Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

01-01-2016 to 04-01-2016

Break away work zone sign

Question

State: WI
Date: 01-04-2016

I have been asked to review a sign design. I believe that TTI tested the design to NCHRP 350. But they only ran the truck tests. They way I see the testing and what TxDOT has in their drawings, I'm concerned about the sand bags and 4" on a 5ft chord issue.

I have indicated that the drawings may not meet the MASH requirements because they would likely need a small car test.

If you could review this it would help.

Thanks

Attachment: http://mwrsf-qa.unl.edu/attachments/ad6c23b5078c487bd177ee4a51832cd9.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/44a8785312784bba4977d52025c584b6.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/ab1ddf84b80b8c9b2438eb2ba56ffb7e.pdf

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Response

Date: 01-11-2016

The sign support from test 453360-3 is documented in TTI Report No. 5388-1F.

A temporary, skid-mounted temporary sign with a mounting height of 7 ft that was recently tested to MASH with a small car at the Texas A&M Transportation Institute had an unsuccessful performance due to excessive occupant impact velocity (test no. 467824-2 in report 0-6782-2). However, with some modifications to the breakaway features of the system, the system performed satisfactorily with the both the small car (test 467824-4) and pickup truck (test 467824-3). Also, additional guidelines were developed based on wind loads to design temporary sign systems to select sign supports based on the sign size desired.

This MASH testing indicated that the breakaway mechanism of sign supports is critical to a successful small car performance. As such, it unknown how the small car would interact with the particular sign support you are inquiring about and how that system would break away. An alternative would be to utilize the skid-mounted, temporary sign support evaluated to MASH safety performance criteria and the guidelines based on wind loads (TTI Report No. 0-6782-2).

Attachment: http://mwrsf-qa.unl.edu/attachments/d71876eeac78fc01c5a5eb69cb8cee78.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/e234f579d647b40851beea7211cc24c5.pdf

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Signs

Question

State: WI
Date: 01-04-2016

I have been asked to review some more signs for crashworthiness. Please review the attached signs and provide comments.

Attachment: http://mwrsf-qa.unl.edu/attachments/e9a7f75c02928bd050e1d4bcd099d680.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/ecbda2783a4c0a73e1348c72c7d3cc61.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/318cf1689cb46784778965403992e32f.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/d176bc23345d5c6f858e026a83a18953.jpg

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Response

Date: 01-11-2016

There are some design variations between the photos shown. It does not appear that there are breakaway holes or another breakaway mechanism in the posts in any of the designs. With a variety of signs and mounting heights, as well as multiple vertical wood posts and sometimes horizontal wood beams, it is unknown how the system will fracture and deform upon impact. Sign systems can be sensitive to minor design changes that may present a problem for windshield and occupant compartment deformation and penetration, depending on the how the system fractures upon impact. At this time, we would recommend crash testing to evaluate crashworthiness of this sign.

Long Span Question

Question

State: WI
Date: 12-23-2015

Please review questions in the PDF. I have a region that does not like having water flow over the top of the box culvert. They are wondering if it is possible to do something like what I have attached.

What I have is a situation where there is a significant elevation difference between the shoulder by the beam guard long span and where the culvert headwall is located. The region staff wants to keep the headwall to prevent water from flowing over the head wall (I don't know why.).

The truck or car can only drop so much during an impact into a long span. For the sake of illustration, I'll pick an imaginary number of 2' of vehicle drop during a crash test. If the culverts headwall is 3' below the shoulder surface, the vehicle cannot interact with the headwall during an impact. It should be O.K. for the head wall to remain in place.

The question becomes, What is the truck or car's drop during an impact into a long-span. I pick the imaginary number of 2' out of a hat for the sake of illustration. But you smart people have the slow motion cameras, computer models, and other sorts of wizardry and probably could actually come up with a more accurate, scientific guess than I could.

Attachment: http://mwrsf-qa.unl.edu/attachments/5d6b008e826752b7be41b471c92b47dd.pdf

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Response

Date: 01-04-2016

If I understand correctly, you are wanting to install the long span at an offset from the head wall larger than what we tested, but with a slope and drop to the head wall behind it.

I don't see this as much of an issue as the long span was tested with a vertical drop off approximately 34" behind the face of the rail. I believe that we require the long span to have 24" behind the posts prior to a 2:1 slope. Assuming that you have continue that grading behind the unsupported span prior to the slope and head wall, I don't see this installation as more severe than what was tested. I believe that the slope to the head wall would only provide additional vertical support to the vehicle and improve stability.

If you plan to start the slope closer to the rail face we may need to discuss it further.

Response

Date: 01-05-2016

2' of grading behind the post is not a problem. But where the span is plan to be the 2:1 starts close to the back of rail.

Attachment: http://mwrsf-qa.unl.edu/attachments/62c828a478a7e0f5fd6cee01be5d938c.jpg

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Response

Date: 01-06-2016

I don't believe we can recommend the system as shown due to concerns with the drop of the slope relative to the rail position in the unsupported span.

In the long span testing, the posts were aligned with the back of the post flush with the face of the headwall. This placed the face of the guardrail in the unsupported span area approximately 34" in front of the headwall. Thus, during an impact in the unsupported span, the ground was supporting the wheels of the vehicle for approximately 3' prior to the vehicle dropping behind the headwall.

In the installation you have shown below, the slope begins directly behind the face of the guardrail. This would allow wheel drop much earlier than the tested system. Beginning of the wheel drop sooner may allow the vehicle to fall farther as it extends over the culvert which could compromise vehicle capture, vehicle stability, and cause issues with the vehicle climbing back over the headwall as the vehicle is redirected.

As such, we cannot recommend the installation shown below.

Portable Barrier and Rock Fall

Question

State: MN
Date: 01-05-2016

We have a temporary work zone situation, where the designer would like to use portable concrete barrier for a potential rock fall concern. Most of the rock fall (95%) will be contained within the ditch section, but 5% could make it to the traveled lane.

The designer understands the concern over pinning barrier on the opposite side of traffic. But feel that it will be placed far enough away from the 30 mph work zone traffic to be a concern.

This is an interesting question and would appreciated you guidance on how the barrier would react to the forces describe below.

Currently our rock fall analysis is finding that a maximum of 5000 ft/lbs of total kinetic energy (translational plus rotational) could impact the barrier at 90°, typically at the base of the barrier or as high as 1' above the base of the barrier. Would J-barrier, which is pinned on the side being impacted, be able to withhold that impact? If it was not pinned what would the deflection be?

Response

Date: 01-25-2016

I have briefly reviewed the inquiry regarding the rockfall concerns near roadways and the use of PCBs for debris containment. Below, it was noted that the impact energy for the design scenario would be approximately 5 kip-ft (6.8 kJ) when applied perpendicular to a portable concrete barrier (PCB) system. Initially, this kinetic energy seems to be rather low. However, I would like to give some perspective to a known quantity that is similar to the roadside safety community.

Let us for the moment consider the AASHTO MASH impact conditions for evaluating roadside barrier hardware. A Test Level 1 (TL-1) impact condition involving a small car would provide an impact severity of 14.0 kip-ft (18.9 kJ), which consists of a 2,425-lb sedan striking at 31.1 mph and 25 degrees. Normally, a TL-1 condition would also include a 5,000-lb pickup truck impact event as well, which is of course the strength test. Most PCB systems are tested and evaluated at a TL-3 condition, which includes a 62 mph (100 kph) impact speed at a 25-degree angle. PCBs are designed to meet the TL-3 impact condition, which provides an impact severity of 115 kip-ft (156 kJ) for the pickup truck with up to 80 in. of dynamic displacement.

It should be noted that the design condition of 5 kip-ft (6.8 kJ) is significantly lower than that provided by the lower of two impact scenarios for TL-1. The design condition would be somewhat represented by a 2,425-lb small car impact the PCBs at a speed of 18.6 mph (30 kph) and 25 degrees.

Based on this fundamental comparison and knowledge of PCB performance under impact events, I believe that existing PCB systems could contain the rockfall material under design conditions when un-pinned to a foundation. Further, dynamic deflections would be estimated as 3.5 in. for this design condition.

In addition, I also reviewed a research report that was conducted by the University of Akron for the Ohio DOT, titled Rockfall Concrete Barrier Evaluation and Design Criteria. Within this study, the OHIO DOT PCB system was evaluated. Researchers determined that the Ohio PCB system could contain rockfall material with a kinetic energy of 18.2 kip-ft (24.7 kJ) with observed sliding of approximately 12 to 15 in. Since the Ohio PCB is similar to that used by the MnDOT, this research confirms that a rockfall slide with material having 5 kip-ft of energy would be safely contained. For a reduced impact energy, deflections may be estimated as 3.3 to 4.1 in.

Using both methods, I can confidently state that the deflections should be less than 6 to 8 in. when un-pinned and connections pulled taught. Also, I do not see a need to pin the barriers to a foundation due to this low estimated deflection.

Please let me know if you have any questions or comments regarding the information contained above. Thanks!

MGS Long Span Deflections

Question

State: IA
Date: 01-05-2016

I'm working on a project where the designer would like to consider whether to use a long-span system (attached eba211.pdf) or connect to the culvert (attached eba210.pdf). There are a number of culvert locations containing a mixture of headwall heights and widths and the designer has inquired about the deflection distances for each of the long-span layouts. I've attached what I could find from recent research, as well as a thought on how we might find another, but I don't have any inclination as to what deflection distances should be used for the two post layout.

Please review and comment on what I could find and provide guidance on what I couldn't.

As always, thank you!

Attachment: http://mwrsf-qa.unl.edu/attachments/d5d6020093c77313c249f96d08097d9f.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/0c145a28afa0c2862c15c6c4f4f49788.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/f8a2aa8c66f6a8a855779a766cbad5a9.pdf

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Response

Date: 01-07-2016

We have not done any detailed analysis of intermediate working widths for the MGS system. However, we do have data points at the 25 ft unsupported span as you noted in your table and data points for the standard MGS system. It seems reasonable that our best path to estimating these working width values would be to linearly interpolate between the know values.

I have added a table below that does just that. For the MGS with standard post spacing, I have assumed a working width of 60.3 in. based on the highest working width value we have observed for an MGS system (this is test MGSDF-1 of the MGS with Douglas Fir posts). This should provide a conservative starting value and better represent the CRT posts used in the long span in terms of system stiffness and barrier deflections. I then used your suggested upper bound of 96 in. based on the results of test no. LSC-1. The results come out relatively nice in that the recommended working widths are essentially even foot values.

While this analysis is not substitute for testing, it should provide you with reasonable guidance given our best available information.

Thanks

Attachment: http://mwrsf-qa.unl.edu/attachments/41089e587c27ab9118e7d47a08c0d4ff.jpg

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Response

Date: 01-08-2016

Looks like a reasonable approach. Thank you.

As a follow-up question – in the TRP-03-187-07 report on page 101 where it mentions a 2" max height, why would this value be lower than the typical 4" height where we start to view an object as a hazard per Roadside Design Guide?

Response

Date: 01-09-2016

The 4" you mention below is a stub height requirement that is intended to prevent/reduce damage to the vehicle undercarriage and the underside of the occupant compartment if the vehicle drives over.

The 2" height for the headwall noted in the long span research is listed for a different reason. The is some concern that interaction of the impacting vehicle with headwalls taller than 2" may affect vehicle stability it is redirected. Previous research with curbs have found that 2" curbs affect vehicle motion significantly less than 4" or 6" curbs. Thus, we limited the headwall extension above grade to 2" or less to improve vehicle stability.

Hope that answers your question. Let me know if you need anything else.

Thrie-beam connection to twin steel tube bridge railing

Question

State: OH
Date: 01-06-2016

I believe you worked with our group on standard drawing MGS-3.1. From past e-mails, I see that she used Iowa's standard drawing as a template. I looked at the Iowa website, but didn't see a connection detail like the one shown below. This is the MGS thrie-beam connection to twin steel tube bridge railing. I received a call from a contractor asking if the washers (highlighted below) are necessary. Do you know if this connection detail was based on research from Midwest Pooled Fund or another state? I would normally assume that the washers are necessary, but this particular contractor does a large amount of work for us and his question makes me wonder whether I'm missing something.

Thanks,

http://www.dot.state.oh.us/Divisions/Engineering/Roadway/DesignStandards/roadway/Standard%20Construct%20Draw
3.1_7-18-2014.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/31c23aa168a137dc8cc1e93d7b318036.jpg

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Response

Date: 01-07-2016

This issue has come up in the past regarding the use of the washers underneath the end shoe and nested thrie beam attachment. We have been recommending that washers be used based on the size and length of the slots in the current thrie beam end shoe pieces. We have some concerns that the size of the slots in the end shoe might allow the nuts to pull through. TTI has been using washers as well based on similar concerns. I believe that they have been using rectangular washers, but I do not have the spec.

In our recent tests, we have been using 5/8" F844 washers on the backside of the end shoe. That has worked fine to date. These are more standard parts and are easier to obtain.

http://www.portlandbolt.com/products/washers/standard-flat/

Thanks

Attachment: http://mwrsf-qa.unl.edu/attachments/305ffe7ebae49c4a49642d43ea1fe963.jpg

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TRP-03-277-14

Question

State: WI
Date: 01-14-2016

Can I get the CADD for the hardware in TRP-03-277-14

The box that we are planning to attach to had some concrete surface issues. Is it possible to increase the embedment of the connection to account for concrete surface issues?

Response

Date: 01-22-2016
I have attached a ZIP file containing CADD file of the final system drawings.

If your culvert has concrete surface damage, increase the embedment depth of the anchor rods by the depth of the surface damage:

New EMB = Design EMB + Surface Damage Depth.

Attachment: http://mwrsf-qa.unl.edu/attachments/ba9431ed592a46c36f93ae644babf4ee.zip

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TRP-03-226-10

Question

State: WI
Date: 01-22-2016

We are interested in use the MGS bridge rail in TRP-03-226-10

Is it possible to install blocks between the post and rail to minimize the likelihood of a plow snagging the bolt on the deck?

My guess adding a block would likely improve performance. It would also get rid of the possibility of rail tearing on the steel post.

Response

Date: 01-25-2016

We have looked into this question previously. See below.

http://mwrsf-qa.unl.edu/view.php?id=760

I don't believe that the response has changed.

Thanks

Triple Blockouts

Question
State: OH
Date: 12-01-2015

We have a guardrail replacement project that is wrapping up on Interstate 70 in Columbus. Because of an inlet, the contractor has used triple 12" blockouts at several approach locations to a structure. The inlet is in poor condition and the project engineer is concerned that the structure is too weak to properly support the load of an impact. From past questions I see that a triple 8" blockout can be used under certain conditions. Is a triple 12" blockout ever an option? If so, can it be used at a transition section such as this? Could some combination of triple blockout and steel post attached to the inlet be used? Is the only option in this case to remove the old inlet and use steel posts attached to the top of the new inlet? Thanks!
Attachment: http://mwrsf-qa.unl.edu/attachments/cb4b50311442ec93e99e898dbcb36188.JPG

Attachment: http://mwrsf-qa.unl.edu/attachments/acd0ce95fff7f85ff91908655a94eceb.JPG

Attachment: http://mwrsf-qa.unl.edu/attachments/2134370d30a7a9f7d5859cff9cac2a9e.JPG

Attachment: http://mwrsf-qa.unl.edu/attachments/cd11ed3a2b8e7b5f549c8b978ca7078d.JPG

Attachment: http://mwrsf-qa.unl.edu/attachments/c5c6cc210086ce2a99b5771a37c00c48.JPG

Response
Date: 01-25-2016
In the past, we have recommended no more than one triple 8" blockout installation very 50' for guardrail installations. This is based on concerns that the ability of the triple blockout to transmit load to the post would be compromised for large deflections. With regards to transitions, we have used a similar rationale and have limited the installation of triple blockouts to a single post in the transition at limited locations. For your installation shown, we believe that the number of consecutive triple blockouts is likely too many.

The MGS utilizes 12-in.deep blocks for standard applications as well as for special applications. For example, the MGS long span design utilizes one 12-in. wood block with three CRT posts instead of two stacked 12-in. deep blocks. For the MGS, it would seem reasonable that the use of two 12-in. deep stacked blocks could be accommodated at a few locations as well, thus also resulting in a rail offset of 24 in. However, it is uncertain as to whether the use of two 12-in. deep blocks may be too excessive when used continuously with the MGS.

Thus, based on previous testing of systems with deep or extended blockouts and an analysis of the contact lengths of typical MGS testing, MwRSF would recommend the following:

1. Double standard blockouts or combinations of blockouts up to 16-in. deep may be used continuously in a guardrail system.
2. Triple standard blockouts or combinations of blockouts up to 24-in. deep should be limited to one in any 75 ft of guardrail.

There is currently a problem statement in the Year 27 Pooled Fund Program to address this issue specifically, " Additional Options for Post Conflicts within the Approach Guardrail Transition".

With respect to attachment to the top of the inlet, that would depend on the connection to the inlet and the relative stiffness of that post configuration. One would also need to be sure that the inlet attachment had sufficient structural capacity as the short, stiff post may overload the top of the inlet.

MGS Rail stiffness transition with Curb

Question

State: MO
Date: 01-26-2016

In the email below is a question on the amount of curb required along the transition area from a bridge end thru the thrie beam, transition section and into the MGS. Basically, our new bridge end design will likely have curb extended into the transition section area or just past that into the nested MGS. But as we read report TRP-03-291-14, the design has curbing that ends either before post 11 OR extends past the first section of MGS that has been double nested with W beam to between posts 4 and 5 (Fig 54, pg 74). The question comes, is there any way to have a curb from the bridge end that ends within the transition section or within the nested MGS section that has been found as acceptable? This only saves a few feet of curbing at each approach, but multiplied over and over at locations around the state, it adds up.

The recommendations on pages 135 and 136 include nesting the W-beam rail at the upstream end of the W-to-thrie transition AND carries the curb past that nested section into the normal MGS. If that is the option, we will go with it. But, if there is some other design option that allows curbing to end in the thrie-to-W beam transition area, please let us know where to look for recommendations and design guidance. If there is nothing, please confirm that the design in Figure 54 with curb extended into the normal MGS rail in report TRP-03-291-14 is current and the best practice for this transitional area with curb.

Thanks for the help with this question and for providing a written reply that we can share with other staff to assure we have considered the options with lesser curb. Please call or email with questions.

Through my research and understanding of the MGS guardrail system and transitions related to the system, I have ran across report TRP-03-291-14 “Dynamic Evaluation of MGS Stiffness Transition with Curb." This report details the testing of MGS rail and the stiffness transition to thrie-beam where a mountable curb exists. The report summary indicates the need for nesting of the first 12'-6" section of MGS rail when curb extends upstream of post No. 11 (beginning of the thrie-beam transition section to w beam). The report also goes on to indicate that if curb is present beyond post No. 11 then it should be extended to the end of the 12'-6" stiffness transition. This distance is stated to be approximately 37.5' from the vertical parapet connection (Bridge end).

My question for MwRSF is – is this still a valid issue? Has there been any additional developments related to stiffness transitions with curb that does not require extending the curb 37.5' from the bridge end. MoDOT has recently modified our designs for Bridge Approach Slabs and Concrete Approach pavements which will cause the curb to fall within this zone. If we can just double nest the first 12'-6" of w-beam MGS then keep our standard curb lengths that's great, but if we must extend that curb to the end of the double nested MGS section that is something I would like to see in writing from MwRSF. This will affect every major road we modify if this is the case.

Response

Date: 01-29-2016

The introduction or removal of roadway geometric features has shown to cause critical differences in safety performances for some barrier systems. Approach guardrail transitions (AGTs) in particular have been shown to fail crash tests after a curb is either introduced or removed from otherwise crashworthy systems, as demonstrated in the noted report (TRP-03-291-14) and in other AGT tests. In addition, the termination or transition between adjacent features can cause their own issues with system crashworthiness. Thus, to be conservative and avoid any potential snag and/or stability issues, we have recommended that the curb (or lack thereof) be consistent through the critical area of this MGS stiffness transition – specifically the area under and upstream of the W-to-thrie transition segment. That has produced the following recommendations:

- If you want to terminate the curb within the AGT system, we recommend doing so prior to the W-to-thrie transition element (or as you stated, before post 11 – from TRP-03-291-14 page 74).
- If the curb needs to be carried further than this location, we recommend carrying it past the entire AGT system. This extends another 18 ft – 9 in. to the upstream end of the nested W-beam rail (or the mid-span between post 6 and 7 from trp-03-291-14 page 74).

Terminating or transitioning within this 18.5-ft region of the AGT may be crashworthy, but we simply don't have the research or testing to confirm that.

MGS Long Span installed on 2:1 Slopes

Question

State: WI
Date: 01-26-2016

We are currently looking at options for installation of the MGS long span at the slop-break-point of a 2:1 slope.

What are your recommendations regarding this type of installation?

Response

Date: 01-26-2016

I looked through past guidance we have given regarding the MGS long span and its use adjacent to slopes.

In the past, we have fielded inquiries regarding the use of the long span at the slope break point. In those discussions, we have noted that the previous testing of the MGS with steel and wood posts suggests that there is potential to use the long span installed at the slope break point if 8 or 9 ft W6x8.5 steel posts are used or 7.5 ft SYP posts are used. Both of these options previously were evaluated to MASH and believed to be acceptable for use on 2:1 slopes. Similarly, we have recommended that the CRT posts used in the long span be extended to 7.5 ft long posts to ensure their proper breakaway function similar to the previously tested systems. All of this guidance assumed standard post spacing and assumed that the lateral offset between the back of the post and the culvert head wall and the height of the headwall relative to the unsupported span section was maintained.
It should be noted that this guidance was provided using our best engineering judgment in the absence of full-scale crash testing, computer simulation, dynamic component testing, or combination thereof. If new information becomes available, MwRSF may deem it necessary to revise this guidance.

You had inquired about the potential to a half post spacing MGS long span system in a similar manner. Previous research at MwRSF has suggested that 7' long posts at half post spacing adjacent to a 2:1 slope should perform in a similar manner to the standard posts and spacing on level terrain. Thus, from a stiffness standpoint, the use of half post spacing seems reasonable. The issue with half post spacing comes from the CRT posts. In order to maintain similar stiffness and deflection of the system, we would likely need to extend the half post spacing to the CRT's. However, the use of energy dissipating CRT posts at half post spacing may affect the function of the long span. The extra CRT's may provide similar deflection, but they may also change the behavior of the CRT's in terms of preventing pocketing and how the CRT's affect the entry and exit of the vehicle with respect to the unsupported span. Without further study, it is difficult to assess how much of an affect the half post spacing of the CRT's may have. Thus, while there is potential for a half post spacing pf the MGS long span to work it would likely require further study.

Thanks

Response

Date: 03-29-2016

I was working with you about a long-span issue the region put together some drawings (see attached). I was wondering about the holes for the CRT posts. Are they located correctly? I'll probably need to call to explain.

Attachment: http://mwrsf-qa.unl.edu/attachments/31c0984173ee05a799316bc706bcd60c.pdf

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Response

Date: 03-31-2016

We don't have a definitive answer for you question regarding the holes in the CRT post. It is true that the lower hole in the CRT post is designed to fracture to post below grade for weaker soils. However, our previous discussion recommended increased post embedment to account for the reduced soil forces due to the slope. Thus, it is currently our best guidance to leave the holes in their standard location.

While it may be possible to optimize that hole location when used adjacent to slopes, determining that hole location would require further research.

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w-beam back-up plate

Question

State: WI
Date: 01-29-2016

In TRP-03-226-10 Development of a Low-Cost Energy-Absorbing Bridge Rail, the back up plate shown is 6" wide.

I believe that at a pooled fund meeting there was some discussion about the need to make the back up plate wider. What width should the back-up plate be?

Response

Date: 01-29-2016

You are correct that recent testing as part of the MGS installed in mow strips project led us to recommend revised backup plates for weak post MGS systems using the S3x5.7 post. The use of 12-in. (305-mm) long backup plates behind the rail was recommended. The partial rail tearing observed during test no. MGSMS-1 was caused when the test vehicle impacted a post and caused it to deflect downstream and twist such that its flange contacted the bottom of the rail directly below the downstream splice bolts. Then, as the vehicle's right-front bumper and fender loaded the splice, the tear propagated to span half of the rail height. If a long backup plate had been installed at this location, the tear may have never occurred.

The original MGS bridge rail utilized 6-in. (152-mm) long backup plates at every post, including splice locations since the splice bolts are 8 in. (203 mm) apart. Unfortunately, the design drawings for the full-scale test specified 12-in. (305-mm) backup plates (taken from the non-blocked MGS drawings) instead of the 6-in. (152 mm) backup plates, and these larger backup plates could not be installed over the splice bolts, which are 8½ in. (216 mm) apart, without additional holes in the plate. As such, backup plates were not installed at locations where posts coincided with rail splices. The lack of backup plate material may have contributed to the partial rail tearing in test no. MGSMS-1. However, the tearing would have likely still occurred had 6-in. (152-mm) backup plates been utilized, because the 6-in. (152-mm) backup plates do not extend below the splice bolts where the tear initiated. Similar rail tearing has been observed in other 2270P testing on S3x5.7 (S76x8.5) weak-post guardrail systems that utilized 5â...•-in (143-mm) backup plates at all post locations.

To prevent rail tearing due to post contact near rail splices, a longer backup should be utilized to protect the rail around all posts, especially at splice locations. Therefore, the utilization of a 12-in. (305-mm) long backup plate is recommended for the MGS weak-post guardrail systems, including the original MGS bridge rail and the weak-post guardrail attached to culverts.

Since 12-in. (305-mm) long backup plates are unable to be installed at guardrail splices, holes or slots need to be cut into the backup plate to allow the guardrail bolts to pass through the plate. The backup plates could utilize the same splice bolt slot pattern that is currently punched into the ends of every guardrail segment. Utilizing this design, the backup plate could be attached to the guardrail and assembled as a part of the splice. Alternatively, a backup plate could be configured to fit over the back of assembled guardrail splices at the time of mounting the rail to a post. Under these conditions, the slots would need to be enlarged to fit around the splice bolts and nuts. Both of these design options are shown in the attached figure and should be equally effective in reducing the

risk of rail tearing.
Attachment: http://mwrsf-qa.unl.edu/attachments/c41092b4068e1c81e23d4cabf813c2ed.jpg

Weak-post guardrail on culverts installation issues

Question

State: IL
Date: 01-29-2016

We are dealing with a few installation/compatibility issues with the weak-post guardrail attachment to culvert system described in TRP-03-277-14. Without compromising MASH compatibility with the system, and without introducing a stiffness transition between the MGS and the socketed weak post system:

1. How can the weak post socket system be modified to fit culvert installations where the thickness of the top slab plus the height of the headwall (T+H) is as little as 13 inches? Note the socket would extend past the top of the culvert in these installations.

2. If this is not feasible, how can the required T+H dimension be reduced, and by how much?

3. Our standards dictate that a 6" radius be used of the bottom edge of the top culvert slab on the upstream side of the culvert. How can the weak post socket system be adapted to our rounded top slab detail? For the as-designed T+H value of 17 inches? For some reduced T+H values as noted above.

4. For precast culverts, the headwall is often part of the band or collar joining the wings and barrel of the culvert. What concerns or constraints arise?

5. Are there constraints on how close bolt holes could be placed to the barrel/headwall horizontal joint when the headwall is part of the band or collar joining the wings and precast barrel.

Please see the attached file for details

Attachment: http://mwrsf-qa.unl.edu/attachments/211f47bd7ff2608164f89afbac6d26ef.docx

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Response

Date: 01-29-2016

Questions 1 &2:

This weak post attachment to culvert design was adapted from a bridge rail design that was previously developed here at MwRSF [reference report TRP-03-226-10]. In that design, the bridge deck was only 8" thick, so the socket extended past the bottom of the deck. To anchor the base of the socket, a steel angle was bolted to the inside face of the socket and to the bottom of the deck (see pages 162 – 164 of the report). I think utilizing this concept would work perfectly for culverts with short headwalls. ½" diameter bolts can be used to attach the socket baseplate to one side of the angle, while the 3/4" diameter anchor rods should be used to anchor into the bottom of the culvert slab. The anchor rods should be increased in size because they now transfer all the load in shear (as opposed to the base plate bearing directly to the culvert headwall. The angle should be at least 3" high (matches base plate), 6" wide (to extend under the culvert), 3/8" thick (to match original bridge rail angle), and 8" long (matches base plate). Also, the 3/4" anchor rods should remain 6" apart – same as the original base plate attachment. A gusset plat similar to the one utilized in the original bridge rail should be welded in the angle to prevent deformations. Note, the exact height and/or orientation of the angle would change based on the actual height of the headwall (the angle could be flipped around to open toward the culvert for headwall heights near or just exceeding the depth of the socket).

Question 3:

We have come up with 2 reasonable options to retrofit these curved edges at the bottom of the culvert slab. Simple sketches of these options are shown in the attached file.

Option 1 utilizes a bent plate with an inside radius of 6" to match the radius of the culvert edge. This bent plate could also be cut from a pipe with an inside radius of 6". The plate would contain two holes that would allow ¾" anchor rods to be inserted through the plate and into the curved face of the culvert, thus attaching the plate to the culvert. A gusset plate would need to be used to bridge the gap and attach the outside of the plate to the inside face of the socket. The gusset plate would be welded to both the socket and the plate.

Option 2 (preferred) utilizes the steel angle concept discussed above. The only difference would be that the steel angle would need to be wide enough to extend past the curved portion of the culvert and few inches into the flat section of the slab. The ¾" dimeter anchors should be utilized to attach the angle to the flat portion of the culvert slab. Note, the angle could also be fabricated from a bent plate to obtain the desired dimensions. The same ½" diameter bolts can be used to connect the angle to the socket base plate, and a gusset should be welded within the angle/bent plate as noted above. Dimensions and orientation of the angle/bent plate would be dependent upon the culvert dimensions.

Question 4:

I believe all dimensional concerns were discussed in the previous questions and answers, so I assume that the sockets have the space needed to be mounted on the headwalls. As you have stated, if no headwall is present, then another barrier option needs to be utilized. Additionally, you would want to avoid any rebar/bolts/rods/etc. critical to joining these components together. Thus, the holes for the epoxied anchor rods should be drilled with care as to not cut through any critical connection hardware.

Question 5:

Anchor bolts/rods do lose capacity when placed adjacent to the edge of a concrete member. However, none of the anchors should be loaded in shear in a direction toward a horizontal joint. Thus, there would be minimal effect. Anchors could be placed within a few inches of a joint – as long as the concrete isn't damaged/split/spalled when the holes are drilled.
Attachment: http://mwrsf-qa.unl.edu/attachments/0b432155d6ca6af7dfacd021af2c03fa.pdf

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Response

Date: 04-05-2016

We are working on updating Illinois Department of Transportation Highway Standards related to guardrail, and have a few questions to resolve in order to finalize the draft highway standards to forward to Design and Environment.

MGS Attached to Culverts

Q1: For the MGS attached to culverts (weak post), in Option 2, provided by Scott, (see attached “socket to culvert attachment sketchs.pdf") there is a leg of the bent angle that goes under the culvert top slab radius. Because the radius at the inlet end of the top slab can be about 6 inches, the leg of the angle into the culvert could be 10 inches or so. In the original design referenced by the MGS attached to a bridge deck, the corresponding leg of the angle was just 7 inches. The thickness of the original angle with a 7 inch leg was 3/8". For a 10 inch leg should the material be thicker?

Q2: What is the minimum bend radius we should use for this bent angle? The radius used here slightly affects the bottom plate gusset dimension. See the following related question.

Q3: Please review the attached weld type and dimension we've shown for the bottom plate gusset.

For questions 2 and 3, please see the “MGS attach culvert bottom bracket assembly.pdf" and “MGS attach culvert bottom bracket parts.pdf" attachments.

Q4. Please review the IDOT spec for “Chemical Adhesive" and the related materials testing procedures (attached “TestingChemicalAdhesives.pdf"), and advise us if this satisfies the intent and requirements of the adhesive used in testing the MGS attached to culverts. Our current list of approved materials (“chemicaladhesives.pdf") is also attached. We note that the AC 100+ Gold by Powers Fasteners is listed, but the AC 50 Silver, apparently a lower grade, is also listed. (Or we might refer this to our Bureau of Bridges and Structures, and/or to our Bureau of Materials, but we need to understand the “bond strength" definition and requirements.)

Note: We do need to look at threaded rod embedment, because IDOT culverts use 3500 psi concrete, while the MwRSF testing used 4000 psi pcc. Also, we need to investigate if any special spec is required for the overhead use of chemical adhesive.

SECTION 1027. CHEMICAL ADHESIVE

1027.01 Chemical Adhesive Resin System. The chemical adhesive resin

system shall consist of a two part, fast-setting resin and filler/hardener. The system

shall meet the requirements of the ITP for Chemical Adhesives and be on the

Department's qualified product list.

Attachment: http://mwrsf-qa.unl.edu/attachments/6b0cf602b9d2bd19b5770e794b57d38d.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/243428957985c6e527af9e1cb2748cab.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/2502fa9b62de5f4a6a4f40debf80e8c2.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/05d09be50da796f4ce6be7e9b73f6346.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/a50a22b9988cb40e4e2550652f4ab9bb.pdf

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Response

Date: 04-05-2016

Q1: The thickness of the bottom angle can remain at 3/8". The compression force the socket transfers to the angle should be equivalent to the bridge rail system, so the same material thickness and cross section seems reasonable.

Q2: Typically, the minimum bend radius for a steel plate is considered to be equal to the plate thickness. 3/8" is getting pretty difficult to bend, so I understand if a fabricator wanted a larger radius. The gusset can handle a radius as large as ¾" without needing modifications, so I would place the maximum bending radius at ¾", or 2 times the plate thickness.

Another option would be to turn this bent plate into a welded assembly of two flat plates (eliminates the bend if you are having difficulty with it). A welded angle assembly should also work just fine.

Q3: The weld and gusset appear to have the same specifications and dimensions as the original bridge rail attachment. I see no issues here

Q4. The strength of an epoxied anchor can be calculated through procedures provided in ACI-138 (Appendix D in 2011 version, or Chapter 17 in 2014 version). Using the equations provided in these sections along with the anchor size and strength, concrete strength, embedment depth, and adhesive bond strength, you can calculate the tensile and shear strengths of a particular anchor. The strength of the epoxy will only be applicable anchors loaded in tension. The anchors on the under-side of the culvert slab (overhead installation) would be loaded in shear and therefore only critical for concrete breakout and steel failure. The top mounted socket designs for culvert attachment also load the headwall anchors in shear, so again the adhesive is not critical.

Only the epoxy anchored version of the side-mounted sockets (originally design concept D2 in report TRP-03-277-14) for culvert attachment are loaded directly in tension. So, if you are utilizing that particular attachment design, you should check your anchorage strength (embedment and bond strength) against those of the tested design.

Since your culverts were constructed with a 3500 psi concrete, you will want to utilize the noted ACI-318 sections to ensure that the embedment of the anchors will develop sufficient concrete breakout strength and concrete pry out strength (tension and shear loading) for all anchors.

Curb termination in MGS stiffness transition

Question

State: MO
Date: 01-29-2016

We have a question on the amount of curb required along the transition area from a bridge end thru the thrie beam, transition section and into the MGS. Basically, our new bridge end design will likely have curb extended into the transition section area or just past that into the nested MGS. But as we read report TRP-03-291-14, the design has curbing that ends either before post 11 OR extends past the first section of MGS that has been double nested with W beam to between posts 4 and 5 (Fig 54, pg 74). The question comes, is there any way to have a curb from the bridge end that ends within the transition section or within the nested MGS section that has been found as acceptable? This only saves a few feet of curbing at each approach, but multiplied over and over at locations around the state, it adds up.

The recommendations on pages 135 and 136 include nesting the W-beam rail at the upstream end of the W-to-thrie transition AND carries the curb past that nested section into the normal MGS. If that is the option, we will go with it. But, if there is some other design option that allows curbing to end in the thrie-to-W beam transition area, please let us know where to look for recommendations and design guidance. If there is nothing, please confirm that the design in Figure 54 with curb extended into the normal MGS rail in report TRP-03-291-14 is current and the best practice for this transitional area with curb.

Response

Date: 01-29-2016

The introduction or removal of roadway geometric features has shown to cause critical differences in safety performances for some barrier systems. Approach guardrail transitions (AGTs) in particular have been shown to fail crash tests after a curb is either introduced or removed from otherwise crashworthy systems, as demonstrated in the noted report (TRP-03-291-14) and in other AGT tests. In addition, the termination or transition between adjacent features can cause their own issues with system crashworthiness. Thus, to be conservative and avoid any potential snag and/or stability issues, we have recommended that the curb (or lack thereof) be consistent through the critical area of this MGS stiffness transition – specifically the area under and upstream of the W-to-thrie transition segment. That has produced the following recommendations:

1. If you want to terminate the curb within the AGT system, we recommend doing so prior to the W-to-thrie transition element (or as you stated, before post 11 – from TRP-03-291-14 page 74).

2. If the curb needs to be carried further than this location, we recommend carrying it past the entire AGT system. This extends another 18 ft – 9 in. to the upstream end of the nested W-beam rail (or the mid-span between post 6 and 7 from trp-03-291-14 page 74).

Terminating or transitioning within this 18.5-ft region of the AGT may be crashworthy, but we simply don't have the research or testing to confirm that.

Weak post culvert mount

Question

State: WI
Date: 02-03-2016

Attachment: http://mwrsf-qa.unl.edu/attachments/26acd91bf9c4443074ece153eb77ce77.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/95371badd78b1ff4a1746d91c4578e1d.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/a876ccd14d537f20609c33463363ad5d.pdf

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Response

Date: 02-12-2016
I reviewed the details you provided.

The modification to the post to prevent the post from dropping through the socket during installation should pose not performance issues.

In terms of how to address limited post deflection area for strong posts adjacent to the culvert mounted rail, we would recommend that you extend the use of S3x5.7 posts at half post spacing past the end of the culvert. The S3x5.7 posts will not be affected by lack of soil displacement as they typically yield at groundline. You could use soil plates or post with extended embedment to ensure sufficient resistance of the post-soil forces to yield the post.

TL-2 minimum installation length

Question

State: IA
Date: 02-25-2016

We are working on putting together standard drawings for our lower speed (45mph or under) as well as our lower volume (400 vpd or under) roadways. Doing so potentially involves aspects of both TTI and MwRSF research, so I'd like to address this as a shared conversation.

For the bridge connection, we would like to go with TTI's MASH 31" TL-2 transition (Report 9-1002-8).

For the tangent w-beam section, we would like to go with MwRSF's nested w-beam recommendation (borrowed from their TL-3 test 03-291-14) to cover all cases regardless if there is a curb or not, as we would rather nest when it wasn't needed than not nest when it was.

For the end terminal, we are looking at the MASH TL-2 Softstop (38'-3.5" long) as well as NCHRP 350 TL-2 ET-Plus and SKT (25') options.

My questions to both research groups are: What is the minimum length needed for a TL-2 system to function when attached to a bridge? What would it be if it was a free standing installation, say protecting a point hazard?

My question to MwRSF research group is: Does a TL-2 situation pose the same rupture risk with a curb under the transition/w-beam connection?

Potential installations would then be the following:
1. Assuming curb is not present at all, ends before transition/w-beam connection, or does not pose rupture risk:
a. System length of approximately 35' (10' transition + X' of w-beam to meet minimum length + 25' terminal)
b. System length of approximately 48' (10' transition + X' of w-beam to meet minimum length + 38' terminal)
2. Assuming curb is under transition/w-beam connection and does pose a risk:
a. System length of approximately 48' (10' transition + 12.5' nested w-beam + X' of w-beam to meet minimum length + 25' terminal)
b. System length of approximately 60' (10' transition + 12.5' nested w-beam + X' of w-beam to meet minimum length + 38' terminal)

Please let me know if you have any questions. Thank you for your assistance.

Response

Date: 02-26-2016

You have several good questions below. I will try to give you my best response based on our current knowledge.

1. Your first question seems to be what is the minimum system length for a the TL-2 approach guardrail transition system. For the TL-3 system, we made several recommendations related to the upstream system length based on our testing and interaction with terminals for both the transition with and without the curb. The placement of the upstream end anchorage too close to the stiffness transition may negatively affect system performance, thus

potentially resulting in excessive barrier deflections, vehicle pocketing, wheel snagging on posts, vehicle-to barrier override, or other vehicle instabilities. For the transition without the curb, we recommended:

A recommended minimum length of 12 ft – 6 in. 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.

Or

A recommended minimum barrier length of 46 ft – 10½ in. 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.

For the transition without the curb, we made similar recommendations:

The length of W-beam guardrail installed upstream of the nested W-beam section is recommended to be greater than or equal to the total system length of an acceptable TL-3 guardrail end terminal. Thus, the guardrail terminal's interior end (identified by stoke length) should not intrude into the nested W-beam section of the modified MGS stiffness transition.

A recommended minimum barrier length of 34 ft – 4½ in. is to be installed beyond the upstream end of the nested W-beam section, which includes standard MGS, a crashworthy guardrail end terminal, and an acceptable anchorage system.

With respect to the TL-2 transition tested by TTI, the system used the same 46 ft – 10½ in. length upstream of the asymmetrical W-beam to thrie beam transition section. Thus, while the overall system length was shorter due to the simplified transition section, the amount of guardrail upstream of the asymmetrical W-beam to thrie beam transition section and the corresponding distance to the upstream anchor was identical.

While we would expect that anchor loads are lower for the TL-2 system, we do not have data that confirms the anchor loads and the effect of a shorter system length at this time. Thus, we cannot shorten these length recommendations at this time. It seems reasonable that one could reduce the distance between the asymmetrical W-beam to thrie beam transition section and the end anchorage by 6 ft – 3 in. or 12 ft – 6 in. and still have acceptable system performance. However, that cannot be verified without further analysis and/or testing.

2. Your second question is what is the minimum system length for a standard TL-2 MGS system. We have conducted previous research into minimum system lengths for the MGS under TL-3 (http://mwrsf.unl.edu/researchhub/files/Report281/MGSLENGTH_R8.pdf). In that study we successfully evaluated a 75' long MGS system. Although the 75-ft MGS performed successfully, several factors, including Lateral Extent of the Area of Concern and the Guardrail Runout Length, must be considered when determining the overall barrier length for shielding a roadside hazard. Only a few roadside hazards can be properly shielded by short guardrail installations. Thus, longer guardrail installations are still required for shielding many hazards.

   BARRIER VII simulations were conducted to investigate system lengths of 62 ft – 6 in. and 50 ft. The 62-ft 6-in. model showed promising results with rail forces, barrier deflections, vehicle behavior, cable anchor forces, and anchor displacements similar to those observed in the validated 75-ft MGS model. Thus, a 62-ft 6-in. MGS showed potential for successfully meeting MASH TL-3 standards. BARRIER VII simulations of the 50-ft system produced erratic results and model instabilities once the vehicle contacted end anchorage posts. It was concluded that the simplified BARRIER VII models of the end anchorages were limited in their ability to accurately simulate BCT posts during vehicle contact.

   The 50-ft MGS was further investigated with LS-DYNA simulations. The LS-DYNA simulations provided more realistic wood post fracture behavior and insight into vehicle roll and pitch tendencies. The simulations showed successful redirection of the 2270P vehicle for impacts between post nos. 3 and 4, while the system gated for impacts at post nos. 5 through 8. The 62-ft 6-in. and 50-ft models both exhibited the potential for successfully redirecting an errant vehicle at the MASH TL-3 test conditions. However, these reduced-length systems would have a narrow window for redirecting vehicles and would only be able to shield limited size hazards. Due to limitations associated with the computer simulations, full-scale crash testing is recommended before these shorter systems are installed.

   The scope of the research did not include evaluation of the performance of end terminals on a reduced-length guardrail system. Further study may be needed to evaluate reduced system length in conjunction with guardrail end terminals in redirective impacts as well as end-on terminal impacts. Guardrail end terminals may have different post sections and/or anchorage than what was utilized in test no. MGSMIN-1. Thus, shorter guardrail lengths may not have the same redirection envelope found in this study. Additionally, for compression based terminals, the system post must develop the compressive forces required for the terminal to function. Very short systems may not provide sufficient resistance to the rail forces in end-on impacts.

   Thus, our current recommendation for TL-2 MGS lengths would still be the 75 ft length until further research can be conducted.

3. Your third question was whether a TL-2 situation pose the same rupture risk with a curb under the transition/w-beam connection. We believe that the rail rupture potential would still exist if a curb was used with the transition. During the testing of the MGS transition system with a curb to TL-3, the rail rupture appeared to occur due to a combination of to heavy upward and lateral forces on the lower region of the guardrail in advance

of the splice between the W-beam and asymmetrical transition segments. While a TL-2 impact would result in lower lateral guardrail forces, there is concern that the upward forces produced by the vehicle wedging between the curb and the W-beam could be very similar in a TL-2 impact. In fact, if you look at the TL-2 transition test conducted at TTI, it appears that the 1100C vehicle did extend underneath the W-beam and lift up on the rail, and similar behavior would thus be expected when the curb was present.

As a side note, we have had several states that have simply incorporated the nested rail in the transition when the curb is present or not as you noted above. This, may be prudent as a means to ensure the transition always is within the tested limits.

Thus, from your email and based on the comments above, I would get the following installation lengths (shown in red). I should note that here and in the discussion above we are referring to maximum of the tested end terminal stroke lengths or the paid system length quoted by the manufacturer. We are concerned with interaction of the terminal with the transition and paid lengths may be shorter than the actual head travel. Thus, the lengths below are assuming the 25 ft and 38 ft terminal lengths you mention reflect the maximum of the head travel or terminal length. Thus, the lengths may change slightly based on the terminal lengths. Again, there may be potential to shorten these somewhat as noted above, but these values are based on our current guidance.

Potential installations would then be the following:

1. Assuming curb is not present at all, ends before transition/w-beam connection, or does not pose rupture risk:
    a. System length of approximately 35' (10' transition + X' of w-beam to meet minimum length + 25' terminal)
        i. For a 25' terminal length (8.67 ft from end of bridge to end of W-thrie section + 46.875 ft including 25 ft terminal = 55.545 ft)
    b. System length of approximately 48' (10' transition + X' of w-beam to meet minimum length + 38' terminal)
        i. For a 38' terminal length (8.67 ft from end of bridge to end of W-thrie section + 12.5 ft of standard MGS + 38 ft terminal = 59.17 ft)
2. Assuming curb is under transition/w-beam connection and does pose a risk:
    a. System length of approximately 48' (10' transition + 12.5' nested w-beam + X' of w-beam to meet minimum length + 25' terminal)
        i. For a 25' terminal length (8.67 ft from end of bridge to end of W-thrie section + 46.875 ft including 25 ft terminal = 55.545 ft)

b. System length of approximately 60' (10' transition + 12.5' nested w-beam + X' of w-beam to meet minimum length + 38' terminal)

i. For a 38' terminal length (8.67 ft from end of bridge to end of W-thrie section + 12.5 ft of nested MGS + 38 ft terminal = 59.17 ft)

Lance, if you have further thoughts on this, feel free to chime in. We are being conservative based on lack of more knowledge on minimum length systems, and you may have arguments for revising what I have here.

Response

Date: 02-27-2016

In case I forgot to send it along, thank you for that response.

Attached are three of our initial markups based on that information.

1. els-tl2-250 will detail a situation where we are protecting a point hazard with minimal to no concern regarding a secondary hazard.

2. eba-tl2-250 will detail a situation where there is concern for a secondary hazard.

3. eba-tl2 transition will detail the Barrier Transition Section shown in both layouts. This includes the short thrie-beam, asymmetrical section, and nested w-beam.

My question today is related to eba-tl2-250. In reportTRP-03-291-14 (page 137 – statement 3) it mentions that when a TL-3 installation is flared, there needs to be an additional 12.5' section of single w-beam added upstream of the nested w-beam before the flare (as stated in our currentBA-250 circle note 5). What I'm wondering is whether or not that remains true for our TL-2 situations.

Attachment: http://mwrsf-qa.unl.edu/attachments/f60dfb82ddfb97f24eafa67e2681f63e.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/79b9f9895976d2ad8034c90181709e63.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/9420f303d45d7ac0efbd055a8c35012a.pdf

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Response

Date: 02-28-2016

We would recommend keeping that 12.5' tangent section upstream of the nested section for TL-2. The two concerns are:

1. Having a flared section directly adjacent to a stiffened region of the barrier may increase pocketing, rail loading, and vehicle instability.

2. We are somewhat concerned that small cars impacting in the flared region upstream of the transition could extend further under the stiffened transition. This may cause increased underride, vehicle snag on posts, increased vehicle decelerations, and increased rail loads.

While we would agree that this potential should be reduced for TL-2, without further analysis or testing, we want to take a more conservative approach.

Thanks

MGS Weak Post Questions

Question

State: WI
Date: 03-01-2016

We would like to add small anti-drop plates to the MGS weak post bridge rail system to prevent the posts from falling through the sockets during installation.

Is what I am showing acceptable for the welding that anti-drop plates to the s3x5.7

Attachment: http://mwrsf-qa.unl.edu/attachments/3182e65e0d790ab3706489211f2128b8.jpg

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Response

Date: 03-02-2016

I think the welding you have listed is fine. I did not realize that you had plates on the front and back side of the S3x5.7. Welding plates to both sides effectively makes the base of the post a tube. This may potentially stiffen the base of the post and change the location that the post yields at.

Do you have any issues with just welding one plate across the back flange? See Below. This would keep things out of the way and still support the post while limiting the effect on the post behavior.
Attachment: http://mwrsf-qa.unl.edu/attachments/7e3cdcd1c98f295e1abcebd7fe29af1c.jpg

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Response

Date: 03-03-2016

More questions for the side mounted assembly. See attached.

Attachment: http://mwrsf-qa.unl.edu/attachments/0a9018e6b6b3113fd3d1dc0a432372da.pdf

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Response

Date: 03-04-2016

Answers to your questions.

1. In terms of minimum headwall thickness for bolting through, there is no minimum thickness. Instead, we would recommend that the headwall have equal or greater capacity to the design we evaluated. We selected a headwall that was on the lower end of headwall strengths.

2. In terms of clear cover for the end of the rod, the main concern would be blowout of the back side. The effect of end cover on epoxy would be minimal. We would recommend that the embedment depth is met. If you were referring to the clear cover of to the top of the headwall, that would be defined by the geometry of the headwall

and the attachment bracket and should not change.

3. In terms of the post length and embedment, we would recommend that a post without a soil plate as shown have a minimum of 42" of embedment. We have used this embedment in the cable median barrier work and it has proven to develop proper soil resistance without the soil plate. If you use the soil plate version of the post, the post can be 36" long I believe.

4. In terms of distance from a slope, we recently did testing of S3x5.7 posts with the soil plate adjacent to steep slopes for NDOR. In that work, it was found that offsets as low as 1' were acceptable, but a 6" offset resulted in blowout of the post through the slope. Thus, we would recommend a minimum offset of 1'. We also believe that the 1' offset would be acceptable with the 42" embedded post without the soil plate.

5. For the post installed 4" from the wingwall, the wingwall will provide additional resistance and help develop the forces to yield the post. As the posts in the weak post MGS system are intended to yield at groundline with minimal rotation in the soil, this should not be an issue.

Thanks

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bridge rai on a timber bridge

Question

State: WI
Date: 03-01-2016

We have road project going past an existing timber bridge.

1. Has MwRSF see a bridge rail like this crash tested?

2. Project staff are asking if they should nest the beam guard on the bridge.

Attachment: http://mwrsf-qa.unl.edu/attachments/24301b56d685dac0d1421d95f2bce130.docx

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Response

Date: 03-03-2016
We have observed similar post and beam bridge rails in use in the past, but we do not have knowledge of them being crash tested to any recent standard.

Nesting the beam would be a positive step in terms of reducing the potential for rail rupture. However, it may not guarantee the crashworthiness of the barrier system.

New Bridge / 2" overlay the following year

Question

State: NE
Date: 03-07-2016

Do you have a good solution for a new Bridge w/ 2" overlay the following year

A: Could we build the bridge approach section at 34" the first year?

or

B: Could we place a steel plate with a 2" offset below the bolts into the bridge where the 5 bolt pattern is.

then remove the following year when the overlay is placed

Response

Date: 03-08-2016

On new construction, it would seem reasonable that one could install a concrete buttress with two different anchor patterns for the five end shoe bolts or studs, which allow for a 2" height adjustment. Some consideration could be given toward mitigating vehicle engine hood/quarter panel snag on the end of the new buttress after the approach thrie beam is lowered by 2" after one year of service. As such, there could be a downward slope or taper to reduce snag on the concrete corner. I like this option more than option A.

There also could be an option where a steel plate is used to make the switch, similar to the options discussed last winter.

Transition between G4(1S) and MGS.

Question

State: MI
Date: 03-11-2016

Per our phone conversation earlier today, I've attached a file containing MDOT's proposed MGS details. If you refer to sheet 10, I've highlighted our proposed details for transitioning from MGS guardrail to conventional (28" tall) strong-post, w-beam guardrail and thrie-beam guardrail, respectively. We would like to introduce a single 3'-1.5" post spacing in order to transition from the splice being located between posts (MGS style) to the splice being located at post locations (conventional guardrail style). By using a single 3'-1.5" post spacing, we can use standard 13'-6.5" long beam elements throughout the entire transition. We are trying to avoid the use of w-beam elements that do not have a standard length (13'-6.5") for ease of maintenance.

Do you foresee any issues with these details, or do you have any comments or suggestions concerning these details?
Attachment: http://mwrsf-qa.unl.edu/attachments/8203ce243d3cc31f446e822f8797388b.pdf

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Response

Date: 03-29-2016

Ron asked me to look at you details and provide some guidance regarding the transitioning from the older G4 type W-beam systems to the 31" tall MGS and dealing with the movement of the splice location. Your plans current show a height transition of the W-beam from the old G4 barrier height to the MGS height over 25'. We have previously recommended that this transition occur over a distance between 25'-50'. Thus, you plan looks acceptable there. We have more concern with the use of the additional post when relocating the splice to the midspan. Adding the post directly downstream of the splice location will tend to increase barrier stiffness and rail loads near the splice location. In previous testing, this has been shown to create a potential for rail rupture. Thus, this would not be our first choice in this area.

We believe that there are two options for transitioning the splice that may work better.

1. As you noted, there is the option to use a non-standard rail section. This would work well, but as you noted can be undesirable in terms of inventory and maintenance.

2. A better solution may be to proceed with the posts at the splices for one full 12.5' rail section at 31" height. Then rather than add an extra post, we would recommend omitting the post at the next splice to make a 9.375' long span between posts. This should place the posts in the correct spacing for the midspan splices used in the MGS. We believe this can be done because we recently conducted a successful MASH TL-3 test on the MGS with a single omitted post which created a 12.5' long span between posts. This would suggest that using a 9.375' long span to get the correct post alignment for the MGS should not be an issue. However, as noted previously, we would recommend not omitting the post until a full 12.5' of 31" tall barrier is in place. See the sketch below.

Please let me know if you have any questions about this. We can provide more thoughts if needed.

Attachment: http://mwrsf-qa.unl.edu/attachments/a34293ed02c9032411dfe885dbca3f5b.jpg

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Response

Date: 03-30-2016

Thanks for looking into this and replying to my request. I agree that your suggested approach of omitting a post and creating a 9.375' long clear span is advantageous, and MDOT will definitely look into this option.

Has your group prepared a report for the MASH TL-3 test where a post was omitted resulting in a 12.5' long clear span? If so, could you share that report? Did your group use 12" or 8" offset blocks when you conducted the test? If your group used 12" offset blocks in your recent test, do you believe the same treatment could be applied to MGS with 8" offset blocks without jeopardizing the crashworthiness of the system?

Response

Date: 03-31-2016

We have not quite completed the omitted post report at this time. I can send you a link to the document when it is complete. As a side note, all of our research reports are currently archived online at the link below. You can always look there for any of our completed research.

http://mwrsf.unl.edu/researchhub.php

As for you question regarding the use of 8" blockouts, we did evaluate the system with 12" blockouts as we feel that they improve vehicle capture and stability. That said, we do not believe that the use of 8" blockouts would degrade the performance of the omitted post system sufficiently to jeopardize the crashworthiness of the system.