

Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

01-01-2014 to 04-01-2014

NJ Safety Shape Median Barrier Curb

Question

State: NJ

Date: 01-17-2014

Thanks you for returning my phone call. I think when I spoke to you this morning, you may have thought I was talking about the NJ safety shaped parapet. I was talking about the Safety Shape Median Barrier (Hardware Guide SGM11a-b), see attached detail. The last time it was approved was via the FHWA memorandum entitled Report 350 Nonproprietary Guardrails and Median Barriers dated Feb, 14, 2000, see attached memo. In this memo FHWA summarizes and describes all non-proprietary longitudinal roadside and median barriers that have met Report 350 requirements. Where applicable, the reference page number for each barrier included the 1995 AASHTO-AGC-ARTBA "Guide to Standardized Highway Barrier Hardware." As you can see on page 2, the 32" high safety shape barrier was approved for TL-4 and the reference page "SGM11a" is next to it. They deferred to the 1995 hardware guide details.

The current hardware guide details basically contain the 1995 version. Both versions state:

AS PAGE 2 OF 4 OF THE FIGURE STATES UNDER "INTENDED USE", SECOND PARAGRAPH, THE "10 FT LONG, 10 INCH DEEP REINFORCED ANCHOR FOOTING SHOWN SHOULD BE PROVIDED AT BOTH ENDS TO PROPERLY SECURE THE BARRIER. OTHER COMMON METHODS OF SUPPORTING THIS BARRIER INCLUDE SETTING THE BARRIER IN A CONTINUOUS KEYED FOUNDATION OR DOWELING THE BARRIER TO A FOUNDATION."

Based on the hardware guide language above, I interpret that to mean that each state will design the footing for the SGM11a-b. Only one version is shown in the highway guide. I looked at the entire length of Federal acceptance letters on the web (http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/listing.cfm?code=long) and have not found any federal acceptance letters concerning other footing designs for the SGM11a-b. These letters go back to 5/31/1985. If crash testing of the various state footing designs were required, why are there no acceptance letters for the different footing types the hardware guide referred to?

Another reason why I believe we do not need crash tests and acceptance letters for footing design is the same reason for the Transpo Breaksafe Sign Support System. The Breaksafe was crash tested and approved for the Breaksafe base and parts. Transpo told us that the footing design is up to each state. Each state did not have to get their footing designs crash tested and approved.

The SGM11a-b has no reinforcement except for some rebars near the top to prevent large pieces of barrier breaking off and falling into the traveled way in a severe collision. I believe the barrier reinforcement criteria you referred to in the LRFD specs in Chapter 13 are for bridge parapets, not median barrier.

I will basically give this info above to the Feds in NJ and hope that they agree with me that NJ's 9" deep continuous keyed foundation does not have to be crash tested and approved. If so, where are the other approved foundations that the hardware guide referred to? Also as you have stated, approval letters began from 1985 on. Since the original NJ barrier was crash tested prior to 1985, no approval letters exist.

Attachment: <https://mwrsf-qa.unl.edu/attachments/eb0fee5b04e8b243fe01dbcae064dfc6.pdf>

Response

Date: 01-17-2014

I am not sure when the original version of this barrier was crash tested with the noted 10" footing. However, if you like, I can dig up some old testing references (possible from Southwest) to try to find the original tests. Just let me know if this would be of use to you. If so, it would also help to have a copy of your current state standard – reinforcement configurations for both interior and end sections. There have been a number of tests utilizing various anchorage designs. We would just need to find crash tests of a comparable barrier with a particular anchorage to justify the anchorage you desire.

Response

Date: 01-17-2014

Thank you very much for your help. The information contained in the on-line hardware guide pertaining to the footings for SGM11a-b appears to be the same info that was in the 1994/1995 published hardware guide. The 1995 hardware guide SGM11a-b (drawing dated 1994) and the current SGM11a-b (drawing dated 5/16/2005) cite the same research references (March 1976 and May 1986). The 1986 reference mentions Dean Sicking. Our DOT library had a copy of the March 1976 reference which turned out to be FHWA-RD-77-3-4 dated June 1976 Final Report. The NJDOT ordered a hard copy of the May 1986 reference. The March 1976 (June 1976) reference contained info on the various footing designs for the 32" barrier. A Math model was developed to evaluate the Concrete Median Barrier (CMB) foundation restraint for stability. From this, they created Figures 52 and 53 which determines footing depth, dowel depth and keyed footings, see attached pages.

I also attached a pdf of NJDOT construction details CD-607-3 and 607-2.1

Attachment: <https://mwrsf-qa.unl.edu/attachments/62841c1c0bbbf954c6e7c6c39c436b36.PDF>

Attachment: <https://mwrsf-qa.unl.edu/attachments/ed55f06f4237e6125f584219108b9cdd.PDF>

Response

Date: 01-23-2014

A few comments as I have glanced through the figures you have provided

1. The original research report was from 1976, thus it predates even NCHRP 230 (which predated NCHRP 350). Therefore, the impact loads used to evaluate the barrier and anchorage options may be significantly lower than the impact loads to which barriers are evaluated under the current MASH standard, or even the old 350 standard. As such, any recommendations made from the report should be taken as absolute minimum requirements for strength.

2. The report's recommendations for using asphalt keyways are for segment lengths of 30 ft. However, your construction details call for joints every 15 ft. Therefore, an asphalt keyway alone may not be enough to anchor the barrier.
3. The report's recommendations for using dowels to anchor the barrier are to utilize a #8 bar every 18". Your construction details call for the dowels to be spaced at 4 ft intervals. Again, using dowels alone may not be enough to anchor the barrier.
 4. I don't see a discussion within the report of the strength of the anchorage if both an asphalt keyway and dowels are utilized. Without conducting additional analysis or a crash test, it would be difficult to prove adequate anchorage compared to the original recommendations.
 5. In terms of the Hardware Guide's recommended minimum embedment depth of 10" for a footing within 10 ft of a discontinuity (end or open joint), your 9" deep footing is very near this mark, but still below the minimum requirement. I'm not sure where this requirement was established, but it may be hard to get around without a crash test proving a shallower embedment will suffice.

I am curious as to what FHWA will have to say about your request to utilize this anchorage scheme. We have been unsuccessful in requesting an approval letter for other concrete barrier systems in recent history based on anchorage details. One barrier in particular utilized an 18" x 18" footing reinforced with torsion and longitudinal steel to support an 8" wide vertical barrier. The system was even successfully crash tested to NCHRP 350, but FHWA still would not grant an approval letter. Please keep me posted on FHWA's response to your request.

Let me know if you have additional questions.

Aluminum NJ or F-shape Parapets

Question

State: IL

Date: 01-21-2014

I am aware of two websites that may have materials which may be helpful. First, the AASHTO Task Force 13 Committee has been slowly working on updating an online bridge railing guide. Subcommittee No. 3 volunteers have been trying to review the original materials and slowly add new materials. I have provided a link to the page, including a few pages for aluminum rails that are included for now but may change as the review process continues. Please let me know if you have questions about this website.

<https://www.aashtotf13.org/Bridge-Rail.php>

<http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=browse>

<http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&railing=4>

Next, I am providing a link to FHWA's bridge railing site. You may find some interesting information here.

http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/bridgerailings/index.cfm

Finally, I am providing a link to FHWA's list of accepted longitudinal barriers, which has some bridge rails included. One would need to search through the posted items.

http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/index.cfm

Can you provide details for the existing aluminum railing system so that I know what you are seeking to repair/replace.

Ron

Response

Date: 01-21-2014

I am the IDOT Planning Unit Chief for the Bureau of Bridges and Structures in Springfield, Illinois. We have a few of these aluminum shapes parapets on truss bridges within the state. We are preparing to rehabilitate one of these trusses. The posts of this parapet need to be replaced due steel deterioration in the lower portion of the posts. The consultant has indicated to me that the existing post connection is very weak and would not satisfy typical test level criteria. I would like to meet a minimum TL-3 safety level after the rehabilitation of this bridge.

I researched the various web sites for TL data on these types of barriers and did not find much. Would you be able to direct me to a site that has this barrier and TL equivalent safety level or similar?

Response

Date: 01-21-2014

Thank you for your reply and reference website locations.

The post and connection to the deck system warrants our concern with the proposed rehabilitation since we plan to reconnect the existing aluminum barrier based on the high cost and good condition of this barrier.

I attached the details of the barrier and connections for your information. We need to provide an attachment for a typical bridge deck and a concrete-filled steel grid deck as indicated by the drawings.

The existing aluminum barrier is an older design with different connections that probably do not correlate to the testing in the attached documents. It may be difficult to demonstrate the existing barrier would satisfy TL-3.

Thanks again for the information,

Tim Craven

Response

Date: 01-22-2014

I have briefly reviewed your details. The aluminum system is really too complex to analyze via hand calculations. To evaluate the capacity of the system to meet Test Level 3 of NCHRP 350, one would likely need to perform 1 of 2 operations – (1) utilize FEA in combination with some limited dynamic component testing to determine capacities for connections and components or (2) construct the railing and anchorage system and conduct full-scale testing with a pickup truck.

Your system is similar to the system I noted below. I have gone to the AASHTO TF13 website to download a

few files. They are attached. FHWA approved an alternative configuration several years ago. For your system, some of my concerns pertain to the post-to-deck anchorage capacity as well as the anchorage for the front barrier base. In addition, it is unknown for now as to the local crush resistance of the aluminum space truss as well as weakened regions nears joints. I believe that one of the two options would be needed to explore the capacity of this existing aluminum bridge railing system.

As such, I do not have a good answer right now to suggest or inform you that the system will meet TL-3 of NCHRP 350. Let me know if you want to further discuss this matter.

Temp. Concrete Barrier Installation Requirement Research

Question

State: SC

Date: 01-22-2014

We typically recommend the following for temporary barrier installations.

1. The barrier should be placed on a relatively flat surface with a grade of 10:1 or less.
2. We recommend that the barrier be placed on a paved (asphalt or concrete) road surface. We do not recommend that the barriers be placed on soil or grass surfaces as they can dig into the ground. This prevents barrier translation and promotes barrier rotation, which are undesirable.
3. We also recommend a clear area behind the barrier to allow for barrier deflection without interference. The width of the clear area will depend on the dynamic deflections of the barrier you choose to employ. Deflections can vary significantly between barrier types.

NCHRP Report No. 358 provides a great deal of guidance on TCB installation procedures and recommendations. I have attached the document below.

The file 'NCHRP Report No. 358.pdf' (84.8 MB) is available for download at

<http://dropbox.unl.edu/uploads/20140205/5d25d5153b62d561/NCHRP%20Report%20No.%20358.pdf>

for the next 14 days.

It will be removed after Wednesday, February 5, 2014.

If you have further questions, let me know.

Response

Date: 01-22-2014

I am currently tasked with researching the detailed requirements necessary for installation of temporary concrete barriers on a South Carolina highway. Our SCDOT standards are slightly vague on the subject, I just need to know if you have any research material that states what

type of surface is needed to install temporary concrete barrier walls (flat surface, any slope allowed, etc.). Please email me back with any questions or if you have any materials that I may find useful!

Bridge Rail End and Curb with Thrie Beam Transition

Question

Date: 01-22-2014

Can you review the attached detail and comment on the placement of the curb and the geometry of the vertical end section with respect to the approach transition?

Attachment: <https://mwrsf-qa.unl.edu/attachments/64ac3b00beddbb1b9b0df5fc5827ac34.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/d413ac8cee62f68d17583d3d109ce2c0.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/2dff5a49b3e5029d1643e25d29e67bd.pdf>

Response

Date: 01-22-2014

1. With respect to the placement of the curb, that system was designed with the curb located with the toe of the curb flush with the backside of the thrie beam. This would be consistent with the schematic you sent.

2. The second issue we discussed was the use of a vertical end section and the need for a tapered corner on the vertical end. We received FHWA approval for the vertical end section with a tapered corner from FHWA and would recommend that the tapered corner be included in the parapet designs if possible. See attached.

Let me know if that addressed your questions.

Attachment: <https://mwrsf-qa.unl.edu/attachments/b988f5472f9c4e89b9bda216b5959f06.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/0e80b14eb549f9d623422c76cddfb7fc.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/515ca882bec1e7112ef305a26e98b707.pdf>

MGS questions

Question

State: IL

Date: 01-27-2014

My name is Mark Bell and I am a guardrail contractor in Illinois.

The question came up relating to the wood block outs that are positioned between the steel post and the W Beam / Thrie beam rail elements.

The nature of wood is that it is not milled to exact tolerances as is the case with fabricated steel products. Certainly the wood mills can get close to specified sizes.

In reviewing your website I came across two pdfs that may assist in answering my questions.

Reference 1: MGS Standard drawing page 3 of 7 dated 7/20/11 illustrates a 6" wide x 12" deep x 14" tall wood block with a double nail driven through the flange of the post to hold the block from spinning. (the double nail suggests that the wood spacer / block out is un-routed)

Reference 2: FPL_MGS_SP_RD drawings for Southern Yellow Pine details both wood posts and block outs and page 9 of 9 provides specifications for the wood that include tolerances for Timber Spacers (block outs) It states the following: The material may be rough sawn or surfaced, full size, hit or miss, with a tolerance of 6 mm (1/4 inch) for all dimensions.

My questions are as follows:

What is the acceptable tolerance PLUS/MINUS for any dimension that wood blocks and posts are allowed before it becomes un-usable ?

Will the acceptable tolerance also apply to wood block outs that have a routed edge to prevent it from spinning on the post ?

I have a meeting on February 3rd, 2014 and would like to be prepared to answer these questions.

I greatly appreciate your reviewing this and getting back to me as soon as it is possible.

Response

Date: 01-27-2014

MwRSF has successfully crash tested the MGS in blocked (12" deep) and non-blocked (no offset block but with backup plates) applications. In these testing programs, improved performance was observed when using the offset blocks. TTI later successfully crash tested the MGS with an 8" offset block. As a result of these programs, I would not get too concerned with minor deviations in the block dimensions when comparing full-sawn, rough sawn, and dressed blocks. As a matter of fact, some fabricators start the process with larger wood sizes so that the dressed block actually measures 6"x12" or 6"x8".

Previously, manufacturers and other State DOTs also inquired into the MGS block tolerances. Since similar inquiries have been made and responses have already been complied, I am forwarding to you information from the 2007 discussions with the Pooled Fund members states. Hopefully, my response above and the attached information will sufficiently answer your question.

Generally speaking, The states set the block tolerances they will accept based on their preferences. They may have additional considerations for the blockouts outside those we use in our design and testing procedures. In addition, any guidance we could provide is not "acceptance" in particular. Our comments would be based on our design and testing and our best engineering judgment, but would not provide acceptance or compliance with any type of standard.

Thanks

Attachment: <https://mwrsf-qa.unl.edu/attachments/082083803a233257b5a5f8acc10b6b6e.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/e044a40d29e37f42a1515de9ee52121e.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/e9a9b2aa9e6558626cee8d6a0ea81738.zip>

Response

Date: 01-27-2014

Thank you for your response. I found it very helpful.

Based on your comment below, the MGS that has a block thicknesses between 8" and 12" will successfully pass the test.

To clarify one point...your comments apply to guardrail that is utilizing a 'steel' posts...correct ?

One additional question: the MGS details illustrate nailing the block with a one double headed 16d nail through a 1/4" hole through the web of a steel post...to prevent the block from spinning. If a wood block manufacturer provides the block with a routed surface while maintaining thicknesses from 8" to 12" , is the depth of the routing considered a critical dimension to meet adherence to the MGS system as tested ?

I'll look forward to hearing back from you.

Thank you

Mark Bell

Response

Date: 03-07-2014

See my comments below in red.

Thank you for your response. I found it very helpful.

Based on your comment below, the MGS that has a block thicknesses between 8" and 12" will successfully pass the test.

This is true. The MGS has also been tested successfully without blockouts. Thus, shorter blocks would likely work too. However, we do not recommend shorter blockouts unless roadway width is an issue.

To clarify one point...your comments apply to guardrail that is utilizing a 'steel' posts...correct ?

The MGS has only been tested with reduced blockout depths on steel posts. We have tested several wood post versions of the system, but they have all had the 12" blockouts. Shorter blockout depths may work, but would likely need further investigation.

One additional question: the MGS details illustrate nailing the block with a one double headed 16d nail through a 1/4" hole through the web of a steel post...to prevent the block from spinning. If a wood block manufacturer provides the block with a routed surface while maintaining thicknesses from 8" to 12" , is the depth of the routing considered a critical dimension to meet adherence to the MGS system as tested ?

We have tested the blockouts nailed and with no nailing. For testing purposes, it makes little difference. Spinning of the blockout during a vehicle impact is not an issue. The nails and the routing are more of a in-service feature to prevent blockout rotation over time, as you noted. Thus, depth of the routing would have little effect on safety performance. However, if the routing was not sufficient to prevent block rotation over time on field installations, it would become an issue. I don't have information on what routing depths are effective in the field. My suggestion would be to work with the state DOT's to determine the depth that they prefer for the routing based on field experience.

Charpy V-Notch Requirement for Crash Tested Steel Post & Rail System

Question

State: WI

Date: 12-11-2013

For a crash tested steel post and rail system for bridges or culverts, are any of the materials used in the components of this system (post, rails, base plates, etc.) required to pass a Charpy V-Notch test for the temperature zone they are to be used in? Does this help provide a more crash-worthy system. I have found some states that call for this on their standard details and was wondering if this is a requirement that all states should be implementing. Thanks for any information you can share.

Response

Date: 01-29-2014

Charpy V-notch testing is used to evaluate the toughness of a given steel. Toughness is generally defined as the energy absorption of the steel during impact loading. The Charpy test evaluates this by impacting a notched specimen with a pendulum type impactor and recording the energy absorbed during fracture. These tests can be run at lower temperatures to evaluate the affect of ductile to brittle transition temperatures on steel. So I can see why some states would have an interest in these types of tests.

Notched-bar impact tests are used to determine the tendency of a material to behave in a brittle manner. This type of test will detect differences between materials which are not observable in a tension test. The results obtained from notched-bar tests are not readily expressed in terms of design requirements, since it is not possible to measure the components of the triaxial stress condition at the notch. Furthermore, there is no general agreement on the interpretation or significance of results obtained with this type of test. For example, different types of notched specimen tests can generate different transition temperature values, thus making it hard to use as an evaluation criteria.

The Charpy specimen has a square cross section and contains a 45° V notch. The specimen is supported as a beam in a horizontal position and loaded behind the notch by the impact of a heavy swinging pendulum. The specimen is forced to bend and fracture at a high strain rate .

The principal measurement from the impact test is the energy absorbed in fracturing the specimen. After breaking the test bar, the pendulum rebounds to a height which decreases as the energy absorbed in fracture increases. The energy absorbed in fracture is rendered directly from a calibrated dial on the impact tester.

That said, it would be difficult to determine what would be an acceptable performance for the Charpy testing. The Charpy test does not measure ductility directly rather only the drop in fracture energy as temperature drops. Thus, it cannot determine what the ductility of a steel is at a given temperature without further testing.

In addition, I don't believe that we have a clear definition of what type of drop in ductility would be detrimental to the performance of our safety hardware. While there would be some limit, it would likely take further research to define that accurately. We have not defined that to date, and I have no knowledge of barrier failure in accidents that were directly attributable to reduced ductility at low temps. A basic limit could be that the ductility should remain above the limitations for the steel spec for the component. However, additional testing beyond the Charpy test would be needed to define the ductility accurately. Thus, it would be difficult to show

that Charpy testing would provide for a more crashworthy system without further research to determine the limits of safe ductility and toughness in our hardware and additional testing outside of the Charpy tests to measure that ductility. It would be recommended that research would need to be conducted to define the parameters for the Charpy test evaluation for the roadside safety hardware components in question.

Two of the more critical components in a guardrail are the beam rail and the post. Ductile to brittle transition in steel is largely related to carbon content. I looked at some of our recent material certs from guardrail testing and found carbon contents for those components (A992 for the post and M180 for the guardrail) between 7-20%. These would classify as low carbon steels. While I don't have exact data on these specific steels, general data available on Charpy impact tests for low carbon steels (attached) indicate that carbon contents below 20% will not induce reduced toughness until temperatures around 0 degrees Fahrenheit. Thus, the issue may not even be relevant unless extremely cold temperatures are prevalent.

Attachment: <https://mwrsf-qa.unl.edu/attachments/63da3c09ff0b29a454060e8fa1dbb092.jpg>

Information on Midwest Guardrail System use behind sidewalk

Question

State: CA

Date: 02-04-2014

I work as a designer for Caltrans. One of the challenges my team is facing now is designing ADA improvements on and near Caltrans facilities.

One of our designs proposes a 4-foot wide sidewalk in front of guard rail. I have seen test reports and photos of MGS behind 6-inch curb. Are there any data or recommendations when the MGS is placed behind 6-inch curb and a sidewalk ?

Response

Date: 02-04-2014

We have done some work with investigation of the MGS system with larger offsets from the curb. We did not achieve a comprehensive set of curb guidelines, but we did get some results that may help you.

I have attached references to that work. They may not answer all of you questions, but they should give you a start.

<http://mwrsf.unl.edu/researchhub/files/Report72/TRP-03-205-09.pdf>

<http://mwrsf.unl.edu/researchhub/files/Report56/TRP-03-221-09.pdf>

<http://mwrsf.unl.edu/researchhub/files/Report47/TRP-03-237-10.pdf>

Let me know if you have further questions.

Provide Expansion Joint in a Short End Section of Concrete Barrier

Question

State: WY

Date: 02-07-2014

we have a bridge with a New Jersey Concrete Bridge Barrier. In the original construction, they extended the transition section of the concrete barrier monolithically beyond the bridge end to eliminate the protruding toe of the barrier. Unfortunately, this barrier crosses an expansion joint at the bridge end. Our bridge folks would like to cut the barrier off at the bridge end, then re-pour the last 10 feet of barrier as a free standing element. This would then attach to box beam rail via the transition MwRSF developed for us several years ago. They would leave a two inch gap between the barrier on the bridge deck and the free-standing end section (called a transition rail in the details). To provide stability for that short section, they propose to dowel it into a heavily reinforced approach slab beyond the bridge deck. The first sheet is the original plan sheet with notations in pencil on the proposed modifications. Sheets two and three show additional details.

I am concerned whether just doweling this short end section will provide adequate stability. Even if we add three smooth #8 bars in the adjacent bridge rail to facility expansion and contraction between the two barrier units, I am not sure if we have enough stability. Please provide us with recommendations on how to proceed. Let me know if you need more information on clarity on what is proposed.

Thanks!

Attachment: <https://mwrsf-qa.unl.edu/attachments/5f262b3e989cb6cb839b4bc7edb59eba.pdf>

Response

Date: 02-12-2014

The design scheme you are suggesting seems like a good retrofit. My calculations show that the #6 bars spaced at 9" should develop equal or greater overturning moment resistance than the #4 stirrups spaced at 12" within the barrier segment. With the bottom attachment able to handle this load, I would move forward with this attachment method.

A few things to ensure the #6 dowels will develop full tensile capacity. First, I would put a few longitudinal rebar in the base of the barrier to tie the dowels into the barrier. This will help develop the #6 dowel within the short depth of the barrier foundation by preventing concrete breakout failure (see attachment for barrier sketch with these bars). Additionally, the correct embedment depth should be used to extend the dowels into the approach slab. This will be a function of the strength of the epoxy, but most manufacturers specific min. embedment length to ensure complete development.

The addition of #8 bars used to connect the bridge rail to this 10' approach segment would definitely help to prevent any relative lateral displacement between the two concrete barriers. You may want to include these to eliminate the risk of snag between barrier segments. It never hurts to error on the conservative side of safety, especially if it involves the simple addition of 3 dowel bars.

Attachment: <https://mwrsf-qa.unl.edu/attachments/25423e7a16ff3f6a3374bc25a13e4531.pdf>



Portable concrete barrier

Question

State: ID

Date: 02-10-2014

Hello, My name is Zach Cooper, I work for Knife River, a construction/materials company headquartered in Bismarck, ND. I work out of our Southern Idaho division, located in Boise, Idaho.

I received your contact information from a few gentlemen at the FHWA, but most directly from Mr. Artimovich, as noted in the email string below.

We have a contract with the State DOT here in Boise. The State has specified some Montana barrier, for scupper size, that we feel is not cost effective to make given the resources typical for Idaho. The quantity of barrier is relatively low, 6700 lf, and use on future projects is not highly likely.

We have proposed modifying standard barrier by increasing the scupper lengths to provide the same surface opening square footage that is provided by the Montana barrier. The state has denied this request based on NCHRP-350 compliance, stating, more or less, that modifications to the barrier would negate the existing NCHRP-350 testing that was done on the barrier and making it unsafe for use. The State left an opening saying that if NCHRP-350 compliance can be provided to substantiate our proposed modifications to barrier they will entertain the proposal.

The modification I am proposing is fairly straight forward, and by all accounts (all unofficially) the consensus is the modification would not change the "crashworthiness" of the barrier.

The proposed modification is as follows (drawing attached):

1. Typical 20' Jersey style barrier, in this particular case the precast outfit is in Utah, so our hope is to use the Utah Standard BA 2A barrier. This barrier has 2 each, 2 ft long by 2" tall scupper openings, one on each half of the barrier, separated by 5'10". That 5'10" separation is concrete.
2. The proposed modification is to block out that 5'10" span, connecting the two existing scuppers to make one large scupper measuring 9'10".

Essentially, what I am trying to do is:

1. Get an engineering determination that states these proposed modifications are insignificant to the "crashworthiness" of the barrier.
2. Have that determination approved by the FHWA
3. Take a letter to the State DOT of Idaho, and hopefully have them approve the use of the modified barrier.

This seems all very doable to me, however time is a barrier. In order to have approval in time to actually make the barrier, this whole process would need to be wrapped up by February 17th, roughly.

I am hoping you can help me with step 1 of the 3 step plan, a sound engineering decision that that modifications do not change the integrity of the barrier enough to negate the existing NCHRP-350 rating of the barrier.

Please contact me and let me know your thoughts on the viability, cost and timing on this issue.

Thank you for your time.

Response

Date: 02-10-2014

Can you send to us full details of the crash tested design, including where the evaluation was performed, report, etc.? Once all of the information is acquired, I will see if one of our engineers can review the content. Thanks!

Response

Date: 02-11-2014

Attached is the information I was able to obtain through various sources.

I was in a little error in my last information, our intent is to modify the Idaho barrier, not the Utah barrier.

So here is the list of the attachments.

1. July 17, 2000 letter. This is the letter from FHWA that approves the Idaho barrier, and it also has some test data (test 13-4300-001 & 002)
2. The current standard drawing for the Idaho barrier.
3. The submittal to the State requesting modification to the barrier. This includes some "conceptual" drawings of the proposed scuppers.

To be clear, our intent is to modify the 20' Idaho barrier.

Hopefully this is enough information.

Your time is greatly appreciated.

Attachment: <https://mwrsf-qa.unl.edu/attachments/51f81533801be241262356ec9c9a499d.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/223ad8e8453048543f37a1c6a287686e.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/2626ebebaa60567fc9d66e50e81d8338.pdf>

Response

Date: 02-11-2014

We have reviewed the proposed details for revising the portable concrete barriers (PCBs) used by the Idaho Transportation Department. Several proposed modifications were provided.

First, you inquired as to whether the two proposed revisions would be acceptable for use with the free-standing option of Detail G-2-A-1, Sheet 1 of 2. A height change in the drainage slot from 2 in. to 2¼ in. was made. We do not believe that an increased height of ¼ in. would noticeably degrade the safety performance of Idaho's free-standing PCB system that is configured with 20-ft long segments. Second, you inquired as to whether the drainage slots could be altered in the longitudinal direction. At the present, two 1-ft 11-5/8-in. long drainage slots are located near the one-third points along the segment. You inquired as to whether two 4-ft long slots could be used in lieu of the small slots but with an inner blocked region in excess of 26 in. long. At this time, we are not concerned with the length and position of the two drainage slots as prior testing has been conducted with

an interior drainage slot of 4-ft long within a 10-ft PCB system for the Ohio Department of Transportation. The two drainage slots also maintains an inner support at the midpoint of the 20-ft long PCB. In addition, we do not believe that the use of two 4-ft long drainage slots would noticeably degrade the safety performance of Idaho's free-standing PCB system that is configured with 20-ft long segments. Note that MwRSF's guidance does not include tied-down applications.

Second, you again inquired as to whether the two proposed revisions would be acceptable for use with the free-standing option of Detail G-2-A-1, Sheet 2 of 2. A height change in the drainage slot from 2 in. to 2¼ in. was made. As stated above, we do not believe that an increased height of ¼ in. would noticeably degrade the safety performance of Idaho's free-standing PCB system that is configured with 20-ft long segments. Second, you inquired as to whether the drainage slots could again be altered in the longitudinal direction but differently than noted above. At the present, two 1-ft 11-5/8-in. long drainage slots are located near the one-third points along the segment. You inquired as to whether one 8-ft long slot could be used in lieu of the two small slots. At this time, we do not have experience with using one very long slot in the inner region of 20-ft long PCBs. When PCBs are impacted in a free-standing configuration, several PCB segments are pushed backward, while the front base is lifted upward. As such, these shifted barrier segments often become supported by the backside toe or edge during the impact event. With an 8-ft long drainage slot in the inner region, the interior support (and backside edge support) would be removed in each segment, thus increasing the unsupported length of each tipped PCB. For the passing dynamic load on the front face of a tipped PCB, increased bending moments may be observed. Although this configuration may still provide adequate safety performance and not compromise barrier capacity, we do not have experience with using an 8-ft drainage slot even though additional steel reinforcement is provided above the drainage slots. As such, we have concerns regarding the use of a single 8-ft long drainage slot configuration within the Idaho Transportation Departments 20-ft long PCBs. It should be noted that full-scale crash testing and/or computer simulation modeling may later demonstrate that its use is acceptable. Note that MwRSF's guidance does not include tied-down applications.

Third, you inquired as to whether the two proposed revisions would be acceptable for use with the free-standing option of Detail G-2-A-2, Sheet 1 of 2. A height change in the drainage slot from 2 in. to 2¼ in. was made. We do not believe that an increased height of ¼ in. would noticeably degrade the safety performance of Idaho's free-standing PCB system that is configured with 10-ft long segments. Second, you inquired as to whether the drainage slots could be altered in the longitudinal direction. At the present, two 6-in. long drainage slots are located near the one-third points along the segment. You inquired as to whether one 3-ft 10-in. long slot could be used in lieu of the small slots in the 10-ft long PCBs. At this time, we are not concerned with the length and position of the single drainage slot as prior testing has been conducted with an interior drainage slot of 4-ft long within a 10-ft PCB system for the Ohio DOT. In addition, we do not believe that the use of a single 3-ft 10-in. long drainage slot would noticeably degrade the safety performance of Idaho's free-standing PCB system that is configured with 10-ft long segments. Note that MwRSF's guidance does not include tied-down applications.

Please let me know if you have any questions or comments regarding the information contained herein! We look forward to hearing from you on this matter in the near future. Thanks!

LRFD Requirements for Guardrail Posts in MSE Walls

Question

State: WY

Date: 02-13-2014

Section 11, page 77 of the AASHTO LRFD requires a post embedment of five feet into an MSE wall, and placed back three feet from the edge. Was this configuration crash tested or was there any research to establish these requirements? Does this adversely affect the performance of the guardrail? Is this something that needs to be added to the Roadside Design Guide? Also, I'm not sure how you can drive the posts without cutting some of the geo-textile reinforcement? Any light you can shed on this subject would be helpful.

Attachment: <https://mwrsf-qa.unl.edu/attachments/80f2587f97c61b306c1f9050637f58bd.pdf>

Response

Date: 02-24-2014

Let me start by recommending you review a recent report that MwRSF put together for FHWA - Central Federal Lands Highway Division. It is report number TRP-03-235-11 and can be obtained from the Research Hub link on MwRSF's website. As part of this project, 2 MASH TL-3 full-scale crash tests were conducted on steel post MGS systems placed on top of MSE walls. The barrier system performed successfully, and the wall sustained no visible damage or displacements. Final recommendations for the minimum lateral offset were to center the post 4'-3" from the face of the MSE wall. This offset (detailed in the report) was selected in part to avoid the larger rock of the MSE wall face in which the posts can not easily be driven through. The large rock typically extends 3' laterally from the face of the MSE wall.

I am unsure where these recommendations listed in the AASHTO LRFD specs originated from as they preceded the MwRSF study noted above as well as some work that TTI has been doing with concrete barriers placed on MSE walls. However, the 3' minimum lateral offset may have been specified to avoid the larger rock facing of the MSE wall (since the rock typically extends 3' from the face) - similar to the MwRSF recommendations. The 5' embedment depth specified by AASHTO is interesting. I do not know why this would be specified as the embedment depth should be a function of the barrier post strength. Testing with standard steel guardrail posts with 40" of embedment proved adequate in the MwRSF project for both redirection strength and damage prevention for the MSE wall. Further, the posts bent over in the full-scale crash tests showing the surrounding soil was strong enough to support the posts. Extending the posts deeper into the ground will not have any significant effects on the system or the MSE wall.

To answer your question about interference with wall reinforcement, AASHTO section 11.10.10.4 addresses the issue of reinforcement obstructions and provides a few design methods that can be implemented. I would recommend following those. Any additional reinforcement that may need to be added for the wall should be minimal as the post obstructions have a small footprint and would only be placed every 6 ft.

Cable barrier posts with anchored base plates

Question

State: NE

Date: 02-13-2014

We have observed high tension placement of cable posts across a concrete structure using anchored base plates.

I don't know of a detail for the low tension cable attached similar to this, do you?

Are there concerns with this type of installation.

Response

Date: 02-13-2014

I have not seen a similar detail for common low-tension cable barrier, except for the S3x5.7 slipbase posts used in the terminal region. If the cable barrier design performed in an acceptable manner with line posts embedded in either compacted soil or foundation sleeves with posts plastically deforming at the ground line, then alternative anchoring to rigid pavement should perform in a similar manner. If steel line posts were needed to release out of the foundation system in order for achieve satisfactory performance, then a rigid mounting to pavement could alter barrier performance.

31" Midwest Guard Rail w/ No Blockouts

Question

State: LA

Date: 02-14-2014

Louisiana has a unique situation on one of our projects and we are interested in using the 31" Midwest Guard Rail System without block outs. However, this will not be a typical installation as we intend to use this system on a 4 lane divided highway median. I have attached a picture of the existing route. As you can see, space is extremely limited. Ideally, we would use a concrete barrier here and that IS our intention when we reconstruct this corridor. However, that project is only in the feasibility stages so we need to do something in the meantime to address the protection system in the median.

Like I said, we would like to use the Midwest System because it is non-proprietary but I had a few questions about it before I make my recommendations:

1. Can the Midwest System without block outs be used in a median application like this? Assuming we mirror the detail and place another section of rail along the back of the post, would this negatively affect its performance?
2. Has the non-blocked out system been accepted by FHWA? I found their letter for the standard 31" system but was wondering if they had reviewed it without the block outs.
3. Do you know of any median end treatments / terminals that could be used with the type of installation I am proposing? All of the median end treatments that I have seen use block outs on each side of the post. Does Midwest have a special transition detail to go from a non-blocked out section to one that uses blocks? I'm guessing we will have to make some kind of transition like that in order to use a crashworthy end treatment.

I would certainly appreciate any help or guidance you can give me on these issues.

Please let me know if you have any questions or need more pictures or information.

Response

Date: 02-14-2014

I reviewed your questions regarding the MGS median barrier and have put comments below.

1. Can the Midwest System without block outs be used in a median application like this? Assuming we mirror the detail and place another section of rail along the back of the post, would this negatively affect its performance?
 - a. Currently, there are two approved versions of the MGS median barrier system. MwRSF sought approval of an MGS median barrier with 12" deep blockouts based on the performance of proprietary 31" tall median barrier systems that were tested under NCHRP 350. These include the NUCOR NU-GUARD 31 and the Gregory GMS 31 which have both been given FHWA approval in double-sided median configurations under the NCHRP Report No. 350 safety requirements (FHWA Approval Letters B-150B and B-162). Neither of these systems used blockouts in their median barrier configurations, and the Gregory system was tested with the

guardrail splices at the posts locations. In the approval, we argued that a MGS median barrier system would be specified with splices at the midspan locations between the posts and using 12-in. deep blockouts, as used on the standard roadside MGS design. Previous testing and analysis of guardrail systems had shown that the use of blockouts and placement of the guardrail splices away from the posts tends to increase the capacity of guardrail systems and reduce the potential for vehicle snag. Thus, a MGS median barrier system would have improved safety performance as compared to the existing approved 31-in. high, median W-beam guardrail systems.

Since that time, TTI has performed research into the use of 8" deep blockouts with the MGS median barrier system. TTI successfully tested this system with both the 1100C and the 2270P vehicles under the MASH requirements, which validated our original approval. No issues were noted in those test with respect to reduced blockout depth from 12" to 8".

No research has been done to date with respect to non-blocked MGS median barrier. However, the previous performance of the MGS median barrier with 8" blockouts and NCHRP 350 testing of non-blocked proprietary median barrier designs would suggest that the non-blocked MGS median barrier has a good likelihood of performing acceptably. The concerns with a non-blocked MGS median barrier are that the reduced offset block may increase the snag on the posts which are supported on both sides by W-beam rail. This may increase the forces developed by the snagged posts and the loads transferred to the vehicle. However, occupant ridedown accelerations (ORA) and occupant impact velocities (OIV) from previous MGS testing with and without blockouts involving wheel snag have not indicated concern and it is not believed that the additional support of the post by the backside rail would prove sufficient to increase the snag forces to unacceptable levels. Review of the 1100C testing conducted by TTI on the reduced blockout median system would seem to support this conclusion, as occupant risk values were not increased to significantly when compared to previous testing of the roadside MGS system with 8" blockouts. However, testing of non-blocked MGS did indicate that removal of the blockouts tended to increase the ORA and OIV levels observed. While these increases were noted, they were not near critical levels. Thus, we believe that a non-blocked version of the MGS median barrier would likely perform acceptably, but there is some concern regarding the effect of vehicle snag on the posts that has not been accurately quantified. You may want to consult with FHWA and see if they would concur with this opinion.

2. Has the non-blocked out system been accepted by FHWA? I found their letter for the standard 31" system but was wondering if they had reviewed it without the block outs.
 - a. The roadside version of the MGS without blockouts has been accepted. I have attached the approval. The report detailing the effort can be found at the following location.

<http://mwrsf.unl.edu/researchhub/files/Report9/TRP-03-262-12.pdf>

3. Do you know of any median end treatments / terminals that could be used with the type of installation I am proposing? All of the median end treatments that I have seen use block outs on each side of the post. Does Midwest have a special transition detail to go from a non-blocked out section to one that uses blocks? I'm guessing we will have to make some kind of transition like that in order to use a crashworthy end treatment.
- a. I do not know of any median terminals that do not utilize blockouts.

As far as transitioning blockout depth from existing median terminals to non-blocked guardrail, we would recommend the following approach. Begin transitioning blockout depth no sooner than 12.5' from the end of the terminal system. At that point, you can transition the blockout depth from 8" to 0" over 25 ft of guardrail using gradually reduced blockout depth. This should only generate a 1.5 degree flare in the rail which should have minimal effect on performance.

Let me know if you have further questions.

Attachment: <https://mwrsf-qa.unl.edu/attachments/e5ca40d332a6ff3fca67cc2b24ffbbda.pdf>

Box Beam Adaptation of the MGS Box Culvert Side Mounted Railing System

Question

State: WY

Date: 02-19-2014

We have a box culvert with about a 44 foot span and low fill over the top. A copy of the plan sheet is provided. We would like to use box beam guardrail due to blowing snow issues. To my knowledge no box beam attachment to a box culvert has ever been tested.

This appears to be an ideal case for adapting the new MGS Box Culvert Rail to a box beam rail. As you know, box beam uses the same post (S3x5.7) as the MGS Bridge and Box Culvert Rails. Box beam has been demonstrated in redirection crash tests (mounted at 28 inches) to perform in a much more forgiving manner than conventional w-beam and may resemble redirection tests with the taller, more robust MGS.

Would it be possible to adapt the MGS Culvert System, only using box beam (presumably still mounted at 28")? It would appear that the post spacing might be the biggest issue to determine since I am guessing the deflection of a box beam bridge rail should probably not exceed the MGS Bridge Rail in order to assure there would be no greater wheel snagging from an impacting vehicle as it rebounds back onto the box culvert curb. I would assume that using a box beam with a 3 foot spacing would be quite a bit stiffer than the MGS Bridge Rail, but perhaps at a four foot or even the standard six foot spacing, it might replicate the stiffness of the MGS Bridge Rail. Would this require crash testing or could an adequate "paper" case be made for using such a system?

Attachment: <https://mwrsf-qa.unl.edu/attachments/fe1cbc51d86c46f7cd683043f1e4dcfa.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/bb2f446c098f660d27e6b3348632e6bb.docx>

Response

Date: 02-20-2014

We would agree that there is a significant potential to adopt box-beam guardrail to a culvert mounted system similar to what we have done with the MGS bridge rail based on the similarities of the posts and the fact that the box beam system has successfully met the MASH criteria when tested with the 2270P vehicle at TTI. However, it is not likely that we could recommend the system be used without further research and full-scale testing due to several concerns.

1. The dynamic deflection of the box beam system tested at TTI was 57.7 in. This is significantly higher than the 40-50 in. deflections that are typically observed for the MGS and the MGS bridge rail under TL-3 impacts. This additional deflection may pose some concerns for increased vehicle overhang of the culvert and potential vehicle instabilities. As you noted, reduced posts spacing could be used to address the increased deflection. However, that changes the system from what was previously crash tested and would require further study to determine the level of deflection reduction and the effect of the reduced spacing on the barrier performance. Alteration of the post spacing may also affect the need for stiffness transitions in the system, although that is unlikely.

2. The 28" height of the box beam system and the shape of the rail element may provide a different level of vehicle capture than the 31" high W-beam used in the MGS bridge rail system. The differences in the vehicle capture aren't full known, but it is possible that the box beam rail capture would not be as effective as the MGS W-beam, especially when considering the extension of the impacting vehicle over the edge of the culvert.

3. In order to get approval of a box beam culvert mounted system, small car testing with the 1100C vehicle may be required. It was required with the MGS bridge rail. We would have to contact FHWA and get feedback on if such testing was required for any proposed design.

Concrete tapered end section

Question

State: KS

Date: 02-20-2014

We use a concrete tapered end section for speeds 40 mph or less on permanent and temporary. We show on our drawings a 5:1/6:1 type transitions vertically. I can't find the history on this and thought you might have some history on the tapered end section. I know the RDG indicates 20' minimum length down to 4" from probably 32". This is around an 8:1 transition.

I appreciate your help!

Attachment: <https://mwrsf-qa.unl.edu/attachments/e53b5c50aa9d37c337fb3127c837d7a5.png>

Attachment: <https://mwrsf-qa.unl.edu/attachments/6e718571fb7f154aab0a8c96d895ec22.png>

Response

Date: 03-04-2014

You are correct that the RDG does allow this on lower speed facilities when space is limited. However, this vertical taper has never been crash tested successfully to my knowledge.

Vertical tapers have been tested at TL-2 for low profile PCB designs at TTI. However, these were at TL-2 speeds and had barrier heights of 20" or less, not the 32" specified here.

The taper guidance in the RDG is likely based on information in NCHRP Report 358. Previous computer simulations of the sloped end treatment, as shown in NCHRP Report 358, found that the sloped end treatment for a 32" high barrier can cause overturn of the vehicle for impact speeds of 30 mph or greater.

We have answered a similar question for MoDOT in the past. See the response below as it details the testing of sloped end treatments and our recommendations.

<http://mwrsf-qa.unl.edu/view.php?id=490>

Epoxy Adhesive Specifications

Question

State: WI

Date: 02-25-2014

NCHRP 757 - Long-Term Performance of Epoxy Adhesive Anchor Systems suggests that epoxy adhesives follow the ACI 355.4-11 specifications. However, epoxy adhesive anchor manufacturers typically list the ICC-ES AC308 standard.

What would be an appropriate epoxy specification for our standards?

Response

Date: 03-07-2014

I spoke with some folks at Hilti regarding your questions about what specifications to meet for the epoxy. They stated that generally the adhesive industry has been set to meet the ICC-ES AC308 standard for the last several years. When ACI 381-11 came out with new methods for design of epoxy adhesive anchors, a push was made to adopt ACI 355.4-11 as the standard spec for epoxy adhesives. The ACI 355.4-11 spec was based on ICC-ES AC308 with some small modifications.

Currently, the adhesive industry is working to harmonize those two documents to alleviate any specification issues. They believe that they will do so by 2015. However, according to Hilti, the majority of the epoxy manufactures are going to meet the ICC-ES AC308 spec at this time. Thus, you may want to stay with that spec for the time being so you can obtain adhesive until the specs are harmonized.

See the attached document from Hilti.

Attachment: <https://mwrsf-qa.unl.edu/attachments/d57fa1fc2dcde69887aeb32cd6d2c2d1.pdf>

Removable Guardrail Post

Question

State: MO

Date: 02-25-2014

The Missouri boot heel (extreme southeast) consists entirely of flat farmland drained by a network of levees and ditches. In places, these ditches run parallel to one another, separated by as little as 50 ft. In one location on US 412, there are five parallel ditches in close proximity (See Aerial photo below).

Shielding the bridge ends and the ditches are critical, but so is maintenance access to the levees. In that vein, previous designers have placed wood posts in sleeves so that a single post and rail section can be periodically removed for access (See Opening photo below). I am unaware of any crash testing for this arrangement, but Missouri has had good experience with it for a couple of decades.

The Westbound lanes of 412 are about to receive guardrail upgrades and the levee district has requested that the DOT use the single socketed post again, only with a steel post to avoid the swelling that occurs with wood.

Here are the questions:

1. Has a socketed guardrail post (wood or steel) ever been tested?
2. If not, is the driven socket a reasonable variance of a driven post?
3. Is there any proprietary product that will afford the needed access?

Attachment: <https://mwrsf-qa.unl.edu/attachments/f6436a8bfe7cf0984cf38de0600af6a0.jpg>

Attachment: <https://mwrsf-qa.unl.edu/attachments/0e62fc701aeb53722514949845ed2348.jpg>

Response

Date: 02-28-2014

Strong-post guardrail systems installed into rigid sockets (or rigid pavements) have shown negative results through full-scale crash testing. The restriction on rotation led to increased forces, rail tearing, and vehicle penetrations. Thus, strong-post systems are not recommended for use within a rigid socket system.

On the other hand, if the socket was simply a steel tube/sleeve for the post to sit in, it would translate within the soil similar to the post itself. Similar flange widths and embedment depths between the steel socket and the original post will result in similar soil resistances and similar barrier performance. However, it is recognized that the socket will have to be slightly larger than the post itself, and will subsequently create higher soil resistances. Thus, it is important to size the socket such that the post fits snugly within the socket and the socket width is as close as possible to the width of the post.

Further, it is recommended to utilize steel posts over wood posts for these socketed installations. Within a socket, the post will be subjected to hard points and stress concentrations at the top of the socket. This may lead to premature fracture of a wooden post and degradation of the barrier's safety performance. The same hard points and stress concentrations will only cause a steel post to yield and bent over – maintaining lateral resistance while doing so. Thus, switch to steel posts has more benefits than simply eliminating the wood post swelling issue.



ZOI for TL-5 barriers

Question

State: KS

Date: 03-12-2014

What is the Zone of Intrusion (ZOI) for TL-5 barriers?

Response

Date: 03-12-2014

The following were the ZOI measurements from the TL-5 bridge rail and the TL-5 median barrier tests conducted at MwRSF between 2004 and 2007. The critical ZOI value was found to be 88.5" - measured from the median barrier crash test.

Bridge Rail - Working width = 79.5"

Top corner offset from front of barrier = 5.5"

ZOI = 74"

Median Barrier - Working Width = 86.5"

Top corner offset = 2" (lower slope) + 8" (head offset) = 10"

ZOI = 76.5"

The ZOI values were calculated from the working widths (measured from high speed video) and the offset distance between the top-front corner of the barrier and the front of the barrier. Both of these barriers incorporated a head slap prevention geometry, so the top of the barrier was offset above 35" in height. If you wanted to apply these ZOI numbers to a vertical parapet, conservatively you could add back in the offset distances noted above.

Field Bending of Thrie Beam

Question

State: OH

Date: 03-14-2014

Can you tell me if there is a maximum allowable *field* bending of thrie-beam out in the field? Correct or not, our specifications indicate that we have allowed w-beam to be curved to a maximum of a 70' radius... Thoughts?

Response

Date: 03-17-2014

I have a couple of thoughts on field bending of thrie beam.

1. We have previously bent thrie beam to a radius as small as 34' here at the facility using a press and incremental bending as part of the bullnose project. We found that this was a pretty difficult process as it required many small incremental bends along the length of the beam to produce a consistent radius rather than hinging the beam. I believe that when we did it in the shop with a press, it required bending the rail every 9"-12" along a 12'-5" section.
 2. We would be concerned that this type of process may not be very controllable in a field situation and would lend to thrie beam that was plastically hinged or kinked in one or two locations rather than actually bent in a radius. Formation of a hinge in the rail could weaken the rail section. Thus, we believe that field bending could be done, but we would recommend that it be done gradually and that hinging of the rail be prevented.
-

Precast Concrete Barrier

Question

State: MT

Date: 03-14-2014

I will summarize the issues that were brought up by the MDOT and what I was hoping you would be able to provide a professional opinion on or facts relating to those issues. Again, the issues brought up are by the MDOT in comparing what our supplier had done when pouring the barrier to the Standard Drawing/Specification for the precast Truck Barrier. I make the assumption that the MDOT does not have crash testing for the truck barrier per the March 7, 2014 e-mail from Matt Strizich. How or where the MDOT came up with the dimensions, rebar placement, slope angles, or any other element of the Truck Barrier is yet un-identified. (I have included a picture we took of a form with the cage in it for your reference. Please note the barrier are poured upside down and you are looking at the bottom of the barrier. I will reference the bottom of the barrier even though it is at the top of the picture.)

Issue 1: The opening in the face of the barrier (blockout) is specified to be 2-1/4" high in the MDOT Standard Drawing. Due to differences in form fabrication, the height of the opening varies in dimension from 2-1/4" to 4" high. The concern that was pointed out to me by the MDOT was that if the blockout is 4" high instead of 2-1/4", there is a reduced dimension of concrete coverage for the lowest horizontal bar and bottoms of the hair pin bent bars through that section of the barrier. [In your opinion, does this compromise these barriers?] This ties into Issue #2 in the fact that the supplier was placing the bottom horizontal rebar several inches up on the hairpin bars which actually results in excessive clearance from the form. (See picture - the horizontal bar that is sitting just above the spacer wheel should actually be near the ends of the hairpins or closer to the bottom of the barrier.)

Issue 2: Bottom horizontal straight rebar is not properly spaced. Again referencing the picture, the horizontal bar closest to the bottom of the barrier should be tied to the hairpin bars near the tape measure. [In your opinion, is there any tolerance in the spacing of the rebar and would the barrier be compromised if the horizontal bars are placed as shown in the picture vs the

standard drawing?]

Issue 3: Optional End Loops are being installed upside down. The 90 degree bends are facing towards the bottom of the barrier as you can see in the picture whereas the MDOT Standard Drawing show the bends facing towards the top of the barrier. [Would this have any negative effect on the truck barrier?]

All of these issues have been corrected to meet the MDOT's Standard Drawing moving forward, but we are trying to get the MDOT to accept the barrier previously poured. I will send follow up pictures that we took of the forms with rebar in them waiting to be poured. In these you can see what was happening and hopefully formulate a professional opinion as to the effects it may or may not have.

Response

Date: 03-14-2014

To clarify, you are wanting to use the tall, truck PCB for passenger car applications under NCHRP TL-3? If the 14" SBP issue is not resolved, all other issues noted below are only secondary.

I will summarize the issues that were brought up by the MDOT and what I was hoping you would be able to provide a professional opinion on or facts relating to those issues. Again, the issues brought up are by the MDOT in comparing what our supplier had done when pouring the barrier to the Standard Drawing/Specification for the precast Truck Barrier. I make the assumption that the MDOT does not have crash testing for the truck barrier per the March 7, 2014 e-mail from Matt Strizich. How or where the MDOT came up with the dimensions, rebar placement, slope angles, or any other element of the Truck Barrier is yet un-identified. (I have included a picture we took of a form with the cage in it for your reference. Please note the barrier are poured upside down and you are looking at the bottom of the barrier. I will reference the bottom of the barrier even though it is at the top of the picture.)

Issue 1: The opening in the face of the barrier (blockout) is specified to be 2-1/4" high in the MDOT Standard Drawing. Due to differences in form fabrication, the height of the opening varies in dimension from 2-1/4" to 4" high. The concern that was pointed out to me by the MDOT was that if the blockout is 4" high instead of 2-1/4", there is a reduced dimension of concrete coverage for the lowest horizontal bar and bottoms of the hair pin bent bars through that section of the barrier. [In your opinion, does this compromise these barriers?] This ties into Issue #2 in the fact that the supplier was placing the bottom horizontal rebar several inches up on the hairpin bars which actually results in excessive clearance from the form. (See picture - the horizontal bar that is sitting just above the spacer wheel should actually be near the ends of the hairpins or closer to the bottom of the

barrier.)

For the 12" wide PCB at top, a 4" tall scupper/slot may not significantly affect the large section's capacity or performance in free-standing applications as compared to the 2.25" tall slot. However, the 14" height to slope break point may be the bigger issue.

Issue 2: Bottom horizontal straight rebar is not properly spaced. Again referencing the picture, the horizontal bar closest to the bottom of the barrier should be tied to the hairpin bars near the tape measure. [In your opinion, is there any tolerance in the spacing of the rebar and would the barrier be compromised if the horizontal bars are placed as shown in the picture vs the standard drawing?]

For the over-designed truck barrier used in passenger car applications, it may not be a big deal for minor changes in rebar position. However, the 14" height to slope break point may be the bigger issue. For 6" wide PCBs at top, changes in rebar position may have a more dramatic effect on barrier capacity due to a much reduced level of reserve capacity. Inward movement of longitudinal bars will reduce a barrier's bending capacity above a vertical axis.

Issue 3: Optional End Loops are being installed upside down. The 90 degree bends are facing towards the bottom of the barrier as you can see in the picture whereas the MDOT Standard Drawing shows the bends facing towards the top of the barrier. [Would this have any negative effect on the truck barrier?]

I do not believe that a 180 degree change in bend will greatly change anchorage capacity or barrier performance.

All of these issues have been corrected to meet the MDOT's Standard Drawing moving forward, but we are trying to get the MDOT to accept the barrier previously poured. I will send follow up pictures that we took of the forms with rebar in them waiting to be poured. In these you can see what was happening and hopefully formulate a professional opinion as to the effects it may or may not have.

Bridge Rail Retrofit to Bridge Curb

Question

State: KS

Date: 03-17-2014

KDOT has an existing bridge deck and bridge rail curb that they would like to retrofit. Can you review the proposed configuration attached and provide feedback.

Attachment: <https://mwrsf-qa.unl.edu/attachments/f72d9def48d516c237a1d85fd2ec6a04.jpg>

Response

Date: 03-17-2014

We looked over you attached bridge rail retrofit that we discussed on Friday and we have a couple of thoughts.

1. The proposed attachment of a blackout and W-beam poses a couple of concerns. First, we do not know the reinforcement of the deck and curb combination, so we cannot comment on whether the attachment has sufficient strength to develop the attached post. A bigger concern is the combination of the W-beam with the sloped 10" tall curb. The combination of the sloped curb face and the deflection of the W-beam under load creates the potential for vehicle climb which may lead to instability and/or override. Thus, we would not recommend that configuration as shown.
2. We also discussed the MGS bridge rail system. Currently we have mounting systems for sockets off the edge of the deck and culvert mounted designs. We cannot recommend attachment to the top or back of the 10" curb due to the stability and reinforcement concerns noted above. We had discussed removal of the curb and mounting the posts to the bridge deck. However, your deck design is only 6" thick and may not have the capacity to handle the loading of the socket mounted off the edge of the deck that we originally tested.
3. That leads to our best option. TTI has tested the MGS bridge rail at TL-2 with top mounted posts that bolt through the deck at standard 6'-3" posts spacing. See attached. We believe that this system would likely meet TL-3 at the half post spacing that we tested with using the TTI top mounted attachment. Again, there are concerns with mounting the posts close to the edge due to the thickness of the deck and the unknown level of deck reinforcement. However, we believe that if the curb was removed and the MGS bridge rail posts were mounted approximately 1' in from the edge of the deck, there would be potential for the deck to be able to handle the attachment loads. This would reduce your lane widths, which is less than desirable, but it may be the best option we have.

Look through this and let me know what you think. We can discuss it further.

There is another option. You could install the TL-2 concrete railing that we developed for Nebraska a while back. This would require removal of the curb as well and would not be TL-3. It would also require looking into the deck capacity. However, I wanted to throw it out there as an option.

Attachment: <https://mwrsf-qa.unl.edu/attachments/d124aa06a3bac58edbca63170e5a0271.pptx>



retro fitting thrie beam approaches

Question

State: WI

Date: 03-18-2014

I have some material questions. See attached

Attachment: <https://mwrsf-qa.unl.edu/attachments/ce2bf3f9096672249393d55f901bc2a6.pdf>

Response

Date: 03-18-2014

Part a1: Yes, the beam can be A36 or A992 steel

Part a2: Yes, the posts can be A36 or A992 steel

Part a3: Yes, the plate can be A36 or A992 steel

Part b1: Yes, a standard hex head ASTM A563 grade A nut

Grade2 steel refers to SAE J429 Grade 2 for the bolt

Part b3: Yes, Standard guardrail nut

Part b4: Use F844 washers, I don't know why Grade 2 is included in the B.O.M. may have been a typo

End Anchor Foundation Tube Thickness

Question

State: OH

Date: 03-18-2014

Is it acceptable to use 1/8" thick foundation tubes in the trailing end anchorage for the MGS versus the 3/16" tube that we have typically tested with in the past.

Response

Date: 03-18-2014

I spoke with designers of the FLEAT and SKT systems. They indicated that they have tested the wood post versions of those terminals with the MGS using the thinner foundation tube. It is likely that the 1/8" tubes that you have originated from this work. The 1/8" thick tubes performed acceptably in their testing, which would indicate that they would be acceptable for the trailing end anchorage as well. The 1/8" tubes would also have a higher section modulus than many of the proprietary steel post end terminal anchorages that rely on the weak action section modulus of W section posts to develop anchorage. Thus, we believe that the 1/8" thick tubes are acceptable.

Thanks

Nested Guardrail Recommendation for Thrie Beam Transition - Task Force 13 Presentation

Question

State: FL

Date: 03-18-2014

At the last Task Force 13 meeting in College Station (see attached agenda), Karla gave a presentation on guardrail and discussed crash test failure of the W-beam to Asymmetric Thrie-beam connection. During that presentation she mentioned that subsequent successful crash tests were conducted with the W-beam nested. I am requesting the crash test results which show the successful crash testing of that Critical Impact Point with the nested W-beam.

I have incorporated a nested 12'-6" W-beam into our FDOT Design Standards and require the crash test results and/or research report for my files. If there are issues with this design, then I need to understand what they are and what can be done to rectify the issue. I have included a copy of our Detail "J" from Index 400, Index 402 and Index 430 for your review and input. These designs are based on the TxDOT and TTI crash tested system with 8" block-outs and panels splices set to mid-span. These designs incorporate the addition of a nested W-beam as based on Karla's TF13 presentation and was reviewed by the Washington Safety Office (Dick Albin) with approval for release by our FHWA Division Office. Please, advise on the status of the requested documentation.

Note:

We have released these drawings based on crash test information provided by TTI and discussions with MwRSF staff and final approval from FHWA.

31" guardrail designs are currently underway which depict the nested W-beam and these designs will begin construction for projects to begin on or after July 2014.

I would like to put this documentation request to rest. Your input and support is greatly appreciated.

Response

Date: 03-18-2014

The report is not complete yet. It is still in internal review within MwRSF and then will be sent to the Midwest States Pooled fund states for review. We can forward it to you when it is final. The report along with the CAD and videos will be uploaded to our Research Hub site when it is final. In the mean time, we can forward the videos of that test if you would like.

After reviewing the materials you provided, we wanted to bring a few things to your attention. The transition system that included the nested W-beam was a transition used in conjunction with a curb beneath it. If the transition does not include a curb, you would want to use the details contained in the following report:

<http://mwrsf.unl.edu/reportresult.php?reportId=38&search-textbox=transition>

Within this report are implementation guidelines (Chapter 13) and adaptation recommendations to different approach transitions than what was used during testing (Chapter 14). Within the recommendations are recommendations regarding the use of flared guardrail in the transition. For flared guardrail applications, a minimum length of 25 ft was recommended between the upstream end of the asymmetrical W-beam to thrie beam transition section (i.e., post m on your Index 400) and the start of the flared section (i.e. bend between

flare and tangent sections). No flaring of the actual transition was recommended.

Flaring of the transition itself was not recommended due to limited knowledge regarding the use of flared approach guardrail transitions and their safety performance. At this time, no research or full-scale crash testing of flared approach guardrail transitions has been conducted under the NCHRP Report No. 350 or MASH evaluation criteria. However, previous testing of flared guardrail systems and transitions suggests several concerns related to flared transitions. First, flaring of the transition would tend to increase the effective impact angle on the barrier, which would raise the potential for vehicle snag on both the posts and the end of the rigid parapet. Similarly, an increase in the loads imparted to the barrier would be expected due to the higher effective impact angle and an associated increase in the potential for barrier pocketing. Loads on the rail elements in approach transitions are already increased due to limited barrier deflection, and there is potential for increased flare rates to raise the loads in the rail elements to the point of rail rupture. Full-scale crash testing of the MGS upstream stiffness transition with the 1100C vehicle indicated that wedging of the vehicle under the asymmetrical W-to-thrie beam transition section and snag on the posts in the system resulted in relatively rapid vehicle deceleration and yawing. While the deceleration did not raise occupant risk issues to critical levels in those tests, there may be potential for increased occupant risk values as the flare rate increases for the critical small car impact. Finally, the use of flared transitions may increase the potential for vehicle instability due to the increased impact angle, increased vehicle snag, and the increased potential for pocketing.

In addition, nesting of the W-beam that you show in Index 430 would not be necessary. The reason for the nesting of the W-beam rail in the system with a curb was to stiffen the rail just upstream of the asymmetrical transition section in order to prevent rail rupture. With a double sided system, we believe that the additional stiffness would not be necessary.

Let me know if you need further information.

32-inch Concr Barrier Test Level

Question

State: MN

Date: 03-19-2014

I remember hearing that the 32-inch, Concrete J-Shape Barrier did not pass the MASH TL-4 test. As I remember, it was determined to be mostly a height issue (therefor the F-Shape was also considered to be a likely failure as well).

Do you know where that information is located?

Also, do you know if the 42-inch barrier was looked at?

Response

Date: 03-31-2014

You are correct that TL-4 testing of the 10000S single unit truck vehicle during the NCHRP 22-14 project resulted in the truck rolling over the rail in tests at TTI and MwRSF. The increased mass and velocity of the new TL-4 test criteria in MASH have created a more severe test both in terms of impact loading and vehicle stability.

I have attached the TTI report to the email and placed a link to the MwRSF report below.

<http://mwrsf.unl.edu/researchhub/files/Report146/TRP-03-177-06.pdf>

Subsequent to this work, MwRSF and TTI have done some research into revised barrier heights. MwRSF performed simulation analysis of a vertical shape precast bridge rail design that indicated that 34.5" high vertical shape barriers could potentially meet MASH TL-4. However, this system was never full-scale crash tested to verify those results.

TTI also performed simulation analysis of single-slope barriers that indicated that 36" high single-slope barriers would meet MASH TL-4. TTI tested the 36" high single-slope barrier with the 10000S vehicle and found that it did safety redirect the SUT vehicle.

<http://tti.tamu.edu/documents/9-1002-5.pdf>

No testing of increased height safety shape barriers has been performed to date, so we cannot definitively say what the height for those sections would be. However, based on the previously research, it would be recommended that a minimum height of 36" would be reasonable and that the required height may decrease as the barrier geometry becomes more vertical.

No 42" high barriers have been tested to MASH TL-4 to the best of my knowledge. It is highly likely that these barriers would meet MASH in terms of vehicle stability. Keep in mind that it is believed that the MASH TL-4 impact loads have increased as well due to the increased speed and mass. Thus, barrier reinforcement for existing TL-4 barriers may need to be re-evaluated.

Thanks

Attachment: <https://mwrsf-qa.unl.edu/attachments/c3844d6f3ce3c1bef6b75546fbf67fd2.pdf>

Rock installation for thrie beam bullnose post in steel tubes

Question

State: WI

Date: 03-21-2014

Is there a way to anchor the steel foundation tubes for the thrie beam bullnose into a rock foundation with out fully excavating a 8 ft or 7ft hole for the tube?

Response

Date: 03-31-2014

I believe that this questions was at least partially addressed in a previous consulting question.

<http://mwrsf-qa.unl.edu/view.php?id=612>

As far as anchoring to rock, no such system has been developed to date and may be difficult due to the irregularity of the rock surface as well as the structural strength of the rock itself.

It may be possible to core a hole in the rock to place the foundation tube within and fill around it. This could reduce the overall depth of the tube, but the depth of that cored hole would need some further consideration.

A third option would be to install a reduced depth foundation tube with a large soil plate. However the dimensions of the soil plate and the feasibility of this option would be dictated to some degree by the amount of fill material above the rock obstruction.

Top mounted Post to Box Culvert Design

Question

State: WY

Date: 03-27-2014

Top mounted Post to Box Culvert Design – MwRSF tested a box culvert design for 27 2/4 inch high corrugated beam guardrail using a half post spacing in 2002. The results are described in TRP-03-114-02. A recent project was to simplify the weld detail. I believe the recommendation was to leave the weld the same as the original design. There was no upper bound listed for the amount of fill above the culvert. The lower bound was 9 inches of fill. Is there an upper bound to the amount of fill for the post to function properly? Can this design be used for 31 inch high MGS guardrail as well?

TTI recently (2012) tested a similar design with a 31 inch high w-beam guardrail and posts at a standard spacing. Again, there was no upper bound recommended for the depth of fill on top of the culvert. Is there a limit or does post performance either improve or stay about the same with greater fill depths.

I think I heard TTI also tested another box culvert mount using weak posts at a half post spacing. Is this true and do you have any details about the design and/or contact information?

Any other comments about this design would be appreciated. We love the Midwest side mounted socket design, but there are several existing box culverts where we cannot mount the guardrail that far out due mainly to grading issues for the approach guardrail.

Attachment: <https://mwrsf-qa.unl.edu/attachments/73ac90f4b634701cc86a1fff674bb070.docx>

Response

Date: 03-31-2014

See responses below in red.

Top mounted Post to Box Culvert Design – MwRSF tested a box culvert design for 27 2/4 inch high corrugated beam guardrail using a half post spacing in 2002. The results are described in TRP-03-114-02. A recent project was to simplify the weld detail. I believe the recommendation was to leave the weld the same as the original design. There was no upper bound listed for the amount of fill above the culvert. The lower bound was 9 inches of fill. Is there an upper bound to the amount of fill for the post to function properly? Can this design be used for 31 inch high MGS guardrail as well?

The guardrail attached to culvert design was developed and tested with the minimum allowable fill level. It is believed that increased soil fill levels will function similarly.

This system has been adapted for use with the MGS by KDOT based on guidance provided by MwRSF. We are still recommending that 1/2 post spacing be used at this time. I have attached the KDOT details below.

There is a proposal in the upcoming Year 25 Pooled Fund to evaluate the system with full-post spacing and minimum offset to the culvert headwall.

TTI recently (2012) tested a similar design with a 31 inch high w-beam guardrail and posts at a standard spacing. Again, there was no upper bound recommended for the depth of fill on top of the culvert. Is there a limit or does post performance either improve or stay about the same with greater fill depths.

As with the system developed at MwRSF, we would expect the system performance to remain similar for increased fill depths for the TTI system as well.

I think I heard TTI also tested another box culvert mount using weak posts at a half post spacing. Is this true and do you have any details about the design and/or contact information?

TTI tested a TL-2 version of the MGS bridge rail with full-post spacing and S3x5.7 weak posts that were attached to a bridge deck using through bolted base plates. This system would likely work at TL-3 with 1/2 post spacing. TTI did have some deck reinforcement modifications for this design in order to place it near the edge of the bridge deck. I do not have the report for this system, but I do have a presentation with details and have attached it. William Williams at TTI can provide more information on this system as well.

Any other comments about this design would be appreciated. We love the Midwest side mounted socket design, but there are several existing box culverts where we cannot mount the guardrail that far out due mainly to grading issues for the approach guardrail.

Attachment: <https://mwrsf-qa.unl.edu/attachments/f7a1f06fcf05333a9636f16bf09a402.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/7e29cc92f0cf6b14096f3361c6a57f4a.pdf>

Attachment: <https://mwrsf-qa.unl.edu/attachments/16022894669e90ca9d7313ab939d7478.pdf>
