Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

04-01-2015 to 07-01-2015

Thomas County 2108-01 PCB Anchorage

Question

State: KS
Date: 04-01-2015

We have a question from a contractor regarding the acceptability of the attached anchorage system for use in temporary concrete safety barrier applications. I have not seen a system like this used before. Could you please take a look at the attached information submitted to KDOT and let us know if you have any concerns with this anchorage system? I have also attached our Standard Drawing depicting our typical anchorage system.

Thanks for your help,

Attachment: https://mwrsf-qa.unl.edu/attachments/5d2c84f3d5d7986df28b664ade33d4bd.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/5bf03c80e94717489debf9b933b8b96a.pdf

Response

Date: 06-07-2015

The system that was tested for Kansas to anchor the TCB through the holes in the toe of the barrier used â...›-in. diameter, ASTM A307 anchor bolts with heavy hex nuts and 3-in. x 3-in. x ½-in. thick washers spaced evenly across the traffic side of each PCB segment. Each anchor bolt was epoxied into the concrete with an embedment depth of 12 in. The test installation consisted of sixteen 12-ft 6-in.) long, redesigned F-shape PCB segments placed adjacent to a simulated bridge deck edge with a total system length of 204 ft. During test no. KTB-1, a 4,448-lb (2,018-kg) pickup truck impacted the system 5 ft – 5 in. upstream from the joint between barrier nos. 8 and 9 at a speed of 62.0 mph (99.8 km/h), and at an angle of 25.3 degrees. The system contained and redirected the vehicle with maximum lateral dynamic and permanent set deflections of 11.3 in. and 3½ in., respectively, and was considered successful according to TL-3 of NCHRP Report No. 350.

In the past, we have often been asked what embedment depth was required for the epoxy anchorage of the A307 rods used in that system. Adhesive anchorage capacity depends on many factors, including anchor size, anchor embedment, concrete strength, adhesive bond strength, spacing effects, edge effects, and other factors. Thus, we have typically recommended that the embedment for the anchor rods should be selected to develop the ultimate shear and tensile capacities of the anchorage. For the 1 1/8" dia. A307 rod, the ultimate shear and tensile capacities are 26.4 kips and 45.8 kips, respectively.

In the case of the alternative anchorage shown, we would make a similar recommendation. Thus, the cast in anchor shown would need to develop shear and tensile capacities of 26.4 kips and 45.8 kips, respectively. It appears that the system shown has an allowable anchor capacity of 27 kips and a max capacity of 54 kips in tension. The shear loading is not listed. Thus, I would check the shear loading and confirm it is sufficient.

Do they install these cast-in-place anchors when the concrete is poured and then set the barriers in those exact locations? This seems difficult.

Temporary Barrier Rail questions

Question

State: IA
Date: 04-07-2015

I have two questions regarding the attached Temporary Barrier Rail standard (BA-401) that I received from the field and would appreciate your assistance with.

1. On page 3 we show details of Strap and Stake Anchorages. We state a Strap Anchorage is only allowed on PCC and Bridge Decks and provide the anchor bolt dimensions in circle note 6, while the Stake Anchorage is allowed on Composite, HMA, and PCC. We have a contractor asking if they can use a Strap Anchor on a Composite (say 3" HMA overlay over original PCC) if they use a longer bolt to provide the required depth into the PCC as specified in circle note 6, in this example at least 3" longer. Thoughts on whether the longer bolt should provide the originally intended anchorage?

2. Also on page 3 we have Table A Anchorage Requirements, which indicates to designers and contractors when TBR needs to be anchored. The question has come up as to how many sections upstream and potentially downstream of an obstacle need to be pinned as well. I've attached a mock situation to assist. Let's assume the dropoff is sufficiently great, so the TBR needs to be pinned and only the minimum 6" offset is available. I'm taking the worst case blanket approach here but also realize that pinning an entire run of TBR for one small section with an obstacle is too conservative.

    a. What distance should be pinned upstream of the obstacle if we assume there are at least eight sections of 12.5' long unpinned TBR upstream of these potentially anchored sections to sufficiently redirect a vehicle per other guidance (TRP-03-209-09, page 6 for one)?

b.
What
distance should be pinned upstream if we have less than eight sections
upstream?

Thanks in
advance for your time and assistance.

Attachment: https://mwrsf-qa.unl.edu/attachments/a79689f13efd50da3269471dd92c3c94.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/b1cc124da162744d78c93f710eabd5fc.pdf

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Response

Date: 04-13-2015

Comments below.

Let me know if you have further questions.

I have two questions regarding the attached Temporary Barrier Rail standard (BA-401) that I received from the field and would appreciate your assistance with.

1. On page 3 we show details of Strap and Stake Anchorages. We state a Strap Anchorage is only allowed on PCC and Bridge Decks and provide the anchor bolt dimensions in circle note 6, while the Stake Anchorage is allowed on Composite, HMA, and PCC. We have a contractor asking if they can use a Strap Anchor on a Composite (say 3" HMA overlay over original PCC) if they use a longer bolt to provide the required depth into the PCC as specified in circle note 6, in this example at least 3" longer. Thoughts on whether the longer bolt should provide the originally intended anchorage?

   The use of the steel strap tie-down has been restricted to concrete pavements with overlays due to concerns that installation through asphalt will increase the bending loads on the bolt and the moment on the drop-in anchor that could reduce the capacity of the anchorage. Thus, we would not recommend using a longer bolt with the drop-in type anchor.

We looked into this issue and some potential alternatives previously for Missouri and did not come up with a solution.

http://mwrsf-qa.unl.edu/view.php?id=636

2. Also on page 3 we have Table A Anchorage Requirements, which indicates to designers and contractors when TBR needs to be anchored. The question has come up as to how many sections upstream and potentially downstream of an obstacle need to be pinned as well. I've attached a mock situation to assist. Let's assume the dropoff is sufficiently great, so the TBR needs to be pinned and only the minimum 6" offset is available. I'm taking the worst case blanket approach here but also realize that pinning an entire run of TBR for one small section with an obstacle is too conservative.

    a. What distance should be pinned upstream of the obstacle if we assume there are at least eight sections of 12.5' long unpinned TBR upstream of these potentially anchored sections to sufficiently redirect a vehicle per other guidance (TRP-03-209-09, page 6 for one)?

    With respect to the upstream side, the length of anchored barrier on the upstream side would be based on your deflection to the hazard. It could be as simple as the anchored barrier only being needed directly in front of the hazard as that is where the deflection needs to be reduced. It should be noted that an approach transition is needed for the steel pin and bolt through tie-down options.

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

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

    b. What distance should be pinned upstream if we have less than eight sections upstream?

    I am not sure I follow. The pinned/anchored sections need to be directly in front of the reduced deflection area of the hazard. Then an stiffness transition must be placed on the upstream end as noted above. We would recommend 8 free-standing TCB segments on both the upstream side of the transition and following the downstream end of the pinned/anchored TCB's. On the downstream side of the pinned/anchored TCB's, a transition is not needed, but we still recommend that 8 free-standing barrier be used. This distance may be able to be shortened following ongoing research on TCB LON requirements being conducted through NDOR, but we are recommending a conservative approach for now.

Thanks in advance for your time and assistance.

Temporary Barrier Rail opening

Question

State: IA
Date: 04-08-2015

We are developing a detail to allow an opening in TBR for contractor access and I'm struggling to find guidance on what buffer length we should be using upstream of the opening for both the anchored and unanchored conditions. Attached is the current draft version.

The downstream unanchored distance comes from the eight sections of TBR needed to develop strength per page 6 of TRP-03-209-09. The anchored distance comes from an assumption that work is not taking place behind the flared TBR sections. That assumption may or may not be correct in the field, but from an ideal situation, it seemed like a reasonable one.

Please advise and thank you for your time.

Attachment: https://mwrsf-qa.unl.edu/attachments/fe4368b0ad7cfea8e53d62c941e55abd.PDF

Response

Date: 04-13-2015

I have reviewed the detail you sent.

Currently, we have recommended that a minimum of 8 barrier segments be used prior to the beginning of length of need or downstream of the end of the length of need for TCB installations. This recommendation is based on testing of these systems typically being performed on 200' long (16 barriers) systems. Thus, what you have shown in the detail for free-standing TCB is appropriate based on our current knowledge. We do not have a clear definition in the performance of the TCB when impacted near the ends of the system, so we have been conservative.

We currently have a project with NDOR to evaluate the actual beginning and end of LON lengths for the F-shape TCB, but that work is not yet completed.

In the past we have also recommended that openings in runs of TCB be overlapped. I am not sure if that is possible for your purposes, but this option is a little more well defined, as the overlap of the PCB runs creates less uncertainty near the ends of the installations. For overlapping TCBs, we have recommended an overlap of at least 8 or 9 barrier segments for each run - front and back. The gap between both barrier runs could be reduced to 6 to 12 in. or so due to both barrier systems being freestanding, thus reducing the propensity for vehicle snag/pocketing. If limited space exists at the roadside edge for the overlapped option, one may consider the slight flaring of the rearward (shielded) TCB system in order to save space near the shoulder. In your case, a larger gap could be used to facilitate vehicle access. In addition, overlapping of the TCB systems eliminate the need for the sand barrel array.

Some states have asked about using the steel pin tie-down to anchor the segments and shorten the beginning and end of LON, however, we have not investigated that at this time and it may need to be further investigated prior to our full endorsement of it.

For the downstream end of the system, we have developed an end anchor for the TCB system that could be applied. This anchor was tested to MASH TL-3 and allows for the LON to start at the first barrier segment. See report below.

http://mwrsf.unl.edu/researchhub/files/Report63/TRP-03-209-09.pdf

You refer to this below, but I don't believe that you would need the 50' denoted in the detail for the anchored system. If the TCB were unanchored, I would recommend sticking with the 100' shown.

Let me know if you need anything else.

Bullnose in Gore Area

Question

State: WV
Date: 04-10-2015

The Internet Police kicked me off Linkedin and you guys apparently had some server issues yesterday.

What would be Midwest's thoughts of using the new Thrie Beam Bullnose in the gore of an exit ramp?

Everyone here thinks it's a great idea, but I remind them that at what point are we going to begin a flare to match the existing mainline and ramp guardrail.

I would propose to flare at Post 8 as shown on the Wisconsin drawings. Is this Wisconsin detail a result of the Midwest research?
The flare at Post 8 I propose is based on the beginning of “Unbent Standard Thrie Beam".

I am trying to address an existing ramp where the mainline and ramp guardrail was terminated with a pair of Tangent End Terminals essentially beside each other. None of us believe this is an acceptable design, since to our knowledge there is no testing of TET's in this placement.

The TET's do not look like this today, I have some pics if you would like to see them.

Attachment: https://mwrsf-qa.unl.edu/attachments/c3fe5b66f5d364d6e1752a2a99c96857.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/510db0b878eb3098880217307445c5c7.pdf

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Response

Date: 04-14-2015

The detail you sent from WisDOT is based on our bullnose system that was tested to NCHRP 350 criteria. We do believe that it can be used in gore areas and it has been done in the past. You can get the reports and other details for the system at the links below.

http://mwrsf.unl.edu/researchhub.php?search-textbox=bullnose&submit=Search

With regards to the start of the taper or flare, we have allowed flaring of the system to begin at post no. 5 with a flare rate of 15:1 based on RDG guidance. That flare may change based on your conditions and the RDG guidance for them.

With regards to the grading, we have addressed this issue in the past by stating that the bullnose itself should be on a maximum grade of 10:1. This applies to cross slopes and v-ditches. We also recommend that the 10:1 slope area be applied for at least 60' in front of the system to provide for more stable tracking of errant vehicles prior to impact. For the longitudinal slopes prior to the 10:1, we have come up with several options in conjunction with the Minnesota DOT. Can get you those if you need them.

Only one other thing to note. Because the gore installation has similar traffic flow on both sides, you will have to make sure that the thrie beam splices are lapped correctly with the traffic flow on each side of the system. For two way traffic, lapping all of the guardrail the same as it moves around the system is fine. However, with the gore installation, this would cause the splices to be setup wrong along one side. I would suggest simply switching the splice on either side of the nose as needed.

Thanks

Black Rebar vs. ECR or Calcium Nitrite in PCB

Question

State: WI
Date: 04-15-2015

MnDOT
needs MwRSF's opinion on the following Portable F-Shape Concrete Barrier specification (reinforcing bar specification).

We currently specifically the following on our plan (attached):

> *Reinforcing steel shall be Grade 60 and shall conform to either of the following:*
>
> *Epoxy-coated deformed bars as specified in Spec 3301.*
>
> *Spec 3301: deformed and plain billet steel reinforcing bars for use with calcium nitrite corrosion inhibitor (30% calcium nitrite solution.)*

The spec 3301 is a MnDOT specification concerning Reinforcement Bars (attached).

A question has come up from a manufacturer. They are asking if the epoxy-coated steel reinforcing is required for crash test performance. They are also asking the same question regarding the alternative of using use of corrosion inhibitor in the concrete.

The barrier will not be owned by MnDOT, that is, it will remain the property of the Contractor. The barrier is required (and inspected) to be in good condition every time it's placed on a MnDOT project, but it will always be the property of the contractor. Therefore, the long term durability and barrier condition risk is transferred to the contractor if they choose a less durable construction method for the barrier (provided that it does not risk the barrier safety performance).

A few of our neighboring states just call out the following:

> *Use Grade 60, ASTM A615 for these bars.*

Attachment: https://mwrsf-qa.unl.edu/attachments/59614fa63b15ba696799522152ee100a.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/6e21578ac1e23ffbea95f3d695e59501.pdf

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Response

Date: 04-15-2015
Hello Michael,

I reviewed Minnesota's specs and some additional supporting material regarding corrosion protection for concrete barrier reinforcement.

First, I wanted to highlight a Pooled Fund consulting question answered in 2010 regarding concrete barrier reinforcement specs for the loop bars:

The loop bar steel is the A706 spec because we have found that the small bend diameter can cause reduced ductility and toughness in some grades of steel which compromises the impact strength of the loop. As such, we current specify that the loop steel must have a minimum yield strength of 60ksi, a minimum tensile strength of 80 ksi or 1.25 times the yield strength " whichever is higher, and a minimum % elongation of 14%. A706 and A709 steel can both meet that spec with the correct grade. Others may as well. The bars can be deformed or smooth as long as the steel is within spec. Some of our states prefer smooth, so it is on the drawings that way.

Minnesota's specs are similar for the loop bars with regards to strength. However, I did not see a note for how the loop bars are protected from corrosion. Some states have used galvanization or stainless steel for loop bars. Epoxy-coated loop bars may experience cracking or peeling during installation or impacts, and exposure to the environment may create a galvanic circuit.

Several online resources have proven valuable for evaluating effectiveness of epoxy coated reinforcement (ECR) and generalized corrosion resistant reinforcement (CRR), compared to "black bars" (BB). Those resources are attached.

Based on the available research, I concluded the following:

(1) In the short term, BB has a higher concrete chemical and physical bond than ECR. Thus, use of ECR may decrease reinforcement bond strength slightly. The effect is not particularly pronounced and MwRSF has used both types of reinforcement in concrete barriers with success. The MIT research synthesis indicated that the recommended factor to account for bond slip in reinforced concrete was 1.35x, and the ACI code recommends a 20-50% longer anchorage length to account for reduced concrete-to-epoxy interface strength. In conclusion, the bond strength of BB is slightly higher than ECR, but in general the effect is not extreme.

(2) Service life of ECR reinforcements have been shown to be vastly increased compared to BB. Although there are still maintenance issues that occur over time, particularly in harsh environments with frequent freeze-thaw cycles and in environments with large chlorine concentrations such as locations with heavy use of chlorinated de-icers or salts, ECR may increase service life for permanent concrete structures by 50 years or more. Note a case study in the MIT synthesis in which a parking garage in Minneapolis using BB and with actual clear cover of concrete of 1" resulted in extensive repairs within 12 years. Although other ECR structures in Canada, Florida, and Iowa still corroded over time, and the epoxy lost the chemical bond with the steel during exposure and when surface cracks were present, the predicted increase in service life was 40 years when both top and bottom reinforcing mats of the bridge deck were ECR, compared to ECR in the top mat and BB in the bottom mat. Also note that the ECR may be damaged by aggregate when nozzles are used to pour concrete over a form; damage can be reduced and potentially eliminated by keeping the nozzle close to the form.

(3) Benefits of ECR are heavily dependent on the attention paid in construction, as noted by Dr. Hartt at FAU (note Review_Corrosion_Performance_Epoxy_Hartt_2012.pdf). Hartt noted that all CRR structures benefit from high-quality concrete and good clear cover. Reduction in clear cover or excessive water-to-concrete ratios result in a reduced benefit of all CRR. In general, subject to the high quality concrete and clear cover as noted, CRR generally increase service life by approximately 10x with respect to simple BB.

(4) Lastly, calcium nitrite is effective in increasing the threshold of chlorine concentration at which corrosion begins to appear. It does not prevent corrosion if the chlorine concentrations are higher than the threshold. However, the service life over which this increase in concentration builds up without damage to the structure may be significant, particularly in locations such as Minnesota that does not heavily use chlorinated de-icers.

In conclusion, the use of BB in temporary barriers offers a short term increase in reinforcement-to-concrete bond strength compared to ECR, but does not offer any other short or long term benefits. In contrast, CRR does increase service life of the barrier by approximately a factor of 10 if adequate clear cover and good quality concrete is maintained. After an impact, if the concrete cracks, the benefits of the CRR will be significantly reduced. Lastly, if all bars, including loop bars, are BB, it may be appropriate to inspect the loop bars for corrosion and estimate internal damage based on the corrosion of the loop bars. This method may not be perfect as it is impossible to accurately identify which elements corrode faster with the current state of knowledge, but BB loop bar inspection is at least a minimum service inspection that could indicate if additional problems may be present.

I recommend that any barriers cast with BB only should have a date stamp indicating the casting date. and any barriers found to have corrosion with the same or similar casting date be fully inspected. Both corrosion resistant reinforcement (CRR) and black bars (BB) may be used and will develop the appropriate strength, given consideration for the increase in lap length and anchorage length for ECR.

Attachment: https://mwrsf-qa.unl.edu/attachments/f878a0d70a5f7af516f52e8e9897b68a.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/27ab659bb9ca385b368787bb710910d2.pdf

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Bridge Overlay - Concurrence requested

Question

State: NE
Date: 04-16-2015

NDOR is planning 2" overlays on Bridge structures & your concurrence is requested:

NDOR has existing concrete bridge railings at 29" across the deck of approx. 1000 bridges, these raise at the end of the bridge railing to 32" for the guardrail to attach at 31". We plan to overlay these with a thin water proof membrane and a 2" asphalt overlay across the bridge.

The FHWA has a list of acceptable bridge railings

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

Under Vertical Concrete Parapet (Open or Closed) / General [PDF 1.08 MB]; the Concrete 27" railing similar to NDORs is here. Do you concur with the below statements?

1. NDORs concrete bridge railing (open) @ 27" meets NCHRP 350 testing which is confirmed by FHWA.
2. The thrie-beam bridge approach at 29" meeting the 30" end of bridge rail, is NCHRP 350 approved.
3. The 29" thrie-beam bridge approach section will be raised to 31" over the first 75' of the guardrail run.
4. The end of the 31" w-beam will use an approved end treatment.

Response

Date: 04-16-2015

Hello Phil,

MwRSF has examined the proposed resurfacing you identified. As we understand it, the roadway will be resurfaced with a water-repelling membrane and an additional 2" of asphalt overlay. The pavement overlay would reduce the effective heights of:

- The open concrete bridge rail from 29 in. to 27 in.;
- The approach guardrail transition (AGT) from 31 in. to 29 in.; and
- The guardrail leading up to the AGT from 31 to 29 in.

In reviewing relevant research, MwRSF researchers felt that the flared distance over which the effective rail height changes is acceptable, and the 31-in. tall W-beam (e.g., MGS) end treatments are acceptable. We have some concerns with other aspects of the proposed resurfacing project. Please note below.

1. No thrie beam AGT has yet been approved at NCHRP Report No. 350 for use with a 29 in. top mounting height. All approved thrie beam transitions were designed to connect to the end of the parapet / end buttress of the concrete barrier with a top mounting height of 31 in. Researchers are unaware of any successful or unsuccessful tests with a thrie beam top mounting height of 29 in. Crash testing is recommended to confirm the adequacy of the 29-in. thrie beam AGT top mounting height according to NCHRP Report No. 350 TL-3 specifications. It should be noted that successful crash testing has been observed for some stacked W-beam with W-beam rubrail/curb approach guardrail transitions using a 29-in. top rail height.

2. Testing at TTI in the early 2000s indicated that there may be an increased propensity for rollover associated with an open concrete bridge rail with a top mounting height of 27 in.; please refer to the attached report "TTI_Report_GFRP_Bars_Concrete_Bridge_Rail_0-4138-3.pdf". As such, an 27-in. tall open concrete rail has demonstrated unacceptable behavior according to TL-3 of NCHRP Report No. 350.

These concerns may be resolved using one of the following approaches, in order of recommendation.

1. Modify the bridge rail height to be similar to what was shown in "TTI_Report_GFRP_Bars_Concrete_Bridge_Rail_0-4138-3.pdf" with a 3-in. tall tube mounted to the top of the bridge rail. Of course, other practical methods for increasing height are acceptable. Raise the height of the thrie beam transition by using modified blockouts and field-drilled holes in the posts, and re-attach the thrie beam to the end of the concrete parapet / end buttress per the original design drawing specs.
2. Modify the bridge rail height to be similar to what was shown in "TTI_Report_GFRP_Bars_Concrete_Bridge_Rail_0-4138-3.pdf" with a 3-in. tall tube mounted to the top of the bridge rail. Also, modify the AGT to be consistent with one of the stacked W-beam designs in FHWA approval letters b-65, b-77, or b-83; crash testing details may be found in the attached report "NCHRP_350_Testing_404211-F.pdf". These designs permitted the use of lower-height, W-beam transitions to the bridge rail with additional rub rails and/or curbs installed.These systems may require the use of alternative posts and structural elements in the transition region, but will not require new holes to be drilled in the end of the existing concrete parapet / end buttress.
3. Limit resurfacing projects that result in 29-in. tall AGTs and 27-in. tall vertical parapet bridge rails to TL-2 applications only.

Attachment: https://mwrsf-qa.unl.edu/attachments/98cda0b1934ac46685df767da008f72b.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/347703374d70184a40669e23bfc8e8d6.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/dfe68beefa83b88201899b1c47121378.pdf

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Concrete median barrier

Question

State: IL
Date: 04-22-2015

Greetings,

Are you aware of any TL-5 crash
tests that have been performed on 42" high non-reinforced concrete barrier
(either Jersey or F-shape)?

Response

Date: 04-27-2015

There has been testing on the Ontario Tall Wall. However, we do not promote its use do to the lack of reinforcement, the formation of shrinkage and temperature cracks over time, and the shifting ballast observed in the testing program which potentially may have reduced the impact loading. This barrier was listed in the prior versions of the Roadside Design Guide.

Mash working width values for temporary barrier

Question

State: WI
Date: 04-23-2015

I am in the process of updating the
working width information for out temporary barrier to MASH. So I have to
do some adjustment of the NCHRP test to MASH.

Could you take a look at what I have
to see if I am on the right track?

Attachment: https://mwrsf-qa.unl.edu/attachments/d4da243623f0f37878b804a3d53d076f.pdf

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Response

Date: 05-06-2015
I have looked through the detail and have the following comments. In general, they look acceptable.

1. For the free-standing barrier critical location, the MASH TL-3 working width is 8.5 ft rather than the 6'-9" shown.
2. For the asphalt tie-down, the working width shown for NCHRP 350 are based on the testing we conducted adjacent to a 3' vertical drop-off that moved a significant section of supporting asphalt and soil. If used farther from the drop-off or with a less severe drop-off, it is expected that the deflections of the asphalt pin system would only be marginally larger than that of the bolted tie-down.
3. For the asphalt tie-down, the working width shown for MASH is currently unknown. Increasing it slightly as you have to account for the higher impact severities in MASH is rational, but it is difficult to say how accurate it may be.
4. Similarly, for the bolted tie-down, the working width shown for MASH is currently unknown.

Thanks

Response

Date: 05-11-2015

If the DOTs have to go to MASH. It would probably be a good idea for MwRSF to come up with recommendations on what the working widths should be (until we test the alternatives to MASH). You are probably going to be asked the same question over and over and a consistent answer would help all the states.

I attached a spreadsheet I put together to do the calculations an pick a “rational" number. I'm not saying it is right, but this looks to be the direction I'm heading in.

Response
Date: 05-12-2015

I would agree that this probably needs some thought. Mostly along the lines of deflections for the tie-down applications. I see that you are currently scaling up the tie-down system deflections based on the free-standing MASH versus 350 deflections. I think that may be too conservative as we don't expect that kind of jump in deflection for the anchored barriers.

I will try to give this some more thought and come up with some numbers. I believe that TTI has some MASH testing of tie-down barriers from with similar 350 testing that could be looked at.

Bullnose modification

Question

State: MN
Date: 05-01-2015

Hello Michael,

The design you provided appears to be consistent with guidance provided by FHWA in approval letter CC68, which may be found at http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/term_cush.cfm. Both wider-bullnose FHWA-approved designs utilized a curved nose piece, a curved transition piece, and a length of straight rail on the departure side of the bullnose that could be flared at a high angle. In-line with straight guardrails can produce instability and rollover, as observed during MwRSF short-radius testing. Thus it is recommended to avoid long, straight segments of guardrail extending from bullnose and short-radius systems which could be subjected to head-on impacts. The Minnesota design has two different flares on the back side of the system, which should increase the probability that an impacting vehicle will deflect the rail laterally and not subject it to potential instability and rollover. MwRSF concurs that the design is therefore consistent with FHWA-approved modifications.

Other bullnose designs provided by MwRSF are preferred in general, in part because of the concerns raised above. Preferred bullnose designs are shown in MwRSF research report on page 149 of no. TRP-03-95-00, found at http://mwrsf.unl.edu/researchhub/files/Report120/TRP-03-95-00.pdf. The designs utilize a larger, broader nose piece and variable tapers on the front and back side rails. It is believed that this design will function better when an impacting vehicle strikes parallel with the guardrail on the opposite-side rail.

The guardrails may connect to bridge rails via an approved thrie beam approach guardrail transition (AGT) after the third rail segment on each side (excluding the nose). The bullnose may be transitioned to other barrier types for connection to bridge ends using approved transitions. The guardrail on the front side may be flared at 15:1 starting at post no. 5, and back side guardrail flare rates may be larger. Note that the maximum back-side flare rate approved by FHWA is approximately 3.5:1, for FHWA Approved Wider Bullnose System (15.6-degree flare). Slopes in front of and up to 40 ft behind the barrier must not be steeper than 10:1. Design details for constructing the systems are shown in the FHWA approval letter and MwRSF report.

Let me know if you have any additional questions.

Attachment: https://mwrsf-qa.unl.edu/attachments/d61744f672b8e910af3e060aac606a1e.pdf

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Response

Date: 05-01-2015

We are updating our bullnose standard (to the USBP design)
and just have a few questions to run by you.

MnDOT's current standard for a
widened median (see attachment, Standard Plan 611, sheet 3 of 3) shows guardrail tapers on both the approach and the opposing sides of the median. We haven't been able to find our background information to support these tapper designs yet and though we should just ask you.

We are specifically questioning the
angle turn locations at the posts (#5 & #11 on the approach side and #5
& #8 on the opposing side). We are wondering what the appropriate angles
are and how best to get back to a parallel alignment with the bridge rails?

Also, the MwRSF Drawing MDN01,
Sheet 6 of 6 (see attached) shows 2 designs for wider bullnose systems.
How would these designs connect/transition into bridge rails? What are
the guidelines for laying out these designs in widened medians when attaching
to a bridge rail?

Your
help is appreciated.

Attachment: https://mwrsf-qa.unl.edu/attachments/d61744f672b8e910af3e060aac606a1e.pdf

Crash Cushion Question

Question

State: IA
Date: 05-04-2015

I had a question come up on an interchange project.
Due to soil remediation they were not able to install standards W beam guardrail which would have allowed them to flare the installation. They plan to install a short section of concrete barrier and a crash cushion instead. The problem this causes is the tangent installation causes a sight distance problem. Would it be possible the flare the concrete barrier at a 20:1 rate and install the crash cushion along that same line? We could flare only the concrete portion but that would require a greater length of concrete rail to maintain the desired sight distance.

Attachment: https://mwrsf-qa.unl.edu/attachments/7e0fac2f6dee8bcff2c39e2c65000719.jpg

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Response

Date: 05-06-2015

With respect to the situation below, we would not have an issue with flaring of the permanent concrete barrier as long as it falls within the guidance in the RDG for barrier flare rates based on the shy line and the design speed.

In terms of the crash cushion, we would likely recommend that the crash cushion be extended parallel to the roadway rather than in line with the flared concrete barrier due to potential concerns for impacting the crash cushion at angles and severities outside the range of its original testing. To the best of my knowledge, there has been little research or testing on crash cushions installed on flares. Minor flares have been allowed for terminals in many cases.

That said, I would recommend that you contact the crash cushion manufacturer directly in order to get their input. They likely have better insight than we do with regards to their systems in flared installations.

TCB/Guardrail Transition

Question

State: IL
Date: 05-05-2015

We are evaluating a proposal submitted by one of our District offices regarding placement of temporary concrete barrier (TCB) across and on both ends of a bridge deck. We have some questions about options for transitioning the TCB once it leaves the bridge deck.

1. Section B-B in the attachment complies with internal guidance to provide 2.0' between the back of TCB and the curb on the bridge deck. This scenario could be for freestanding TCB, or at most, short pins (approximately 4.5" embedment into the bridge deck) could be provided.

2. Section A-A does not comply with internal guidance since we indicate that the face of the TCB should be between 2" and 1' from the face of the guardrail; and the underlying surface on which the TCB rests needs to be paved and in the same plane extending to the face of the guardrail.

We feel that the TCB could be shortened by using a taper of not steeper than 1:12 once the installation is off of the bridge deck. At that point a section or two could be placed parallel to the guardrail and the end units could be anchored to the underlying paved shoulder using anchor pins in all six holes. Placement of a Test Level 3 (the posted speed on the roadway is 55 mph) temporary narrow, redirective impact attenuator would be required at both ends.

A.

Are there any transition devices available that would allow another option for realigning the guardrail to attach to the TCB on both ends? We could anchor the end unit of the TCB into the underlying paved shoulder (or we could anchor multiple units, provided that they are not on the bridge deck, since our Bureau of Bridges and Structures does not allow anchoring on bridge decks). Such an arrangement would not require temporary impact attenuators on both ends; however, we are unaware of potential performance issues for TL 3 connecting guardrail to TCB.

B.

Are there other options that we might consider?

Any thoughts or guidance that you could provide would be appreciated.

Attachment: https://mwrsf-qa.unl.edu/attachments/5ca3f1a1f0ffee35ec21e4e42688ccf6.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/97fc9c09e384c50c5107c23d1ad6da2c.pdf

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Response

Date: 05-29-2015

We have reviewed you detail for the temporary barrier installation on bridges and believe that it is acceptable based on current safety practices.

The overlap of the PCB on the guardrail and the shielding of the end of the PCB with an attenuation system is currently about all that can be done for this type of installation. There currently is no tested and approved transition between TCB and guardrail. However, we currently have research underway with NDOR to address exactly this situation. The system was designed in the report below and the testing of the system is scheduled for this summer. If all goes well, the system should be sent in for FHWA eligibility before the end of 2015.

http://mwrsf.unl.edu/researchhub/files/Report299/TRP-03-300-14.pdf

The only other comment we would have is that the 2 ft offset between the bridge rail curb and the back of the barrier may need to be increased. The concern is that the motion of the TCB's during impact may cause them to strike the curb and rotate back about the rear corner of the barrier. If this happens, the propensity for vehicle climb and instability can increase. Increasing the offset another foot to 3 ft or using the steel strap tie-down system for the F-shape TCB (or other tie-down systems) could help alleviate that concern. The steel strap tie-down uses less embedment and should restrain deflections sufficiently.

http://mwrsf.unl.edu/researchhub/files/Report219/TRP-03-115-02.pdf

Thanks

W-beam attachment to Culverts

Question

State: NE
Date: 05-06-2015

The post within the weak-post w-beam attached to culverts (TRP-03-277-14) is specified as 44" long. A few situations may call for variable post lengths. What post lengths will function properly within this system

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Response

Date: 05-06-2015

Without doing more analysis, we are not too comfortable with changing the dimensions of the post, as post bending is the main provider of the systems lateral support. We could live with +/- 1", but aren't really big fans of recommending further changes without additional evaluation.

I'm not too sure why you would want a longer post anyway.Headwall and/or slab dimensions can change, but the socket has a defined length of 16.5" and a top mounting height of 2" above the top of the culvert. Thus, a 44" post will provide the specified rail mounting height of 31" above the ground/headwall. Variations to the culvert structure will not affect this rail mounting height. The only thing that would affect mounting height would be if the top of the culvert is not at ground line. However, when we reviewed each pooled fund States' culvert standards, it was common that the top of the culvert headwall was even with the groundline. Subsequently, the need for a longer post would only occur if your headwall was below the ground line. I don't know how this would occur as nothing would be preventing erosion/runoff if the headwall was not at ground line.

Note, within the conclusions section of the report it states that the system is not recommended for use with approach slopes greater than 10H:1V as the system (and the original bridge rail) were never designed for, or evaluated with, an approach slope. A steep approach slope could significantly affect the performance of the barrier system.

Response

Date: 05-06-2015

If the parapet is above the shoulder or If this is used on a bridge with a curb: is a curb over 9" too high?

Response

Date: 05-06-2015

This design has not been evaluated with a curb. As such, we do not recommended the system to be installed in combination with a curb or a headwall that extends more than an inch or two above the ground line / shoulder.

Response

Date: 05-06-2015

Is the 1' W-beam Backup Plate required at the splice joints?

Response

Date: 05-06-2015

Yes. As shown at the meeting last week, rail tearing has been evident in multiple systems utilizing S3x5.7 posts and w-beam guardrail – typically at splices. Thus, the 12" backup plate is recommended for use on all similar systems – including the original bridge rail which was only tested with 6" plates. The 12" plates will not fit between the splice bolts, so they will have to be modified to fit over/around the splice bolts. Oversized slots should do the trick.

Response

Date: 05-08-2015

So, I am stuck with only one length of post … if so, I will need to adjust most headwall parapets to meet the 4% to 6% slope of the shoulder, either cutting it to lower it or extending it to raise it? Even a few inches?

But we expect the MGS leading into this area to work at both 31" & 27.75"? why don't we think this will work properly?

Response

Date: 05-08-2015

This weak-post w-beam system was designed and tested for level terrain applications without curb. Thus, we recommend it be installed in such a manner. We have recently shown that the addition of a curb below an otherwise acceptable system can lead to rail rupture and failure (the MGS stiffness transition to thrie beam AGT). So until the system is evaluated with the addition of a curb, we don't recommend installation of the system with a curb. If the culvert headwall extends more than an inch or two above the ground, it should be cut down or the a different system should be considered. If the roadway is significantly higher than the headwall, that would mean that there is a roadside slope leading into the headwall/barrier system. Steep roadside slopes may cause vehicle instability issues and negatively affect the performance of the system. Since this system has not been evaluated to use on steep roadside slopes, it is not recommended for installation on slopes greater than 8:1.

What we are left with are culvert sights were the roadway and the headwall are level with each other or near the same elevation. I understand that there may be installation sites were the adjacent w-beam guardrail is at a height of 27.75". However, TTI has conducted recent testing of w-beam treatments for culverts and short bridge structures with this lower height w-beam and has observed rail rupture and testing failures with the MASH vehicles. As such, we recommend keeping the installation height of this system at 31" and transitioning the guardrail adjacent to this weak-post w-beam up to 31" over a distance of 25 ft on both sides of the installation.

Parapet Heights at Thrie Beam Transitions

Question
State: IA
Date: 05-08-2015

We have a project showing the following detail (77-1631-087_BridgeEnd.pdf), where it is a bridge deck overlay and they are upgrading the bridge ends from Type C (page 3 of BA-202) to a more updated Type B (page 2 of BA-202). New bridge ends are normally specified at 34" (per BA-107) but the project in question is showing 32" to match the height of the existing bridge rail.

On past projects where we are only replacing guardrail, we've allowed the bridge side of our BTS (BA-201) to sit at 30" or 31" to match the old transition height as reconstructing new bridge ends was beyond the scope of the project. However, since this project is updating the bridge ends, in which order are the alternatives from most preferable to least?

a) Stay at 32" to the top of the bridge end and install 32" BTS. This leaves only 2 3/8" from center of top bolt hole to top of rail and puts the top of the BTS at the top of rail.

b) Stay at 32" to the top of the bridge but lower the bolt hole pattern by 1" or 2" to gain clearance and still fall within adjusted tolerances for the BTS (30" or 31").

c) Taper the new bridge end from 32" existing up to 34" by the first bolt holes (three vertical) and install BTS at normal height. The 2" transition would have to take place over 20.5".

d) Another option?

Thanks for your assistance on this and future applicable instances.

Attachment: https://mwrsf-qa.unl.edu/attachments/1d0f83df90972c75bad80fbab0d8b6f5.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/14a87bc232568941d850c27b6aff0b29.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/2aba5128232f8afa7a5ace3cfb6aa4a5.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/085462edc6403bb933fdeb36aa6c4623.pdf

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Response
Date: 05-08-2015

We had a similar discussion to this just last week at our annual Midwest States Pooled Fund meeting. At that time, multiple State DOTs had said they have transitioned the ends of the bridge parapet up a few inches to provide a few more inches of cover to the attachment bolts/hardware of the thrie beam transition. This is very similar to your Option C, and I recommend this method for use. You vertical height transition of 1/10 seems well within reasonable bounds to prevent snag and vehicle instability during reverse direction impacts.

CMB in Wide Medians

Question

State: CA
Date: 05-13-2015

I'm writing to ask if the Midwest States Pooled Fund Crash Test Program has done any research on the use of cable barriers in wide medians (greater than 75 feet). I'm conducting some research on behalf of California Department of Transportation into practices and guidance for the use of barriers in wide medians. Is that a subject that the pooled fund has investigated? I'd appreciate any information you can offer.

Response

Date: 05-13-2015

There is some available data that suggests there is a benefit to placing CMBs in medians wider than 70-75 ft, but there is a distinguishable reduction in the benefit-to-cost ratio. I have attached a report, **Reducing Median Crossover Crashes in Wisconsin** , which is abbreviated "CMC in Wisconsin.pdf". In it there is a chart showing the total cross-median crash data collected in Wisconsin by median width on Page 11. Several crashes occurred in medians of 75 ft wide or greater.

Similar analysis of crashes in medians in North Carolina shown on Page 7 of the attached PDF titled **Median Barriers in North Carolina** and denoted as "NCDOT - Summer TRB Presentation on Cable experience.pdf" indicated that even at moderate to low traffic volumes for divided rural interstates, cross-median crashes (CMCs) still occurred when median widths were in excess of 75 ft:

Dr. Dean Sicking, Karla Lechtenberg, and Dr. F. Daniel B. Albuquerque conducted a study evaluating cross-median crashes and cable median barrier warrants based on interstate data in Kansas. That report is titled **Cable Median Barrier Guidelines** and denoted as "TRP-03-206-08-Final-Revised.pdf", and is available from the Pooled Fund website under TRP-03-206-08. Although the frequency of crashes occurring in medians wider than 70 ft was limited, there was at least a limited risk that crashes could occur in wider medians.

Likewise, TTI issued some guidance for the state of Texas in 2006 titled **Median Barrier Guidelines for Texas** and denoted as "TTI Report No. 0-4254-1.pdf". On page 33, TTI noted that the majority of TX's cross-median crash data occurred in medians larger than 60 ft wide. The disproportionate representation of wide median CMC data is partially because TX is proactive in shielding medians narrower than 70 ft, but it is reflective of the need to consider shielding even in wide medians. This was reinforced in a 2009 report titled **Development of Guidelines for Cable Median Barrier Systems in Texas.** denoted as "Development of Guidelines for CMB in Texas - TTI 2009.pdf".

Other data may also be available, but the brief synopsis suggests that for large traffic volumes, CMBs may be warranted for medians wider than 75 ft. Unfortunately there are currently no recommendations available to establish guidelines for usage, and states may determine if there are locations in which barriers placed in these wide medians are justified.

Attachment: https://mwrsf-qa.unl.edu/attachments/18e8711559c8b5570d05cb84d88c70cc.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/936f8f4d33a434ed76a316b121f8f38f.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/a854835ed12d067f5506626090266b27.pdf

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Brittle Test For Steel Bridge Rail Tubes

Question

State: WY
Date: 05-18-2015

When Wyoming's TL-3 and TL-4 Bridge Rail was approved to NCHRP 350, FHWA expressed concerns about addressing brittle failure for ASTM A 500 tubes, something they had seen in other bridge railings. To address this concern, Wyoming adopted a requirement for testing in accordance with ASTM E 436 using criteria similar to the State of New York. The specification calls for a Drop-Weight Tear Tests and reads, "the percentage shear area shall be determined by testing six specimens from the 6-inch side or sides of the structural tube not containing a weld. If the average percent shear area falls below 50, the material represented by these tests shall be rejected."

Trinity is in the process of providing railing for a project and they observed the following results:

Of the six tests, three showed 100% shear and three tests showed 10 % shear. The average of all six tests is 55% which according to the specification would pass. We are concerned however about the spread of the results. Should this be considered a pass, or should additional criteria be applied? Any light you can shead on this mater would be helpful. We are in the process of approving the material, so time is of the essence.

Thanks!

Response

Date: 05-27-2015
ASTM E436 appears to be a shear test for evaluation of the fracture propagation in the temperatures where the steel behavior transitions from ductile to brittle. In your question, you noted that three showed 100% shear, which would indicate ductile response, and three showed 10% shear, which would indicate a brittle response. However, there is no mention of the temperatures for these tests. According to the ASTM spec, these tests should be run at varying temps to determine how the temperature is affecting ductility.

As such, in order to answer this question, we would need to look at the temperatures that the tests were run at and compare them with your operational temps in order to decide if the ductility of the bridge rail was an issue. I don't believe that you can average the results as the samples were likely run at different temps.

If you have some additional data from the testing that you can share, we can look into things further.

Thanks

Condition of Temporary Concrete Safety Barrier

Question

State: KS
Date: 05-27-2015

I had a question come in from KDOT field personnel regarding the condition of TCSB and at what point the TCSB should be rejected on a KDOT project. KDOT doesn't currently have any published criteria, but I gave our field personnel the direction that if the shape of the barrier or the structural integrity of the barrier is compromised then it should not be allowed on the project. I talked with Scott King and he seemed to think MwRSF might have looked into this topic previously. I didn't find anything on the Pooled Fund site directly related to the condition/damage to TCSB. Do you know of any research that MwRSF has done previously on this? I have some information I've found from a few other states, but just wanted to check and see if you had anything MwRSF had done. Please give me a call to discuss if needed or just send an e-mail response.

Response

Date: 05-27-2015
There has not been any formal research in this area, but it has come up as a research need several times in the past.

As such, I don't have hard data or guidance, but I can note some areas for concern that we would consider when looking at barrier condition.

1. Deformed or damaged connections – If the barrier loops or the connection pin are significantly deformed or damaged, we would recommend replacing the barrier. Deformed loops or pins have lost some of the toughness/ductility. As such, a second impact may be sufficient to fracture those components. This would include deformation or partial tearing of those parts. Additionally, if the loop bars are necked or decreased in cross-section. Similarly, if the loops have been pulled out of the end of the barrier face partially, this would indicate that previous impacts have moved the bars in the concrete and that there is the potential for loss of development.
2. Damage to the barrier toes or ends – We often see damage to the toes of the barrier segments due to moving and placement, especially on the ends of the barriers . This is a significant problem as the engagement of the adjacent barrier toes when the segments rotate during impact is critical for generating the forces in the joint needed to redirect the vehicle. Thus, if the toes of the barrier segments are spalled off or broken, it would be a concern.
3. Structural cracking – If you observe any through cracking of the barrier or cracks wider than 1/8", I would be concerned about barrier integrity.
4. Large sections of disengaged concrete – If large chucks or pieces of the barrier have been disengaged anywhere along the segment, it would be cause for concern. This missing concrete would degrade the structural capacity of the barrier and potentially act like asperities or snag areas along the face of the barrier that could promote vehicle instability or snag. I don't know if I have an exact size for those disengaged pieces off the top

of my head.
5. Exposed rebar – Exposed rebar would be cause for concern as it would indicate large regions of disengaged concrete and it would expose the reinforcing steel to corrosion.

Again, these are not hard guidance, as that kind of effort would take more research, but these are the types of things I would look out for.

Steel Plate Grades

Question

State: WI
Date: 05-27-2015

mso-fareast-font-family:"Times New Roman"">I've once again reviewed my materials specifications.

mso-fareast-font-family:"Times New Roman"">For steel plates can I substitute ASTM A572 grade 50, or ASTM A A572 Grade 50 KSI Max. for ASTM A36?

mso-fareast-font-family:"Times New Roman"">

Response

Date: 05-27-2015

That would largely depend on the application. A36 has lower specified yield and tensile strength and marginally higher elongation. For almost all structural applications, I don't see any issues unless you desired a lower strength or slightly higher ductility. There may be some applications where this is necessary, for some kind of fracture critical or energy absorbing component.

Thanks

Temporary Concrete Barrier

Question

State: IL
Date: 06-11-2015

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

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

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

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

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Response

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

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

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

Extra Blockouts / Bridge Guardrail Near Side Roads

Question
State: IA
Date: 06-15-2015

Extra Block outs

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

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

Restricted Length Bridge Guardrail

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

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

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Response

Date: 06-17-2015

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

This holds true for the standard MGS system as well.

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

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

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

"Thus, the following implementation guidelines should be followed:

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

2. A recommended minimum barrier length of 46 ft – 10½ in. (13.3 m) is to be installed beyond the upstream end of the asymmetrical W-beam to thrie beam transition section, which includes standard MGS, a crashworthy guardrail end terminal, and an acceptable anchorage system. This segment includes one halfpost spacing for Design K and three half-post spacings for Design L.

3. For flared guardrail applications, a minimum length of 25 ft (7.6 m) is recommended between the upstream

end of the asymmetrical W-beam to thrie beam transition section and the start of the flared section (i.e. bend between flare and tangent sections). This segment includes one half-post spacing for Design K and three half-post spacings for Design L."

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

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

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

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

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

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

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

Thanks

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

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Curb in front of thrie-beam transition

Question

State: IA
Date: 06-22-2015

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

As you can see on BA-201, we show possible curb running through the end of the BTS as report03-291-14 *Dynamic Evaluation of MGS Stiffness Transition with Curb* suggests in the second full paragraph on page 136. We denote this 4" curb via circle note 2, which states to see project plans. Project plans in this case typically refer to our Bridge Approach Standards (BR Series), and therein lies the problem. On the Bridge Approach Standards that show abutting pavement (BR-102, 103, 104, 105, 106, 107, 112, 205, 211, 212, 231), we have a circle note that states something to effect of "Build 4 inch Sloped Curb to end of Double Reinforced Section." This note has been shown on our approach standards since April 1999 when we switched to the 4" curb. A typical double reinforced section extends 20' out from the center of the roadway (see image below). I would say that in almost no case would the double reinforced section extend 37.5' and meet the report's suggestion. What I would like to know is which of the following is the real issue with curb location:

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

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

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

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

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

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Response

Date: 06-22-2015

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

Response

Date: 06-23-2015

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

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

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

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

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

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

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

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

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

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

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

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

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

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Response

Date: 06-24-2015

I have commented below in RED

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

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

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

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

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

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

Requirement #1

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

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

Requirement #2

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

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

Requirement #3

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

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

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

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

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

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

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

Response

Date: 06-25-2015

Two additional questions for you then.

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

Part A

BTS = 37.5'

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

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

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

Part B

BTS = 25'

w-beam = 12.5'

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

Total install = 87.5'

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

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

Response

Date: 06-26-2015

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

Concerning the 12.5 ft for requirement B-1:

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