine. I read it as the start of attachment to an approach guardrail transition to a bridge.

Standard 506.__ (Transition from MGS 31" to GR-2 27 ¾" height)

This detail was based on the Washington DOT and Caltrans design. The height transition would probably begin at the off-post splice since there is a little flexibility there.

I understand. We have typically not recommended the splice in the height transition area, but we cannot say that it will not function that way. We liked the idea of keeping the splice relocation and post spacing outside of an area where the rail is tapering vertically. Basically limiting alteration of the system to a single variation of the standard system at a time. Certainly the detail you have ma work as well. This was just what we have recommended.

Response
Date: 12-11-2016

One more quick question.. We are still working on our pieces / parts details for the MGS. Can you address Mr. Cross' question below? This is in regards to a conflict at one or multiple posts.

Can you get an opinion on double 12" block out and a 4" with a 12" blockout.

Response
Date: 12-12-2016

We have looked at this issue previously with the state DOTs. I have placed a link to the response on the consulting site below. Let me know if you need anything else.

http://mwrsf-qa.unl.edu/view.php?id=267

In transitions, we have given slightly different guidance and have allowed more blockout depth. Let me know if you need that information as well.

Response
Date: 12-13-2016

Another small detail for clarification on the MGS… When we are showing the height of the MGS in conjunction with curb AND gutter, should we extend the pavement height over for to account for the typical 2" drop (see attached detail) at the flow line of the gutter pan? Otherwise the MGS will be 2" short in relation to the pavement.

It appears that most states take the dimension from the lowest point on the flow line but we want to get this right (the vehicle will not drop the 2" when impacting the curb / MGS
Attachment: https://mwrsf-qa.unl.edu/attachments/9207d379037a7d273447baf0d7dc4df0.pdf

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Response

Date: 12-15-2016

This is a good question. Our recommendation would be to install the MGS relative to the edge of pavement rather than the flowline of the curb. When we tested the MGS we placed the height of the barrier relative to the tow of the curb. However, for the drop shown, that would effectively lower the barrier height from 31" to 29" relative to the roadway. We believe that the height of the MGS is an important feature with respect to its ability to be placed adjacent to an offset curb. As you noted, for high angle impacts, we would not expect much vehicle drop for higher angle impacts over the 2' length shown. Thus, by setting the barrier height relative to the edge of the roadway, we maintain a barrier height similar to what has been tested.

If a low angle impact occurred which allowed the vehicle to drop 2", the effective rail height of 33" relative to the flowline of the curb should not be an issue as we have successfully conducted 1100C tests on the MGS with barrier heights of 34" and 36". The 2270P vehicle response is not expected to be adversely affected by the effective increase in rail height either.

Let me know if you need anything else.

Thanks

Temporary barrier deflections at less than TL-3 speeds

Question

State: IA
Date: 12-21-2016

As mentioned in the Year 28 Problem Statements by Wisconsin, we frequently have to use TBR for projects with less than TL-3 speeds. Up until six months ago, we had defaulted to using full eight sections for an unanchored run before we could expect the 48" deflections used in our standards. In answering a unique design question recently, I had to dig into that guidance to see what modifications we could make to fit the project's needs. Part of my reply was the following:

> For 62 mph, previous guidance (page 6 of TRP-03-209-09 and others) has suggested that 8 sections (100') upstream and downstream of the impact point (or protection area) are needed to properly “anchor" a free-standing section so that the deflection relates to the Unanchored column in Table A of BA-401.
> Using that information, we could presume that for similar deflections, a 50% reduction in speed would allow for at least a 50% reduction in length of need. As it turns out, the attached shows that it's more like a 66% reduction in length of need (35 mph = 33% of TL-3). The equation and values come from page 11, equation 2-1, in 2009 MASH.

My question to you is, are the presumptions made above consistent with other available information you've been able to find since TRP-03-209-09 and TRP-03-113-03 were published or in preparation for the Year 28 Problem Statements packet? The above situation was focused on minimizing TBR length rather than deflection, but perhaps the opposite approach could be used keeping the eight sections upstream/downstream and focus on reduced deflection. See attached for calculations.

Any additional
thoughts or comments will help guide our decisions until such time these
layouts are actually tested.

Thank you for
your time.

Attachment: https://mwrsf-qa.unl.edu/attachments/6c2924c6d607decc1eb86510825f63b5.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/07bd1f00f1cf35358d2a49f0d270e34f.pdf

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Response

Date: 12-22-2016

I have just finished writing up sections of a report on this subject for NDOR where we looked at reduced PCB lengths an potential deflections for TL-3 and the 85th percentile impact conditions. That report is currently in draft review internally at MwRSF. It should address a lot of your questions.

The work for NDOR does not go down to TL-2 or 1, but does look at speeds around 51 mph.

As we discussed in November at the Pooled Fund meeting, if states want to look at the lower speeds, we can run some simulations under a very small year 28 project to look at the lower speeds.

Bolt holes in guardrail posts

Question

State: NE
Date: 12-22-2016

Multiple people have contacted MwRSF to discuss the proper sizing and location for bolt holes in W-beam guardrail posts. Within the Hardware Guide, some system details show a 3/4" hole diameter, while others show a 13/16" hole diameter. Which size hole should be utilized? Also, some drawings show the hole as centered 1 1/8" from the center line of the post, while other drawings show the hole centered 3/4" from the edge of the flange. Which is correct? Finally, discrepancies have existed for the dimension of the hole to the top of the post. Some drawings indicate 7" while other indicate 7 1/8". What should the distance be?

Response

Date: 12-22-2016
Hole diameters: We believe the two different hole diameters came about from the conversion to metric dimensions, rounding, then a conversion back to English and rounding again. The 1/16" difference in hole diameters that has resulted is so minor that MwRSF believes that both hole diameters function and perform appropriately. Thus, either bolt hole diameter could be utilized in the fabrication of guardrail posts. Construction standards typically call for a hole to be 1/8" larger in diameter than the bolt. Guardrail bolts are 5/8" diameter, so MwRSF utilizes a 3/4" diameter hole in all of its drawings and details.

Placing the hole as a distance from either the center line of the post or the edge of the post flange will result in a difference of 1/16" in the lateral position of the hole. MwRSF views this difference as within construction tolerances and inconsequential to the performance of the guardrail. Thus, either placement/measurement may be utilized. MwRSF has typically detailed the hole as 3/4" from the flange edge for simplicity reasons.

The distance from the center of the bolt hole to the top of the post has varied due to a couple of factors. First, the height of W-beam guardrail has been shown as the nominal 12 1/4" or rounded to 12". Second, wood blockouts typically extended 1" above and below the rail, while steel blockouts typically extended 3/4" above and below the rail. Finally, the nominal height to the top of the rail has been shown to vary depending on rounding and which units are shown (US empirical vs. SI). All of these minor variations have lead to some post drawings showing the hole centered 7" from the top while others show it 7 1/8" from the top. MwRSF feels that this 1/8" difference is within standard construction tolerances, and thus either dimension can be utilized. Note, the targeted (nominal) height to the top of the W-beam rail should remain 31". MwRSF has typically utilized the more precise dimension and detailed the hole as being centered 7 1/8" from the top of the post.

Steel Specifications for Guardrail Posts

Question

State: NE
Date: 12-22-2016

MwRSF has received multiple questions referring to the steel specifications and grades for guardrail posts. The Barrier Hardware Guide lists multiple steel specifications for standard guardrail posts: ASTM A36, ASTM A709 grade 36, and A709 grade 50. Additionally, the drawing details for many new systems are specifying the use of ASTM A572 and/or ASTM A992. Are all of these steel specifications considered equivalent, or which specification should be utilized for guardrail posts?

Response

Date: 12-22-2016
Over the bast few decades the standard steel specifications for rolled structural shapes has changed. The older steels with yield strengths of 36 ksi have been replaced with higher strength materials with Fy = 50 ksi. Please refer to the attached documents describing this transition.
There is a lot fo overlap between these material specifications which allows for a particular steel lot/heat/batch to qualify under multiple of these specifications. For example, A36 requires only a minimum yield and tensile strength, there is no maximum values. As such, it has been easier to manufacture steel at grade 50 and label it as A992, A572, and A36. It may still be possible to obtain grade 36 steel posts, but it becoming increasingly difficult to find. Over the past 5-10 years, MwRSF has typically received A572 or A992 when ordering guardrail posts (even when A36 was requested).

The performance of standard guardrail systems should not be negatively affected by this change. Systems designed and tested with 36 ksi steels should still be crashworthy utilizing 50 ksi steel posts. However, there is a possibility for increased deck damage from bridge rails designed/tested with 36 ksi steel. The increased strength of the post will transfer more load to the deck. Damage to the deck would also depend upon the deck strength, concrete strength, and rebar configuration, so increased damage is not a certainty. Increased deck damage is a concern and systems specifying A36 posts should be evaluated on an individual basis.

Within drawing sets and system details, MwRSF has been listing multiple specifications as equivalent steel materials for use as guardrail posts. However, we have made the decision to stop listing grade 36 steels in current and future projects due to a lack of 36 ksi steel availability. MwRSF will only be listing grade 50 steel (A992).

ASTM 709 is available as a grade 50 steel, but A709 was developed for use on bridges. Accordingly, standard roadside and median barrier systems will likely not require A709. Instead, A992 is a better steel specification for these in-ground installations. Agencies may desire to utilize A709 in bridge rails. Thus, it should remain a viable option for use in bridge rail components with attention being given to ensure the proper grade is utilized to match the desired strength of the post/system.

This change is steel materials should also be reflected in the Barrier Hardware Guide. These recommendations will be given to Task Force 13 in hopes that the specifications within the guide can be updated to reflect current material standards.

Attachment: https://mwrsf-qa.unl.edu/attachments/fe79189f7dc96d4eae7f5221141a9158.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/f655a61fe45173f605b5bc8bdb8078f7.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/db97177b9d73fb17a1d97ef10e2d445a.pdf