Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

07-01-2016 to 10-01-2016

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

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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

Bolts in asymmetrical transition section

Question

State: IA
Date: 07-12-2016

When testing, does MwRSF use two bolts for the center post of the asymmetrical section or just one? I would default to assuming two or there wouldn't be a need to have the bottom hole, but I've seen other states with just one in the top slot and I can't find a definitive answer.

I've seen some drawings where there will be a nail near the bottom hole to prevent rotation with a wood blockout and wood post but since MwRSF typically tests with steel posts, that wouldn't apply. However, I'm not sure if the nail would be needed as it would be a thrie-beam blockout and the weight distribution may keep the blockout from rotating anyway. Thoughts?

Response

Date: 07-13-2016

All of the MASH testing of the the asymmetrical W-to-thrie beam transition piece was conducted with only the top post bolt installed. This was done partially because standard thrie beam posts will not have a post bolt hole in the correct location for the lower slot in the rail, and to allow for the blockout in that location to rotate and reduce snag. Thus we would recommend using only the single bolt in that location.

We don't have evidence that using both bolts is an issue, but we recommend using only the top bolt to be consistent with the full-scale crash tests.

MGS Long-Span post depth

Question

State: OH
Date: 07-12-2016

We have a location where we are trying to use MGS long-span to its maximum 25' span length. Even at that length, we are unable to embed the posts adjacent to the culvert (posts 3 and 4) more than 2 - 2.5'. Do we have any post options (adding concrete, additional posts, ...) that could be substituted for a fully embedded post?

Thanks!

Attachment: http://mwrsf-qa.unl.edu/attachments/99a1f86bdf929f32fa29943abaceae75.pdf

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Response

Date: 07-13-2016
Currently we do not have reduced CRT post depth options for the MGS long span system. CRT posts are designed to develop forces in the soil that fracture the post at one of two hole locations, one at groundline and a second 15.75" below groundline.

In the long span system, the post functions by supporting the barrier adjacent to the unsupported span and then releasing to prevent barrier pocketing as the vehicle moves from the unsupported span to the standard MGS. The concern would be that reducing the embedment of the first post adjacent to each end of the span could allow increased deflection of the MGS which may affect barrier performance negatively.

Thus, we do not recommend using shorter than tested embedment depths for the CRT posts at this time.

Non-Blocked MGS and Terminals

Question

State: IL
Date: 07-14-2016

The Illinois Department of Transportation is developing a new Highway Standard to implement the non-blocked MGS. We are also updating design guidance in the Bureau of Design and Environment Manual according to the Implementation Guidance section of “SAFETY PERFORMANCE EVALUATION OF THE NONBLOCKED MIDWEST GUARDRAIL SYSTEM (MGS)", TRP-03-262-12. In that guidance we do not find any limitations or guidance on transitions to end treatments (anchors or crashworthy ends.) We do note a requirement for placing at least 25 feet of MGS with blockouts between the asymmetrical transition from thrie beam to w-beam at an approach guardrail transition and the non-blocked MGS. From this, our interpretation is that the non-blocked system does not introduce constraints for connection to an MGS downstream anchor or to a proprietary crashworthy end terminal. However, review of manufacturer literature for various proprietary crashworthy end terminals shows some variance and uncertainty.

From this, it appears that we should consult with the various manufacturers regarding their guidance for connection and/or transition to the non-blocked MGS. Do you think this is the appropriate course, and what are your comments and suggestions?

Response

Date: 07-14-2016

We cannot make recommendations regarding the omission of blockouts within proprietary terminal systems. Typically these systems have used blockouts in there as-tested configuration, so it is difficult to anticipate their performance without them in place and the performance may vary between different types of terminals. Thus, we would recommend keeping the as-tested blockout configuration.

As far as transitioning blockout depth from existing terminals to non-blocked guardrail, we would recommend the following approach. Begin transitioning blockout depth no sooner than 12.5' from the end of the terminal system. At that point, you can transition the blockout depth from 8"or 12" to 0".

With respect to trailing end terminals, we believe we need to be conservative as well. Previous MASH testing we conducted on the trailing end terminal impact 31.25' from the end with the pickup truck and 9.375' from the end with the small car. We believe that the presence of blockouts may affect the results of both of these tests in terms of vehicle stability, vehicle capture, vehicle snag, and/or occupant risk measures. As such, we would recommend not converting to the non-blocked system until 50' from the final anchor post in the trailing end terminal.

Thanks

mounting beam guard on top of box culvert

Question

State: WI
Date: 07-28-2016

We have a contractor that wishes to use 3/4" x 6.25" carbon steel wedge anchors verses epoxy anchors for the details MwRSF developed in TRP-03-114-02 and TRP-03-278-13.

Contactor indicates the following in their submittal

Arbor Green would like to use a mechanical wedge anchor for the mounting of guardrail posts to the top of box culvert between the WB Beltline ramps and Broadway St. Specifically, we would like to use a 3/4"x 6.25" carbon steel wedge anchor. The data sheet I have provided shows pullout strengths well in excess of the plan notes per bolt, and there will be 4 bolts in each plate.

Attachment: http://mwrsf-qa.unl.edu/attachments/23b224d3baf75a7dac55af2c12d83477.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/3457a84322f52f1b34c43c2305ac9318.docx

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Response

Date: 07-28-2016

The attached detail showed an anchorage that was dynamic component tested in report TRP-03-278-13. It consisted of 1" dia. A307 bolts anchored into a minimum of 4,000 psi concrete to a depth of 8" using an epoxy with a minimum bond strength. The dynamic testing found that this configuration was capable of developing the moment capacity of the post and was acceptable for use with the previously crash tested culvert mounted guardrail system.

The original anchorage consisted of 1" dia. A307 bolts that were through bolted into the top of the culvert. In order to be conservative and in lieu of dynamic testing, we have required that any alternative anchor must meet the ultimate capacity of the as-tested design. For the 1" dia. A307 bolts, that would correspond to an ultimate tensile load of 36.4 kips.

The proposed TruBolt wedge anchors do not appear to have sufficient capacity. 3/4"dia. TruBolt wedge anchors have a maximum capacity of 17.7 kips at the length/embedment noted above in 4,000 psi concrete. Even the largest diameter TruBolt wedge anchors only develop 26.5 kips in 4,000 psi concrete. Thus, we are unable to recommend the alternative anchor at this time.

It may be possible that the TruBolt anchor could function acceptably, but some form of dynamic component testing would be needed to verify the performance similar to report TRP-03-278-13.

Attachment: http://mwrsf-qa.unl.edu/attachments/221cbd0d54482cda86c44ee89f82e882.pdf

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Curbs with Transitions and Guardrail

Question

State: IA
Date: 08-04-2016

Somewhat of a follow up question to our conversation regarding curb heights (bottom of Q/A 1079):

We have situations where an existing 6 inch standard curb is adjacent to a lane receiving a 3 inch overlay, ultimately making it a 3 inch curb. We have typically treated curb heights less than 4 inches as a non-issue regardless of their offset to face of rail, but your statement that headwalls (and I'm expanding to curbs here) greater than 2 inches affect vehicle stability brings that into question. What aspect of height is causing the issue; the height itself or the vertical face? To express it in another way, which of the following scenarios should be considered an issue?
1. Two inch or less curb in front of guardrail without a height transition or a transition of less than three feet
2. Two inch or less curb in front of guardrail with a height transition of at least three feet
3. Between two and four inch curb front of guardrail without a height transition or a transition of less than three feet
4. Between two and four inch curb front of guardrail with a height transition of at least three feet
5. Two inch or less obstacle behind face of guardrail but within working width, with a vertical face and without a height transition or a transition of less than three feet
6. Two inch or less obstacle behind face of guardrail but within working width, with a vertical face and with a height transition at least three feet
7. Between two and four inch obstacle behind face of guardrail but within working width, with a vertical face and without a height transition or a transition of less than three feet
8. Between two and four inch obstacle behind face of guardrail but within working width, with a vertical face and with a height transition of at least three feet
9. Four inch obstacle or less behind face of guardrail but within working width, with a height transition of at least three feet and with a sloped face (essentially slope the headwall face and ends to mimic a four inch sloped curb – doesn't exist but just a thought of how we could make them non-hazardous)
10. Other combinations we should consider as issues…

I'm wondering if your response in Q/A 1079 necessitates a change in how we deal with curbs, such that our guidance (subject to change based on above response) might read:
1. Curbs less than two inches in height
a. Can be ignored and do not need a transition
2. Curbs between two inches and a typical four inch sloped curb
a. Require a transition of at least three feet and that transition cannot occur within:
i. Fifty feet upstream of the end terminal,
ii. Any point within the end terminal, or
iii. Within the nested w-beam and asymmetrical transition piece of a barrier transition section
3. Standard six inch curbs
a. Require a transition of at least three feet down to a four inch sloped curb and that transition cannot occur within:
i. Fifty feet upstream of the end terminal,
ii. Any point within the end terminal, or
iii. Within the nested w-beam and asymmetrical transition piece of a barrier transition section

b. Cannot exist within:
i. Fifty feet upstream of the end terminal,
ii. Any point within the end terminal (Iowa flares all end terminals), or
iii. Within the nested w-beam and asymmetrical transition piece of a barrier transition section
iv. Any location where the offset from gutter line to face of rail is more than six inches
4. Headwalls less than two inches in height
a. Can be ignored and do not need a transition
5. Headwalls greater than two inches
a. Cannot exist within guardrail working width

Thanks for working through my interpretation of your intent, and as always, thank you for your assistance.

Response

Date: 11-02-2016

Let me try to answer things in a general sense and then we can work down to specifics.

First, the response in Q/A 1079 was specific to the long span system and not intended to be applied to all guardrail in general. The long span system has significantly larger deflections than typical guardrail and the vehicle tends to traverse the headwall and extend over the drop-off. Thus, the height of the head wall become more critical for this application. Additionally, we don't have data for curbs that first impact guardrail and then impact a curb offset that distance behind the rail. Typically curb studies have focused on placement of the curb at or slightly offset behind the face of the rail or with the curb offset at larger offsets in front of the rail. As such , we limited the headwall height for the long span systems to provide a conservative approach.

With respect to the broader aspects of curb and guardrail installations, we believe that the Roadside Design Guide still likely provides the most appropriate guidance based on our current knowledge. Essentially, the RDG states that for high speed facilities guardrail should not be offset from curbs unless crash testing has shown that it is acceptable. If guardrail offset from curbs is needed, it recommends a 1.5" laydown curb that would have minimal effect on the barrier performance. If does note that W-beam can be used with 6" tall curbs if the rail is installed flush with the curb up to 8m km/h and gives guidance for other sloped curbs for at higher speeds. It also notes exception to these guidelines for crash tested systems like the MGS which was tested with a 6" curb and a 6" offset behind the curb. As noted above, the 2" headwall height was specific to the long span and would not supersede the RDG guidance with respect to guardrail systems in general.

You note transitioning or tapering of the curbs. Specifically when to do it with respect to AGTs and end terminals.

Currently there is no set guidance with respect to curbs and end terminals other than it is not generally recommended due to lack of knowledge and testing of the combination. We currently have a limited study funded by WisDOT to investigate this issue, but it is not yet complete. A previous study by CALTRANS with respect to curbs and inertial sand barrels was conducted in the 60's and it does provide some guidance w/r/t placement of curbs in front of the barrels, but it was done using sedans and my not be as relevant as it originally was.

In terms, of transitions, our recent testing of the MGS stiffness transition with curb (http://mwrsf.unl.edu/researchhub/files/Report295/TRP-03-291-14.pdf) found that a 4" tall sloped curb could be used in the region of the AGT. In order to ensure the safety performance of the MGS stiffness, the 4-in. tall curb should be placed through the entire length of the stiffness transition. Thus, the curb should be extended a minimum of 37.5 ft from the bridge parapet before either being terminated or transitioning to a 6-in. high AASHTO Type B curb. Additionally, it was recommended to utilize a minimum length of 3 ft for any curb shape transitions or terminations (e.g. transitioning from 4-in. curb to no curb).

Hopefully this gets us started down the path of answering your questions. Take a look at what is above and see if it addresses the situations you have below. Then if we need to discuss some specific items, we can go over those together.

Thanks

Expansion Gap Joint Width

Question

State: DE
Date: 08-17-2016

MwRSF was contacted through FHWA for the state of Deleware regarding our thoughts on the allowable gap lengths
for unshielded expansion joints in permanent concrete bridge rails.

Response

Date: 11-02-2016

There are two main concerns with expansion joints in concrete bridge rails. The first is the lack of continuity in the rail and the need for increased rail capacity adjacent to the opening to maintain the rail strength. For this discussion, I will assume that the gaps we are discussing are designed with appropriately reinforced rail end sections to address the lack of rail continuity. The second concern is vehicle snag on the downstream end of the gap as the vehicle is redirected. Snag in this area can potentially result in increased vehicle deceleration, vehicle damage, and instability.

Determination of a maximum unshielded gap can be looked at in several ways. First we can look at available test data. MwRSF tested a Nebraska open concrete bridge rail with a 4.5" gap under the PL-2 criteria. We evaluated this barrier at the gap with a 2,449 kg pickup truck at 61 mph and 20 degrees and a 8,165 kg SUT at 51.9 mph and 16.8 degrees. Both of these tests were successful, but snag was evident in both tests. Additionally, the pickup truck test was conducted at a lower angle than NCHRP 350 or MASH TL-3 and TL-4. More recently, TTI tested the Texas T224 bridge rail to MASH TL-4. As part of that evaluation, they successfully tested the 10000S vehicle across a 2" wide expansion gap. The passenger vehicle tests were not conducted across the expansion gap.

http://mwrsf.unl.edu/researchhub/files/Report262/TRP-03-51-95.pdf

I don't have the TTI report. It may not yet be published.

Similarly, temporary barrier designs in free-standing and anchored configurations have had gaps as large as 4" that have been successfully traversed by vehicle in TL-3 testing. This would suggest that the potential for larger gaps may exist. For example, the Midwest States F-shape PCB was tested to NCHRP 350 TL-3 anchored to a bridge deck with a 4" barrier gap. This test did have some snag across the PCB joint, but the vehicle was safely redirected. The snag may have even been exaggerated in this type of system as compared to a rigid bridge rail as the upstream barrier could translate more laterally prior the vehicle traversing the gap, thus exposing the face of the adjacent barrier at the downstream end of the gap even more.

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

http://mwrsf.unl.edu/researchhub/files/Report223/TRP-03-134-03.pdf

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

http://mwrsf.unl.edu/researchhub/files/Report151/TRP-03-173-06.pdf

When we have considered this in the past, we have looked at the potential vehicle overlap on the downstream gap edge. We know from previous research that vertical asperities can cause problems with barrier performance. NCHRP 554 concluded that vertical asperities of ¼" or less were recommended to maintain vehicle stability and safe redirection. Previous testing conducted at MwRSF on a portable steel barrier for IaDOT noted similar concerns when a 3/8" thick vertical plate on the face of the portable barrier was sufficient to snag a pickup truck rim and cause the vehicle to roll. However, these two examples may not be completely analogous as NCHRP 554 dealt with aesthetic bridge rail designs and the portable steel barrier had a plate that extended from the face of the barrier

http://mwrsf.unl.edu/researchhub/files/Report245/TRP-03-120-03.pdf

http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_554.pdf

In previous discussions with the state DOT's, the issue of the allowable level of lateral misalignment of the barriers has come up. With regards to permanent concrete barrier, we recommended keeping the lateral offset or alignment offset minimized to eliminate snag. Variations of 1" or less would be preferred. We also recommended that the edges of the gap be chamfered to reduce the severity of any vehicle snag on the gap. The 1" offset was larger than the ¼" or 3/8" noted above based on the fact that these gaps were specific to concrete barrier overlaps where the concrete would be expected to fracture and give when snagged. If we use a similar rational and apply it to the expansion joint problem, we can use a gap length “L" and a 25 degree impact angle or intrusion angle of the impacting vehicle to estimate snag. This would make the snag or overlap on the downstream end of the gap approximately equal to L*sin(25). This results in the following snag for various gap lengths.

Gap Length “L" (in.)	Estimated Snag (in.)
1	0.42
2	0.85
3	1.27

4	1.67

This would seem to suggest that a 2" gap will limit snag to less than 1".

As you can see, there may not be a perfectly defined answer. Previous anchored PCB testing and the PL-2 open concrete rail tests suggest that 4" gaps may be permissible. However, a more conservative approach may be to limit the gaps to 2". In both cases, we recommend chamfering the edges to limit the snag severity.

Thanks

Strong Post Deck Mounted Guardrail

Question

State: IN
Date: 08-23-2016

We have had a lot of question about guardrail over large culverts adjacent driveways or roadways. With the close proximity of the roadways, our standard nesting system is too long. We are in the process of possibly moving toward the MGS system but are still showing strong post guardrail with an 8" blockout in our standard drawings. Could you recommend a good connection system for culvert mounted strong post guardrail? Would it still be an option to use the SRG05 Culvert Mounted W-Beam Guardrail in the Guide to Standardized Highway Barrier Hardware, see attached.

I also have one other question, a lot of designers have proposed to just cross these large culverts with our modified post that we use in our nested guardrail system, see attached standard drawing sheets. We believe these modified post should only be used as part of a system, modified posts (E 601-NWGA-03, attached) should be proceeded and followed by 4-6'-0" length steel posts w/routed wood blockouts (E 601-NWGA-01 and -02, attached). They should not stand alone. Do you know of any mounted post systems that are similar to our nested guardrail modified post that can stand along over a bridge?

Attachment: http://mwrsf-qa.unl.edu/attachments/59c7d8a68fae9813bedab822ac88ebf3.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/22a2bcbb93192dbd6cf10c395c397d5f.pdf

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Response

Date: 08-23-2016

We have looked into similar questions previously for several states. I have placed links below to simialr questions that have additional details that you may wish to refer too.

http://mwrsf-qa.unl.edu/view.php?id=322
http://mwrsf-qa.unl.edu/view.php?id=799

It appears from the question you submitted that you cannot use a long span guardrail and need to attach posts to the culvert. MwRSF previously developed system for this purpose that was evaluated to NCHRP Report 350. The system used metric height W-beam with 1/2 post spacing and special posts that were mounted to the culvert. Subsequent research on that system also evaluated an epoxy adhesive anchorage for the posts. This system has not been tested to MASH, nor has it been evaluated with the MGS. However, it is believed to be likely that the system would perform acceptably under MASH. As such, Kansas DOT has adopted the system for use with 31" guardrail. Details are attached. The reports on the MwRSF research and testing of this system are located below.

http://mwrsf.unl.edu/researchhub/files/Report144/TRP-03-114-02.pdf
http://mwrsf.unl.edu/researchhub/files/Report280/TRP-03-278-13.pdf

TTI recently (2012) tested a similar design with a 31 inch high w-beam guardrail and posts at a standard spacing. This design uses a slightly different base plate and uses full post spacing. This design passed MASH TL-3, but partial tearing of the W-beam guardrail was noted. They also tested a similar system under NCHRP Report No. 350 that was installed at 27".

http://www.roadsidepooledfund.org/files/2012/02/405160-23-2-box-culvert-rev2.pdf
http://www.roadsidepooledfund.org/files/2011/03/405160-5-1-box-culvert.pdf

Thus, several options exist for attachment for strong post guardrail to culverts. Please review the attached information and let me know if you have further questions.

As for the modified detail you attached, I have not seen any similar system tested or evaluated. There are some concerns that the use of the BCT post in a foundation tube would limit the energy absorption of the posts in the system. This could potentially increase deflection and rail loads to undesirable levels. Thus, we would recommend using one of the tested systems noted above.

Attachment: http://mwrsf-qa.unl.edu/attachments/e1b1bbea41823a4bcdb2b0725e7604cd.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/a1932f3b77f3973bc4d2f0d9d540fdc4.pdf

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MGS Blockouts and Slopes

Question

State: VA
Date: 09-01-2016

We are beginning our development of the VDOT MGS standard drawings. There is a little confusion on the exact dimensions of the 12" blockouts. Is there an allowable tolerance for the depth dimension? Some of the wood samples are routed and are 11" deep. The composite sample is 11 ½" deep.

In addition, what is the standard dimension from the back of the post to the hinge point? We have seen the research for the post being installed at the hinge point but want to establish our standard layout for our geometrics.

Response

Date: 09-02-2016

With respect to the MGS blockouts, we don't typically provide tolerances for the states unless it is a safety performance issue. For the MGS, the nominal blockout depth is 12". The blockouts on the MGS serve two purposes. First, they space the rail away from the post which reduces vehicle snag on the post. Second, the blockout helps maintain the rail height as the post rotates and promotes improved vehicle capture and stability.

We conducted all of our MGS with non-routed blockouts. However, we understand that several states prefer routed blocks and we do not see any issue with the slight loss of depth for the routed system.

Similarly, there would be no issue with using a 11 ½" deep composite blockout. The MGS has been successfully tested with 12" blocks, 8" blocks, and non-blocked configurations. Thus, small variation in the blockout depth should not be an issue.

I assume when you refer to the hinge point below, you mean the distance from the slope break point. That answer is somewhat dependent on the slope being shielded. We have may many recommendations regarding this issue over the years. Our standard answer has been that we believe that a 2 ft offset from the back of the post to the slope break point should provide similar performance for the MGS as when it is installed on level terrain. We have tested other versions of the MGS at the slope break point. One version had 9' long posts at the slope break point of a 2:1 slope. TTI tested a similar system with 8' long posts. Thus, another option has been to install the MGS with 8' or 9' posts at the slope break point. This provides similar levels of dynamic deflection as the level terrain system.

More recently, we did test the MGS at the slope break point of a 2:1 slope with standard 6' long posts. This system met MASH but had significantly higher dynamic deflections than the standard MGS.

If you have specific slopes you are shielding, we may be able to provide a better answer.

We currently have a research project in the pooled fund to develop guidelines for the placement of the MGS adjacent to slopes. The background research is complete, but I need to finish the guidance. I have attached the proposal right up for you to review. It also contains some of our previous slope guidance to reference.

Let me know if you have further questions.

Attachment: http://mwrsf-qa.unl.edu/attachments/63f8b038ec5fc4a9d1fbea0f566ca745.pdf

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Response

Date: 09-03-2016

Thanks for your assistance. I have a few other questions..

Has a double sided (median) version of the MGS been developed and tested?

How about a weak post version of the same?

Response

Date: 09-04-2016

The MGS has been approved as a median barrier system. MwRSF reviewed previous testing of 31" tall W-beam median barriers and received an approval letter for the system without full-scale testing in 2010. See attached.

Since then, TTI tested a MGS median barrier with 8" blockouts that met MASH as well.
http://tti.tamu.edu/documents/9-1002-12-8.pdf .

Weak post W-beam guardrail (G2) was tested to MASH under NCHRP 22-14(3) using the 2270P vehicle only. It was successful. To the best of my knowledge, a median version has not been evaluated. However, there would seem to be potential for such a system to work in terms of the capacity of the system to contain the pickup truck. One concern would be the override of the S3x5.7 posts by the 1100C vehicle. We have observed floorpan cutting in cable barrier systems when the small car vehicle overrides these posts. Similar concerns would exist for the G2 system or a median version of it. The 1100C test was not conducted during the TTI research. The G2 system was tested under NCHRP Report 350 and puncture of the gas tank of the 820C vehicle was noted. However, the test was still a pass.

Let me know if you need anything else.

Attachment: http://mwrsf-qa.unl.edu/attachments/347121b505d6a9a32ec0d383aecc83ae.pdf

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Curb Height Question

Question

State: IL
Date: 09-07-2016

I work in the District One offices of the Illinois Department of Transportation.

District One is comprised of the 6 County Chicagoland Region.

The reason for my email is that the suburban bus company for the region, Pace, is looking for permission to install 12-inch high curb along approximately 20 routes in the Chicagoland that are under the jurisdiction of the Department.

Having read through their literature it is clear that the 12-inch high curb has benefits to the transit provider however it is unclear to me on its impacts to vehicular roadside safety.

Is 12-inch high curb TL1 crash worthy?

Will it increase the likelihood of vehicles flipping or vaulting further when they leave the roadway?

I look forward to hearing back from you.

Response

Date: 09-08-2016

In general, moderate to tall curbs are not always desired along moderate to high speed roads due to some safety concerns. Of course, other benefits are provided by curbs, including pavement edge stiffening, road edge delineation, hydraulic drainage, etc. When curbs are struck by non-tracking vehicles, taller curbs can accentuate tendencies for increased roll angles, and possible rollover. Eventually and at some increased height, vehicle rollover tendencies go down. However, a tall curb would then be stopping and/or redirecting a vehicle if non-tracking, resulting in higher vehicle decelerations and changes in velocity, also possibly rapid yaw. At any rate, we do not currently perform crash tests under non-tracking impacts.

A 12-in. tall curb may not provide redirective capability for tracking impacts with higher center-of-mass vehicles. In a recent study performed in the early 1990s, we found that 12- to 14-in. tall timber curbs could only redirect a 2000P pickup truck at 15 mph and 15 degrees. This condition is lower than provided by Test Level 1 (31 mph and 25 degrees), which we deemed sub-TL-1 at the time. Later, we also developed two different curb-rail timber systems under TL-1 of NCHRP 350. For those systems, rail heights may have ranged from approximately 18 to 20 in. If desired, I could provide more information regarding these research efforts.

Response

Date: 09-08-2016

Attached is the research that was sent to me by the Chicagoland suburban transit provider,Pace, in order to justify the use of 12" high curb for certain transit stops along their system.

In my review of the literature it does not justify the use of the 12" high curb from a roadway safety perspective, but it does suggest that there might be some positives to vehicular traffic flow if the bus doesn't have to stop & kneel at the transit stop.

Since I first emailed you the Department has agreed to allow Pace to implement the 12" high curb as a Pilot Project which means that no other routes along their system will be allowed to have 12" high curb until after their first route is implemented and monitored/studies for at least 5 years. The roadway that the Department has agreed to allow as a Pilot Project is Milwaukee Ave from theJefferson Park Transit Center in Chicago north to Golf Mill Shopping Center in Niles. The roadway's Posted Speed Limit varies from 30mph to 35mph and its ADT ranges from 20,000vpd to 30,000vpd. The cross section of the roadway is 4-lane and goes from divided to undivided depending on the location.

Per what research I have done there is no curb height and design (e.g. curb face slope) that meets TL1 crash test worthiness while some 18" high curb designs do meet TL2 crash test worthiness.

That being said what you'll find in the attached is that some entities, notably Grand Rapids, MI; Las Vegas, NV and Phoenix, AZ are placing the 12" or higher curb adjacent to the traveled way for transit accommodations.

The transit accommodations along Grand Rapids, MI (Silver Line) are rather new but have shown no adverse impacts per my discussions with them. As such they are proposing similar transit stop accommodations/treatments along Lake Michigan Dr (MI 45) to/from Grand Valley State University and Grand Rapids. Due to the context of the surroundings along MI 45 vehicles should operate at a much higher speed than the roadways that the Silver Line runs along.

Another example is Cleveland's Health Line (MAP) which has a similar treatment to Grand Rapid's Silver Line.

This topic may be of interest to you and the MwRSF since, in my opinion, there is a big push to improve non-motorized facilities within this country potentially at the expense of the safety of motorized facilities. In short the transit agencies want near level boarding, like a train stop, to improve their reliability. With no research to prove the effects of higher than Department or AASHTO standard curbs this gives me pause from a liability perspective.

Attachment: http://mwrsf-qa.unl.edu/attachments/fea0c99dbafb05916e54ace45b1d95d8.pdf

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Response

Date: 09-09-2016

From your email below, it would appear that there is a recent trend or desire to use 12" tall curbs associated with transit operations, even along roads with moderate speeds. I guess my main thought may pertain to the increased risk of vehicle instability for passenger vehicle contacting tall curbs in tracking and non-tracking scenarios. If speeds are low enough, then I may agree that these elements are reasonable. However, the speeds noted below may still seem high enough to post those risks.

Non-cracking vehicle orientations may actually pose the greater risk as tripping could ensue upon impact. It would be interested to know whether any crash data has been analyzed for the recent sites with tall curbs to evaluate risk as compared to standard curb use. The risk of non-tracking vehicle impacts may be accentuated in locations where increased rain, snow, or ice may likely occur on road throughout the year. Grand Rapids, Michigan, should have those types of conditions more often than Las Vegas and Phoenix. Has anyone reviewed crash reports in these locations to actually see if tall curbs were struck? What percent of curb impacts resulted in rollovers?

I realize that only short segments of curbing would be mounted at 12" between the transitions to standard height. With short segments, the exposure is reduced somewhat. However, there still remains increased safety risk. In the end and in the absence of supporting study, I would agree that agencies could potentially be exposed to greater tort risk.

Compressive Strenght of Temporary Barrier

Question

State: WI
Date: 09-07-2016

What is the spec. compressive strength for the temporary barrier that MwRSF developed for the pooled fund?

What was the design compressive strength for the temporary barrier that MwRSF developed for the barrier?

Response

Date: 09-08-2016
The original NCHRP Report 350 testing of the F-shape barrier used by the Midwest Pooled Fund states specified a minimum concrete strength of 4,500 psi.

Over time the original design has evolved somewhat. The current barrier system has been tested to NCHRP MASH in several different configurations including free-standing and reduced deflection applications. For the more recent testing the minimum concrete strength was specified as 5,000 psi.

Based on this, we would recommend the use of 5,000 psi concrete with the F-shape PCB in order to be compliant with the most recent testing.

Response

Date: 09-08-2016

What was the designed breaking strength of the barrier.

A lot of the concrete we spec. has a 28 day breaking strength between 4,000 and 4,500 PSI. ASTM C825-06 indicates a 28 day design strength of 4,000 PSI for precast barrier. Is this difference in 28-day breaking strength significant?

Response

Date: 09-09-2016

Slightly lower compressive strength should not have a drastic effect on barrier capacity. Most of the impact damage we see is flexural cracking of the barrier due to the impact loads. We would not expect the flexural cracking or the moment capacity of the section to change much due to the compressive strength reduction as it is largely controlled by the reinforcing steel.

The reduced concrete strength will cause increased spalling, damage at the joints, and disengagement of the barrier toes. Shear capacity would be reduced as well. These effects may cause some additional issues with barrier performance, but that is difficult to quantify.

Hardware Component Steel Grades

Question

State: WI
Date: 09-15-2016

I know that we have gone through this a number of times. However every time I think I got this handled, I find something weird on my drawings or somewhere else that causes me to relook at this

In most cases, things that are fabricated out of steel plates, some examples:

Anchor Bracket
Anchor Bracket - Bearing Plate
Retrofit Thrie Beam Cantilever Blunt End - Backup Plate
Retrofit Thrie Beam Cantilever Blunt End - Base Plate
Retrofit Thrie Beam Cantilever Blunt End – Beam
Square Washer - Non – Galvanized (temporary barrier anchoring into AC or concrete…)
Temporary Barrier - Anchor Asphalt - Asphalt Pin
Temporary Barrier - Anchor Asphalt - Asphalt Pin - Top Plate
Terminal Connector - Single Slope Connector Plates and Stiffeners

And posts like

Post - Strong Post – Steel w6x9

Are fabricated out of A36 Steel.

If that is the case, the following steel grades are acceptable?

Or ASTM A529 Max. Strength 50 KSI,
Or ASTM A572 Max. Strength 50 KSI
Or ASTM A709 Max. Strength 50 KSI,
Or ASTM A992 Max. Strength 50 KSI

Response

Date: 09-15-2016

When we design new systems or components here at the facility, we typically specify those parts using the appropriate current material specifications. However, older components may have been specified years ago and the original specification may have changed, or is no longer available. Thus you will tend to see them change. This is true for many of the older hardware guide components.

We run into this issue as well and plan to try and bring it up at the TF 13 meeting. We know that we specify things according to the hardware guide but get some form of equivalent when we order it. That may not be a huge issue, but we should try to update things. As TF 13 has a mission to work towards standardization, it seems the place to best address this issue.

As far as the many components you list below, they would need to be looked at individually to make sure that the specs are up-to-date.

The AISC Steel Manual has some nice tables that list materials specs for various structural shapes. They also put out a pdf on the website that explains things better than I am. See attached. The website also allows you to look up component availability interactively.

For example, you note that post sections are A36 below, but we typically get them in A992 and that is currently the preferred designation in the AISC manual.

Let me know if this helps you out or if you want to discuss things further.

Attachment: http://mwrsf-qa.unl.edu/attachments/76fb4fe67de33dbf1ba097a00728bf5f.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/62b9580054ec622bff7b02380d1212c8.pdf

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MGS Bridge Rail - Steel Backup Plate

Question

State: UT
Date: 09-15-2016

The MGS bridge rail system uses W-beam backup plates between the S3x5.7 post and the guardrail. Are there any concerns with extending the length of that backup plate.

Response

Date: 09-15-2016

I do not have any concerns with using a 12-in. long steel backup plates with the MGS bridge rail system. The original MGS bridge rail system was designed, configured, and successfully crash tested under MASH using 6-in. steel backup plates between the W-beam rail and the S3x5.7 steel posts. Longer steel backup plates would likely reduce the propensity for the flange ends of support posts to contact the backside of the rail after bending and twisting during impact events. As such, we are supportive of using a longer steel backup plate with this system.

On another recent research project involving the 31-in. tall, weak-post, W-beam guardrail system installed in mow strips, we recommended that the 6-in. long backup plates be increased to 12 in. A weblink is provided below to access this research study, report, drawings, and videos. See pages 145 through 148 of the associated research report.

http://mwrsf.unl.edu/reportresult.php?reportId=315&search-textbox=mow%20strip

Based on this recent guardrail research, we also noted within this report that there would be benefit to increasing the length of the backup plate to 12 in. on other similar systems, such as the MGS bridge rail and the MGS post-socketed system adapted to culvert headwalls.

Please let me know if you have any further questions regarding the enclosed information. Thanks!

Top Mounted Thrie Beam & Steel Post Bridge Rail

Question

State: WI
Date: 09-20-2016

A guardrail to bridge rail transition was crash tested to MASH (TL-3), and is shown in eligibility letter (B-231), and has Taskforce 13 Designator (STG03a). The bridge rail is shown on the sketches in this letter (sheets 8 & 9 of sketches), and has a steel post that is top mounted to a concrete slab. The rail element attached to the post is a thrie-beam. Could you tell me if this bridge rail was ever crash tested to MASH criteria? If not, are there any plans to crash test it?

We are looking for a top mounted steel post and thrie beam bridge rail that has been crash tested to MASH criteria, that we can use to replace the old Missouri Thrie-Beam & Channel bridge rail. Are you aware of any crash testing on this type of bridge rail being done by other testing facilities?

Thanks for any assistance you can provide.

David Nelson - (WisDOT) - Bureau of Structures

Attachment: http://mwrsf-qa.unl.edu/attachments/908bc9fa039225c609ae7614fd7a8cb6.pdf

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Response

Date: 09-26-2016
At this time, MwRSF is not aware of any thrie beam bridge rail systems that have been evaluated to MASH criteria (pass or fail). It may be a good idea to submit a problem statement concerning the evaluation of this type of system for next year's Pooled Fund Program.

Wood Bridge Railings on Concrete Decks

Question

State: WI
Date: 09-22-2016

In 1998, a document was published entitled "Plans for Crash-Tested Wood Bridge Railings for Concrete Decks". Could you tell me if this bridge rail was ever crash tested to MASH criteria? If not, are there any plans to crash test it?

Are you aware of any crash testing on this type of bridge rail being done by other testing facilities? We have been using this timber railing and the attachment details since 2000, and would like to find out its future.

Thanks for any assistance you can provide.

Attachment: http://mwrsf-qa.unl.edu/attachments/2fce5320b083d0212ef2c3d6ba1acb7c.pdf

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Response

Date: 09-22-2016
Currently, no timber bridge rail systems have been evaluated to MASH, including the design attached. Additionally, there has been no plans made to evaluate this timber bridge rail to MASH at this time.

To the best of our knowledge, no other agencies or labs have conducted or plan to conduct testing of timber bridge rails, but the need to do so likely exists.

This is an area that we could work with you and other states to investigate and study if you are interested.

thanks
