Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

10-01-2012 to 12-31-2012

MGS Low-Fill Culvert Attachments

Question

State: KS
Date: 09-29-2012

The two attached draft standard drawings are related to another topic you and Scott may have been discussing; attachments to low fill culverts. KDOT is working on developing standard drawings illustrating MGS attachments to low fill culverts wider than 22'-6". We've adopted a different base plate for our epoxy attachments which corresponds to a plate tested by TTI for MGS guardrail (see attached report). Please review the attached PDFs and let me know if you have any comments/concerns. Essentially we are planning to use the same plates we've used in the past when bolting through the top of the RCB. Typically the top slab of the RCB would be a minimum of 6" thick. Our primary concern is whether or not the increase in height to 31" will affect the bolt performance because of the increased moment from a guardrail impact. I appreciate your time and look forward to hearing from you.
Attachment: https://mwrsf-qa.unl.edu/attachments/508c507e69566a51207e4979adf5aff4.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/e56b0f915723f58df5de7bb3889fa456.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/bf687e6feec16a9aa811d6a2f9603fcd.pdf

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Response

Date: 09-29-2012
Historically, most researchers have had the opinion that the W6x8.5 or W6x9 steel posts with steel base plates anchored to the top of the culvert slab would allow the W-beam guardrail system to perform in an acceptable manner when embedded either into shallow soil fill as well as full depth soil fill. In addition, these types of guardrail designs have utilized various sizes and configurations of welded steel base plates at the bottom of the posts for bolted attachment to the top of concrete culvert slabs. Further, different diameters of through-bolts have been successfully used for the attachment. Over the years, these crashworthy designs have generally used 27-in. or 27¾-in. top rail mounting heights and post spacings of 6 ft – 3 in. or 3 ft – 1½ in., depending on the lateral post offset relative to the front face of the headwall.

Recently, TTI researchers successfully crash tested and evaluated a modified W-beam guardrail system for attachment to culverts using a 31-in. tall W-beam guardrail system. For this recent design, W6x9 steel posts were welded to 7/8-in. thick steel base plates and spaced 6 ft – 3 in. on centers with midspan rail splices. The posts were attached to the culvert using four 7/8-in. diameter rods that were epoxied into the concrete with a 6-in. minimum embedment depth and a Hilti chemical adhesive anchoring system.

At this time, the Kansas DOT is exploring revisions and alternatives to the currently-used W-beam guardrail system for attachment to concrete culvert slabs based on the recent TTI test results and the desire to utilize the MGS barrier system. As such, there is a desire to increase the guardrail height from the old standard to the new 31-in. mounting height while still maintaining the W6x9 steel post and welded base plate measuring 5/8 in. x 6 in. x 10 in. The original KsDOT post/plate configuration was likely designed to allow for plastic post deformations to occur, thus contributing to the energy dissipation capacity of the guardrail system. The new TTI post/plate was also likely designed to serve a similar purpose. Therefore, if similar dynamic behaviors and capacities exist for the two slightly different post/welded plate combinations, then similar guardrail performance would also be expected using either anchor post system with 31-in. tall guardrail. At this point, it would seem reasonable to allow the use of either post/base plate alternative, I currently do not have specific force-deflection and energy-deflection curves for the two options. These dynamic curves would helpful in making a final determination.

On another matter, the CAD depicts the post in Detail A having its front flange welded very close to the bolt heads (Section A-A). Is sufficient clearance available to attach the post? Are washers used on the top and bottom surfaces? Is the 3/8-in. single-pass fillet weld applied to both flanges and web of a W6x9 post? The web and flange thicknesses are much thinner than this weld size? Where did the guidance come from regarding a safe working load of 8,000 lbs of tension for alternative anchors?

I look forward to hearing from you on this matter. Thanks!

Response

Date: 09-29-2012
I wanted to take some time to respond to your questions.

Q: Is sufficient clearance available to attach the post?
A: KDOT has been using this detail for our low fill culvert attachments for several years and have no reports of any issues attaching the post to the plate.

Q: Are washers used on the top and bottom surfaces?
A: The washers are only located on the bottom surface.

Q: Is the 3/8" single-pass fillet weld applied to both flanges and the web of the post? The web/flange thicknesses are thinner than the weld size.
A: From our standard drawing it appears the weld is only applied to one side of the web. The web and flange thicknesses are 3/16" while the weld is 3/8".

Q: Where did the 8,000 lbs of tension for alternative anchors come from?
A: The 8,000 lbs for alternative epoxy anchored bolts is related to accomodating pull out strengths for various epoxy manufacturers. Essentially KDOT would not be excluding/specifying a specific epoxy manufacturer as long as they meet the minimum tensile load requirment of 8,000 lbs. It's my understanding, per information from Rod Lacy, 8,000 lbs was selected due to the pull-out force the bolts would experience during a TL-3 barrier impact.

Given this information can you offer any additional guidance regarding whether you feel the attachments seem appropriate for this application?

Response

Date: 09-29-2012

Thanks for the follow up on this issue. My comments are contained below.

Ron

Ron, I wanted to take some time to respond to your questions.

Q: Is sufficient clearance available to attach the post?
A: KDOT has been using this detail for our low fill culvert attachments for several years and have no reports of any issues attaching the post to the plate.
**The CAD detail appears to depict the bolt head touching the front flange in Section A-A. In this configuration, the bolt head would be positioned on the front fillet weld, thus making it difficult to turn the head and fit a socket wrench. Maybe the post is scaled to an incorrect size in Section A-A? Thecenterline of slotted holes are 1.75 in. away from right side of plate, while the front flange is about 3 in. away from right side of plate (without considering 3/8-in. weld. The bolt could be shifted inward per the use of slotted holes. As such, the head could be positioned even farther inward.

Q: Are washers used on the top and bottom surfaces?
A: The washers are only located on the bottom surface.
**Okay.

Q: Is the 3/8" single-pass fillet weld applied to both flanges and the web of the post? The web/flange thicknesses are thinner than the weld size.
A: From our standard drawing it appears the weld is only applied to one side of the web. The web and flange thicknesses are 3/16" while the weld is 3/8".
++The 3/8-in. fillet weld is shown all the way around (i.e., both sides of web and both sides of each flange). As such, the toe of the fillet weld on the outside of the traffic-side flange would be 2.625 in. away from the right side of the plate. This 3/8-in. weld size is rather large for a single pass weld per side when considering the flange/web thicknesses. Industry would not likely want to fabricate it in a single pass. We have worked with three-pass 5/16-in. fillet welds on the traffic-side flange (both sides) and ¼;-in. fillet welds on the web (both sides) and back side flange (both sides).

Q: Where did the 8,000 lbs of tension for alternative anchors come from?
A: The 8,000 lbs for alternative epoxy anchored bolts is related to accomodating pull out strengths for various epoxy manufacturers. Essentially KDOT would not be excluding/specifying a specific epoxy manufacturer as long as they meet the minimum tensile load requirment of 8,000 lbs. It's my understanding, per information from Rod Lacy, 8,000 lbs was selected due to the pull-out force the bolts would experience during a TL-3 barrier impact.
++A ¾-in. diameter ASTM A307 anchor through-bolt has an ultimate tensile strength of about 20 kips without applying reduction factors. Previously, I missed the fact that the alternative 7/8-in. diameter threaded rods conform to AL 39 material, which are to be epoxied into the slab. I am unfamiliar with this material. Can you elaborate on the steel grade so comparisons can be made to the two capacities and later to determine whether the epoxy rating is sufficient? Thanks!

Response

Date: 10-01-2012
Ron, see myhighlighted responses to your comments/questions below.

Tom

Ron, I wanted to take some time to respond to your questions.

Q: Is sufficient clearance available to attach the post?

A: KDOT has been using this detail for our low fill culvert attachments for several years and have no reports of any issues attaching the post to the plate.

**The CAD detail appears to depict the bolt head touching the front flange in Section A-A. In this configuration, the bolt head would be positioned on the front fillet weld, thus making it difficult to turn the head and fit a socket wrench. Maybe the post is scaled to an incorrect size in Section A-A? Thecenterline of slotted holes are 1.75 in. away from right side of plate, while the front flange is about 3 in. away from right side of plate (without considering 3/8-in. weld. The bolt could be shifted inward per the use of slotted holes. As such, the head could be positioned even farther inward.

See response to third question.

Q: Are washers used on the top and bottom surfaces?

A: The washers are only located on the bottom surface.

**Okay.

Q: Is the 3/8" single-pass fillet weld applied to both flanges and the web of the post? The web/flange thicknesses are thinner than the weld size.

A: From our standard drawing it appears the weld is only applied to one side of the web. The web and flange thicknesses are 3/16" while the weld is 3/8".

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The weld we have been discussing is actually and 3/8" x 3/8" beveled weld (from a construction practices perspective I'm not sure if that makes a difference). The weld is intended to be placed all the way around the outside of the flanges and web (as you indicated is currently shown on the drawing). If the ¾" diameter hex bolt is placed in the slotted hole closest to the weld the bolt head does overlap the weld location. However, if the hex bolt is placed in the slotted hole farthest from the weld there is approximately 5/8" b/t the outer most edge of the hex head and the base of the weld. This should allow enough room for construction. (Please see attached detail for clarification, it appears the previous drawing may not have been shown to scale.) Given this information would it still be appropriate to specify fillet weld sizes and locations you wrote in blue above in lieu of the 3/8" bevel?

Q: Where did the 8,000 lbs of tension for alternative anchors come from?

A: The 8,000 lbs for alternative epoxy anchored bolts is related to accomodating pull out strengths for various epoxy manufacturers. Essentially KDOT would not be excluding/specifying a specific epoxy manufacturer as long as they meet the minimum tensile load requirment of 8,000 lbs. It's my understanding, per information from Rod Lacy, 8,000 lbs was selected due to the pull-out force the bolts would experience during a TL-3 barrier impact.

++A ¾-in. diameter ASTM A307 anchor through-bolt has an ultimate tensile strength of about 20 kips without applying reduction factors. Previously, I missed the fact that the alternative 7/8-in. diameter threaded rods conform to AL 39 material, which are to be epoxied into the slab. I am unfamiliar with this material. Can you elaborate on the steel grade so comparisons can be made to the two capacities and later to determine whether the epoxy rating is sufficient? Thanks!

The A139 is a typo. It's supposed to be A 193 threaded rod per TTI's report. I apologize for the confusion. Attached is information related to the Epoxy used in the TTI report for low fill culvert attachments. Please review the material to determine if it seems appropriate. Would the tensile strength of the epoxy govern the design in this case?

Given this information can you offer any additional guidance regarding whether you feel the attachments seem appropriate for this application?

Response

Date: 10-25-2012

Tom:

See my comments below!

Ron, I wanted to take some time to respond to your questions.

Q: Is sufficient clearance available to attach the post?

A: KDOT has been using this detail for our low fill culvert attachments for several years and have no reports of any issues attaching the post to the plate.

**The CAD detail appears to depict the bolt head touching the front flange in Section A-A. In this configuration, the bolt head would be positioned on the front fillet weld, thus making it difficult to turn the head and fit a socket wrench. Maybe the post is scaled to an incorrect size in Section A-A? Thecenterline of slotted holes are 1.75 in. away from right side of plate, while the front flange is about 3 in. away from right side of plate (without considering 3/8-in. weld. The bolt could be shifted inward per the use of slotted holes. As such, the head could be positioned even farther inward.

See response to third question.

****I will leave the weld issue to your bridge and structural group noting the desire/need to develop the full structural capacity of the post. Various weld details can be used to make this connection, whether noted as bevel welds, full/partial penetration welds, fillet welds, etc. Certainly, the fillet welds that I have noted**

****With regard to clearances, I trust that your group will ensure that adequate clearance is provided and that the contractors will let you know if that is not the case.**

Q: Are washers used on the top and bottom surfaces?
A: The washers are only located on the bottom surface.
**Okay.

Q: Is the 3/8" single-pass fillet weld applied to both flanges and the web of the post? The web/flange thicknesses are thinner than the weld size.
A: From our standard drawing it appears the weld is only applied to one side of the web. The web and flange thicknesses are 3/16" while the weld is 3/8".
++The 3/8-in. fillet weld is shown all the way around (i.e., both sides of web and both sides of each flange). As such, the toe of the fillet weld on the outside of the traffic-side flange would be 2.625 in. away from the right side of the plate. This 3/8-in. weld size is rather large for a single pass weld per side when considering the flange/web thicknesses. Industry would not likely want to fabricate it in a single pass.**We have worked with three-pass 5/16-in. fillet welds on the traffic-side flange (both sides) and ¼;-in. fillet welds on the web (both sides) and back side flange (both sides).**
The weld we have been discussing is actually and 3/8" x 3/8" beveled weld (from a construction practices perspective I'm not sure if that makes a difference). The weld is intended to be placed all the way around the outside of the flanges and web (as you indicated is currently shown on the drawing). If the ¾" diameter hex bolt is placed in the slotted hole closest to the weld the bolt head does overlap the weld location. However, if the hex bolt is placed in the slotted hole farthest from the weld there is approximately 5/8" b/t the outer most edge of the hex head and the base of the weld. This should allow enough room for construction. (Please see attached detail for clarification, it appears the previous drawing may not have been shown to scale.) Given this information would it still be appropriate to specify fillet weld sizes and locations you wrote in blue above in lieu of the 3/8" bevel?
****If used, fillet welds would be provided on each side of the flange.**

Q: Where did the 8,000 lbs of tension for alternative anchors come from?
A: The 8,000 lbs for alternative epoxy anchored bolts is related to accomodating pull out strengths for various epoxy manufacturers. Essentially KDOT would not be excluding/specifying a specific epoxy manufacturer as long as they meet the minimum tensile load requirment of 8,000 lbs. It's my understanding, per information from Rod Lacy, 8,000 lbs was selected due to the pull-out force the bolts would experience during a TL-3 barrier impact.
++A ¾-in. diameter ASTM A307 anchor through-bolt has an ultimate tensile strength of about 20 kips without applying reduction factors. Previously, I missed the fact that the alternative 7/8-in. diameter threaded rods conform to AL 39 material, which are to be epoxied into the slab. I am unfamiliar with this material. Can you elaborate on the steel grade so comparisons can be made to the two capacities and later to determine whether the epoxy rating is sufficient? Thanks!
The A139 is a typo. It's supposed to be A 193 threaded rod per TTI's report. I apologize for the confusion. Attached is information related to the Epoxy used in the TTI report for low fill culvert attachments. Please review the material to determine if it seems appropriate. Would the tensile strength of the epoxy govern the design in this case?
****Yes, MwRSF is very familiar with the alloy steel specification of ASTM A193 B7, which is often utilized for threaded steel rods as anchors in roadside safety applications. Next, our structural engineering staff reviewed the recent TTI R&D report and your alternative post-plate option when through-bolts cannot be placed, such as over interior/exterior wall supports. There feedback is provided below:**

^^^
It's unclear whether TTI used A 193 Gr. B7 bolts or ISO 898 Class 5.8 bolts. The drawings and the text state 2 different types. Either way, the epoxy would be the weak link in the anchorage, not the steel rods.

METHOD 1:
Calculation only approach

1. Assume the post transfers its full plastic moment into anchorage. This magnitude will depend on the post material, e.g., A36 (36 ksi) or A992 (50 ksi).
2. Calculate tensile force in front anchors by dividing by moment arm in the anchorage system. I would use distance from front bolts to back of the post (happens to be the same for both post types), but it could also go to back of the plate or back row of bolts.
3. Design epoxy anchorage to satisfy force requirement calculated in step 2. Epoxy anchorage design should follow ACI-318-11 procedures or the manufacturers guidelines. Design calculations MUST consider epoxy strength, embedment depth, spacing, and installation methods.
4. Using these methods, the required tensile force is **30 kips** for an A36 post and **41 kips** for an A992 post.

METHOD 2:
Compare to previous testing of epoxy anchorages:
Using the epoxy anchorage procedure described in ACI and the Hilti technical guide, the bond capacities are:
37 kips (full-scale test passed) = TTI's anchorage with 7/8" rods spaced @ 9 in. and embedded 6 inches
32 kips (BOGIE TEST FAILED) = MwRSF bogie tests with new anchors, 1" rods, 5 in. spacing, embedded 6 inches
46 kips (Bogie test passed) = MwRSF bogie tests with new anchors, 1" rods, 5 in. spacing, embedded 8 inches
****No reduction factors (or dynamic increase factors) were included in either of these calculations.
****It is uncertain what grade posts were used by TTI, but MwRSF ran recent bogie tests with 50 ksi posts.

From the test comparison, we would recommend any anchorage system with an ultimate tensile capacity above 37 kips (unfactored). Again, the design calculations MUST consider epoxy strength, embedment depth, spacing, and installation methods.
^^^

****Thus, the 8,000-lb safe working tension load for the chemical-adhesive could be replaced to state to use a minimum ultimate bond strength of 37,000 lbs. The Hilti HIT-RE-500 epoxy system chart appears to provide approximately a 4:1 ratio between ultimate and allowable bond/concrete capacity. I assume that your 8000-lb safe working load may have considered allowable bond strength with a reduction factor but not sure. If I apply a 4X magnifier to your 8000-lb load, I potentially achieve 32,000 lbs if factor correct. This 32,000 lbs is close to 37,000 lbs. If no reduction factor used in your note, then the safe working load would need to be increased slightly so that the 4X multiplier would exceed 37,000 lbs. Your colleagues would know how your 8,000-lb safe working load was obtained. I recall Scott King or Rod Lacy asking about epoxy anchors over the last several years. Can you verify this number and what it included? Otherwise, I think that we are close to stating if your alternative is acceptable.**

Ron

Response

Date: 10-30-2012

Attached is a PDF version of the updated low-fill culvert standard drawing. I spoke with Scott and we adjusted the plate sizes, weld types/locations and the epoxy ultimate bond strength. Please review the attached PDF and let me know if you have any comments as soon as you are able.

Thanks,
Tom Rhoads

Response

Date: 10-31-2012

My comments are provided below!

(1) Fillet weld symbols are to be shown with triangles pointing to the right. In Section A-A, they need to be reversed. The weld size is then to be shown on the left side of the triangles where an extended horizontal line is provided before the arrow angles up or down.

(2) The MGS posts are attached to the culvert with half-post spacing versus full-post spacing for the MwRSF design. If you desire to use full-post spacing, then you would use the TTI design in its entirety – through bolts when allowed and epoxy anchors over vertical walls.

(3) The MwRSF design used 1-in. diameter vertical bolts to anchor the post/plate assemblies.

(4) The Post Details side view does not seem to be drawn to scale and match what was depicted in Section A-A.

(5) The anchor specification for the TTI system is still incorrect. ASTM A193 Grade B7 rods were used. Recall that you noted the typo of "A139".

(6) The TTI anchors are threaded rods which are epoxied into concrete near vertical wall locations or which could be through-bolted as well with a lower bearing plate. Your Alternate Post detail should depict threaded rods with nuts and washers on the top plate. It appears that you use a hex bolt which cannot be removed after it is epoxied into slab.

(7) I need to speak to Scott/Bob on the tensile specification/bond strength for the TTI anchors.

Response

Date: 11-06-2012

I think I may have sent you an older version of the standard drawing in my previous e-mail. Attached is the updated copy I meant to send. I made a few comments/had a few questions which are listed on the drawing. Please share any thoughts you may have and I appreciate the time you're taking to assist me with developing this standard drawing. I noticed looking at the drawing the threaded rods seem to be a little large in the diagram. I'll adjust the scale for the next version of the drawing.

Thank you,
Tom

Attachment: https://mwrsf-qa.unl.edu/attachments/5c7ac75ca22dbae5d7a7cf6f14e4d8e5.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/2e522e8323d6f97e7518c95926d084a4.pdf

Response

Date: 11-12-2012

Thank you for sending the revised CAD detail. My general comments are provided below:

(1) At this time, you are showing two different bolting patterns for the attachment assembly of the post/base plate system in Detail A (top center to right). I am not sure how you will fit the washer plate with the anchor spacing of the base plate. Should the bolt spacing be 7" versus 5"?

(2) MwRSF utilized a half-post spacing with its post/base plate system, while TTI utilized a full-post spacing with its post/base plate system. TTI had a chemical-adhesive option for their design. Recently, MwRSF utilized bogie testing to develop a chemical-adhesive option for its design. For now, it would be recommended that you utilize the specific anchor option that pertains to a specific post/base plate configuration; since, individual testing was performed on each specific system which had different plate/post stiffness, prying action under loading, and plastic deformations. I might suggest that you consider using our plate and epoxy anchor detail to be consistent. However, I understand if you desire to show both; since, TTI had a 6-in. embedment depth with a different rod size and grade. It may be confusing to show both options though.

(3) The chemical-adhesive system utilized by MwRSF consisted of 1" diameter threaded rods meeting ASTM A307 Grade A. An 8" embedment depth was used for the epoxy rods. The adhesive specifications are shown in the attached pdf file. Additional specifications for the weld details are contained on pages 1 and 2 of the pdf file. Also note that our CAD details provides a minimum bond strength for alternative epoxies used with the MwRSF configuration.

(4) The detail depicts a minimum 10" lateral offset between the back of the post and the culvert headwall. Crash testing was successfully performed with an 18" offset and unsuccessfully performed with a 1" offset. Later, an analysis of the crash videos/film, post-test barrier damage, & vehicle trajectory guided us to allow a 10" minimum lateral offset for the metric-height W-beam rail. If one were using MGS height of 31", one would expect increased barrier deflections. As such, the 10-minimum recommended lateral offset would likely increase, thus guiding one to likely use 18 in. for now if considering 8-in. deep blockouts.

(5) Years ago and while working with FHWA to seek acceptance, several conversations took place regarding transitions. Dick Powers of FHWA desired a more extensive transitioning, while MwRSF did not believe that extensive transitioning was necessary as the post-soil behavior for guardrail posts reasonable resembled that provided by the culvert-mounted posts. In the crash testing effort, MwRSF utilized six (6) half-post spacings beyond the culvert, including the first span from the culvert to a post in soil. In the absence of further computer simulations or crash testing, it may be reasonable to utilize a similar configuration beyond the culvert structure. However, it should also be noted that we generally have recommended that a rail splice not occur at the same location where half-post spacing begins or ends for MGS applications. Instead, we have recommended that at least one half-post spacing be provided before encountering a post at a splice location. I also believe that there would exist an opportunity to reduce the number of half-post spacings on each end from six to some smaller number based on future research and analysis.

Please let me know if you have additional questions or comments regarding the information provided above.

Response

Date: 11-13-2012

Ron, I've made some adjustments to the details on the draft standard drawing. I think our conversations have been beneficial and we are working our way towards converging on an acceptable design. See my responses to your comments below inred and see an updated version of the draft standard drawing attached to this e-mail. Thanks for the continued help. Please let me know if you have any additional comments or if I've missed or misunderstood anything.

Tom

From: Ronald K. Faller [mailto:rfaller1@unl.edu]
Sent: Monday, November 12, 2012 11:23 AM
To: Thomas Rhoads
Cc:rfaller@unl.edu;rbielenberg2@unl.edu; Scott Rosenbaugh
Subject: RE: MGS Low-Fill Culvert Attachments - additional comments!

Tom:

Thank you for sending the revised CAD detail. My general comments are provided below:

(1) At this time, you are showing two different bolting patterns for the attachment assembly of the post/base plate system in Detail A (top center to right). I am not sure how you will fit the washer plate with the anchor spacing of the base plate. Should the bolt spacing be 7" versus 5"? I believe that was a typo. I agree the spacing should be 7". The detail has been changed.

(2) MwRSF utilized a half-post spacing with its post/base plate system, while TTI utilized a full-post spacing with its post/base plate system. TTI had a chemical-adhesive option for their design. Recently, MwRSF utilized bogie testing to develop a chemical-adhesive option for its design. For now, it would be recommended that you utilize the specific anchor option that pertains to a specific post/base plate configuration; since, individual testing was performed on each specific system which had different plate/post stiffness, prying action under loading, and plastic deformations. I might suggest that you consider using our plate and epoxy anchor detail to be consistent. However, I understand if you desire to show both; since, TTI had a 6-in. embedment depth with a different rod size and grade. It may be confusing to show both options though.The reason for my question relating to the plate sizes was to determine if we could simplify the details of the attachment. I've adjusted the details and notes to match the details of the MwRSF details you had attached to the previous e-mail. I agree providing a single set of details for the attachment helps clarify the intention of the details.

(3) The chemical-adhesive system utilized by MwRSF consisted of 1" diameter threaded rods meeting ASTM A307 Grade A. An 8" embedment depth was used for the epoxy rods. The adhesive specifications are shown in the attached pdf file. Additional specifications for the weld details are contained on pages 1 and 2 of the pdf file. Also note that our CAD details provides a minimum bond strength for alternative epoxies used with the MwRSF configuration.Thanks for the info. I've changed the standard drawing to reflect these details. I'm not concerned with the embedment depth being increased to 8" because the epoxy attachment will only be used when a post is located over a vertical wall so the contractor should have the ability to embed the rod 8".

(4) The detail depicts a minimum 10" lateral offset between the back of the post and the culvert headwall. Crash testing was successfully performed with an 18" offset and unsuccessfully performed with a 1" offset. Later, an analysis of the crash videos/film, post-test barrier damage, & vehicle trajectory guided us to allow a 10" minimum lateral offset for the metric-height W-beam rail. If one were using MGS height of 31", one would expect increased barrier deflections. As such, the 10-minimum recommended lateral offset would likely increase, thus guiding one to likely use 18 in. for now if considering 8-in. deep blockouts.KDOT is recommending the use of the 12" blockouts (unless otherwise noted in our thrie beam details). I increased the min offset to 1'-10" from 10" (which is 4" greater than the 18" since the 12" blockouts are 4" deeper than the 8"). Try saying that five times fast.

(5) Years ago and while working with FHWA to seek acceptance, several conversations took place regarding transitions. Dick Powers of FHWA desired a more extensive transitioning, while MwRSF did not believe that extensive transitioning was necessary as the post-soil behavior for guardrail posts reasonable resembled that provided by the culvert-mounted posts. In the crash testing effort, MwRSF utilized six (6) half-post spacings beyond the culvert, including the first span from the culvert to a post in soil. In the absence of further computer simulations or crash testing, it may be reasonable to utilize a similar configuration beyond the culvert structure. However, it should also be noted that we generally have recommended that a rail splice not occur at the same location where half-post spacing begins or ends for MGS applications. Instead, we have recommended that at least one half-post spacing be provided before encountering a post at a splice location. I also believe that there would exist an opportunity to reduce the number of half-post spacings on each end from six to some smaller number based on future research and analysis.I've revised the detail to show additional half post spacing. Per the guidance related to the splice locations we actually end up having seven (7) half post spacings beyond the limits of the culvert (including the first span from the culvert to a post in soil).

Attachment: https://mwrsf-qa.unl.edu/attachments/c4295d3643ecf1df74f70b335f78190c.pdf

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Response

Date: 11-13-2012

I think that we are close. My comments are below!

Ron

Ronald K. Faller, Ph.D., P.E.
Assistant Director and Research Assistant Professor

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From: Thomas Rhoads [mailto:trhoads@ksdot.org]
Sent: Tuesday, November 13, 2012 11:40 AM
To:rfaller1@unl.edu
Cc:rbielenberg2@unl.edu; Scott Rosenbaugh
Subject: RE: MGS Low-Fill Culvert Attachments - additional comments!

Ron, I've made some adjustments to the details on the draft standard drawing. I think our conversations have been beneficial and we are working our way towards converging on an acceptable design. See my responses to your comments below inred and see an updated version of the draft standard drawing attached to this e-mail. Thanks for the continued help. Please let me know if you have any additional comments or if I've missed or misunderstood anything.

Tom

From: Ronald K. Faller [mailto:rfaller1@unl.edu]
Sent: Monday, November 12, 2012 11:23 AM
To: Thomas Rhoads
Cc:rfaller@unl.edu;rbielenberg2@unl.edu; Scott Rosenbaugh
Subject: RE: MGS Low-Fill Culvert Attachments - additional comments!

Tom:

Thank you for sending the revised CAD detail. My general comments are provided below:

(1) At this time, you are showing two different bolting patterns for the attachment assembly of the post/base plate system in Detail A (top center to right). I am not sure how you will fit the washer plate with the anchor spacing of the base plate. Should the bolt spacing be 7" versus 5"? I believe that was a typo. I agree the spacing should be 7". The detail has been changed.

****Agree.**

(2) MwRSF utilized a half-post spacing with its post/base plate system, while TTI utilized a full-post spacing with its post/base plate system. TTI had a chemical-adhesive option for their design. Recently, MwRSF utilized bogie testing to develop a chemical-adhesive option for its design. For now, it would be recommended that you utilize the specific anchor option that pertains to a specific post/base plate configuration; since, individual testing was performed on each specific system which had different plate/post stiffness, prying action under loading, and plastic deformations. I might suggest that you consider using our plate and epoxy anchor detail to be consistent. However, I understand if you desire to show both; since, TTI had a 6-in. embedment depth with a different rod size and grade. It may be confusing to show both options though.The reason for my question relating to the plate sizes was to determine if we could simplify the details of the attachment. I've adjusted the details and notes to match the details of the MwRSF details you had attached to the previous e-mail. I agree providing a single set of details for the attachment helps clarify the intention of the details.

**Agree with single set of details.

(3) The chemical-adhesive system utilized by MwRSF consisted of 1" diameter threaded rods meeting ASTM A307 Grade A. An 8" embedment depth was used for the epoxy rods. The adhesive specifications are shown in the attached pdf file. Additional specifications for the weld details are contained on pages 1 and 2 of the pdf file. Also note that our CAD details provides a minimum bond strength for alternative epoxies used with the MwRSF configuration.Thanks for the info. I've changed the standard drawing to reflect these details. I'm not concerned with the embedment depth being increased to 8" because the epoxy attachment will only be used when a post is located over a vertical wall so the contractor should have the ability to embed the rod 8".

****Agree.**

(4) The detail depicts a minimum 10" lateral offset between the back of the post and the culvert headwall. Crash testing was successfully performed with an 18" offset and unsuccessfully performed with a 1" offset. Later, an analysis of the crash videos/film, post-test barrier damage, & vehicle trajectory guided us to allow a 10" minimum lateral offset for the metric-height W-beam rail. If one were using MGS height of 31", one would expect increased barrier deflections. As such, the 10-minimum recommended lateral offset would likely increase, thus guiding one to likely use 18 in. for now if considering 8-in. deep blockouts.KDOT is recommending the use of the 12" blockouts (unless otherwise noted in our thrie beam details). I increased the min offset to 1'-10" from 10" (which is 4" greater than the 18" since the 12" blockouts are 4" deeper than the 8"). Try saying that five times fast.

****I believe that you should show an 18" minimum so that you are not required to always provide 22". Road width can be hard to achieve at times. In my opinion, both 8" and 12" blocks could be used here.**

(5) Years ago and while working with FHWA to seek acceptance, several conversations took place regarding transitions. Dick Powers of FHWA desired a more extensive transitioning, while MwRSF did not believe that extensive transitioning was necessary as the post-soil behavior for guardrail posts reasonable resembled that provided by the culvert-mounted posts. In the crash testing effort, MwRSF utilized six (6) half-post spacings beyond the culvert, including the first span from the culvert to a post in soil. In the absence of further computer simulations or crash testing, it may be reasonable to utilize a similar configuration beyond the culvert structure. However, it should also be noted that we generally have recommended that a rail splice not occur at the same location where half-post spacing begins or ends for MGS applications. Instead, we have recommended that at least one half-post spacing be provided before encountering a post at a splice location. I also believe that there would exist an opportunity to reduce the number of half-post spacings on each end from six to some smaller number based on future research and analysis.I've revised the detail to show additional half post spacing. Per the guidance related to the splice locations we actually end up having seven (7) half post spacings beyond the limits of the culvert (including the first span from the culvert to a post in soil).

****In 2003, FHWA was requesting that we provide two half-post spacings beyond what was shown in our original design details and used in the crash testing program, even though our original details with 6 spacings was likely excessive. Regardless, your detail is incrementally closer to what FHWA desired years ago even though we disagreed with excessive use of half-post spacing.**

****Additionally, you should show hole size in lower plate and also denote with specifications when galvanized hardware is used. I assume that you do this somewhere.**

****Finally, I want to reiterate that an entirely different sheet could be prepared for the TTI system, which differs from this design.**

Ron 11-13-2012

Response

Date: 11-14-2012

Ron, I updated the offset behind the post to 1'-6" and called out the dimension of the holes on the lower plate. I'm going to maintain the post spacing beyond the limits of the culvert (as shown on the standard drawing I sent you previously). I understand another alternative could be provided showing the TTI installation details, but in order to keep things simple KDOT has decided to provide only the one alternative at this time. The details on the drawing we've been discussing will be copied to another drawing for parallel installations. Would you like to review the drawings one more time before we submit them to FHWA for approval?

Tom

Response

Date: 11-14-2012

Yes, please send to me your final version. Thanks!

Response

Date: 11-14-2012

Ron, attached are the two PDFs for our low-fill details.

Tom Rhoads

Attachment: https://mwrsf-qa.unl.edu/attachments/c8a5fb3b5d6edb462e53aaf940c1d894.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/c8a5fb3b5d6edb462e53aaf940c1d894.pdf

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Response

Date: 11-15-2012

Here are my final minor comments:

(1) The weld sizes should be depicted below the line instead of above the line.
(2) The 5/16" 3-pass fillet weld could have additional clarification that was identified in our most recent dynamic component testing program (draft report in progress). It is as follows:
**Welding is to be completed using the Gas-Metal Arc Welding (GMAW) process with ER70S-3 welding wire and argon-oxygen or CO2 cover gas.
(3) On note for epoxy rod – spell out washer and remove period after “WSHR.".
(4) I just want to note that alternative culvert lengths may create scenarios where the MGS rail splices occur at different locations than shown in your two details. Recall that our testing program utilized 6 half-post spacing beyond the first or last post on the RC structure. Thus, there may be situations where you only need to provide 6 half-post spacings on each side of the structure. However, we have given guidance that the MGS splice should not fall at the start of the half-post spacing but instead a minimum of one half-post spacing away from full-post spacing. For this original study, no crash testing was performed on the transition region on each side of the culvert.
(5) For your documentation purposes and in 2003, I want to reiterate that FHWA wanted 8 half-post spacings on each end, even though we believed that 6 were conservative as is.

I have no further comments beyond those noted above. Please let me know if you have any other questions or comments.

Replacement Criteria for Broken Strands of Cable

Question

State: WY
Date: 09-27-2012

Our maintenance personnel are asking how many strands of cable can be ruptured before cable replacement becomes necessary. I believe that Trinity has advised that if 3 or fewer strands in a single bundle are ruptured, the cable will still have adequate reserve. I believe the cable is 3 bundles, with 7 strands each (3x7). Please advise.

Response

Date: 09-28-2012
The decision on when to replace damaged 3x7 wire rope (3 strands of 7 wires each) in a cable system should be kept conservative due to the critical function it serves. In the past we have noted research into wire rope damage for rigging applications that suggested that several wires could be damaged or fractured in a strand as long as the fractured wires were relatively far apart. The argument for this approach was that the friction developed by the weave of the cable wires and strands should help develop the fractured wire over significant cable length.

However, this type of recommendation does not apply readily to wire rope used in cable barriers because of the difference in the application. For rigging applications, there is generally a significant factor of safety and the wire rope is not stressed near its ultimate capacity. In addition, the wire rope must have the ability to absorb energy in order to develop higher loads when used in a barrier impact. Due to these requirements, we would recommend replacing wire rope on a cable barrier if there is a single fractured wire in a strand or if there is visible plastic deformation and necking of individual wires in a strand. If wire fracture or necking of one wire in a strand is observed, it is safe to assume that the other wires in the strand were loaded at or near their plastic limit as well. Thus, the remaining ductility, internal energy, and capacity of the strand with the damaged wire is likely very low and subsequent loading of the cable will reach the limit of the strand capacity more quickly due to the reduced internal energy dissipation in the strand.

We would not recommend that you follow the Trinity recommendation of 3 damaged wires in the strand. This would indicate a strand that had very little remaining capacity and we would not consider it fit for service.

Response

Date: 09-28-2012
Thank you for your response. One additional point of information. The fractures don't appear to be caused by yield strain, but by some feature of the impacting vehicle actually cutting strands.

Response

Date: 10-22-2012
end of response

Codecs for Viewing Crash Tests Videos

Question

State: WY
Date: 09-27-2012

I just got a Windows 7 computer at work and have experienced problems viewing older crash test videos, most of which are taken with the high speed cameras. Sometimes the first few seconds will run, then they shut down. After doing some research on the internet, I found that apparently Microsoft has felt that some codecs pose a potential security threat to their operating systems so they furnish less out of the box. I have noticed this on several different computer manufacturers. Do you have codecs that can run these older videos and if so, how do you install them?

Response

Date: 10-23-2012
Attached is a zip file that contains two different files. The first executable file "iv5setup" is usually the one that needs to be installed. If you are still having problems after installing that one, then install the other executable file, "K-Lite_Codec_Pack_640_Full".

Attachment: https://mwrsf-qa.unl.edu/attachments/dfc39ceeb80ff6e1f84f75e714a5b51e.zip

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How much of a 2:1 do we need behind a beam guard post

Question

State: WI
Date: 11-01-2012

There is a project that may have to install beam guard near a body of water. The are asking how far down the slope do they need to carry the 2:1?

I know that the attached drawing show 2.5:1.

Attachment: https://mwrsf-qa.unl.edu/attachments/253137118ac5245d984b54d01f4b950a.pdf

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Response

Date: 11-01-2012
We would recommend that a minimum of 4 ft of the 2:1 slope be maintained prior to switching to a steeper slope. This length should provide sufficient soil at the 2:1 slope to resist post rotation in a manner similar to the continuous 2:1 slope that was tested.

Tube Spec for PCB Tie-Down through Asphalt

Question

State: WI
Date: 09-06-2012

Could you review the questions below.

I don't think that there is an ASTM for the cold drawn DOM steel tube. I think that the information provide for this tube is adequate enough to get the correct steel.

Please see the attached detail for the USH 41 temporary barrier wall on structures with an asphaltic overlay. One comment we have recieved from the regional materials folks is that the mounting hardeware, in particular the steel tube. Is there an ASTM/AASHTO standard for the steel sleeve. We currently have a Cold Drawn DOM Steel Tube (Min 72 ksi Yeild Strength). Does this type of tube relate to an ASTM/AASHTO standard to ensure that the 72 ksi yeild strength is being met?

We also need information for the ASTM standards for the nut.

Attachment: https://mwrsf-qa.unl.edu/attachments/aa0f159570f3e77ffd2c0f7ef1c10795.pdf

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Response

Date: 09-06-2012

The material for that tubing is labeled wrong on the original drawing I sent (sorry). It lists the tubing as having a 72 ksi yield strength. It should have read a 60 ksi yield strength and a 72 ksi tensile strength.

The material used for the tube is Cold Drawn Seamless ASTM 519 tubing and the tube material should be AISI 1026 (UNS G10260) cold drawn steel tubing. Cold Drawn Seamless is made from 1026 (UNS G10260) steel in sizes through 9 ½" OD.

http://www.matweb.com/search/DataSheet.aspx?MatGUID=f3c08781eced413ebd167d9a9d1211f2

Let me know if you need anything else.

Response

Date: 11-05-2012

Allowable exposed offset for PCB to median gate attachment hardware

Question

State: WI
Date: 11-07-2012

WisDOT is planning on installing some median gates with temporary barrier. The manufacturer's drawings indicate that the face of thier gate anchorages and the face of our temporary concrete barrier will not be flush. One manufacture indicated that the difference could be up to 1".

How much of a difference between the face of our barrier and their anchorage would be considered a snag issue?

I believe that there was some crash testing done at MwRSF indicating that 3/16 or 1/4 inch plate caused a vehicle to roll over. Would this be a good rule of thumb with steel?

Shouldn't manufacturers be able to provide gates that don't have snag issues with out doing a crash test or significant amount of engineering?

Thanks

Response

Date: 11-14-2012

I have some comments regarding the use of median barrier gates with TCB segments as well as the snag issue you brought up.

First, you are planning to install a median gate system on a run of TCB's. This type of installation poses some concerns as median gate systems were typically designed and tested with permanent concrete parapets. Thus, you will likely need to anchor both sides of the TCB segments adjacent to the gate in order to provide similar deflection performance as compared to a rigid parapet. Currently, we do not recommend anchoring on the front and back sides of a TCB system. However, we have seen in past testing that pins on the backside of a barrier may cause excess rotation and tipping of the barrier which in turn can produce vehicle instability. Thus, we currently do not recommend pinning on both sides of the PCB when placed in the median except for the transition section which we tested. In addition, anchoring both sides of the TCB system will still allow some level of deflection which will be greater than the rigid parapets the median gates were designed and tested with. Thus, there are concerns that the use of a median gate system may be affected in some manner when used with anchored TCB's.

That said, your original question was regarding the level of vertical asperity that can be present on the face the barriers due to the attachment of the steel gate hardware to the TCB segment. Previous research has shown that vertical asperities can affect vehicle stability in a negative manner.

1. MwRSF conducted testing on an anchored steel temporary barrier section formed by welding together stacked steel H-sections. The H-sections were connected using 3.5"x15"x0.375" vertical steel plates. In the first test of the system, test no. HTB-1, the 2000P vehicle impacted the barrier and rolled. From an analysis of the test

results, MwRSF researchers believe that the rollover was caused by snagging of the pickup truck on the barrier. More specifically, the steel rims and sheet metal snagged on the 3/8" thick vertical straps holding the barriers together, the separation between the barrier joints, and the top of the barrier section. This conclusion was based on the damage to the vehicle's right-side sheet metal and steel wheel rims as well as the right-front fender being pulled down during the test as observed on the high-speed film.

Following this investigation, MwRSF researchers determined that the safety performance

of the H-section temporary barrier tie-down system (Design No. 1) could be significantly improved by reducing the potential for snag. In order to eliminate the snag potential, two modifications were made to the barrier tie-down system. First, the vertical steel straps positioned on the traffic-side face of the barrier were removed and replaced with a longitudinal seam weld. Second, the anchor bolts used in conjunction with the drop-in anchors were changed from ASTM A325 to ASTM A307 grade bolts. The bolt grade reduction was made in order to reduce the load capacity of the tie-down attachments and allow for a slight increase in the deflection of the system. It was believed that allowing slightly higher deflections would potentially reduce any additional vehicle snag on the top of the barrier section and at the joints.

A second test, test no. HTB-2, was performed on the modified system with

a ¾-ton pickup truck and was determined to be acceptable according to the TL-3 safety performance criteria presented in NCHRP Report No. 350.

Based on this research, it would appear that vertical asperities of 3/8" or more can contribute to vehicle instability.

The file 'HTB.zip' (23.5 MB) is available for download at

http://dropbox.unl.edu/uploads/20121128/6cb3d21f9c7e7a96/HTB.zip

for the next 14 days.

It will be removed after Wednesday, November 28, 2012.

2. Previously, MwRSF has provided guidance with respect to the allowable offset for alignment of permanent concrete parapets and temporary concrete barriers.

   With regards to permanent concrete barrier, we would recommend keeping the lateral offset or alignment offset minimized to eliminate snag. Variations of 1" or less would be preferred.

   For the temporary barrier installation shown in your photo, we would prefer that the alignment gap be 1" or less, but we believe that gaps as large as 2" are likely permissible. The rationale behind the larger alignment gap allowance is that temporary barrier segments will move when impacted and cause changes in the alignment gap as the impacting vehicle reaches the barrier joint. Thus, a joint that has a given initial alignment will move change alignment as the barrier is impacted. This allows for more tolerance for the temporary barrier gap. Alignments gaps larger than 2" would indicate problems with the temporary barrier joint and would require investigation.

   Keep in mind that these gaps were specific to concrete barrier overlaps where the concrete would be expected to fracture and give when snagged.

3. TTI conducted research in NCHRP 554 regarding aesthetic barrier design and the size of vertical asperities allowable for concrete barriers. This research found a range of performance for vertical asperities dependent on

the angle, depth, and the width between asperities. Crash testing conducted as part of this project found that vertical concrete ridges as deep as ½" could result in failure. Further simulation analysis found that vertical steps of ¼" were acceptable.

In addition to the above studies, there is some concern that gaps between the sheet metal of the gate system and the concrete barrier could be opened further during impact and increase vehicle snag as well as compromise the structure of the gate connection. As such, we would recommend keeping the gap to a minimum. This should be achievable through using steel plates to bridge from the exposed edges of the gate hardware until it is flush with the surface of the PCB. The thickness of these plates should likely be limited to ¼" based on the issues with 3/8" plate in the HTB testing and the results of NCHRP 554.

Retrofitting brush curb

Question

State: WI
Date: 11-16-2012

I was investigating concrete retrofits for brush curb. I was able to find in the RDG Iowa's concrete block retro fit.

Are there other concrete retrofit designs?

The Iowa detail in the RDG only shows a mid barrier cross section. Does the end sections of the Iowa design have more reinforcement?

Response

Date: 01-03-2013

I was unable to find any other tested designs for concrete barrier retrofits. There are other designs for retrofitting bridge rails, but they include steel and/or aluminum beam sections. Additionally, there are other designs out there that have been / are being used as concrete barrier retrofits, but they are untested.

You are correct in thinking that the end sections of the barrier have to be strengthened due to the lack of continuity. The original crash test report shows a 7 ft long end section that includes a thicker cross-section as well as additional reinforcement. Please refer to MwRSF report no. TRP-03-19-90. I believe the full drawing set for the barrier is available on Task Force 13's online Bridge Rail Guide also.

CRT posts behind a curb

Question

State: IA
Date: 11-20-2012

We have an MGS post conflict at an existing curb intake. As you have recommended, we will be installing the MGS long-span system to skip one post at the intake, placing three CRT posts upstream and downstream of the unsupported section.

Since the CRT posts will be installed behind a curb, must we adjust the location of the weakening holes so that the center of the top hole is flush with the ground line? Or can we still use the CRT posts that were utilized in the long-span crash tests (top hole centered 32" from top of post)?

Response

Date: 11-21-2012
In this situation, we would recommend that both the CRT holes be adjusted upward to account for the additional soil behind the curb. Thus, if a 6" curb was used in the installation, we would recommend that both the upper and lower holes in the CRT post be shifted 6" up on the post. Post embedment can remain the same (in this case slightly increased from a standard installation due to the curb).

The rational for shifting the holes is related to the function of the holes in the post.The holes reduce the cross-section of the post and act to weaken the post in bending and in shear. However, if the hole that is typically at groundline is buried, then the cross-section of the post is not weakened for shear loads across the base of the post at groundline, and the post may not fracture as designed. The bending section would still be reduced even with the hole being placed below groundline. Because the CRT is used with a curb in this case, the vehicle bumper may impact the post lower than a typical installation. Thus, the shear fracture of the post may become more critical. Thus, we recommend relocating the holes in the post as noted above.

MGS Long Span - TRP-03-187-07

Question

State: WY
Date: 12-12-2012

Two questions:
Question1 1 - Slope Requirements Question:
The first test conducted under this study was in a configuration shown on page 15 of the report with the headwall of the box culvert 1 foot behind the back of guardrail post line. No specific slope criteria was specified in this drawing.
The second test conducted was with the headwall of the box culvert flush with the back of post line. In this drawing on page 65, it shows a 3:1 slope 2 feet behind the back of post line for a distance of six post spaces upstream and downstream of the installation.
Was the 3:1 slope a requirement for the installation to work properly, or would any slope at the slope break line (located 2 ft. behind the back of post line) be acceptable? e.g. 2:1 or steeper, and/or flatter than a 3:1?
There is no mention of this in the summary, conclusions and recommendations, so perhaps this was only to document the installation as it was tested?
Question 2 - CRT Details:
In preparing our standard plans for the MGS, we noticed Road Systems is has created a "universal CRT post" as shown in the attachment. This way they can use the same post for w-beam SKT's as well as MGS SKT's.
The hole locations are midway between what is used for conventional w-beam an that specified for the MGS as detailed in Midwest's report (and also attached). Is it O.K. to use this "universal" CRT post for the long span as well? Every chance we can get to reduce the total number of stockpiled components really helps our maintenance folks as well as fabricators.

Attachment: https://mwrsf-qa.unl.edu/attachments/59659e5f006239b78c88568b251549d3.docx

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Response

Date: 12-13-2012

Question 1

With regards to the slope used in the second MGS long span guardrail test, a 3:1 slope was started 610 mm (24.0 in.) behind the back face of the guardrail posts of the MGS long-span, and the wingwalls were modified to match the soil slope. The choice of the slope profile was based on choosing the flattest slope of the typical culvert installations submitted by the sponsoring states. The choice of the flattest slope maximized the potential for vehicle interaction with the wingwalls of the culvert during the impact event.

Based on this, it is reasonable to assume that steeper slopes would be acceptable as long as sufficient offset was provide to the slope point in order to provide proper soil resistive forces for the posts. We have recommended a 2' offset distance in the past from the back of the post to the slope break point. For areas where you cannot get the required 2' offset we have provided additional guidance on post lengths. See the link below.

http://mwrsf-qa.unl.edu/view.php?id=322

At this time, we do not have data to determine the effect of flatter slopes with the long span. Flatter slopes than

3:1 may be acceptable, but there is concern that vehicle snag on the downstream wingwall would increase significantly as the slope flattened. Thus, at this time it may not be possible to recommend flatter slopes with the MGS long span without further study and analysis.

Question 2

We would not recommend switching to the RSI CRT post. The RSI design would place the CRT holes approximately 2" higher that the CRT posts used in the design and testing of the MGS long span. This would put the top CRT hole slightly above groundline. There are a couple of concerns with respect to the change in the location of the CRT hole. First, the change may adversely affect the loading and fracture of the hole and change the breakaway forces and energies of the post. Second , the higher hole would increase the amount of post sticking above groundline after then post fractures and potentially increase vehicle interaction/tripping/snag on the fractured lower post section. The effect of these concerns on the performance of the long span is difficult to quantify. Thus, we cannot recommend changing the posts.

Tie Down Straps on thin HMA Pavements

Question

State: MO
Date: 12-14-2012

Like most states, Missouri has been faced with an increasing need to tie down barriers on Portland cement concrete pavement (PCCP) that has been overlain with hot mix asphalt (HMA) pavement. The MwRSF has developed methods to tie to PCCP, or HMA on base, but a solution to the composite pavement still eludes the industry.

An upcoming project in the St. Louis area will require that freeway traffic on an 8-lane freeway to be run head to head (separated by Type F concrete barrier) in 6 of the lanes. This particular segment of highway has a 3-3/4 in. overlay of HMA.

Questions:

1. Would it be a reasonable variance of the tested and approved tie-down strap method to remove the asphalt pavement at each joint and pin directly to concrete by way of an elongated strap? (see attached diagram)

2. If this is a possibility, what would be considered a practical limit as to the thickness of asphalt layer for which this anchorage would be feasible?

3. A similar proposal involves milling a 2 ft. wide trench the entire length of the barrier run and pinning directly to the concrete by way of the conventional length strap. Would the Type F barrier still be functional if its effective height is decreased by 3-3/4 in.?

Attachment: https://mwrsf-qa.unl.edu/attachments/fab63f23199066f51e802bbdae64a334.jpg

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Response

Date: 12-17-2012
1. There are several unknowns with this type of installation of the tie-down strap. using the straps with a mill out as you have shown would prevent loading of the anchors through a layer of asphalt and provide similar anchor capacity to the tested design. However, doing so would require lengthening the sides of the strap. This change in geometry would likely affect the force-deflection and energy absorption of the strap to some degree and modify the loading of the anchors. The effect of these changes cannot be quantified without further study, but could potentially increase barrier deflections and the anchor capacity as compared to he tested system. Thus, while the potential exists for this type of modification to work, we cannot accurately quantify the impact performance without further study.

2. As noted above, changing the length/geometry of the strap would affect the force-deflection and energy absorption of the strap. The deeper the asphalt thickness, more prominent those effects would be and the greater the expected change in system performance. Determination of an acceptable asphalt depth limits would require further study.

3. We cannot definitively say that the barrier system will or will not work with the reduced height when anchored, but our experience in testing the F-shape PCB's in anchored configurations leads us to have concerns for vehicle stability at the reduced height. If you look at our testing of the anchored F-shapes with 32" heights, you will note the degree of instability present. Reduction of the height to 28.25" would be significantly lower than previously tested TL-3 F-shape barriers, and would not be recommended.

Response

Date: 12-27-2012

Thank you for the prompt reply to my inquiry. I have shared your analysis with the project team and have the following questions by way of follow up.

1. Your response mentioned the change in geometry of the strap leading to an (as yet) incalculable effect on the force-deflection and energy absorption of the strap as well as the loading of the anchors. Would a thicker strap, perhaps 3/8 to 1/2 in., be sufficient to allay that effect?

2. Would my entire original inquiry perform sufficiently under TL-2 conditions?

Response

Date: 01-02-2013

Responses in Red.

1. Your response mentioned the change in geometry of the strap leading to an (as yet) incalculable effect on the force-deflection and energy absorption of the strap as well as the loading of the anchors. Would a thicker strap, perhaps 3/8 to 1/2 in., be sufficient to allay that effect?

   Increasing the strap thickness is not a viable option. During the development of the strap tie-down, we investigated various strap thicknesses. It was observed that thicker straps tended to pry the anchors out of the concrete with very little energy absorption and made the system less effective.

2. Would my entire original inquiry perform sufficiently under TL-2 conditions?

There is increased potential for the proposed modifications working under TL-2 impact conditions. We believe that the first option to remove the asphalt pavement at each joint and pin directly to concrete by way of an elongated strap has a very good chance of performing well under TL-2. With the lower impact energy of the TL-2 impact, the effect of modifying the strap geometry becomes much less critical.

We would not be as confident in the second option to mill underneath the barriers and effectively lower the height 3 3/4". There has been very little research done on reduced height barriers with sloped faces for TL-2. The majority of the reduced height sections for TL-2 have been vertical face designs. As such, we would be more wary of this option, especially when considering high CG vehicles and the potential for barrier climb. That is not to say that it cannot work, but that our confidence is lower than the first option.
