TRANSPORTATION POOLED FUND PROGRAM
QUARTERLY PROGRESS REPORT

Lead Agency (FHWA or State DOT): Nebraska Department of Roads

INSTRUCTIONS:
Project Managers and/or research project investigators should complete a quarterly progress report for each calendar quarter during which the projects are active. Please provide a project schedule status of the research activities tied to each task that is defined in the proposal; a percentage completion of each task; a concise discussion (2 or 3 sentences) of the current status, including accomplishments and problems encountered, if any. List all tasks, even if no work was done during this period.

Transportation Pooled Fund Program Project # *(i.e, SPR-2(XXX), SPR-3(XXX) or TPF-5(XXX)* TPF-5(193) Suppl. #83		**Transportation Pooled Fund Program - Report Period:** ☐ Quarter 1 (January 1 – March 31) ☐ Quarter 2 (April 1 – June 30) ☑ Quarter 3 (July 1 – September 30) ☐ Quarter 4 (October 4 – December 31)
Project Title: Annual Consulting Services Support		
Name of Project Manager(s): Ron Faller, John Reid, Bob Bielenberg	**Phone Number:** 402-472-9064	**E-Mail** rbielenberg2@unl.edu
Lead Agency Project ID: 2611211115001	**Other Project ID (i.e., contract #):** RPFP-15-CONSULT	**Project Start Date:** 8/1/2014
Original Project End Date: 7/31/17	**Current Project End Date:** 7/31/17	**Number of Extensions:** 0

Project schedule status:

☑ On schedule ☐ On revised schedule ☐ Ahead of schedule ☐ Behind schedule

Overall Project Statistics:

Total Project Budget	Total Cost to Date for Project	Percentage of Work Completed to Date
$50,001.00	$29,962.00	60%

Quarterly Project Statistics:

Total Project Expenses and Percentage This Quarter	Total Amount of Funds Expended This Quarter	Total Percentage of Time Used to Date
	$9,693.00	

Project Description:

This project allows MwRSF to be a valuable resource for answering questions with regard to roadside safety issues. MwRSF researchers and engineers are able to respond to issues and questions posed by the sponsors during the year. Major issues discussed with the States have been documented in our Quarterly Progress Reports and all questions and support are accessible on a MwRSF Pooled Fund Consulting web site.

Progress this Quarter (includes meetings, work plan status, contract status, significant progress, etc.):

In the past quarter MwRSF has responded to a series of state inquiries. The Quarterly Progress Report summarizing these responses has been attached to this document. The summary will also be available for download at the recently completed MwRSF Pooled Fund Consulting web site - http://mwrsf-qa.unl.edu/

We are continuing to work with and improve the MwRSF Pooled Fund Consulting web site as our experience with it grows. We would ask that all Pooled Fund member states use the new site from this point forward for their inquiries and to contact us with any issues they experience with the web site.

Anticipated work next quarter:

MwRSF will continue to answer questions and provide support to the sponsors during the upcoming quarter.

We would ask that all questions be submitted through the web site so that they can be answered and archived therein.

http://mwrsf-qa.unl.edu/

Significant Results:

A quarterly summary of the consulting effort was provided and users can use the web site to search and find responses as well.

Circumstance affecting project or budget. (Please describe any challenges encountered or anticipated that might affect the completion of the project within the time, scope and fiscal constraints set forth in the agreement, along with recommended solutions to those problems).

None.

Potential Implementation:

None.

Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

06-10-2015 to 09-09-2015

Temporary Concrete Barrier

Question
State: IL
Date: 06-11-2015

As a follow-up to the response of November 25, 2013 regarding considering increasing the optional chamfer from 1/2" to 1" on all edges, we have received a suggestion regarding an additional modification in an effort to reduce the likelihood of temporary concrete barrier damage related to handling/placing/removing. The attached Standard 704001 shows a sketch of a 2" x 6" modification at the bottom of both ends of the temporary concrete barrier across the entire 22-1/2" width of the base of the barrier.

How would this modification affect the performance of the IL F-Shape temporary concrete barrier?

[I will send the attachment via e-mail since I was having difficulty in successfully being able to provide an attachment through this method earlier this week.]

Attachment: http://mwrsf-qa.unl.edu/attachments/d04395074eceb4388930b0927e05ff90.pdf

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Response
Date: 06-12-2015
We would not recommend the removal of the 2"x6" section at the end of the TCB segment. The performance of the TCB during impact is partially dependent on the interlock of the toes of the barrier segment and development of moment continuity across the barrier joint. Removal of a portion of the toe of the barrier may reduce the effectiveness of the barrier toe contact and alter the barrier performance. Additionally removal of the concrete in that area would create more concentrated loading of the toes during impact and make them more likely to be damaged.

We do understand that the toes of the barriers get damaged during moving and placement. However, we would recommend placing more steel reinforcement in those areas rather than removal of portions of the toe near the end of the barrier.

There is potential that this alteration may still work, but it would need to be crash tested in order for us to be able to recommend its use.

Extra Blockouts / Bridge Guardrail Near Side Roads

Question
State: IA
Date: 06-15-2015

Extra Block outs

http://mwrsf-qa.unl.edu/view.php?id=205

On the link above it discusses the use of double and triple
block outs. I read it that double block outs are OK for any number of posts. I thought that would be for the standard 12" block out in the W beam, but it seems to indicate 8". Would the 8" block out, only apply to the Bridge transition section? Are there areas of bridge guardrail that double block outs should be avoided?

Restricted Length Bridge Guardrail

A question that arises often is one related to placing
guardrail that conflicts with a side road or entrance. We have developed a document for guidance, Short Radius Guardrail, and would like your input. We realize some of the shorter choices are less desirable but thought they were better than not doing anything. This is a tough subject that should probably require research and analysis, but we really just want to make sure we are not giving some blatantly bad direction to designers. Any input would be greatly appreciated.

Attachment: http://mwrsf-qa.unl.edu/attachments/8eb8ee9b0fc3bae65990600d82f3dbf8.docx

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Response
Date: 06-17-2015

First, the blockouts guidance below is based on 8" blockouts. Thus, we believe that 6" blockouts are acceptable as we have used them in certain special applications without a problem. 24" blockouts are allowed in limited locations as noted in the response.

This holds true for the standard MGS system as well.

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

For the short-radius document, I have a couple of comments.

1. We have provided guidance for attachment of terminals and minimum system lengths for the approach guardrail system and terminals. I would review that relative to the guidance on the first page, as I believe that some of them may be in conflict.

"Thus, the following implementation guidelines should be followed:

1. A recommended minimum length of 12 ft – 6 in. (3.8 m) for standard MGS is to be installed between the upstream end of the asymmetrical W-beam to thrie beam transition section and the interior end of an acceptable TL-3 guardrail end terminal. This segment includes one half-post spacing for Design K and three half-post spacings for Design L.

2. A recommended minimum barrier length of 46 ft – 10½ in. (13.3 m) is to be installed beyond the upstream end of the asymmetrical W-beam to thrie beam transition section, which includes standard MGS, a crashworthy guardrail end

terminal, and an acceptable anchorage system. This segment includes one halfpost spacing for Design K and three half-post spacings for Design L.

3. For flared guardrail applications, a minimum length of 25 ft (7.6 m) is recommended between the upstream end of the asymmetrical W-beam to thrie beam transition section and the start of the flared section (i.e. bend between flare and tangent sections). This segment includes one half-post spacing for Design K and three half-post spacings for Design L."

http://mwrsf.unl.edu/researchhub/files/Report38/TRP-03-210-10.pdf

2. When discussing the radius options for the short-radius guardrail, we would suggest a minimum radius of 8'. No radius smaller than that has ever been crash tested.

3. It appears that you are using the Washington short-radius design. This is likely based on the FHWA memo that previously recommended that design for use until a better, crash-tested system is developed. Recently, TTI got TL-2 approval for the Yuma County short-radius design. Some states have moved to this design, and I just wanted to bring it up in case you were unaware.http://www.roadsidepooledfund.org/files/2010/11/T-Intersection-final_2010-08-17.pdf

4. TTI has also done some recent research into a MASH TL-3 short-radius system. The system did meet the crash test criteria, but we have some concerns about impacts on the system in locations not specified in the crash test matrix. I thought you might want to look at that information as well.

   http://tti.tamu.edu/documents/0-6711-1.pdf

   http://tti.tamu.edu/documents/0-6711-S.pdf

5. On page three, you discuss an area for breakaway posts for capture of errant vehicles. You have chosen an area from 5-15 degrees. We have recently been doing research with NDOR on a new safety treatment for intersecting roadways and have looked into similar issues of necessary capture area. We believe that the angle of impact in the capture area may vary from 0-25 degrees. We defined the potential impact area for errant vehicles based on a runout length calculation. NDOR liked this approach because it provided for a more justifiable definition. The report on this research should be out in a month, but I have attached a draft of the chapter describing this for your review.

Thanks

Attachment: http://mwrsf-qa.unl.edu/attachments/e2d3281ac681d0ecdc4a939be383a1f4.pdf

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Curb in front of thrie-beam transition

Question
State: IA
Date: 06-22-2015

I'm looking for guidance for the following situation regarding the length of 4" curb placed through the end of the nested w-beam of a typical 37.5' Barrier Transition System (BA-201).

As you can see on BA-201, we show possible curb running through the end of the BTS as report 03-291-14 *Dynamic Evaluation of MGS Stiffness Transition with Curb* suggests in the second full paragraph on page 136. We denote this 4" curb via circle note 2, which states to see project plans. Project plans in this case typically refer to our Bridge Approach Standards (BR Series), and therein lies the problem. On the Bridge Approach Standards that show abutting pavement (BR-102, 103, 104, 105, 106, 107, 112, 205, 211, 212, 231), we have a circle note that states something to effect of “Build 4 inch Sloped Curb to end of Double Reinforced Section." This note has been shown on our approach standards since April 1999 when we switched to the 4" curb. A typical double reinforced section extends 20' out from the center of the roadway (see image below). I would say that in almost no case would the double reinforced section extend 37.5' and meet the report's suggestion. What I would like to know is which of the following is the real issue with curb location:

1) If curb is present along any length of the thrie-beam portion, asymmetric transition portion, or nested w-beam, it must be run from the bridge end out to at least 37.5', and thus typically ending at the nested-to-single w-beam splice, or

2) If curb is present under the 7' (approx.) asymmetric transition, it must be run out to the nested-to-single w-beam splice.

The first option forces a 37.5' minimum install length of curb. The second option says to end the curb before the asymmetric transition (somewhere in the first 18' or so) ***OR*** carry it out to the nested-to-sing w-beam splice (at least 37.5').

I ask because both our “old" Barrier Transition Section standard (older BA-201) and “new" Barrier Transition System (BA-201) have the asymmetric transition ending 25' away from the bridge end. With a typical bridge end section of 7' (BA-107), that placed the asymmetric transition 25' to 32' from the edge of the deck/beginning of double reinforced section. Perhaps on the long side of a strong skew we could have seen the edge of the double reinforced be in line with the asymmetrical transition, but it was likely a rare occurrence, which is great if #2 above is what was meant. If #1 is the real issue, then I'll have a follow up email about the immediacy of change for our current standards.

Attachment: http://mwrsf-qa.unl.edu/attachments/615314ed3792e3351fb4e9051968de56.jpg

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Response
Date: 06-22-2015

Your option 2 is more accurate. If you wish to have curb extend from the bridge rail/buttress and terminate prior to the asymmetric rail section, this would be acceptable. In fact, prior to the success of the full-scale test you are referencing, we had recommended terminating the 4" curb within the thrie beam section of the approach guardrail transition. I apologize for this not being clearer in the report.

Response
Date: 06-23-2015

Given that clarification, here is what I plan to propose to designers. Please let me know if this is incorrect.

Using BA-201 as an assumed Barrier Transition Section and a standard 5' curb transition as shown on page 2 of PV-102 (which meets your 3' min curb transition recommendation on page 136 of TRP-03-291-14):

A) If any part of the 4" sloped curb (including the curb transition) extends into the asymmetrical transition section, the minimum curb length must be extended to 37.5' and they are to use the new 37.5' BA-201 showing the nested w-beam and also meet the general layout requirements as shown on page 137 of TRP-03-291-14 *Dynamic Evaluation of MGS Stiffness Transition with Curb*:

B) If the 4" sloped curb (including the curb transition) is terminated prior to the asymmetrical transition section, the older BA-201 (25' instead of 37.5') may be used but must also meet the general layout requirements as shown on page 154 of TRP-03-210-10*Development of the MGS Approach Guardrail Transition using Standardized Steel Posts*:

I ask for confirmation because the potential installation length differences between the two are significant. Assuming all other things equal, the first has a minimum install of (37.5 BTS+50.0 w-beam+3.125 connection+50.0 terminal) 140.625'. The second has a minimum install of (25.0 BTS+12.5 w-beam+3.125 connection+50' terminal) 90.625', or a 50' installation length increase for not terminating the curb early. This may not matter on the interstate or higher volume roadways where the runout length (according to Roadside Design Guide) is well beyond that, but certainly would come into play on our lower volume roadways and especially our county/local roadways. It may also cause us to rethink requiring the curb to extend to the end of the double reinforced section (discussed below) as this would essentially guarantee the longer installation.

Another situation we run into frequently is having to drop down from a TL-3 Barrier Transition Section to a TL-2 where we essentially eliminate the thrie-beam downstream of the asymmetrical transition and attach the asymmetrical transition piece to the Bolted End Anchor (BA-202). This TL-2 system was tested at TTI (available here with detail attached). This typically comes into play when we have an entrance/side road within the normal guardrail installation. Since I wouldn't expect for you to speak for another research facility, TTI in this case, I will make the following statement and ask that you agree or disagree based on the general principles at play.

A) Since the failure in TRP-03-291-14 was due to the existence of curb in the asymmetrical piece, any curb coming off of the bridge end would extend into that section and thus a 12.5' section of nested w-beam should be added to the upstream end of the transition, along with w-beam equal to the length of the end terminal (25' TL-2 typically).

B) If there is no curb, no nested w-beam is needed but a length of w-beam equal to the length of the end terminal should be included, OR

C) If there is no curb, no nested w-beam is needed but a 12.5' section of w-beam should be included before the end terminal.

Again, please confirm my understanding on both points. And as always, your assistance is appreciated.

Attachment: http://mwrsf-qa.unl.edu/attachments/1f0350b249e192392307be9bd1da7a8e.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/119a1b57e82737ead02789dbd7bd8964.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/f7aa283ca4a3b630eb26f906b0490a0b.pdf

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Response
Date: 06-24-2015

I have commented below in RED

Given that clarification, here is what I plan to propose to designers. Please let me know if this is incorrect.

Using BA-201 as an assumed Barrier Transition Section and a standard 5' curb transition as shown on page 2 of PV-102 (which meets your 3' min curb transition recommendation on page 136 of TRP-03-291-14):

A) If any part of the 4" sloped curb (including the curb transition) extends into the asymmetrical transition section, the minimum curb length must be extended to 37.5' and they are to use the new 37.5' BA-201 showing the nested w-beam and also meet the general layout requirements as shown on page 137 of TRP-03-291-14 *Dynamic Evaluation of MGS Stiffness Transition with Curb*:

B) If the 4" sloped curb (including the curb transition) is terminated prior to the asymmetrical transition section, the older BA-201 (25' instead of 37.5') may be used but must also meet the general layout requirements as shown on page 154 of TRP-03-210-10*Development of the MGS Approach Guardrail Transition using Standardized Steel Posts*:

I ask for confirmation because the potential installation length differences between the two are significant. Assuming all other things equal, the first has a minimum install of (37.5 BTS+50.0 w-beam+3.125 connection+50.0 terminal) 140.625'. The second has a minimum install of (25.0 BTS+12.5 w-beam+3.125 connection+50' terminal) 90.625', or a 50' installation length increase for not terminating the curb early. This may not matter on the interstate or higher volume roadways where the runout length (according to Roadside Design Guide) is well beyond that, but certainly would come into play on our lower volume roadways and especially our county/local roadways. It may also cause us to rethink requiring the curb to extend to the end of the double reinforced section (discussed below) as this would essentially guarantee the longer installation.

I agree with the installation options A) and B) above. However, the minimum total lengths of the systems should be identical. The only difference between the installation length recommendations was the reference point. The Option A reference point is the upstream end of the W-beam section, while the reference point for Option B is the upstream end of the asymmetrical segment. Since the nested region is 12.5 ft long, the lengths should add up:

Requirement #1

A: 37.5 ft (end shoe through nested w-beam) + Terminal = 37.5 ft + Terminal length

B: 25 ft (end shoe through w-to-thrie segment) + 12.5 ft (standard MGS) + Terminal = 37.5 ft + Terminal length

Requirement #2

A: 37.5 ft (end shoe through nested w-beam) + 34.38 ft (w-beam) = 71.88 ft

B: 25 ft (end shoe through w-to-thrie segment) + 46.88 ft (standard MGS) = 71.88 ft

Requirement #3

A: 37.5 ft (end shoe through nested w-beam) + 12.5 ft (MGS) + Flared Terminal = 50 ft + Flared Terminal

B: 25 ft (end shoe through w-to-thrie segment) + 25 ft (MGS) + Flared Terminal = 50 ft + Flared Terminal

Another situation we run into frequently is having to drop down from a TL-3 Barrier Transition Section to a TL-2 where we essentially eliminate the thrie-beam downstream of the asymmetrical transition and **attach the asymmetrical transition piece to the Bolted End Anchor** (BA-202) **There should be 37.5" of either 10 ga. thrie beam (as tested) or nested 12 ga. thrie beam between the asymmetrical segment and the end shoe** . This TL-2 system was tested at TTI (available here with detail attached). This typically comes into play when we have an entrance/side road within the normal guardrail installation. Since I wouldn't expect for you to speak for another research facility, TTI in this case, I will make the following statement and ask that you agree or disagree based on the general principles at play.

A) Since the failure in TRP-03-291-14 was due to the existence of curb in the asymmetrical piece, any curb coming off of the bridge end would extend into that section and thus a 12.5' section of nested w-beam should be added to the upstream end of the transition, along with w-beam equal to the length of the end terminal (25' TL-2 typically).

B) If there is no curb, no nested w-beam is needed but a length of w-beam equal to the length of the end terminal should be included, OR

C) If there is no curb, no nested w-beam is needed but a 12.5' section of w-beam should be included before the end terminal.

Since we do not have testing of the TL-2 system with curb, it would be conservative to add the nested section of W-beam upstream of the asymmetrical segment to prevent possible rail tearing when a curb is present. As such, I agree with these installation configurations and lengths.

Response
Date: 06-25-2015

Two additional questions for you then.

I agree that requirements 2 and 3 would produce installations that are identical in length. Makes sense that you're simply turning a piece of standard w-beam into nested. What I'm perhaps more confused with now is how requirement 1 is bringing me to the same length for each situation.

Part A

BTS = 37.5'

W-beam upstream of nested to be greater than or equal to end terminal = 50' (or 37.5' for cable connection)

End terminal = 50' (or 37.5' for cable connection)

Total install = BTS + 2 times end terminal length = 37.5' + 2(50') = 137.5'

Part B

BTS = 25'

w-beam = 12.5'

End terminal = 50' (or 37.5' or 25')

Total install = 87.5'

The other question is in regards to the 12.5' section of standard w-beam between the asymmetrical transition piece and the end terminal (requirement B,1 below). What was the underlying concern that introduced this section? We have plenty of existing installations where it is simply a BTS to End Terminal.

Sorry for all the questions, I'm just trying to understand the underlying principles in order to make correct modifications as needed.

Response
Date: 06-26-2015

For requirement 1, the length upstream of the nested w-beam section is to include the terminal itself. Thus, you only need your BTS length (37.5 ft) plus the length of the terminal (50 ft), for a total of 87.5 ft – same as option B. Sorry for the mix up there, we probably could have worded that better.

Concerning the 12.5 ft for requirement B-1:

The overlying reasoning is to separate any transition elements from the end terminal so as not to affect the performance of the end terminal. A different rail segment could easily affect performance, so we need to keep the stroke length of the terminal upstream of the 10 ga. w-to-thrie segment. Additionally, there is a post at ½ post spacing (37.5") upstream of the w-to-thrie segment. Depending on the specific terminal, utilizing a different post spacing (or different post altogether) may also affect the performance of the terminal. Thus, we wanted to stay upstream of the ½ post spacing portion of the transition as well. We could have used a 6'-3" distance in this recommendation (B-1), but many states don't stock 6'-3" segments of w-beam guardrail. Therefore, the length was conservatively extended to 12'-6". If you desire to use the shorter 6'-3" distance for plans, I couldn't argue against it.

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what is the purpose of beam guard in the photo attached

Question
State: WI
Date: 07-01-2015

The RDG indicates that the guardrail shown on the attached file should be used near a short radius system. Any idea on why?

Attachment: http://mwrsf-qa.unl.edu/attachments/53690234fd13a4c0fbf7cfe65ff5dbb9.docx

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Response
Date: 07-02-2015
I would assume that the guardrail shown upstream of the short-radius system in the attached figure is based on two related concerns.
1. All previously developed and tested short-radius systems have focus only on impact locations ranging from the center of the nose of the system and towards the primary roadway. Thus, the behavior of the system farther up on the secondary roadway is unknown.
2. Based on potential vehicle trajectories and runout length calculations, current short-radius systems may potentially be insufficient to shield the range of potential impacts.

Placement of the additional barrier on the upstream side of the primary roadway limits the potential for these concerns. We have currently been working on a research project for NDOR to develop a treatment for intersecting roadways that better addresses the hazard area and has significantly higher capture capabilities. The Phase I report for that should be out this month and Phase II started July 1.

Guardrail flare rate with Bullnose

Question
State: NE
Date: 07-01-2015

With w-beam tested on a 7:1 taper rate, would you expect
thrie-beam to redirect a MASH vehicle properly on a 7:1 taper?

What about thrie-beam on a 8:1 placed in a 45 mph median?

This is attached to a bullnose.

Response
Date: 07-06-2015

We have not recommended increased flare rates on thrie beam at this time. While the work done with the MGS system would indicate that the potential for increased flares with thrie beam exists, there are too many unknowns. The thrie beam system would have different capture, post stiffness, and dynamic deflections as compared to the MGS. As such, it is difficult to ensure that the performance would be similar. Certain design changes might need to be made regarding the post embedment, splice location, and the blockout geometry to ensure that it would work.

As far as the bullnose system goes, we have recommended standard flare rates as recommended by the RDG beginning at post no. 9 on the oncoming traffic side. Flaring prior to post no. 9 may affect the performance of the bullnose. On the reverse direction traffic side of the system, we have allowed more aggressive flares as the guardrail would be flaring away from oncoming traffic.

The RDG list 8:1 flares as acceptable for 40 mph and 10:1 for 45 mph.

Thanks

Curb Offset Behind Guardrail

Question
State: KS
Date: 07-07-2015

When you have time could you please give me a call to discuss the attached? My calendar is full today, but I have some time tomorrow from 11:00 am to 1:00 pm or anytime Thursday works well.

Attachment: http://mwrsf-qa.unl.edu/attachments/8d0fb26a6d0eca56a2fb1a16eeb4799f.pdf

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Response
Date: 07-09-2015

I reviewed the schematic you sent regarding a curb offset behind a guardrail system to help control erosion. You had asked about our thoughts on the offset of the curb behind the barrier and the height of the curb. We have several comments on this setup for your consideration.

1. We would assume that this setup would be made with the MGS system rather than the previous metric height G4(1S) system. Thus, all of the remaining comments are made with respect to the MGS.
2. As you noted, the guardrail posts will be installed in a 4" thick concrete pad with leave outs filled with grout. I assume that this is based on the TTI research on mow strips and leave outs. We would recommend using that research for the leave out details.
3. The main concern with this type of system is the interaction of the vehicle with the curb during deflection of the guardrail system. For an MGS system, that deflection is typically in the 4' range for TL-3. We have seen that vehicle

interaction with curbs can increase rail loads and increase the potential for vehicle instability. However, we believe that the curb can be placed in the later portion of the barrier deflection and still maintain safe performance as the overall barrier stiffness will be lower at that point in the impact.

4. With regards to the barrier offset, the MGS with an 8" or a 12" blockout is approximately 18" to 22" deep, respectively. A 24" offset from the back of the post to the curb would make the offset to the curb 42"-46" depending on the blockout size. This would be at the latter region of the MGS deflection. Thus, placement of the curb at the 24" offset should have minimal effect on the barrier performance.

5. As far as curb height, we believe that a 4" curb would be a more forgiving setup in terms of its effect on barrier performance. However, we believe a 6" curb will work given the 24" offset and the limited vehicle interaction with the curb.

Let me know if that answers your questions or if you need anything else.

Thanks

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top mounted Guardrail to culvert attachments

Question
State: KS
Date: 07-22-2015

I've attached KDOT's current Standard Drawing (RD617E) for attaching MGS to low fill culverts as well as a sketch (Attachment to Low Fill Culverts) from a project currently being constructed. The design engineer on the project has run into an issue where the steel plate that rests on the ceiling of the box is conflicting with the fillet in the waterway opening (this is occurring at several locations). The sketch in the second attachment depicts the issue. We've investigated shifting the guardrail installation to avoid the conflict with the fillets, but unfortunately we are unable to avoid them given the needed post spacing. I'd like to discuss whether or not we could

include washers between the steel plate and the culvert ceiling to shim the plate down an inch or so to avoid the conflict. I marked on the second attachment where the washers would be included.
Attachment: http://mwrsf-qa.unl.edu/attachments/4dd0c01034511e5febc1e90c7dbc5297.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/dee23cc9cc5a36c8c28ab6ba30cb993d.pdf

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Response
Date: 07-22-2015

. I do not see a problem with utilizing washers underneath the culvert top slab. In fact, a similar attachment scheme was full-scale crash tested back with the original evaluation of this system – see report no. TRP-03-114-02 page 111 (available on MwRSF website/research hub). The tested configuration utilized only rectangular plate washers in the attachment of the through bolts to the underside of the culver slab. You could use this concept too, if desired. If you utilize washers of a similar size, you would not need to include the full-size washer plate in addition to the enlarged washers. The individual washers alone would suffice.

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fall protection on parapets

Question
State: WI
Date: 07-22-2015

We have had our contractor's mount hardware on top of our 32" bridge parapets to comply with OSHA fall requirements.

Here is one example.

I have concerns from a roadside design perspective that:

- Vertical struts are snag issue
- cables may interfere with vehicle once it leans over the barrier.

I have heard of 2x4 being used on our parapets and temporary barrier as well.

I wanted to get MwRSF's impression of what our contractors are doing.

Thanks

Attachment: http://mwrsf-qa.unl.edu/attachments/beb3b76612f4a35076adcb4b0a121ff1.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/8e95b8975d4000173df1f12e99ff05fc.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/450f13ee326c40d54504e4b3f3edd5fe.jpg

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Response
Date: 09-03-2015
The concern with the addition of structures on existing barrier systems would be similar to those concerns with barrier performance raised in previous studies regarding pedestrian rails, combination bridge rails, and the Zone of Intrusion (ZOI) study. Additional structure on top of a barrier may pose a snag hazard, adversely affect vehicle stability, potentially create dangerous debris, and pose a hazard for partially ejected occupants or occupant heads among other concerns.

Thus, while there is a need for fall protection in critical areas, we would recommend that the fall protection concern be weighed against concerns for the effect on the safety performance of the barrier.

Development of safe fall protection hardware would likely require further research, design, and potential evaluation testing. additionally. the design of the fall protection may differ significantly depending on the type of barrier it is used on.

Tie-Down Anchor Bolt - Length Change

Question
State: MO
Date: 07-29-2015

Ivan Schmidt at MoDOT has raised a question regarding the anchor bolt length that was used to tie-down the steel straps. The built-up strap material at the hole locations was ½ in. thick. In the original report, a 57-mm (2¼-in.) long bolt was used. Original report link is provided below.

Original Report

http://mwrsf.unl.edu/reportresult.php?reportId=219&search-textbox=tie

In the letter report to NDOR, the CAT anchorage testing
program utilized a 1¾-in. long bolt with the RedHead Drop-In anchors. Letter report link is provided below.

Letter Report

http://mwrsf.unl.edu/reportresult.php?reportId=266&search-textbox=tie

Do you know why the hex head bolts were shortened by ½ in.
and did not use the original bolt length of 2¼ in.? I have quickly looked through the original and letter reports as well as current RedHead anchor information. I was unable to find the published threaded distance within the anchor cavity. Let me know if you recall as to why this both length was reduced. Thanks!

Response
Date: 07-29-2015
The bolt was shortened based on concerns that the longer bolt could bottom out in the drop in and cause it to break the tabs that expand at the base.

Response
Date: 07-30-2015

So, then why was the original report not revised? Also, the 2007 letter reports that the ultimate tensile capacity 17.3 k of the red head anchor is based on “limited testing and review of manufacturer test data of the drop in anchor conducted during the development of the steel strap." Seems like the tensile capacity would be a pretty big deal since these anchors are pulled out of the holes in barriers directly impacted.

And, if the tensile strength is actually based on a 2 1/4" long bolt, then the alternatives would be overdesigned.

Where and when was this change in bolt length documented so we can have for our records and should MoDOT immediately change the 2 ¼" long grade 5 bolt to 1 ¾"? If the 2¼" long bolt was crash tested and worked why change to 1¾"?

Response
Date: 07-31-2015

See my comments below!

So, then why was the original report not revised?

**There was not a problem with the original configuration. The RedHead anchors were used with 2¼" long bolts, and the barrier system performed well. The original report documents the successful crash test and installation details. No hardware problems were encountered in the crash test. As such, there was no consideration to prepare a revised version of the report. However and based on your inquiry, we are wondering whether a notice should be posted to better indicate this bolt length change.

Also, the 2007 letter reports that the ultimate tensile capacity 17.3 k of the red head anchor is based on “limited testing and review of manufacturer test data of the drop in anchor conducted during the development of the steel strap." Seems like the tensile capacity would be a pretty big deal since these anchors are pulled out of the holes in barriers directly impacted.

**Per Bob, the RedHead anchor socket has an internal threaded length of 1¼". Thus, the original longer bolt penetrated farther into the void region below where the internal threads ended within the socket. Those extra threads did not engage the socket and did not provide additional tensile capacity. The new 1¾" long bolt with full threads engaged the entire threaded region of the socket when considering the strap thickness and welded washer plate thickness. Thus, the tensile capacity of the anchor was not changed. The anchor socket behavior was controlled by concrete fracture and/or bond failure.

**If bolts are excessively long, there could be a potential for really long bolts to contact and rupture the deformed tabs at the bottom socket. We do not recall that scenario occurring in our actual field installation that was used in the crash test.

**With additional information from the manufacturer, we chose to use a 1¾" bolt length in the follow-on study that evaluated alternative mechanical anchor hardware. Again, the tensile capacity of RedHead socket was controlled by concrete strength. Both bolt lengths would provide equivalent tensile capacities when considering a 1¼" threaded length within the socket.

And, if the tensile strength is actually based on a 2 1/4" long bolt, then the alternatives would be overdesigned.

**See comments above.

Where and when was this change in bolt length documented so we can have for our records and should MoDOT immediately change the 2 ¼" long grade 5 bolt to 1 ¾"? If the 2¼" long bolt was crash tested and worked why change to 1¾"?

**The 2007 letter report documented this change. Drawings were also prepared for AASHTO Task Force 13 Roadside Barrier Hardware Guide. The latest drawings are attached. Note that the anchor socket did not change. The 2¼" long bolt met crash testing requirements. The 1¾" long bolt will also meet crash testing standards as all bolt threads are engaged in 100% of socket threads. The shorter bolt length can reduce any potential concerns if an excessively long bolt would contact deformed tabs and cause damage. Although we did not have that occur, we took advise and reduce the risk of it occurring.

**It does appear that the original 2003 FHWA eligibility letter, B-112, still depicts the 2¼" long bolt for use in the socketed anchors.

**Alternatively, the online AASHTO TF13 Hardware Guide provides a 2007 SWC10 detail (8/31/2007) that shows the 1¾" bolt length. The link is provided below. However, the latest version of the SWC10 drawing that is located on our internal server is dated, 10-22-2008. Thus, the different dates causes me to raise the question as to why the online hardware does not depict the most current version of the detail that remains on our server as it should. As such, I need to speak to my colleagues to better understand how often revised details are forwarded to TF13 for replacement in the online guide.

http://guides.roadsafellc.com/hardwareGuide/index.php?action=view&hardware=124

**Please let me know if you have further questions regarding my responses to the items noted above. Thanks!

Temporary Barrier on Bridge 9123

Question
State: MN
Date: 08-04-2015

I need your help with this one (See specifics below).

We use the Iowa F-Shaped Temporary Concrete Barrier System (SWC09), and we do have the Tie-Down Strap System for the F-Shape Concrete Barrier in our standards (SWC10).

We essentially will have TL-2 conditions (45 mph Posted Speed).

Is there a configuration/location that would not require anchorage of the barrier for these TL-2 conditions. I realize that a barrier pushing up against the curb would easily tip over as the curb would act like a hinge, but how close is too close?

Is the top of the sidewalk an option at
all? I can visualize barrier deflection followed by vehicle vaulting on the curb (assume an 8" curb).

If we place a system with the Tie-Down Strap on
the shoulder, How close to the curb could it be placed? In the Tie-Down test FHWA letter, I believe that I saw 12 " (305mm) of dynamic deflection for the area of impact, with a TL-3 test. Could we go 6" with TL-2 conditions? We would like to give some room to the vehicles (the barrier is 22.5" wide).

Of course the designers need an answer asap.

Please give me a call if you need more information.

Thanks

I left you a voicemail, but I thought I would follow up with a clearer view of what was needed.

Some information about the site:

- TH 21 over UPRR (Bridge 9123)
- 45 mph on the bridge

- Expect that this condition will last for approximately 5 years until the bridge is replaced

We are looking for guidance on the following questions:

- Can we place temporary barrier on the shoulder or on the sidewalk?

- If so, how far from the edge of the sidewalk or from the gutter line does it have to be?

- What sort of anchoring is needed?

- Any other guidance we need?

Attachment: http://mwrsf-qa.unl.edu/attachments/ec81acdbe36a98bfbf5393571fd8a16a.jpg

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Response
Date: 08-06-2015

I will try to provide my thoughts and be as brief as possible due to MnDOT's time constraints.

First, the duration of the work zone is noted to be five years, which is a long duration . Thus, a conservative approach may be justified using best available information.

Second, the placement of a single row of F-shape PCBs on top of the sidewalk may create some concerns. Although the F-shape PCBs will displace laterally, I wonder whether impacting passenger vehicles will reach a higher effective climb height on the barrier face relative to the lower bridge deck surface, thus potentially leading to slightly greater vehicle instabilities. Granted, I believe that this risk would be much reduced at TL-2 conditions as compared to TL-3 conditions. However, I just want to denote this low to moderate potential safety concern.

Third, the laterally-deflecting PCB on the sidewalk could create a scenario where a partially-redirected vehicle allows for a wheel to catch on the sidewalk, thus leading to increased roll or yaw behavior and increased vehicle instabilities. Although this behavior and risk is uncertain, I still need to raise this potential concern.

Fourth, a free-standing PCB at TL-2 of NCHRP 350 would result in reduced dynamic deflections as compared to TL-3. Unfortunately, we have not previously conducted testing nor simulations at the TL-2 conditions. Instead, MwRSF conducted a study to evaluate conditions at the 85% impact condition – 2000P pickup truck, 36 mph, and 27.1 degrees. Details of this R&D is provided using the link below. Note that the TL-2 impact condition provides approximately 35% more impact severity than the 85% impact condition used for the noted study. Thus, the anticipated free-standing deflection for F-shape PCBs would range between 24 and 43 in for NCHRP 350 TL-2 impact conditions. One might estimate between 32 and 36 in. for TL-2.

http://mwrsf.unl.edu/reportresult.php?reportId=243&search-textbox=concrete%20barrier

Next, I agree with you that free-standing PCBs on the bridge deck and adjacent to the sidewalk would laterally deflect backward and strike the sidewalk. Once the PCBs had bottomed out against the raised sidewalk edge, the PBCs would be prone to rotation without translation, which could increase vehicle climb and subsequent roll and pitch angles as well as vehicle instabilities. As such, adequate space would be necessary between a free-standing PCB and the raised sidewalk, even for TL-2 conditions.

Based on this information, I believe that a reduced-deflection PCB system may be worth considering for your application and located on the bridge deck. There exists the (1) tie-down strap system, (2) vertical through-bolt tie-down system, and (3) WisDOT steel tube and saddle system for use with the F-shape PCB system. All three systems alone, or in combination with one another, may provide a workable solution, especially where you have limited width for PCB placement. If you can accept deeper holes drilled into the bridge deck for either through-anchor bolts or rods epoxied into partial-depth holes within the deck, the through-bolt system offers a low-deflection system, especially at TL-2. If that option is not acceptable, the tie-down strap system could be used with a larger gap between the back of barrier base and face of raised sidewalk, say 6 to 9 in. and at TL-2. Finally, it might seem reasonable to use a combination of the tie-down strap system and the WisDOT tube/saddle system at an even closer offset to front face of sidewalk, say 3 to 6 in., for TL-2 impact conditions of NCHRP 350. Actually, I might consider the latter option more preferred when considering the 5-year work-zone time period. Of course, it should be noted that these options have not been tested under TL-2 conditions and/or when positioned this close to a raised sidewalk edge.

Finally, one last consideration would be to further reduce posted speed limits from 45 mph to 35/40 mph to help control potential impact speeds.

If you have any further questions regarding the enclosed information, please feel free to contact me at your earliest convenience. Thanks!

Steel Posts in the Bridge Transition Section

Question
State: IA
Date: 08-06-2015

We received a call from a contractor asking if they could use a longer 6'9" for posts 8-13 in that attached modified road standard. This would keep the same length of embedment but would allow the top of the post to be flush with the blockout.   It sounds like they are having trouble getting the post installed correctly with the method they typically use. The other reason they sighted, for the longer posts, is that the dies their manufacture have need 7 inches at the top of the post to punch the holes. The shorter posts shown are requiring custom punching of the posts and driving costs up. Do you see an issue with allowing the post to be 3 inches longer?   They are waiting to order some posts, so a quick response would be greatly appreciated.

Attachment: http://mwrsf-qa.unl.edu/attachments/6668147e0cd2bef85398813a1ac55ad0.jpg

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Response
Date: 08-07-2015

Brian:

I have reviewed our prior research studies that pertain to this issue. These include:

System No. 1 - ITNJ Test Series – nested thrie-beam AGT with steel posts on ¼-spacing near buttress end

Report can be accessed at:

http://mwrsf.unl.edu/reportresult.php?reportId=61&search-textbox=transition

Test ITNJ-2 on an improved design consisted of successful 2000P pickup truck test at TL-3 impact conditions. The six ¼-spaced posts were installed with a 49" embedment depth and a 2" recess for top of post below top of 31" tall thrie beam rail. The distance between the buttress end and the center of the first post was 11.5". The top of posts were recessed 2" in this stiffer transition region due to concerns for engine hood and quarter panel contact and snag that may lead to instabilities and even rollover. This snag behavior was observed and deemed a contributing factor in the rollover of a pickup truck in a R&D study performed on a Missouri transition system. See System No. 2 discussion below. This testing was performed under NCHRP Report No. 350.

System No. 2 – MTSS Test Series – dual 10-gauge thrie-beam median AGT with steel posts on ¼-spacing near buttress end

Report can be accessed at:

http://mwrsf.unl.edu/reportresult.php?reportId=84&search-textbox=transition

Test MTSS-2 on a modified design consisted of successful 2000P pickup truck test at TL-3 impact conditions. The six ¼-spaced posts were installed with a 43" embedment depth and a 2" recess for top of post below top of 31" tall thrie beam rail. The distance between the buttress end and the center of the first post was 11.5". Following the first failed test, the top of posts were recessed 2" in this stiffer transition region due to engine hood and/or quarter panel contact and snag on top of the median posts and on the upper sloped end of concrete buttress. This testing was performed under NCHRP Report No. 350.

System No. 3 – Test 2214T-1 – nested thrie-beam AGT with steel posts on ¼-spacing near buttress end

Report can be accessed at:

http://mwrsf.unl.edu/reportresult.php?reportId=148&search-textbox=transition

Test 2214T-1 consisted of same system evaluated under System No. 1 above and included a successful 2270P pickup truck test at TL-3 impact conditions. The six ¼-spaced posts were installed with a 49" embedment depth and a 2" recess for top of post below top of 31" tall thrie beam rail. The distance between the buttress end and the center of the first post was 11.5". The top of posts were recessed 2" in this stiffer transition region due to concerns for engine hood and quarter panel contact and snag that may lead to instabilities and even rollover. This testing was performed under MASH.

System No. 4 – nested thrie-beam AGT to T131RC using steel posts on ¼-post spacing **[no recessed posts]**

Report can be accessed at:

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

Two tests - 2270P and 1100C - were successfully run under MASH TL-3. The six ¼-spaced posts were 7-ft long but did not utilize a recessed top of posts relative to the top of the rail. As such, this testing may suggest that the recessed region may not be needed under MASH.

**At this time, I have requested two other crash test reports from TTI. I want to review these tests and determine whether those successful thrie beam AGT systems utilized recessed tops for posts relative to top of rail. I should have those reports by next week. In closing, I believe that there is strong potential for the tops of closely-spaced, steel transition posts to incorporate the same height as the blockouts under MASH testing. However, I want more evidence from a few more cases for confirmation, and then I will get back to you with an updated response.

Thanks again and feel free to contact me with any additional questions or comments!

Response
Date: 08-27-2015

Hi Ron,

Sorry for the delay in getting you this information. It is amazing how quickly things get buried these days. I appreciate the reminder. ☺

The first test was run under NCHRP Report 350. It did not have a curb, and the end shoe was rotated into the slope of the concrete bridge rail parapet to which it was attached. The link to the report for this test is:

http://d2dtl5nnlpfr0r.cloudfront.net/tti.tamu.edu/documents/4564-1.pdf

The second test was run under MASH. It also did not have a curb. However, in this case, a special adaptor block was fabricated for use under the end shoe to keep the thrie beam rail in a vertical plane throughout its length. The link to the report for this test is:

http://d2dtl5nnlpfr0r.cloudfront.net/tti.tamu.edu/documents/9-1002-12-3.pdf

Please let me know if you have any questions or need any additional information. We can forward video, etc. as needed.

Best regards,

Roger

Response
Date: 09-09-2015
I believe that the recess may not be needed based on System No. 4 noted below. Thus, an increase in post length without the use of top recess may seem reasonable.

I agree that you could move forward with the longer post that no longer includes a recessed top region.

NHSX-52-5(31)--3H-96 Curved Guardrail

Question
State: IA
Date: 08-07-2015

As we discussed please see the
curved guardrail layout. I have include the MicroStation files so you can do more detailed measurements.

As I eluded to
below and again on the phone, I think a radial guardrail design needs to be used for both the Pulpit and Madison roadways. Looking at the attached Google Earth image, one can see the (no longer) existing layout changes from steel to wood posts around the curve. We need to effectively mimic that design this time around.

I contacted
WHKS to get their design file as the plan sheets leave a lot to be desired.
Attached is their reply. Those files and all workup files are available at W:HighwayDesignMethodsSectionMethods-SubjectAreasBarriers_DesignModificationsGuardrail_RadiusGuardail_US52

Model
'Guardrail Details' inside **7288.14.dgn** shows that the proposed paved shoulder at Pulpit has an inside radius of 46' (attached image). Using that as the design radius (as it is already poured is effectively becomes the radius), the curved portion of the guardrail layout is shown in attached CurvedDimensions.pdf.

With an Lg of
75, you'll need (75/6.25) + 3 = 13 CRT posts and (75/12.5) = 6 curved w-beam pieces. You'll also need to add a 12.5' VT section between the end of the last curved section and the beginning of the End Terminal. The 3 posts that are used in that section are the 3 added in the sentence above.

Using model
‘Guardrail Detail Shading', the guardrail splits should look something like this:

BTS= 28.125

VF = 75

VT1 = 250

Curved VT = 75

VT2 = 12.5

ET = 50

I'm not
entirely confident in the precision of the first VT as the paved shoulder and guardrail don't seem to run parallel to each other in **the 7288.14.dgn**, but since the paved shoulder is already constructed, drill the holes at 6.25' increments and begin placing CRT posts at the junction of the ~250' VT and the curved VT, continuing through the curve and final VT, ending at the end terminal. The posts in the BTS and the VT1 may be changed to steel as requested as I don't believe that change impacts the radial component.

For the Madison
Road curve, since it is only a partial layout, I can't really design it. The same approach would apply though. Basically, the CRT posts begin at the connection of the VT and the curved w-beam and continue through the curve. Since the guardrail doesn't terminate near the intersection, continue the CRT posts for two posts beyond where the curved pieces flatten out. You'll have to count up the number of CRT posts in the field.

Attached
US52_CurvedGuardrailAtSideRoad.pdf is a highly modified detail of past installations. Most of the information has been stripped out as the tables didn't cover this large of a radius. The CRT hole spacing is borrowed from the current BA-211.

You mentioned
over the phone that because of the severe drop off, additional actions were being added in an attempt to extend the 10:1 grading at least 3' behind the face of guardrail, with the potential of 4' before it broke to 2:1 or steeper. Normally we would introduce longer posts for that situation, but since these are CRT posts, I don't think that makes sense to do so. Brian and Dan may have a comment on this.

I'm out of the
office tomorrow and on the road Monday, so I'll be happy to check back in on Tuesday to see what Brian, Dan, and yourself decided to do for these installations, including any necessary changes to the curved and VT lengths.

Image 806 is where we plan to
begin the ET, which is 802 plus 25' as you note. The side road this
is at is Pulpit. This is an area that rock drilling needs to occur, and
Lovewell is marking out the post locations today so yes we need to know shortly
if the post spacing changes.

I would much prefer starting the end terminal from the end of the rail shown on the attached 806 than the attached 802. It doesn't appear the end terminal even terminates within the shoulder if starting from the 802 measuring tape. I would concur that adding the additional 25' from the 802 to end up at the 806 and then start the end terminal would be the preferred option.

The trailing back around the radius of the side road throws me a bit here. Looking at page D.3 of the plans, it appears that they are running normal guardrail around the corner of Pulpit Rock Road, if that is the location of these images, but I don't see a detail in the plans for laying out guardrail on that tight of a radius. We typically use a short radius guardrail detail that has different posts and spacing at that kind of a radius.

I'm sure you're waiting on an answer, so I'll talk with Brian this afternoon. In the meantime, can you provide the side road these pictures were taken at if it isn't Pulpit/Madison, both of which have potentially the same issue.

The guardrail layout is a long section that extends from
a bridge endpoint on the trailing end back around a radius of a side road. The section shown in the attachments is within the VT-2 section with an additional 25' added. If we didn't add the 25' the ET section would end up flaring toward the side road. Is this acceptable?

These pictures show the last two sections extending the
VT-2 by 25'.The ET would then be in the tangent paved shoulder area.

Attachment: http://mwrsf-qa.unl.edu/attachments/16a1a95a1f898971310814814569c13c.zip

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Response
Date: 08-07-2015

A few years ago, MwRSF conducted a research study for the Wisconsin DOT. This effort explored the performance of W-beam short-radius guardrail systems under TL-2 impact conditions with larger radii. See the link to access a copy of the report. Also, note the Chapters & Sections that are recommended for reading.

http://mwrsf.unl.edu/reportresult.php?reportId=288&search-textbox=radius

See page 199 or PDF page 213!

See Chapter 12 – page 205 – PDF page 219!

See Chapter 13 – page 223 – PDF page 237!

See Chapter 14 – page 227 – PDF page 241!

From this simulation effort, it was determined that rail heights greater than 27 in. and up to 31 in. would improve barrier performance for pickup truck impacts. Although no small car simulations were performed, there is concern that a 31 in. rail height could accentuate small car underride. As such, it was believed that a 29 in. rail height may still provide improved performance for pickup truck impacts but reduce concerns for small car underride. In the absence of an actual crash testing program at TL-2, MwRSF personnel would lean toward the use of a 29 in. rail height versus a 27 in. rail height based on the best available information and results from this study. Of course, the only true evaluation of safety performance would be through full-scale crash testing.

Second, the study noted that blockouts on posts around the radius contributed to improved vehicle capture by better maintaining adequate rail height. Blockouts also showed an ability to reduce vehicle to post contact. Further, CRTs were simulated around the nose through the tangent sections. As such, it would be recommended to maintain the CRTs throughout the entire curve and into tangent for any larger radius system that is implemented.

It should also be noted that the simulation effort was performed with level terrain behind the barrier system. Your real-world scenario will likely feature a gradual slope behind the barrier system for some distance, followed by a steeper slope. Barrier performance can be greatly affected by the presence of various slopes behind the actual barrier. Thus, it is recommended to provide a gentle slope behind the barrier using as much lateral distance as feasibly possible.

Again, these thoughts are provided based on our best available information as well as the research findings from the recent simulation effort. If you have any questions regarding this information, please feel free to contact either myself or my included colleagues at your convenience. Thanks!

Non-Proprietary Bullnose Thrie-Beam System

Question
Date: 08-17-2015

I was in Virginia a week ago and saw the Bullnose Thrie Beam in a gore. They had several.

While I have seen several in medians, this was the first time I saw one in a gore.

Attached are some pictures from Google of the location.

Are the Midwest states doing this design in gores.

Any pointers for this application, concerns?

What are your thoughts?

Response
Date: 08-17-2015

Thanks for the information. I want to give some background. As you know, MwRSF developed and tested this system in the late 90s. At that time, we tested a narrow system with parallel sides and requested eligibility of two wider alternatives with parallel sides to accommodate different median/hazard widths. During the review process, Dick Powers suggested an alternative design that included non-parallel sides (i.e, the back-side rail flaring away from the front rail that runs parallel with the traveled way. As such, the FHWA letter included an alternative layout that included Dick's suggested variation that is similar to the front of the system contained in your photographs.

Again, the as-tested configuration utilized parallel sides. The as-tested design could be used to shield the hazards shown therein and would result in shallower impact angles along the sides of the bullnose. The currently-depicted dual flares would seem to potentially increase approach angles for 1 or both sides.

In such scenarios, I would always prefer to use the bullnose in a parallel configuration if site conditions allow. However, the Power's alternative may allow for a flared version to be used in these settings when traffic is on both sides. One potential item to consider is the effect of flare angle on sides and resulting increased I.S. Historically speaking, there has not been considerable crash testing performed on crashworthy systems that are now installed with the maximum allowable flare angle provided in the AASHTO RDG for highway and WZ applications. We performed some testing on a flared MGS many years ago. However, PCBs/TCBs and some other devices may not always have had the allowable flare built into the testing program on the front end. That topic may be something to reconsider moving forward under MASH.

I have also copied Bob on this reply as he was largely responsible for the original bullnose system. He has fielded the majority of the bullnose implementation questions and may be able to provide additional input into this special scenario. Please let me know if you have any questions regarding the information provided thus far. Thanks!

Response
Date: 08-18-2015

Ron made most of the good points on the bullnose in gore areas, but I have a couple more.

As Ron noted, there are concerns with flaring the bullnose sides on a couple of levels. First, is the increased IS issue Ron noted below. Second, is that we don't want the flaring of the bullnose to negatively affect the capture and energy absorption of the system. Thus we have typically not recommended flaring of the bullnose prior to post no. 5 on each side. Dick Powers did approve a flared version as noted below, but that was intended for medians and not necessarily gore areas where the traffic on both sides of the bullnose is in the same direction.

Also because the system in this application would have traffic in the same direction on both sides, the guardrail splicing would be different than a median bullnose installation in that the overlap of the splice on the left side of the system would be reversed.

We have addressed this issue in the past through our Pooled Fund Consulting efforts and I have attached those responses as well for you to review as they contain some additional thoughts.

http://mwrsf-qa.unl.edu/view.php?id=927

http://mwrsf-qa.unl.edu/view.php?id=909

As Ron noted, if you have any questions, let us know.

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grading behind a MGS transition

Question
State: WI
Date: 08-21-2015

If we have the 2' of relative flat grading behind the post of the MGS thrie beam transition, How steep can the slope behind the 2' of flat grading? I assume a 2:1 would be acceptable.

Response
Date: 08-21-2015
A 2:1 slope located 2 ft behind the posts (on level grading) should not cause any adverse affects to the performance of the guardrail transition.

Vertical Taper Rate for Concrete Barrier

Question
Date: 08-26-2015

I have been
searching through various standards and reports trying to find documentation on acceptable vertical flare rates when transitioning from 32" high safety shape barrier to 42" safety shape barrier, or adjacent to thrie beam structure connections to 42" bridge rail . As you will appreciate, many different slope ratios are used by various agencies ranging from 1.5H:1V to 10H:1V. While reviewing TRP-03-300-14, I saw the photo below of the TCB to permanent concrete barrier, which referenced TRP-03-208-10.

Based on
dimensions of the steel cap rail transition in TRP-03-208-10, the slope is 4.97:1 (1262mm run over 254mm rise), or say 5H:1V. Can you offer any comments on the rationale for this ratio to minimize snagging potential for the vertical transition from a 32" PCB (pinned) to 42" permanent barrier, and whether using 5H:1V transition for a MASH thrie beam connection to 42" bridge rail would be acceptable (ie 32" at end of bridge rail where nested thrie beam overlaps the concrete, and immediately transitioning the top of concrete upward at 5H:1V to 42" bridge rail)?

Attachment: http://mwrsf-qa.unl.edu/attachments/49c33a8d68c89e96f5eac8697b32ec40.png

Attachment: http://mwrsf-qa.unl.edu/attachments/71e2ebccb8564ba33ea8c5563e7613e3.png

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Response
Date: 08-26-2015

You are correct that in TRP-03-208-10 we utilized a 5:1 vertical taper to go from 32" to 42". The 5:1 height transition slope that we tested with the PCB transition used a steel cap to create the vertical transition flare. There is some concern that using concrete to create the 5:1 flare may increase friction and or gouging of vehicle components in the flared region. Thus, the 5:1 is an aggressive approach with some concerns for its use. However it has been adopted by many states. Use of an 8:1 slope has been commonly used as a more conservative approach.

Based on the performance of the this 5:1 flared cap, it would seem reasonable to use slopes as high as 5:1 when transitioning from 32" tall barriers up to higher heights. We could not recommend the use of these higher flares for shorter barrier heights below 32" as the potential for the vehicle to climb the flared section may increase if the starting height of the flare is lower.

For transition from heights lower than 32", the recommendation would be an 8:1 slope. We would recommend this based on the concerns noted above regarding the difference in the slope materials. In addition, we would not want to go to steeper slopes for barrier heights below 32" on the low side due to concerns for increased vehicle exposure to the slope and climb.

Response
Date: 08-27-2015

I note in the package you just sent me that the proposed Standardized AGT Buttress uses all W6x8.5 posts (with last 6 posts immediately adjacent to the concrete rail being 78" long and spaced at 18-3/4"). We are currently using the simplified steel post MGS stiffness transition with three W6x15x84" posts immediately adjacent to concrete rail spaced at 37-1/2" with next four W6x9x72" at 14-3/4", etc (per MWTSP similar to Missouri Transition to Single slope in TRP-03-210-10). Has the post configuration in the proposed Standardized AGT Buttress with all W6x8.5 posts been crash tested before? Were there concerns with the performance of the MWTSP design using the W6x15 posts, or is the proposed change to all W6x9 posts in the proposed AGT buttress being primarily driven by desire to only use one size of posts (albeit two different lengths)?

Response
Date: 08-28-2015

I have a some answers to your questions.

First , the upstream stiffness transition that was developed in TRP-03-210-10 was designed to work with a wide range of AGT designs that were tested on only the downstream end in NCHRP 350. When we conducted the crash tests of that upstream transition, we selected a very stiff AGT design adjacent to the bridge rail. The system that was selected was a Missouri AGT to a steel bridge rail with a cap rail because it was a very stiff design that would accentuate any issues with the upstream end of the transition.

That said, that Missouri AGT with W6x15 posts at 37.5" spacing was never tested and evaluated with a concrete parapet. However, you are connecting to a concrete parapet. Thus, you may want to consider designing the downstream end of the transition to comply with a previously tested and approved transition for the downstream end. We provided guidance for adapting existing transitions for use with the upstream end that we developed in the project report (TRP-03-210-10). Thus you could adapt one of those designs, or we could assist you in adapting something else that you prefer to use.

Another potential option would be to use the W6x15 posts at half post spacing that you originally had and connect to a concrete parapet. I went through some old correspondence that we had with Iowa. They requested guidance on using the W6x15 posts at half post spacing and connecting to a vertical concrete parapet. We had replied then that the system was likely to work, but that there were concerns with slightly more thrie beam deflection relative to the more rigid bridge system. However, we believed that the increased deflection would pose minimal risk for wheel snag, excessive barrier deflections, or vehicle pocketing. This design would also require flaring of the end of the parapet to prevent snag. Use of this design would likely require further investigation into the relative deflection and snag potential of similar systems to justify its use. We recommended to Iowa to run this by FHWA as well. The upcoming testing on the standardized parapet may shed light on that as well.

http://mwrsf-qa.unl.edu/view.php?id=676

The transition being used for the standardized parapet is a previously NCHRP 350 tested AGT design (with a curb) that was done for Iowa for connection to concrete bridge rail. It was selected for evaluation of the standardized parapet because it is among the least stiff of the approved transitions and it has been shown to be sensitive to use without or without a curb. Thus, we believed it will provide a critical test of the standardized parapet such that we can use it with all previously tested AGT's. The Iowa transition was also tested to MASH with the curb and passed during NCHRP 22-14.

http://mwrsf.unl.edu/researchhub/files/Report148/TRP-03-175-06.pdf

http://mwrsf.unl.edu/researchhub/files/Report61/TRP-03-69-98.pdf

So the Iowa transition has been around for a while, and the use of the W6x9 posts was not originally intended for simplification of inventories. However, during the development of the upstream stiffness transition, we were asked to considered simplified post configurations to limit inventories, and the Iowa transition was part of that thought process.

Minimal guardrail lengths to develop tension over culvert

Question
State: IA

Date: 08-26-2015

We have a
situation where guardrail is being attached to a short bridge and is acting as
the bridge rail, though we also see similar issues with culverts, and there is
a question regarding how long of an installation is needed on either side of
the bridge or culvert to develop enough tension in the system. I can somewhat
gather that for a TL-3 system, 62.5' is the minimum needed on the upstream and
downstream ends beyond the obstacle, assuming a typical end terminal as shown
in the attached standards. Please confirm or provide guidance.

The project in
question is leaning towards a TL-2 installation (gravel road, very low volume)
and the same question remains; what would the minimum combined length of w-beam
and end terminal typically need to be to correctly anchor and tension the
system for a TL-2 installation?

For a very
generic layout, see attached.

Thanks for your
assistance!

Attachment: http://mwrsf-qa.unl.edu/attachments/d0b01a764b637e917d16273edb2ea2f7.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/3747c7df94a35634eec4259539f29241.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/d2ed67ce4498074a7b14264d3d3d3d6a.pdf

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Response
Date: 08-26-2015

We have looked into minimum system length for the MGS in the past as well as made recommendation regarding minimum system lengths for long span guardrails over culverts with reduced span lengths from the 25 ft unsupported span that was crashed tested. There are three factors that come into play.

1. Lateral Extent of the Area of Concern, the Guardrail Runout Length, and length-of-need (LON) as determined by the Roadside Design Guide (RDG) must be considered when factoring in minimum system lengths. Often a guardrail system may be able to redirect vehicles at lengths less that that required to adequately shield the hazard. As such, determination of length should start here. If the runout lengths are short enough to consider s shorter barrier system, then a couple of other factors need to be considered.

2. Minimum system length required for capture and redirection.

The MGS Long-Span Guardrail System over culverts was successfully crash tested and evaluated according to the Test Level 3 (TL-3) safety performance criteria found in MASH. For this testing program, the overall system length was 175 ft, including 75 ft of tangent rail upstream from the long span, a 25-ft long unsupported length, and 75 ft of tangent rail downstream from the long span. As part of the final recommendations, MwRSF had noted to provide a minimum “tangent" guardrail length adjacent to the unsupported length of 62.5 ft. While your installation does not appear to use the long span system, similar logic may apply.

A recent MASH crash testing program on a minimum length version of the MGS suggests that there may reason to consider potentially reducing the 75-ft total guardrail length on the upstream and downstream ends of MGS Long-Span Guardrail System.

In the minimum length study, computer simulation and full-scale testing indicated that a 75' long MGS system would be capable of redirecting a 2270P vehicle under the MASH TL-3 impact conditions. Test no. MGSMIN-1, was performed on the 75-ft long MGS with a top rail mounting height of 31 in. A 4,956-lb pickup truck impacted the barrier system at a speed of 63.1 mph and at an angle of 24.9 degrees. The test results met all of the MASH safety requirements for test designation no. 3-11. The tested system had a total of 13 posts.

A performance comparison was conducted between 75-ft MGS (test no. MGSMIN-1) and 175-ft MGS. The dynamic deflection for the 175-ft (53.3-m) MGS was slightly higher than observed for the shortened system, but this difference could be due to variations in soil compaction between tests. The working width was nearly indistinguishable. In general, the 75-ft MGS in test no. MGSMIN-1 performed as desired and closely resembled the standard 175-ft MGS.

A second study regarding downstream anchoring of the MGS found that the MGS would successfully redirect 2270P vehicles impacting at 6 posts or more upstream of the end of the system for a MASH TL-3 impact on a 175-ft long MGS system.

Based on previous testing and the results of test no. MGSMIN-1, MASH TL-3 vehicles impacting between post nos. 3 and 8 of the 75-ft long system should be redirected. Vehicles impacting downstream of post no. 8 may be redirected, but the system would also be expected to gate based on the downstream anchor research.

Based on the MASH 2270P test into the MGS Minimum Length System, we believe that the MGS Long-Span Guardrail System would likely have performed in an acceptable manner with 62.5 ft of rail on the upstream and downstream ends, thus resulting in an overall system length of 150 ft. A 62.5-ft long tangent length adjacent to the unsupported length would still provide adequate space to incorporate a 37.5 ft or 50 ft long energy-absorbing guardrail end terminal.

For unsupported lengths of 18.75 ft and 12.5 ft, it would seem reasonable to consider a reduction in the required guardrail length both upstream and downstream from the unsupported length using the test information and arguments noted above. For two missing posts or an unsupported length of 18.75 ft, we believe that the upstream and downstream guardrail lengths likely could be 56.25 ft each with a minimum overall system length of 131.25 ft. For one missing post or an unsupported length of 12.5 ft, we believe that the upstream and downstream guardrail lengths likely could be 50 ft each with a minimum overall system length of 112.5 ft. However, we believe that the three CRT posts still would be required on the upstream and downstream ends of the 18.75 ft and 12.5 ft long unsupported lengths. In addition, one would need to discuss with and likely obtain approval from the manufacturers as to whether they would allow three CRTs to be used within the last 12.5 ft of a 50-ft long guardrail terminal.

If one were to follow the logic used above and consider the situation of no missing posts (i.e., 6.25 ft post spacing throughout), the upstream and downstream ends would be reduced by 6.25 ft each and include the interior 6.25 ft long span in the middle of the system. As a result, the overall system length would be 43.25 ft + 6.25 ft + 43.25 ft for a total of 92.75 ft. As noted above, MwRSF recently crash tested a 75-ft long version of the MGS with satisfactory results, effectively configured with two 37.5-ft long guardrail segments with tensile anchorage devices and placed end-to-end. This corresponds to the situation in the schematic you sent and would provide conservative guidance on minimum length for the guardrail system over the culvert. Thus, this would correspond to 43.25 feet of barrier on the upstream and downstream end of the system. However, some terminals may require a 50 ft length for installation.

Of course, it should be noted that these design modifications are based on engineering judgment combined with the unpublished results from the MGS Minimum Length System crash testing program. In addition, the opinions noted above are based on the assumption that the currently-available proprietary guardrail end terminals would provide comparable tensile anchorage for the MGS as provided by the common tensile anchorage system using in the MwRSF crash testing program (i.e., two steel foundation tubes, one channel strut, one cable anchor with bearing plate, and BCT posts at positions 1 and 2 on each end). Although we are confident that the modifications noted above would provide acceptable performance, the only sure means to fully determine the safety performance of a barrier system is through the use of full-scale vehicle crash testing.

3. Sufficient length for compression based terminal operation must be considered as well.

To the best of our knowledge, the shortest installation lengths for compression based terminal testing was conducted on 131.25-ft long system. We believe that this length could be shortened some based on our current knowledge of guardrail compression forces. We have used a reduction in longitudinal rail force of approximately 1-1.2 kips at each post in a guardrail due to the connection between the post and the rail. Current terminal designs tend to have impact head compressive forces that average about 15 kips. This would mean that a minimum of 12-13 posts would be needed to develop the compression load. Of course the end terminal takes out some posts during its compression. However, most of the velocity drop occurs in the first 25-31.25 feet of the compression. Thus, we can assume that if we allow for 31.25 ft of compression and 13 posts to develop the compressive load, an estimated minimum system length for the development of the end terminal compressive loads would be 112.5 ft (13*6.25+31.25).

Because we did not have additional funds or terminal testing and evaluation in the above research, we would recommend minimum system lengths of 112.5 ft in order to be conservative.

One last factor to consider with the use of terminals on these short systems is the deflection of the terminal when impacted on the end relative to the hazard. As noted above, we believe that the system will redirect the vehicle beginning at post no. 3 in the system. However, in an end on impact of the terminal, the vehicle may deflect down the rail between 37.5 ft – 50 ft. Thus, hazards near the back of the guardrail may still be impacted by end terminal impacts even when they are in the redirective area of the guardrail system. As such, you have to consider both the deflection of the terminal, the redirective region of the LON, and the runout length considerations when designing the placement of short guardrail system.

Thus, based on the analysis and review of previous research, it seems that the minimum length of the installation may be limited to 112.5 ft based the function of the compression terminals. In answer to your question with respect to guardrail over culvert, we would recommend that the overall system length be at least 112.5 ft, and that a minimum of 43.25 ft be required on the upstream and downstream ends of the system. Again, consideration of Lateral Extent of the Area of Concern, the Guardrail Runout Length, and length-of-need (LON)may trump this guidance.

Thanks
