Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

04-01-2016 to 07-01-2016

31" Guide Rail Curb Offsets

Question
State: NJ
Date: 04-04-2016

What criteria do you have on the 31" MGS placement behind curb? This is what I have so far:

NCHRP TL-3 used 6" Type B Curb (type of vertical curb) which passed 6" behind curb for 31" height measured from gutter line. The designer can use the lay down curb as shown in the Roadside Design Guide Figure 5-35(b) (see Section 5.6.2.1.2 2nd paragraph) in lieu of the 6" AASHTO Type B curb.

TRP-03-221-09 concluded for MASH TL-3 that an 8' offset is not acceptable.

TRP-03-237-10 concluded that for MASH TL-2 that 4' to 12' max is acceptable.

Can we use the sloping curb offsets shown in the 2011 Roadside Design Guide, Section 5.6.2.1.1 which is:

· Design speed less than 45 MPH: Set flush or at least 8 feet behind the face of curb. Use 6" high or shorter sloping-faced curbs.

· Design Speed 45 to 50 MPH: Set flush or at least 13 feet behind the gutter. Use 4" high or shorter sloping curbs.

· Design Speed greater than 50 MPH: Set flush with gutter. Design speeds above 50 MPH, use 4" or shorter sloping face curb. For design speeds above 60 MPH, the sloping face should be 3:1 or flatter and no taller than 4" high.

Response
Date: 04-07-2016

I have added some comments below regarding curbs.

What criteria do you have on the 31" MGS placement behind curb? This is what I have so far:

NCHRP TL-3 used 6" Type B Curb (type of vertical curb) which passed 6" behind curb for 31" height measured from gutter line. The designer can use the lay down curb as shown in the Roadside Design Guide Figure 5-35(b) (see Section 5.6.2.1.2 2nd paragraph) in lieu of the 6" AASHTO Type B curb.

This is correct, and we believe that less severe curbs such as the one sloped curb you note would be acceptable to NCHRP Report 350 as well. The MGS with curb in this configuration has not been evaluated to MASH TL-3 at this time. There is a proposal in the Year 27 Pooled Fund to evaluate this to MASH.

TRP-03-221-09 concluded for MASH TL-3 that an 8' offset is **not**acceptable.

True. We did test this as we believed it was a critical configuration, and found that the 2270P vehicle became unstable and rolled.

TRP-03-237-10 concluded that for MASH TL-2 that 4' to 12' max is acceptable.

We did do additional analysis for larger curb offsets at TL-2 and had a single test of a MGS offset 6 ft from the curb that was successful. I should note that the 6 ft offset test used a MGS installed with a height of 37 in. from the gutter line. Based on this test and our LS-DYNA work, it was concluded that the 37 in. high MGS was acceptable for curb offsets between 4'-12'.

Can we use the sloping curb offsets shown in the 2011 Roadside Design Guide, Section 5.6.2.1.1 which is:

This guidance is based on slightly older research and pertains to the G4(1S) guardrail system and mounting heights. We would expect the MGS to perform as well or better than the G4(1S) systems in almost all circumstances. Thus, the RDG guidance is likely acceptable for the MGS unless we have the previous research noted above to alter it.

Note that none of the RDG guidance appears to consider alteration of the rail height relative to the gutter as we did in our previous studies.

- Design speed less than 45 MPH: Set flush or at least 8 feet behind the face of curb. Use 6" high or shorter sloping-faced curbs.

- Design Speed 45 to 50 MPH: Set flush or at least 13 feet behind the gutter. Use 4" high or shorter sloping curbs.

- Design Speed greater than 50 MPH: Set flush with gutter. Design speeds above 50 MPH, use 4" or shorter sloping face curb. For design speeds above 60 MPH, the sloping face should be 3:1 or flatter and no taller than 4" high.

Weak-post guardrail on culverts installation issues

Question

State: IL
Date: 01-29-2016

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

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

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

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

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

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

Please see the attached file for details

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

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Response

Date: 01-29-2016

Questions 1 &2:

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

Question 3:

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

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

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

Question 4:

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

Question 5:

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

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Response

Date: 04-05-2016

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

MGS Attached to Culverts

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

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

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

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

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

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

SECTION 1027. CHEMICAL ADHESIVE

1027.01 Chemical Adhesive Resin System. The chemical adhesive resin system shall consist of a two part, fast-setting resin and filler/hardener. The system shall meet the requirements of the ITP for Chemical Adhesives and be on the Department's qualified product list.

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

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

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

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

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

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Response

Date: 04-05-2016

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

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

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

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

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

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

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

BCT for MGS Long Span

Question

State: IL
Date: 04-05-2016

For long-span guardrail would it be acceptable to use steel tubes and CRT drop-in wood posts rather than the full length wood CRT posts? What would be a minimum length of the steel tubes?

Our Highway Standard 631011 uses steel tubes and posts, adapted to anchor the w-beam. (Attached "218-631011-09 TrafBarTermType2.pdf)

Attachment: http://mwrsf-qa.unl.edu/attachments/aa12632cee9ad968a195d03e8399ba90.pdf

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Response

Date: 04-05-2016

CRT posts have two weakening holes in them, one at ground line and one 16" below ground line. Thus, CRT can break at two different locations – and did so at various post locations during the evaluation of the MGS Long Span system. Placing them in steel foundation tubes would prevent fracture at the bottom hole and may change the performance of the system.

BCT end terminal posts are typically placed in steel foundation tubes. However, BCTs have a slightly different cross section and only one weakening hole. We do not recommend utilizing BCTs in place of CRTs for the Long Span system.

Use of flared, non-blocked, MGS

Question

State: IL
Date: 04-05-2016

Is it acceptable to flare the
non-blocked MGS? If so, do we use the same guidance as for the MGS with
blockouts? I don't find any mention of this in the report or in the
consulting questions.

Response

Date: 04-05-2016

MwRSF has never evaluated a flared, non-blocked, MGS installation. Both the flared MGS and non-blocked MGS illustrated significant increases in vehicle snag, rail loading, and vehicle instability when evaluated as independent systems. If the two were combined into a single system, these negative characteristics may further increase to unsatisfactory levels. So, without further evaluation, we do not currently recommend installing a flared, non-blocked, MGS.

Departing End Structure for MGS with Curb

Question

State: IL
Date: 04-05-2016

IDOT uses a guardrail attachment
to the downstream end of a structure on a one-way road that attaches the w-beam
to 1 inch diameter by ~2 ft anchor bolts cast into the concrete
structure. Is it acceptable to use a curb with this guardrail near and up
to the structure? (See attached Highway Standard “218-631026-06 TrafBarTermType5.pdf").

Attachment: http://mwrsf-qa.unl.edu/attachments/a7fdd9c6d0136ff81d20e449853744e8.pdf

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Response

Date: 04-05-2016

To my knowledge, guardrail anchorages such as this have never been crash tested. As such, I will comment on the system assuming that the upstream anchorage is crashworthy and wouldn't negatively affect the guardrail system.

The MGS placed 6" behind a 6" tall curb was successfully evaluated to NCHRP 350 standards – see report TRP-03-139-04. Assuming your upstream anchorage doesn't affect the performance of the guardrail system, then the behavior of the MGS with curb should also be crashworthy to NCHRP 350 standards.

One side note here – recent testing with MASH small cars has raised concerns about possible rail tearing when small vehicles impact a curbed MGS installation (the NCHRP 350 evaluation did not include a small car). This issue has been brought to the Pooled Fund Sponsors' attention, and a MASH evaluation (including the small car) is currently a potential project for the 2016 Pooled Fund Program. Thus, we may learn more about the crashworthiness of the MGS with curbs in the near future. Until then, I can only say that the MGS with curbs has been NCHRP 350 approved.

MASH 2016 Bridge Rail Loads

Question

State: IL
Date: 04-06-2016

I appreciated the opportunity to discuss this subject with you yesterday by phone. Illinois is still attempting to understand all of the issues involved with MASH 2016 so we can meet the deadlines and make some informative decisions moving forward with a standard barrier shape. Currently we use both a 34" and a 42" F Shape barrier and the reinforcement we use is designed per Chapter 13 Appendix of the AASHTO LRFD Bridge Design Specifications. We know that any barriers under MASH 2016 need to be a minimum of 36" tall plus any additional future wearing thickness. In Illinois this means we would need a minimum 39" tall barrier. We were considering just using our 42" F Shape barrier for possibly the MASH 2016 TL-4 load but it would likely require a crash test to assure that it could handle the 57% increase in impact severity. You also brought to my attention that while the TL-5 impact severity did not increase from NCHRP 350 to MASH 2016, it is anticipated that the 124 k and other design values for TL-5 will increase significantly because these design values were apparently calibrated way to low.

Ideally AASHTO will publish a revised load chart soon to replace the current chart shown below with MASH 2016 loads. The 2015 FALL presentation showed the 2nd chart below but there aren't any values for the other Test Levels as there currently is in AASHTO. You noted that you have a Manitoba 49" TL-5 barrier test coming up next week and that you anticipated learning a lot towards better TL-5 design values from that test. If there are any new developments regarding the TL-5 design loads I would appreciate if you could send me a link to that information.

Illinois is leaning towards switching to a constant 11 degree slope barrier, but we don't know what height yet.For TL-5, it may likely be 45" tall, but we don't have design values for that yet.

Attachment: http://mwrsf-qa.unl.edu/attachments/7a9b4021c788c481765291553e03fc96.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/0ce35426e51b2200c54a0a28eed2abcc.jpg

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Response

Date: 04-07-2016

I have dug out some information from a few quarterly progress reports on NCHRP Project No. 22-20. During that effort as well a few others, TTI researchers utilized modeling to investigate barrier loading. I have provided details in the attached PDF regarding their suggested design forces for analyzing and designing bridge railings. As I recall, existing procedures largely use the application height at the top of the barrier when considering prior 32" TL-4 and 42" TL-5 barrier heights. Now, they have offered revised loads and application heights based on barrier heights. Note that one design load and application height was suggested for TL-4 to cover a range of barrier heights.

Next, it is not yet clear as to what the minimum barrier height would be required for all TL-4 rigid barriers. Barrier front face shape and top width may impact the minimum height. TTI had a success at 36 in., while failures were observed at 32". We know from past experience that rectangular shapes (vertical faces) allowed a lower height parapet to redirect a 8000S vehicle at TL-4 of NCHRP 350 as compared to safety shapes (sloped front faces). We have conducted LS-DYNA modeling around a decade ago that suggested that a 34.5" tall vertical face barrier may actually be capable of containing a MASH TL-4 10000S SUT vehicle. Testing would be needed to confirm. Thus, the 36" minimum is based on a TTI test on a particular barrier shape and top width. Depending on shape, it may be possible to actually use a 37.5" barrier to account for a 3" overlay for later field use of 34.5" (if proven with testing). If overlays are considered, then the barrier and deck design should consider the increased barrier height and load application height from asphalt surfacing.

I suspect that we will learn more with the Manitoba TL-5 test next week. We also plan to investigate and evaluate existing guidance for deck design based on barrier capacity at base. We will let you know of those results as they are obtained from the crash testing and follow-on analyses.

Finally, several of the topics discussed above are actually addressed in a few proposals under consideration at the April Pooled Fund meeting.

Attachment: http://mwrsf-qa.unl.edu/attachments/ea247f56f6e6f37a1909ab0b8712d6a4.pdf

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Thrie-beam height tolerances

Question

State: IA
Date: 04-06-2016

As a follow-up toQuestion #288, was a decision ever reached about thrie-beam height tolerances for MASH TL-3 levels? We currently show 32" at the concrete bridge end post and then transition down to 31" at the w-beam connection on ourBA-201, but a question has come in about an older bridge rail that is lower than 32" and we're not sure how low the thire-beam connection can be.

Response

Date: 04-07-2016

We have addressed this issue in the past for several states. You may want to look at the responses below and see if they address your question. The short answer is that little research has been done on thrie beam transitions at heights below 31". Thus, there are concerns with the performance as the thrie beam AGT height is reduced.

http://mwrsf-qa.unl.edu/view.php?id=1029

http://mwrsf-qa.unl.edu/view.php?id=246

http://mwrsf-qa.unl.edu/view.php?id=908

Let me know if this helps.

Thanks

AGT Special Curb Design, under the Thrie-beam section

Question

State: MN
Date: 04-14-2016

We have been looking into a new single slope barrier, AGT design option, with a back taper at the bottom to reduce snagging. However, this option is not preferred because of constructability and water drainage (curb flow line consistency) concerns.

One alternative design has recently been proposed. It uses an 8-inch high curb with a front face at the same slope as the single slope barrier.
The gap between the top of the 8-inch curb and the bottom of the thrie-beam would be 3-inches. The curb would tapper to a 4-inch curb (or to zero) before the W-to-thrie transition element.

Attached is a draft drawing showing this design approach. We would like to know if this would reduce the snagging issues at the concrete end, similar to a back tapper design.

Let me know if you need any further information.

Thank you
Attachment: http://mwrsf-qa.unl.edu/attachments/4d2093376bb7b469d8fddf16774e19cb.pdf

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Response

Date: 04-25-2016
Previous full-scale testing has shown that the addition of a curb has generally reduced the amount of snag on the rigid buttress. That being said, the face of the buttress has typically been tapered back in conjunction with the curb. I don't believe any AGT has been successfully tested without a taper of the rigid buttress (of the toe or the entire front face). Thus, snagging may still be an issue. The smallest taper that i could find that was successfully tested on a SS buttress was 4" laterally, by 22" longitudinally - reference TRP-03-47-95. That AGT configuration did not utilize a curb, so there is a chance that adding a curb could help reduce snag, but it has not been tested/evaluated.

My biggest concern with the proposed idea is a potential for vehicle instability with an 8" tall curb. All of the previously crash tested (350 and MASH) AGT designs utilized 4 inch tall curbs. Testing of W-beam guardrail with a curb has used curb heights up to 6". There is no data available on vehicle interactions with an 8" tall curb. As such, my concerns are that excessive climbing and rolling may occur during vehicle impacts. Again, there is no data proving that this is a problem, but there is also no data demonstrating the crashworthiness of an 8" curb. However, I would feel better about leaving the curb height at 4 inches throughout its length - the curb height typical of AGT's that has been shown to be crashworthy.

If you do decide to utilize a taller curb, you are correct in transitioning it down to a 4" tall curb prior to the curb extending below the W-to-thrie beam transition element. Previous testing has demonstrated the sensitivity of this region of the AGT, and we would not want to induce further vertical climb and increased vertical rail loads.

Installation of MGS Long Span on Arched Culvert

Question

State: WI
Date: 05-02-2016

We have an installation proposed for the MGS long span across an arched culvert. The long span was chosen as the arched culvert limited the embedment depth.

However, the culvert has a secondary issue in that the CRT posts are against the culvert headwall.

What are your thoughts on this type of installation?

Attachment: http://mwrsf-qa.unl.edu/attachments/9e9f29dd38dea41f55af92a82ac3de6c.pdf

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Response

Date: 05-05-2016

We are concerned that placing the CRT post against the headwall may adversely affect the performance of the breakaway post and the overall behavior of the long span. CRT's do dissipate some energy through soil rotation and placement of the post against the headwall will affect that rotation significantly.

In previous situations like this we have recommended a minimum offset of 1 ft between the back of the post and the headwall to allow for that rotation.

The only other option I can think of is to use the MGS weak post system. However, it is unlikely that we can get those posts embedded sufficiently over the top of the culvert arch. Thus you would need to move the line of the guardrail closer to the slope and attach the weak posts to the culvert directly.

MGS Deflection Reduction

Question

State: UT
Date: 05-03-2016

Its it possible to reduce the MGS deflection to 12 inches measured from back of post to hazard? Such as nested rail with quarter post spacing or use larger posts similar to w-beam to parapet transition designs?

Thank you for your time,

Response

Date: 05-05-2016

We do not have any data to support the reduction of the MGS deflection to the level of 12 inches measured from back of post to hazard. During NCHRP 350 TL-3 testing of the MGS, we evaluated a ¼ post spacing system that had a dynamic deflection of 17.6" and a working width of 36.7".

The working width value is taken from the front face of the guardrail. As such, this would correspond to a clear distance from the back of the post to the hazard of only 15.4".

There is potential to further reduce that deflection through the use of larger posts and or nested guardrail. However, we have not developed that kind of system to date. In order to determine what the deflection reduction would be and the effect on the overall performance of the barrier, further analysis and/or crash testing may need to be conducted on a modified system.

Double-faced to Single Face MGS Transition

Question

State: MO
Date: 05-05-2016

Please review the attached email regarding how to transition between the median version of the MGS to two separate MGS installations and provide any guidance or suggestions.

thanks

Attachment: http://mwrsf-qa.unl.edu/attachments/26c0bc1094be4b2b2a4b4f7443189529.pdf

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Response

Date: 05-05-2016

I looked at your detail. As you noted, there is more leeway currently to omit a post in the MGS due to the test we ran last year. Thus, I follow the thinking you guys were using. However, we still have two concerns.

1. There is some concern that the deflected guardrail can impact the back side of the offset posts. This loading of the rail across a corner or an edge of a post has shown the potential to cause rail rupture. Thus, there is some concern that the offset posts may degrade system performance.

2. There is also concern that the posts that are offset longitudinally may interfere with each other when one side is impacted. In tested of a PCB transition system and recent research we have been doing regarding minimum light pole offsets for the MGS, we have observed that the deflected post movement can be restricted by objects just downstream of the post as it is deflected. This can potential lead to reduced deflection, pocketing, and snag.

To alleviate these concerns, we would recommend using extended blockouts on a single post as the two rails widen out at whatever flare you chose up to a maximum of 15:1. This should be acceptable up to a blockout depth of 24". Two separate posts can be used as soon as you reach a minimum offset from the face of the guardrail to the backside of the opposite post of 43".

I have attached a schematic of what that looks like for a 15:1 flare starting with 8" deep blockouts.

Let me know what you think and we can discuss things further.

Attachment: http://mwrsf-qa.unl.edu/attachments/9153cfccf615a36a16a48f023aeb3a55.jpg

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Wood and Composite Blocks MGS

Question

State: UT
Date: 05-09-2016

The RDG section 5.4.1.7 MGS,
states that Wood Blocks are used and testing documents I have read
state "White Pine".

Are composite blocks allowed
to be used with the standard MGS system?

Regarding the 25 feet span
that required 12 inch blocks, would composite blocks appropriate as
well?

Thank you for your time,

Response

Date: 05-09-2016

We have largely tested the MGS with 12-in. SYP wood blocks. We have conducted some testing with 12-in. WP wood blocks as well. I do not believe that we have used recycled blocks in our tests but may have conducted component tests on recycled blocks that may be applicable. I somewhat recall that E-Tech Testing Services conducted testing on a recycled MGS blockout several years ago. I will ask my colleagues whether 12-in. blocks have been fabricated and tested with the MGS. The MGS was only tested with wood blocks thus far. Without testing, I am uncertain about using a different blockout with the 3 CRTs on each side of the long span and believe that testing may potentially need to be performed.

Working width of MGS median barrier

Question

State: OH
Date: 05-10-2016

ODOT would like to install a two sided MGS barrier guardrail in a relatively narrow median. We typically use a minimum barrier clearance to an obstacle of 5 feet, measured from the face of the barrier to the obstacle, for both barrier guardrail and normal MGS. Would a two sided barrier type guardrail be stiffer than normal MGS? With 12" blockouts, the width of the system would be around 3 feet. To keep part of the barrier or a vehicle that strikes the barrier from protruding into the opposing traveled way, what would be a reasonable working width? Would there be any problems associated with using half or quarter post spacing with a barrier guardrail in the median?

Response

Date: 05-10-2016
A two-sided or median type MGS system would be slightly stiffer and have lower deflections than a standard MGS system due to the rail on both sides of the barrier.

At this time, the MGS median barrier with 12" blockouts was deemed NCHRP 350 compliant with an FHWA eligibility letter based on a submission we made comparing the MGS system to other previously tested median barrier designs.

Subsequent to that eligibility letter, TTI performed two MASH tests on a version of the 31" tall MGS median barrier with 8" deep blockouts (see attached). Testing with the 2270P found a dynamic deflection of 39" and a working width of 55". The dynamic deflection is about 11% lower than the 43.9" dynamic deflection we observed in the original MASH testing of the MGS. However, the working width is slightly higher, as the original MGS MASH test found a working width of 48.6" The difference in the working widths is likely due to the additional width of the median system as compared to the roadside version.

Based on this, the best working width guidance we have is the 55" number from the TTI testing. We cannot recommend reducing this value using reduced post spacing without full-scale crash testing based on concerns for rail rupture observed in the 1100C test performed on the system by TTI. In the 1100C test of the 31" tall MGS median barrier with 8" deep blockouts. the vehicle was safely capture and redirected, but a tear that extended 2/3 to 3/4 through the first rail splice downstream of impact was observed. We have seen similar tearing in select MGS small car tests due to what we believe is combined loading of the splice due to the lateral rail loading and vertical loading/bending of the splice due to the small car body being wedged under the rail. For example, the first test of the upstream end of the MGS AGT stiffness transition with a 4" tall wedge curb had a similar rupture that caused the test to fail. Thus, there is concern that further stiffening of the barrier through reduced post spacing may increase the rail loads and lead to a complete rail rupture. As such, we would recommend full-scale crash testing to evaluate reduced post spacing versions of the MGS median barrier prior to implementing reduced post spacing.

Let me know if you have further questions or comments.

Attachment: http://mwrsf-qa.unl.edu/attachments/2327ec399e4402d7954403f325fa3916.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/2eec3c45bcdbdddf32c691d396448b96.pdf

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Cover Plate for Gaps in Concrete Barrier

Question

State: WY
Date: 05-10-2016

A question I posed before the Pooled Fund Website in 2013 concerned a method of transferring load between a concrete bridge barrier and an approach concrete barrier. Shown below in red is a part of Midwest response:

"If you feel that the connection is in need of improvement, a number of states have utilized a design that resembles a steel plate/shell that is bent to the shape of the barriers, placed over the top of the adjacent barrier ends, and bolted down on both sides. Of course, the bolts are placed in slots so that the joint can expand and contract. This type of connection would ensure a quicker load transfer as well as prevent vehicle snagging on the barrier ends if the expansion joint opens up."

Could you send me a copy of details of this? What is the maximum gap you can cover with this type of design?

Response

Date: 05-17-2016
We recently had a student look through State DOT standard plans and drawings for such cover plates. I have attached the drawings that were located. Other states do something similar, but it may not be in the standards.

These cover plates are largely untested, so the gap sizes that they can cover is not really known. Allowable gap size would be dependent upon plate and barrier shape, plate thickness, attachment hardware, and test level. We should know a lot more about such devices in a year or two as we currently have a Pooled Fund project to explore PCB gap hardware – YR 26 project.

Attachment: http://mwrsf-qa.unl.edu/attachments/5c9e13ef7d0415c507f7bf60af04c380.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/ffb71e5f1e4da36da3de77d6a663fa97.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/81fdddc49c864bbe54ad53918e3f2c12.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/a8528d3e6fba070354827f7347d959a7.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/8af6d0b4e98afaa56a93e94ddecab6a0.pdf

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Guardrail Transition to MGS

Question

State: IN
Date: 05-11-2016

In Ohio, they have a standard drawing showing a transition from existing guardrail to MGS, see attachment. This transition adjusts for the height difference between the two systems within 25 ft and moves the spice from the post to the midspan. This standard uses a 3'-1.5" post pacing closest to the existing guardrail. I have seen on the Q&A page that you have addressed the transition question before (1076) and did not recommend placing a 3'-1.5" post spacing to keep the 12'-6" standard w-beam sections. Rather you suggested that the height transition be completed within 25 ft with splices at the posts, develop the 31 inch height for one 12'-6" w-beam section, and then drop a post to start the midspan slices. This would make the transition 53'-1.5". Have there been other discussions that have led to the Ohio standard being acceptable?

Attachment: http://mwrsf-qa.unl.edu/attachments/8b6b6d3e18802a106062616484f0ca11.pdf

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Response

Date: 05-19-2016

Hello.

The guidance in 1076 is currently our best guidance in terms of transitioning from G4(1S) systems to the MGS without using a odd length rail section to transition from splices at the posts to midspan splices. Keep in mind that these recommendation our based on our engineering judgment and past knowledge of the impact performance of W-beam guardrails systems in MASH and NCHRP 350 testing. Thus, our recommendations tend to represent our best case for performance as is is difficult to anticipate at what point we reach system limits without testing. Other design iterations may still work. So, we are not suggesting that other transitions are not feasible, but rather that we have more confidence in the recommended transition.

That is not to say that other designs may not work. However, none of these types of transitions have ever been crash tested, so we tend to err on the side of caution when making these recommendations. The transition we suggested in question 1076 is similar to a recently tested MGS system with an omitted post that was evaluated to MASH TL-3. Thus, we have confidence that a similar scenario will work to transition between the splices.

The transition you have shown from Ohio appears to add an additional post at reduced spacing to achieve the splice adjustment. This appears to occur in the G4(1S) section of the guardrail. This may be acceptable, but we might recommend a couple of improvements.

1. Currently, we are not as confident in using the additional post method to transition between G4(1S) systems to the MGS in the G4(1S) section of the guardrail. Based on previous testing, the G4(1S) system is at or near its limits with respect to TL-3 MASH impacts. We recommended transitioning the splices in the MGS region as the MGS has proven more robust that the G4(1S) system in a variety of crash test scenarios, and it seems more conservative to apply any splice transition in the more robust guardrail system.
2. The extra post shown for transitioning the splice is currently shown directly adjacent to a splice. We would adjust this slightly as the splice tends to be the weak point in a guardrail system. Thus, we would place the extra post adjacent to the midspan post and have no reduced post spacing at the splice. This should lower the loading on the critical location of the splice in the rail. - See attached.
3. As noted in 1076, we would not recommend this until you have completed the height transition and have a minimum of 12.5' of 31" rail.

You could also do the additional post splice transition noted above in the G4(1S) section of the guardrail, but locating the transition in that region is less conservative than the approach in 1076 or the 31" guardrail post adjustment noted here.

Please let me know if you have further questions or comments.

Attachment: http://mwrsf-qa.unl.edu/attachments/99cbb020631141d1f57733c1f046b770.jpg

dimension tolerances and W6x9

Question

State: WI
Date: 06-03-2016

To All,

I got a request from a manufacturer to approved there shop
drawings for steel post in the thrie beam transition.

I when through their drawings, my drawings and your
drawings.It appears that the manufacturer's
one of the dimensions are about 3/32" of an inch off.

To me this does not appear to be an issue.But, it does bring up a good questions:

1. What
   dimension tolerances does MwRSF use in their drawings? Should states have similar tolerances in their drawings?
2. We
   allow the contractors to use W6x9 or W6X8.5 in the MGS thrie beam
   transition. MwRSF has only W6X8.5 in the
   MGS thrie beam transitions. Is W6X9
   acceptable in the MGS thrie beam transition?

Response

Date: 06-03-2016
It is not entirely clear what dimension on the post is 3/32" off the detail. However, the discrepancy could could

from a couple of areas.

First, our plans show the actual dimensions of a W6x8.5 post to the nearest 1/16". These vary slightly from the actual decimal values for the post. Additionally, W-shapes are generally referenced or shown using simplified values rather than the actual values. Thus, a W6x8.5 is usually listed as 6" deep by 4" wide when the actual dimensions are 5.83" and 3.94", respectively.

As you noted, the difference could also be due to W6x9 posts versus W6x8.5 posts. We have been using these posts for several years as it came to our attention that they are what is generally supplied for guardrail applications. They are very similar, and you may have received W6x9 posts in reality.

We do not believe that the performance of the barrier is affected in any significant way using W6x9 versus W6x8.5. The overall dimensions are nearly identical and the section properties are only 10% different. Thus, we don't see any issues with using the W6x9 posts in lieu of the W6x8.5 posts.

We do not post dimensional tolerance for standard structural parts in our plans. The AISC steel manual does list acceptable tolerances for W-shapes. This would be the best place to start for tolerances. See attached.

Attachment: http://mwrsf-qa.unl.edu/attachments/9c575806585446aaaee8aa7ea2aceb94.pdf

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Response

Date: 07-12-2016

We have further reviewed the dimensional tolerance issue noted previously. It appears that the discrepancy has to do with the location of the post bolt hole along the width of the post flange.

Review of the plans, previous hardware guide details, and plans from full-scale crash tests suggest that the 3/32" you note is based on how the hole placement is measured. The original 1979 Guide to Standardized Highway Barrier Rail Hardware denotes the location of the hole as 1 1/8" from the center of the post. Later revision of in the 1995 AASHTO Guide to Standardized Highway Barrier Hardware denotes the hole location using both the 1 1/8" from the center of the post dimension and a dimension 3/4" from the edge of the flange.

These two locations for the hole are not the same. Using the 1 1/8" from the center of the post dimension yields a distance from the center of the hole to the the edge of the flange of 0.845". This is the source of the 3/32" difference you noted.

MwRSF has typically detailed our hole locations from the edge of the flange based on standard CAD practices. Also, this provides a slightly lower distance to the flange edge for structural loading. However, most manufacturers specify the hole locations from the center of the flange based on manufacturing procedures. We believe that there is little functional difference between these values and either is acceptable.

It should be noted that the AISC allowable tolerances for W-sections allows for variations in flange width and hole placement that may affect the location of the hole. According the AISC, flange width can vary +1/4" to -3/16" and the web can be offset as much as 3/16" from the center of the flange. These variations along with manufacturing tolerances on the hole fabrication may place the hole closer to the edge of the flange than 3/4". To the best of our knowledge, the affect of the W-section tolerances on the post hole distance to the edge of the flange has not been a safety issue with respect to barrier performance.

Attachment: http://mwrsf-qa.unl.edu/attachments/a846e1326f11538eae630ce5a4ace6ec.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/27ba565d2cceae89d533d6c6d5739229.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/f94c817a1728211f699f40b9a9f29e0f.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/4105044e6607cf840ebcdd163d05102f.pdf

Box beam barrier

Question

State: OH
Date: 06-07-2016

Ohio has a high wind and snow area on a causeway where box beam barrier was installed in a narrow median.
I didn't know we had this system in Ohio until it was damaged and the district office wasn't sure how to repair it.
Which leads us to several questions:

1) This system appears to be common in New York and Wyoming. Do you know the status of this system as far as MASH compliance?

2) Should and can this system be raised to reflect the increased height of other barrier systems and if so how high?

3) Can this system be used with an inlet or post socket instead of a soil plate when continuing over a structure?

Thanks!

Attachment: http://mwrsf-qa.unl.edu/attachments/8f19e9a6a641daba06f034c22312a29b.JPG

Response

Date: 06-08-2016

I have replied to you questions regarding box beam barriers below in red.

1) This system appears to be common in New York and Wyoming. Do you know the status of this system as far as MASH compliance?

Box beam guardrail has been evaluated to MASH TL-3. Previous MASH full-scale crash testing was conducted on the G3 box beam guardrail system and New York's box beam terminal design found that box beam systems with top mounting heights of 27 in. (685.6 mm) were capable of safely redirecting 2270P vehicles under TL-3 impact conditions.

With respect to the 1100C vehicles, recent cable barrier testing with S3x5.7 posts have shown a potential for laceration and penetration of the vehicle floorboard by the free edges of the post. According to MASH, this is sufficient cause for failure of the test based on penetration of the occupant compartment. To the best of our knowledge, this has not been evaluated through MASH testing of box beam with the 1100C vehicle.

http://mwrsf.unl.edu/researchhub/files/Report60/TRP-03-203-10-Vol1.pdf

http://mwrsf.unl.edu/researchhub/files/Report60/TRP-03-203-10-Vol2.pdf

http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_w157.pdf

2) Should and can this system be raised to reflect the increased height of other barrier systems and if so how high?

Box beam median systems were evaluated under NCHRP Report no. 350 with small car vehicles at 28" top mounting heights. Without further analysis, it would be difficult to recommend heights above this even though the potential for them to perform adequately exists

3)
Can this system be used with an inlet or post socket instead of a soil plate when continuing over a structure?

Because box beam is a weak post guardrail system, the main resistive force and energy absorption is derived from yielding and deformation of the post. Weak post systems typically depend much less on displacement of the post through soil. As such, cable barrier systems commonly have substituted sockets in place of deeper embedment posts or soil plates and demonstrated similar behavior. In terms of box beam guardrail, there is some potential that the use of as socketed foundation could alter post deformation and deflection slightly, and we have not observed this practice used box beam in any past crash tests. However, based on the performance of cable barrier systems with sockets, it would seem that the performance of a socketed post box beam system would be very similar to a standard system with posts and soil plates.

SD Concrete End Block and multiple transitions to High Tension Cable Barrier

Question

State: SD
Date: 06-21-2016

See
attachments. I included a .dgn file as the .pdf is not very good.

Since I was instructed that South Dakota MUST utilize a transition from MGS to High Tension Cable Barrier and in the interim time period when there is NO MASH alternative for a transition from MGS to High Tension Cable Barrier, I drew a long transition from the SD concrete bridge end block to Nested Thrie Beam to MGS to W Beam to High Tension Cable Barrier.

Pleaselook at what I have and provide any recommendations as there is probably something that could be made better. We need a transition from the SD concrete end block and this may eliminate one of the 7' long 6"x8" wood posts. We did not want to use the larger (10"x10"?) wood post transition. We are interested in only using wood posts and blockouts. We did not want to have a separation from the MGS to cable barrier as we may have traffic in the opposite direction at times and want the cable attached to the W Beam. The W Beam to high tension cable barrier transition is drawn using the Trinity transition from W Beam to 4 Cable High Tension Barrier.

I can't wait until the pooled fund gets a MGS to Midwest High Tension Cable Barrier Transition MASH tested and approved. Much will need to be done prior to the transition testing though.

On another note, I noticed multiple places in the crash test reports that the wood blockout is dimensioned as 6"x12"x14 ¼" as used in the MGS. I see many states use 6"x12"x14" blockouts. Is it proper to use the 14 ¼" dimension?

Thanks for looking at the suggested long transition,

Attachment: http://mwrsf-qa.unl.edu/attachments/2ec5ebafb672fdb0af660b5779e7b63d.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/5ff898e4ca506d4c6e3d0db6b736767b.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/b93692c6dec989ffe46c9c0e7313c1f5.zip

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Response

Date: 07-11-2016

First, I looked at the cable to W-beam transition that you are proposing to use based on the Trinity design. The detail you sent doesn't show all of the small details of the transition, but I have a few thoughts.

1. We have had concerns about the currently accepted high-tension cable barrier transition designs and have noted them in the past. There may be some advantage to a high tension cable to W-beam transition in that these cable

barriers have a lower deflection than typical cable barriers. This creates a less dramatic stiffness change in the transition and would also reduce the potential for interaction with the terminal end. But we also have concerns.

a. First, the adequacy of the anchorage of the downstream ends of the cable barriers in these systems is largely unknown. Recall that the testing of the South Dakota cable to W-beam transition displayed two instances where cable anchorage was partially lost and reduced. Expectation for the high tension cable to W-beam transition design would have to be even higher anchor loads, yet these anchorages have not been tested.

b. Second, the increased tension in the cables could increase the potential for vehicle snag at the point in the transition where the cable and W-beam barriers come together.

c. Next, cable heights and for the high tension cable barriers are generally significantly higher than the 27" top cable height used on the previously tested transitions.

d. Finally, the hardware pieces used to transition the cables to the W-beam vary greatly and have not been evaluated.

2. If you are basing the design off of the Trinity system that has an FHWA approval letter, then I would recommend that you follow as closely to the accepted design as you can. I have attached the letter. Note that the Trinity system uses 10 gauge W-beam and is for a three cable system in the letter. I cannot see the rail type or the anchorages that you are using, but I would follow these guidelines as they are what is in the letter. I don't have any details or approval for the 4-cable transition.

With respect to the AGT details.

1. What you are showing seems consistent with the MASH upstream stiffness transition we have previously developed and tested with the wood post version of the Iowa AGT. This should be fine. However, if I understand correctly, you wish to omit post 1 due to the shape of your parapet. This may be problematic due to lack of support for the thrie beam and the potential for the omitted post to increase the potential for vehicle snag on the parapet. The original, tested Iowa transition connected the thrie beam to the parapet at approximately 20" from the end of the parapet which left around 11.5" between the center of post no. 1 and the parapet.

2. Your parapet attaches the thrie beam farther back which does not appear to allow for the placement of post no. 1. As such, there are a couple of potential options:

a. Move the thrie beam end show connection closer to the end of the parapet to allow for the placement of post no. 1. This would allow for installation of the transition as tested. However, you may have reasons for not doing this currently.

b. One could omit post no. 1 as you have shown. In order for this to have potential to perform safely, we would recommend that the offset from post no. 2 to the parapet be less than or equal to the 11.5" noted above. This should help reduce snag. There is some concern that omitting post no. 1 will affect the overall deflection and stiffness of the system and may lead to increased snag. However, this can't be easily investigated without further effort. For this installation, we would also recommend that a wood or steel spacer be placed under the thrie beam in the 18" long, flared back portion of the parapet to provide support for the thrie beam. This should help reduce the snag potential and aid in the transition performing closer to the tested design. The rail can be bolted through the spacer and the parapet to keep things in place.

With respect to the blockout dimensions. Our details tend to show the 14 ¼" number, but we don't believe that a 14" blockout height is an issue if you have them in that size.

Thanks

Attachment: http://mwrsf-qa.unl.edu/attachments/d363be4c02f480807db8e9c8ed169ab6.jpg

Repair of posts embedded in concrete

Question

State: OH
Date: 06-27-2016

We have several old existing bridge terminal assemblies on low volume roads which use posts embedded in concrete. When a single post is damaged and requires repair, our maintenance crews pull the old concrete, set a new post and then pour new concrete. They would like to eliminate the concrete work. Would a single post not set in concrete cause a performance problem? Are there other post depth or thickness options that would perform in a similar fashion to the embedded concrete post?
Attachment: http://mwrsf-qa.unl.edu/attachments/206a7336aedfaf871b313d34050b58a7.pdf

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Response

Date: 06-29-2016
To answer your question, changing from a post embedded in concrete to a post embedded into soil only could certainly change the performance of the system. To counter the loss of stiffness when removing the concrete, you would have to increase the post size and/or increase the embedment depth of the post. Unfortunately, I do not have any data on the strength and resistance capacity of these posts embedded in concrete. Consequently, we don't have a target strength/resistance to shoot for when designing a retrofit for these posts. Additionally, the strength provided by a W6x15 steel post will be significantly different than that of a 6"x8" wood post. So, I'm not really sure were to even start with a retrofit post.

After a quick literature search, I cannot find any documentation on the development, analysis, or testing of these guardrail-to-bridge rail transitions. In comparing them to the transitions that have eligibility letters from FHWA, I have concerns about the crashworthiness of these systems to either MASH or NCHRP Report 350 safety standards. As such, I would not recommend new installations utilize these designs. Taking it a step further, if you have to repair one of these transitions, it may be easier to just tear out all of these concrete embedded posts and install a new NCHRP 350 approved transition system (one that doesn't include concrete posts). Transitions that the FHWA had previously noted as meeting NCHRP 350 criteria can be found on their website - in the acceptance letters and two Technical Advisory Memos. See the following links:

http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/
http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/techadvs/archive/t504026/
http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/techadvs/archive/t504034/
