Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

07-01-2014 to 10-01-2014

Questions about the MGS Trailing End Terminal

Question
State: WY
Date: 07-03-2014

We have a couple of questions regarding the MGS Trailing End Terminal as reported in TRP-03-279-13:

1. On pages 155 and 167, a single span rail (6'-3"), part a3 is incorporated into the design. Why? Is this necessary for the terminal to work properly?

2. On sheet 165, the cable anchor is shown to have an overall length of 80 inches. The old metric standard barrier guide calls out 2000 mm (78 ¾"). The old, old guide calls out 78 inches. Obviously some rounding was applied when it was made metric. This cable is not shown in the current on-line guide. Should this figure on page 165 really show 78 inches since it references a part number from the standard barrier guide, or does it require a special cable?

Attachment: http://mwrsf-qa.unl.edu/attachments/acfd0277a8b5f1aabf775d543f735c01.docx

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Response
Date: 07-10-2014

The single 6'-3" rail segment was used for testing purposes to set the overall system length at 175'. For real world installations, this system length is not required and would not necessitate the extra rail segment. It is important to maintain the splice locations at the mid-span between the posts to maintain the benefits of the MGS system.

With respect to the anchor cable length, we believe that either length is acceptable. The hardware guide specifies the 78.75" long anchor cable. We have used that length in these types of end anchorage for many tests. In more recent years, our guardrail part supplier has begun supplying the 80" long version, so we have been testing with that.

One related item to note deals with the location of the cable bracket on the guardrail. According to the hardware guide, the cable bracket should be located such that the first bolt for the bracket is 1250 mm or 49.25" from the center of post no. 1. The downstream anchor report you mentioned above shows this distance as 47.625". Going back through our records, it appears that this anchor bracket has varied slightly in location over time. However, we would recommend that the 49.25" location be used in order to be consistent with the hardware guide and to minimize the angle of the cable as it approaches post no. 1.

What Degree of Slope of a Concrete Barrier Would Require Head Ejection Considerations

Question

State: WY
Date: 07-03-2014

Our Bridge Program designed a 42 inch high single slope concrete barrier to protect some bridge columns underneath the interstate. The design speed of this roadway is only 30 mph. The slope of the barrier face is 5.8 degrees. The barrier the pooled fund tested with head ejection criteria is about 3 degrees. Do we need to consider head ejection for a barrier with a slope of 5.8 degrees (and also considering the speeds are relatively low). As a follow-up question, do you have an idea of what barrier slope face would dictate head ejection criteria for a high speed roadway?

Attachment: http://mwrsf-qa.unl.edu/attachments/429a701a42dffc8abeba0af7a21d840d.docx

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Response

Date: 07-07-2014

Although we have not studied head ejection for impact speeds below TL-3 conditions, I don't believe that an occupants head would extend very far out the side window on a 30 mph roadway. Thus, the risk of head slap is greatly reduced, and I do not think you need to incorporate head ejection into your design for such a low speed roadway.

The head ejection envelope was developed for vertical (or near vertical) barrier geometries. Although single slope barriers show increased vehicle stability during impacts over safety shaped barriers, existing single slope barriers (9.1 and 10.8 degrees from vertical) do cause some vehicle climb. However, head ejection is still present with single slope barriers. In fact, a few impacts with single slope barriers were utilized in the initial development of the ejection envelope. We do not have a set slope angle in which the envelope should be applied as this has never been studied. Though, the answer would probably be more of a sliding scale reduction factor that increased with an increase in slope. Unfortunately, we just don't know this answer at this time. So, on the side of safety and being conservative, you could apply the head ejection envelope as it currently stands to any single slope barrier (MwRSF recommended method). Or, you can choose to use engineering judgment and take a more aggressive approach.

Ballasting Questions for the FHWA/DOT Midwest Work Zone Roundtable

Question

State: MO
Date: 07-16-2014

From: Nick.Artimovich@dot.gov
[mailto:Nick.Artimovich@dot.gov]

Sent: Monday, July 21, 2014 12:25 PM

To: Daniel.Smith@modot.mo.gov

Cc: Ken.Wood@dot.gov;marc.thornsberry@dot.gov;Julie.Stotlemeyer@modot.mo.gov;
Rob.Frese@modot.mo.gov;James.Connell@modot.mo.gov

Subject: RE: Ballasting Questions for the FHWA/DOT Midwest Work Zone Roundtable

Mr. Smith,

Thank you for your inquiry. I will defer to the researchers at Midwest Roadside Safety Facility on this question. They have conducted numerous tests on portable sign stands and would be a better source for information. If their extensive crash testing experience leads them to the opinion that your requested options would not seriously compromise the crash-worthiness of these devices under NCHRP Report 350 then I would concur with their assessment.

While I have reviewed numerous
crash test reports of portable sign stands I have not kept a record of which
used ballast, nor the performance of the ballast upon impact.

You may find contact information
for the MWRSF on our website listing all accredited laboratories:
http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/laboratories/

Regards,

Nicholas A. Artimovich, II

Highway Engineer, Safety Design
Team

Office of Safety Technologies,
Rm E71-322

Federal Highway Administration

U.S. Department of
Transportation

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WebSite http://safety.fhwa.dot.gov

From: Dan Smith [mailto:Daniel.Smith@modot.mo.gov]

Sent: Wednesday, July 16, 2014 1:39 PM

To: Artimovich, Nick (FHWA)

Cc: Wood, Ken (FHWA); Thornsberry, Marc; Julie Stotlemeyer; Rob Frese; JAMES D CONNELL

Subject: Ballasting Questions for the FHWA/DOT Midwest Work Zone Roundtable

Mr. Artimovich: MoDOT has been working on several ballasting ideas as possible alternatives to sandbags. In May, I presented the ballasting ideas to the FHWA/DOT Midwest Work Zone Roundtable and they were interested in the different ballasts and field applications. As a consensus, the roundtable would like your our opinion if they would be acceptable alternatives or would need crash testing. The description and examples are located in the attached word document.

If you have any questions please let me know. Thank you for your time.

Daniel J. Smith, P.E.

Traffic Management and Operations Engineer

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Attachment: http://mwrsf-qa.unl.edu/attachments/9a5f56ae9008b3a02bf46dce864c6891.docx

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Response

Date: 07-31-2014

We have briefly reviewed the material contained in the recent inquiry. In general, if the ballast is not positioned higher than the originally configured sandbags/ballast and does not provide a potentially hazardous condition (based on size, material selection, attachment to base/legs, etc.), then alternative ballast options would likely allow the work-zone device to perform similarly and in a safe manner. If sand ballast bags or sacks are fabricated using a stronger material and protrude farther above the base and legs than original tested and evaluated, then I could envision a condition where small car vehicles encountered increased instabilities after traversing across the system.

http://mwrsf-qa.unl.edu/view.php?id=641

MGS Omitted Post Questions

Question

Date: 07-18-2014

I have several questions regarding the MGS when posts are skipped or omitted.

1. When using the MGS and a post is needed to be skipped due to an underground feature that is in conflict, what happens if just one or two post need to be skipped? Can you place less than the 3 CRT post? If we skip one post can we place just 1 CRT post? If we skip 2 post can we place just 2 CRT post? If we skip 3 post then we need to place all 3 CRT post?

2. If you are on a one way roadway or a wide roadway so that you are not concerned about apposing traffic, do we need to place CRT post on the near side of the missing post or just on the far side (downstream)? Is the number of nearside CRT post the same as the downstream need?

3. Does the steepness of the slope behind the guardrail matter? (Max 3:1 or 2:1 or 1.5:1 or ? or vertical culvert headwall???)

4. If you are skipping a post due to an underground conflict, but there is another fixed object behind the guardrail just downstream, say 12' downstream, but just 5' behind the face of the rail. Should we increase our CRZ behind the rail for some distance downstream and further from the guardrail from where the post needs to be skipped?

5. If you are missing 3 post I understand that the guardrail should remain inline before any flaring of terminals or redirection of the guardrail for an additional 62' but if you are just skipping 1 or 2 post can this be reduced to 50' or 38' or 25' or ???

6. If you have to skip a post or two and then 50' latter need to do this again is there concern or limitation to how often this can repeat?

Response

Date: 07-21-2014

I have some feedback for you regarding the MGS with omitted post questions you gave me at the meeting.

1. When using the MGS and a post is needed to be skipped due to an underground feature that is in conflict, what happens if just one or two post need to be skipped? Can you place less than the 3 CRT post? If we skip one post can we place just 1 CRT post? If we skip 2 post can we place just 2 CRT post? If we skip 3 post then we need to place all 3 CRT post?

The MGS Long-Span system was developed for use to span transverse culverts measuring 24 ft wide or less. In this circumstance, three posts would be removed from the system. This system also utilizes three CRT posts on each side of the culvert structure. For culverts measuring less than 24 ft wide and where one or two posts are omitted, it still would be necessary to utilize the CRTs on each side of the unsupported segment of rail.

Although it may be possible for the MGS to work with one post removed and without CRTs adjacent to the long span, it should be noted that crash testing has not been performed on this MGS system nor to verify that acceptable performance would result. As such and in the absence of test data, we recommend that the CRTs be installed in systems where one, two, or three posts are removed.

2. If you are on a one way roadway or a wide roadway so that you are not concerned about apposing traffic, do we need to place CRT post on the near side of the missing post or just on the far side (downstream)? Is the number of nearside CRT post the same as the downstream need?

Similar to the comment above, we believe that the CRT's are needed on both the upstream and downstream end without further analysis. The CRT's on the downstream side are more critical in terms of rail pocketing and snag, but review of the barrier performance in testing found that the CRT's on the upstream end often fracture as well, which may make the barrier system more forgiving and reduce rail loads and pocketing angles. Thus, we cannot recommend removing the upstream CRT's without further analysis and/or testing.

3. Does the steepness of the slope behind the guardrail matter? (Max 3:1 or 2:1 or 1.5:1 or ? or vertical culvert headwall???)

We recommend providing 2 ft of level, or mostly level, soil grading behind the wood CRT posts. However, we understand that this can be difficult. As such, there is potential that the wood CRT posts could be lengthened to account for the reduction in soil resistance resulting from an increased soil grade behind these six posts, especially when placed at the slope break point of a 2:1 fill slope.

Recently, MwRSF performed limited research to determine an acceptable MGS post length for a 6-in. x 8-in. solid wood post installed at the slope break point of a 2:1 fill slope. MwRSF determined that 7.5-ft long wood posts are an acceptable alternative when considering the 31-in. tall MGS placed at the slope break point of a 2:1 fill slope using 6-ft 3-in. post spacing.

The MGS Long Span system utilizes six CRT wood posts. A CRT post's moment capacity about its strong axis of bending is approximately 81 percent of that provided by the standard wood post. In the absence of dynamic component test results, it is believed that the six CRT wood posts could also be fabricated with the 7.5-ft length when used in the MGS Long Span system. If the steep fill slopes continue beyond the location of the CRT posts, then the guardrail would transition to the MGS for 2:1 Fill Slopes using either 6-in. x 8-in. by 7.5-ft long wood posts or W6x9 by 9-ft long steel posts.

For general installations at slope breakpoints or offsets less than 2' from the slope breakpoint of 2:1 to 6:1 slopes, we would recommend using the 7.5' long CRT posts. For slopes steeper than 2:1, we have little test data or analysis to guide us. Thus, we would recommend maintaining the 2' minimum offset in those locations.

4. If you are skipping a post due to an underground conflict, but there is another fixed object behind the guardrail just downstream, say 12' downstream, but just 5' behind the face of the rail. Should we increase our CRZ behind the rail for some distance downstream and further from the guardrail from where the post needs to be skipped?

In locations where posts are left out, dynamic barrier deflections and working widths would be expected to increase. Test results are available for the case with three posts removed from the MGS. However, data is not available for cases with one or two posts removed. BARRIER VII computer simulations could be performed to estimate barrier deflections and working widths. A small modeling study would be necessary to validate the model for the MGS long-span system and then predict barrier performance with fewer posts removed.

That said, attempting to account for the expected increase in deflection would be a step in the right direction. Full-scale crash testing of the MGS Long Span had dynamic deflections of 92.25". Based on this level of increased deflection over the 60" you note above, the a reasonable approach may be to assume a 12" increase in dynamic deflection for every omitted post. For example, one omitted post = 72" and two omitted posts = 84". However, this guidance is only based on rough approximations on a limited number of tests and it would be better to derive more accurate values through modeling as noted above.

We currently have a project to investigate the omission of a single post from the MGS without the use of the CRT's through full-scale crash testing that may provide further information.

5. If you are missing 3 post I understand that the guardrail should remain inline before any flaring of terminals or redirection of the guardrail for an additional 62' but if you are just skipping 1 or 2 post can this be reduced to 50' or 38' or 25' or ???

The MGS Long-Span Guardrail System was successfully crash tested and evaluated according to the Test Level 3 (TL-3) safety performance criteria found in MASH. For this testing program, the overall system length was 175 ft, including 75 ft of tangent rail upstream from the long span, a 25-ft long unsupported length, and 75 ft of tangent rail downstream from the long span. As part of the final recommendations, MwRSF had noted to provide a minimum “tangent" guardrail length adjacent to the unsupported length of 62.5 ft.

In lieu of a recent MASH crash testing program on a 75-ft long version of the MGS (unpublished at this time), there may reason to consider potentially reducing the 75-ft total guardrail length on the upstream and downstream ends of MGS Long-Span Guardrail System. For example and based on the MASH 2270P test into the MGS Minimum Length System, we believe that the MGS Long-Span Guardrail System would likely have performed in an acceptable manner with 62.5 ft of rail on the upstream and downstream ends, thus resulting in an overall system length of 150 ft. A 62.5-ft long tangent length adjacent to the unsupported length would still provide adequate space to incorporate a 37.5 ft or 50 ft long energy-absorbing guardrail end terminal.

For unsupported lengths of 18.75 ft and 12.5 ft, it would seem reasonable to consider a reduction in the required guardrail length both upstream and downstream from the unsupported length using the test information and arguments noted above. For two missing posts or an unsupported length of 18.75 ft, we believe that the upstream and downstream guardrail lengths likely could be 56.25 ft each with a minimum overall system length of 131.25 ft. For one missing post or an unsupported length of 12.5 ft, we believe that the upstream and downstream guardrail lengths likely could be 50 ft each with a minimum overall system length of 112.5 ft. However, we believe that the three CRT posts still would be required on the upstream and downstream ends of the 18.75 ft and 12.5 ft long unsupported lengths. In addition, one would need to discuss with and likely obtain approval from the manufacturers as to whether they would allow three CRTs to be used within the last 12.5 ft of a 50-ft long guardrail terminal.

If one were to follow the logic used above and consider the situation of no missing posts (i.e., 6.25 ft post spacing throughout), the upstream and downstream ends would be reduced by 6.25 ft each and include the interior 6.25 ft long span in the middle of the system. As a result, the overall system length would be 43.25 ft + 6.25 ft + 43.25 ft for a total of 92.75 ft. As noted above, MwRSF recently crash tested a 75-ft long version of the MGS with satisfactory results, effectively configured with two 37.5-ft long guardrail segments with tensile anchorage devices and placed end-to-end.

Of course, it should be noted that the design modifications for the 25 ft, 18.75 ft, and 12.5 ft long unsupported lengths were based on engineering judgment combined with the unpublished results from the MGS Minimum Length System crash testing program. In addition, the opinions noted above are based on the assumption that the currently-available proprietary guardrail end terminals would provide comparable tensile anchorage for the MGS as provided by the common tensile anchorage system using in the MwRSF crash testing program (i.e., two steel foundation tubes, one channel strut, one cable anchor with bearing plate, and BCT posts at positions 1 and 2 on each end). Although we are confident that the modifications noted above would provide acceptable performance, the only sure means to fully determine the safety performance of a barrier system is through the use of full-scale vehicle crash testing. We are hopeful that these design modifications can be evaluated in the near future and as part of a continued R&D Pooled Fund program involving the MGS Long-Span Guardrail System.

6. If you have to skip a post or two and then 50' latter need to do this again is there concern or limitation to how often this can repeat?

This question has not been answered to date. There are concerns about how close these post omissions can occur. We currently have a project to investigate the omission of a single post from the MGS without the use of the CRT's that may shed some light on this issue. We plan to test the MGS with the omitted post with the 2270P vehicle. Based on the outcome of that test we plan to give guidance on what the minimum allowable offset between omitted posts should be.

South Dakota Road Closure Gate

Question

State: KS
Date: 07-25-2014

We would like your comments on the use of a double-sided road closure gate based on the single arm gate that was tested for South Dakota.

Response

Date: 07-25-2014

The South Dakota Road Closure Gate does not pose any significant hazard for vehicles impacting the gate in a stowed position. The crash tests were conducted on the road closure gate oriented in a stowed position as opposed to a closed position, since it was believed that it would result in the most severe impact. In addition, SDDOT reasoned that vehicle impacts into road closure gates in the closed position rather than the stowed position would not be as likely to occur due to the significant increase in delineation and subsequent lower driving speeds. Head-on tests were conducted since the vehicle would be required to break both the gate support post and the hold back post. The impact location, consisting of the centerline of the hinged connection, was selected because the post and gate weights were approximately equal.

We have some basic comments on the use of the double-sided gate.

1. Switching to a double-sided gate would not affect the performance of the gate in the open position. This would basically be the same as the single-sided gate that was tested.

2. When the double-sided gate was in the closed position, there are some concerns. If the closed gate has too strong of a connection, there is concerns that impacts on the gate in the closed position may not cause the support posts and hinges to breakaway easily enough and may post a risk to vehicle occupants. The concerns would include occupant impact velocity and ridedown acceleration increases and the potential to cause vehicle instability. Thus, any closure of a double-sided gate would have to release under very low loads. With a breakaway or weak connection at the connection between the gate arms, the potential for impact safety issues may exist, but the potential is much less.

3. Gate closure capacity would need to be very small to reduce concerns for impact with the double-sided gate. Ultimate capacities of the closure between 1-2 kips or less would be recommended. This is roughly equal to the capacity of two 1/8" dia. A307 bolts or a single ¼" dia. A307 bolt.

Let me know if you need further information.

Vehicle Impact Protection

Question

State: OH
Date: 07-28-2014

Putting this as simply as possible, do we think a 32" Jersey Barrier can accomplish the intended goal below?

Maria E. Ruppe, P.E.

Roadway Standards Engineer

Ohio Department of
Transportation

Mail Stop 1230

1980 W. Broad St.

Columbus, OH 43223

614.466.2847

From: French, Lynn

Sent: Monday, July 28, 2014 10:38 AM

To: Stargell, Reynaldo

Subject: FW: Vehicle Impact Protection

Hi Reynaldo…

I was referred to you by David
Powers to assist with this question.

Thanks,

Lynn

The Ohio Fire Code (OFC) requires
all aboveground flammable and combustible liquid tanks to be protected from
vehicles using barriers meeting the following code requirement:

Section
312 Vehicle impact protection

(1)
312.1 General. Vehicle impact protection required by this code shall be provided by posts that comply with*paragraph* (L)(2)(312.2)*of this rule* or by other approved physical barriers that comply with*paragraph* (L)(3)(312.3) *of this rule*.

(2) **312.2**
Posts. Guard posts shall comply with all of the following requirements:

(a) Constructed
of steel not less than 4 inches (102 mm) in diameter and concrete filled.

(b) Spaced
not more than 4 feet (1219 mm) between posts on center.

(c)
Set not less than 3 feet (914 mm) deep in a concrete footing of not less than a 15-inch (381 mm) diameter.

(d) Set with
the top of the posts not less than3 feet (914 mm) above ground.

(e) Located
not less than 3 feet (914 mm) from the protected object.

(3)
312.3 Other barriers. Physical barriers shall be a minimum of 36 inches
(914 mm) in height and shall resist a force of 12,000 pounds (53 375 N) applied
36 inches (914 mm)
above the adjacent ground surface.

I have quickly looked thru your Roadside Safety Field Guide
and would appreciate if you could help with a variance request we received.

The request is to allow the barriers (jersey barriers
proposed) to be 32" in height in lieu of the required OFC height of 36".

Any input to these proposed type barriers being equivalent
to the required 36" (3 feet) posts or other barriers would be greatly
appreciated.

Thanks for your input

Response

Date: 07-28-2014

The spec below is specific in terms of height, but not very specific in any other way when regarding the specification for other barrier types. The height is limited to a minimum of 36". Without further knowledge of the intent of the specification, I don't believe that we could justify going any lower.

In terms of the forces, the spec lists a set load at a height of 36" that the barrier must resist. However it is unclear if the barrier is allowed to deflect or not. Free-standing 32" tall barriers would deflect significantly under that type of load at the top of the barrier and would not be a good fit for this application. Permanent parapets would not deflect appreciably depending on the design, as most of the TL-3 or TL-4 parapet design have withstood crash test impact loads of over 60 kips.

Thus, in short, a permanent 36" tall or taller NJ shape, single-slope, or vertical barrier that we use for roadside safety with appropriate anchorage and footing would likely withstand that loading. Vertical barriers would likely be more appropriate for the application as they would produce less vehicle climb and extension and are more typically used in this type of protection scheme.

I am not sure if that answers you question. Let me know if you need more information.

Epoxy Anchorages for F-shape Barrier Tie-Down

Question

State: MN
Date: 07-30-2014

During our teleconference this
morning, we had a discussion regarding appropriate epoxy anchorage depths for
the F-shape PCB tie-down system and related research on epoxy adhesive
anchorages in Wisconsin. Can you review the information we discussed.

Response

Date: 07-30-2014

During our teleconference this morning, we had a discussion regarding appropriate epoxy anchorage depths for the F-shape PCB tie-down system and related research on epoxy adhesive anchorages in Wisconsin.

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

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

MwRSF has also done some recent work to investigate epoxy adhesive anchors for permanent concrete barriers. As part of that research, MwRSF conducted static and dynamic testing of threaded rod and rebar with shallow embedment and attempted to determine design procedures for the epoxy adhesive anchors. The full report can be downloaded at the following link.http://mwrsf.unl.edu/researchhub/files/Report14/TRP-03-264-12.pdf

In that report, we tested the 1 1/8" dia. A307 rod used in the tie-down system in concrete with an f'c = 6,454 psi, an epoxy with a nominal bond strength of 1,800 psi (1,904 psi based on threaded anchor diameter effects in manufacturer literature). In this test, the rod developed 45.3 kips in tension loading and over 40 kips in shear loading prior to anchor fracture. Based on these results, we made the following comments.

“The ultimate tension and shear capacities were calculated to be 45.9 kips (203.6 kN) and 26.4 kips (117.6 kN), respectively. The average ultimate tension and shear loads observed from the dynamic testing program of the 1 1/8 in. (29 mm) diameter A307 rods were 45.3 kips (201.5 kN) and 40.6 kips (180.8 kN), respectively. The failure mode in tension consisted of a pullout of the adhesive core accompanied by a 2 ¾ in. (70 mm) deep concrete cone breakout. The ultimate shear value obtained during the component test is an estimated minimum value because the anchor did not fail in the test and the load was governed by the equipment. Nonetheless, the ultimate shear capacity was determined to be far greater that the nominal shear capacity of the anchor and the ultimate tension capacity was within one percent of the nominal tension capacity for the concrete strength in the component tests. Therefore, the anchorage design with 5 ¼ in. (133 mm) embedment depth utilizing the Hilti HIT-RE 500-SD epoxy adhesive was considered an adequate alternative anchorage design for the 1 1/8 in. (29 mm) diameter A307 rods used in the tie-down temporary concrete barrier developed by MwRSF because the tested capacities met the nominal capacities of the anchorages used in the full-scale crash test. However, the failure in the tension test created significant concrete damage. This concrete damage would be expected to occur to the bridge decks of real-world installations during severe, high-energy impacts. In addition, the compressive strength of the concrete used in these component tests may be higher than the typical strength of concrete bridge decks. Thus, some decrease in the capacity of the anchors would be expected for lower strength concrete. This decrease in strength would likely be offset to some extent by the presence of reinforcing steel in the bridge deck. Thus, it is believed that using the A307 rod with Hilti HIT-RE 500 or Hilti HIT-RE 500 SD epoxy adhesive with a 5 ¼-in. embedment depth should provide similar anchorage to the tested system, but some increased deflection and increased deck damage may result. It should also be noted that epoxy adhesive manufacturer recommendations for torque requirements on threaded anchors should be closely followed for these types of anchors to prevent concerns for anchor creep and associated reductions in anchor capacity."

So while component level testing did indicate that the shallow embedment had the potential to meet the desired loads, it was noted that reduced concrete strengths would reduce the loads and that damage and release of the anchors could occur in high-energy impacts.

MnDOT has different embedment depths listed in their standards. The bridge standard suggests an embedment of 5.5" due to deck thickness concerns, while the roadside standards suggest 9" of embedment. In order to shed more light on the issue, I reviewed the anchor design procedures suggested in TRP-03-264-12. In this report, we calculated anchor capacities based on two methods:

1. A factored, as-tested procedure based on the ACI-11 code that applied dynamic increase factors for the steel and concrete, used as-tested values for the material strengths, and without strength reduction factors. This was

used to compare the analytical procedure to tested values as closely as possible.

2. A design procedure based on the ACI-11 code that applied dynamic increase factors for the steel and concrete, used published values for the material strengths, and included standard strength reduction factors. These were more conservative and recommended for design values.

If the factored, as-tested procedure is used to estimate the anchor tensile capacity in the component test, the procedure returns a value of 43.6 kips. This corresponds very well to the test value of 45.3 kips and predicts the failure mode (concrete failure). Using the factored, as-tested procedure for concrete with strengths of 4,000 psi and 3,000 psi yields tensile loads of 34.3 kips and 29.7 kips respectively. Thus, reduction in concrete strength would be expected to reduce tensile capacity significantly. Shear capacities for all concrete strengths were acceptable.

If the design procedure is used to estimate the anchor tensile capacity in the component test, the procedure returns a value of 27.3 kips and predicts the concrete failure mode. Using the design procedure for concrete with strengths of 4,000 psi and 3,000 psi yields tensile loads of 22.3 kips and 19.3 kips respectively. Shear capacities for all concrete strengths were acceptable. Thus, the design procedure provides more conservative estimates on anchor capacity.

We typically design our hardware and anchor systems near the edge of the ultimate capacities without reduction factors or factors of safety. However, we generally test those systems to verify their capacity. Thus, the method of estimating anchor loads may be dependent on what level of conservativeness the DOT wants to have in their spec.

It should be noted that higher bond strengths won't improve performance as the failure are concrete controlled. In addition, cracked concrete may be very difficult to design a reasonable anchorage for. Published values for bond strength tend to decrease over 50% for cracked concrete. That does not include reductions in strength for the concrete. Thus, it is difficult to recommend anchoring in cracked concrete. The numbers above also consider only individual shear and tensile loads and not combined loading. Estimation of the effect of combined loads on anchor performance is difficult. Thus, the best avenue for addressing this issue may be full-scale testing of the tie-down system with shallow anchor embedment to evaluate its performance.

I also analyzed the 9" embedment listed in the roadside spec. For 3,000 psi to 6,000 psi concrete, 9" embedment is sufficient to generate the ultimate steel rod capacities using both the factored, as-tested procedure and the design procedure.

Please review this information and contact me the any questions and or comments.

Thanks

PL-1/TL-2 Bridge Railings w/ Sidewalk

Question

State: MN
Date: 07-31-2014

We are looking for details for a 32" tall bridge railing that is installed on an 8" tall sidewalk. Do you know of any?

Response

Date: 07-31-2014

I have not yet found a 32" tall bridge railing that is installed on an 8" tall sidewalk. I have found a couple that utilized a 42" height.

TL-2 BR27C Bridge Railing w/ sidewalk – 42" above sidewalk

TL-2 BR27D Bridge Railing w/ sidewalk – 42" above sidewalk

The file 'FHWA-RD-93-058.pdf' (20.3 MB) is available for download at

< http://dropbox.unl.edu/uploads/20140814/258b3211a4258bec/FHWA-RD-93-058.pdf >

for the next 14 days.

It will be removed after Thursday, August 14, 2014.

The file 'FHWA-RD-93-065-1.pdf' (19.6 MB) is available for download at

< http://dropbox.unl.edu/uploads/20140814/a142a3dd65c8a61d/FHWA-RD-93-065-1.pdf >

for the next 14 days.

It will be removed after Thursday, August 14, 2014.

Response

Date: 08-12-2014

Any luck in finding a 32" tall vertical face parapet mounted on an 8" sidewalk that meets TL-2? We've verified that our parapet meets the structural requirements to resist TL-2 loads, but still need confirmation that the geometry meets crash test requirements (32" vertical face parapet mounted on an 8" sidewalk). We've been using this standard for a number years and need to decide if we should discontinue its use, and if retrofitting of past installations is necessary.

Response

Date: 08-13-2014

I had only found the 42" tall railing systems with advance sidewalk. I have emailed Dean Alberson at TTI to inquire whether they have tested shorter variations at 32" with sidewalk.

STH 78; additional guardrail over culvert question 5590-03-30

Question

State: WI
Date: 08-12-2014

Please review the text and
drawing and let me know if the proposed installation is acceptable.

Here is additional information
with a section view.

Notice due to the elevation of the roadway with respect to the elevation of the existing headwall a reinforced concrete section will be added to the existing headwall in order to meet the 10:1 requirement for grading at the guardrail. We did a structural analysis to know that what we have shown will work structurally to retain the earth that needs to be against it for grading. This will also allow us to extend the wings vertically to better grade behind the posts and in which case the posts could be moved closer to the edges of the box, if that will address some of the beam guard deflection concerns. If we meet the 15' horizontal roadway clearance requirement, we still are not meeting typical criteria for placement of the headwall behind the guardrail post installation. For the installation to meet current long span system guardrail installation requirements using the CRT posts the horizontal clearance would only be 14'3" instead of 15'. Horizontal clearance is a controlling criteria. We are asking for an exception to the typical beam guard installation criteria to allow the back of the post to be placed in the same plane as the headwall.
Attachment: http://mwrsf-qa.unl.edu/attachments/b7e55a5cc24348efaf42903a5bc0b6fd.pdf

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Response

Date: 08-13-2014

We have concerns that changing the offset of the posts relative to the headwall from the tested system may increase vehicle extent over the culvert and potential wheel snag on the culvert. In the detail shown, the culvert wingwalls appear to be perpendicular to the headwall, which would further increase the snag concerns.

Thus, we would not recommend the offset shown.

CALTRANS Bridge Rail

Question

State: MN
Date: 08-13-2014

From: Ronald K. Faller [mailto:rfaller1@unl.edu]

Sent: Wednesday, August 13, 2014 1:10 PM

To: Jewell, John R@DOT; Whitesel, David A@DOT

Cc:rfaller1@unl.edu

Subject: Bridge Rail

John and David:

I received an inquiry from one of our Pooled Fund Member States (MnDOT) regarding a CALTRANS bridge railing, specifically B11-54. This concrete parapet utilizes an 8-in. curb/sidewalk in advance of the 27-in. tall vertical RC parapet. A tubular hand-railing is attached to the top of the parapet. What is the height to top of hand-railing? Does the hand-railing contribute to vehicle redirection in the PL-1 or TL-2 tests? Can I get a copy of the test report as well. Thanks!

Response

Date: 08-26-2014

Ron,

Here is the detail for the handrail. The bridge rail you are referring to is the Type 26 and it was never crash tested. I believe it was grandfathered in as a TL-2 bridge rail a long time ago but I can't say exactly how that happened. John Jewell may have more information. That said, we have an ongoing project to test a (hopefully) TL-2 MASH-compliant version of that bridge rail, called the 732SW. All the crash testing is complete on that project and I am currently finishing up the final report. To qualify this bridge rail at TL-2, we conducted the pickup test at MASH TL-3. In that test, it appears the handrail may contribute slightly to vehicle direction and rollover stability as the dynamic deflection of the handrail was approximately an inch. As a side note, we also conducted a TL-3 small car test but had it fail due to high ridedown acceleration when the impact point was the sidewalk edge. Essentially the sidewalk impact caused just enough flail space to be taken up such that the occupant impact occurred near the beginning of the impact with the barrier face, thus resulting in higher than expected ridedown accelerations. It was an interesting phenomenon. Because of this, FHWA concurred that we could use a TL-2 small car test to qualify the bridge rail at TL-2 so we also performed that test. If you would like, I can send you a copy of the report when it is complete, which should be in the next month or two depending on how much time I can devote to it. Please let me know if you would like any more information.

David Whitesel, P.E.

Roadside Safety Research Group

Office of Safety Innovation and Cooperative Research

Division of Research, Innovation and System Information

California Department of Transportation

Cable Median Barrier Statistics

Question

State: KS
Date: 08-26-2014

Kansas was interested in
some information on cable median barrier statistics for crashes (i.e. % pass thrus, number of fatalitites, etc.). I seem to recall an update for ongoing research from this year or last year where some Missouri information crash data was used. The only report I'm aware of that's been completed is TRP-03-275-12 “Cable Median Barrier Failure Analysis and Prevention". We've got a copy of that report. Is there any other data you're aware of for research performed by MwRSF?

Response

Date: 09-04-2014

MwRSF has been involved in a limited number of in-service studies for cable or other barrier types. This type of analysis generally tends to be sensitive and not within the scope of funding of most state DOTs, proprietary companies, or the federal government. Thus, I can't point to any other MwRSF research regarding cable barrier in-service; the closest I can get is TRP-03-265-12, “Test Matrices for Evaluating Cable Median Barriers Placed in V-Ditches", which has simulated trajectories of various vehicles and may help to guide cable barrier placement in V-ditches.

Recently, however, there have been a number of studies regarding the performance evaluation of cable median barrier in various states:

- Florida DOT submitted a paper to TRB regarding CMB performance, indicating penetration rates as high as 18-20% (http://www.dot.state.fl.us/research-center/Completed_Proj/Summary_RD/FDOT-BDK80-977-19-sum.pdf, http://www.dot.state.fl.us/research-center/Completed_Proj/Summary_RD/FDOT-BDK80-977-19-rpt2.pdf, also TRB paper attached –should not be disclosed) that led to a moratorium on cable barrier construction (attached)
- TTI provided a cost-effectiveness and performance evaluation in 2009 (http://d2dtl5nnlpfr0r.cloudfront.net/tti.tamu.edu/documents/0-5609-1.pdf)
- One of the more quoted research reports in Washington in 2003, which really set the ball rolling for cable barriers (http://www.wsdot.wa.gov/publications/fulltext/design/roadsidesafety/cablemedian.pdf)
- Malcom Ray conducted a limited survey in 2003 (attached)
- Auburn University prepared a rudimentary performance evaluation in Alabama (http://www.eng.auburn.edu/files/centers/hrc/IR-05-01.pdf)
- University of Tennessee conducted an in-service performance evaluation (http://www.tandfonline.com/doi/full/10.1080/19439962.2013.812168#.VAhuEWNNiNI; if you would like

additional resources, I can point you to additional papers)

None of the state DOT or independent researcher reviews was as extensive as the MwRSF research, but each does present a snapshot of the state of cable barrier performance in each state. I hope this helps with the consideration of cable barrier in-service performance evaluation studies.

Attachment: http://mwrsf-qa.unl.edu/attachments/6bbd7540eef55be01df86ff5d78bed81.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/b94fa5181c2b0181cd03a7e1fc46a144.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/bae983a9dc37aa049e60182dd1479e23.pdf

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Wood vs steel posts

Question

State: MN
Date: 08-26-2014

I am doing some research for a story about our use of wood vs steel guardrail posts. DeWayne Jones has been very helpful in answering my questions.

I have one other question that perhaps someone else should answer. And if that's not you, will you forward this to that person? Or if it is DeWayne, let me know and I will ask him.

DeWayne provided me with information that indicates steel posts initially cost higher, but they can be recycled and they last longer. The labor to install steel posts is less and crews are safer for that time savings by not being near traffic as long. With the steel posts, One Call is not needed either, resulting in faster response time.

With that said, I am wondering if guardrails with steel posts are stronger and safer than guardrails with wood? Are there any studies that show that?

Thank you!

Response

Date: 08-26-2014

We have done many studies that relate on some level to this topic over the years and did a TRB paper on it specifically last year.

I have attached several reports and papers related to this topic for your review. The TRR paper from last year on wood vs steel posts in the MGS probably has the most direct answers for you, but the others may be useful as well.

The file 'Wood vs Steel Posts.zip' (73.5 MB) is available for download at

http://dropbox.unl.edu/uploads/20140909/18aa46e220d62216/Wood%20vs%20Steel%20Posts.zip

for the next 14 days.

It will be removed after Tuesday, September 9, 2014.

Take a look at this information and contact me if you have any questions.

Long Span Guardrail -- Minimum Lengths with Common Terminals

Question

State: IL
Date: 08-29-2014

Illinois has developed Highway Standard 630106-1 (attached) to implement the MGS Long Span design for crossing culverts. In this design we have shown 62.5 feet of Type A guardrail (MGS with 6'-3" post spacing) beyond the three CRT posts before any other pay item may begin. This means that attachment of a crashworthy end terminal is generally going to add another 37.5 feet of length of need guardrail (according to IDOT pay item definition for these items), plus the gating portion , if any.

We note that a previous question on the consulting website (6/18/2007) inquired specifically about the FLEAT terminal and how it could be used with the long span design. As we read the result, the 37.5 feet of the proprietary FLEAT design plus 25 feet of guardrail item would satisfy the needed 62.5 feet minimum installation beyond the three CRT posts. This was allowed specially for the FLEAT because crash testing in the flared section showed that the FLEAT works for impacts in that region.

We would like to adopt this practice to help make the long span installations more economical and practical, and also wish to explore similar applications for other guardrail terminals to allow for competitive bidding for these terminal applications.

The SRT terminal by Trinity is a flared terminal that includes a version accepted for use with the MGS system. Is it acceptable to use this terminal in a similar layout to that described for the FLEAT system? It does not appear that the length of need point impact was tested in the acceptance for MGS use, but that was considered and agreed to be waived by FHWA.

With respect to “tangent" guardrail terminals, the ET-Plus by Trinity and the SKT by Road Systems are commonly used in Illinois. Neither of these terminals were tested at the length of need point for the MGS version. Could the length of these systems be counted against the required 63.5 feet beyond the three CRT posts? Would the same answer also apply if these terminals were flared at a rate of 1:50 as is the practice to avoid having extruder heads overlap the shoulder area?

Lastly, Connecticut submitted a question on July 5, 2012 regarding the long span installation and you responded soon thereafter July 16th, in which you pointed out MASH testing done on 75' long version of the MGS - we ask, would this also apply to the use of the same flared terminals just discussed above?

Attachment: http://mwrsf-qa.unl.edu/attachments/a09e6b4dc997285a436512aee4db3fe8.pdf

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Response

Date: 09-13-2014

As you note in the email, the MGS long span system was tested to MASH and the implementation of the system was accompanied by recommendations regarding the system length and the amount of guardrail tangent to the long span. Special systems such as the MGS long-span could actually further increase the loading of the barrier system and create higher anchor loads and affect the length of the system and the anchorage. Although it was likely that guardrail lengths shorter than 175 ft could redirect 2270P vehicles impacting at the TL-3 conditions, there was no crash test data to support or recommend the use of shorter lengths at that time. Based on these noted concerns, it was recommended that the minimum installation length of the MGS long-span be set at 175 ft for a long span length of 25 ft. However, if a shorter long span length was used, it was still recommended that the upstream and downstream lengths of the installation including the end anchorage be no less than 62.5 ft beginning at the third CRT post. This length is based on the 175 ft system length that was tested. At that time, there may be a potential to reduce the downstream distance, but this would require further analysis and verification with full-scale crash testing.

Following those recommendations, we were contacted by IaDOT for clarification on the system lengths as well as questions regarding the use of the FLEAT terminal with the MGS long span. At that time we reviewed this question, the FLEAT had been tested near the beginning of length of need with the MGS under the NCHRP Report 350 impact criteria. See attached. Because the FLEAT had been tested under NCHRP 350 in the flared section of the barrier, it was believed that the FLEAT possessed sufficient anchor capacity for the MGS long span even if the flaring of the terminal occurred within the 50 ft of tangent guardrail recommended originally. The overall length of the installation remained 175 ft. Of course, this recommendation was only relevant for NCHRP 350 impact conditions, but it gave IaDOT some additional flexibility in their installations. MASH impacts (with their higher angle and vehicle mass) have not been conducted on the FLEAT or the majority of the other terminal system at the beginning of length of need.

You have requested our thoughts regarding the use of the SRT terminal and its parabolic flare in a similar fashion. There is potential that this configuration would be acceptable.
FHWA acceptance letter HSSD/CC-100, dated August 30, 2007 specific to “NCHRP Report 350 Test 3-35 of the SRT-31" contains details of successful Test 3-35 of the SRT-31. Based on this test, the argument for using the SRT 31 would be very similar to the FLEAT. Thus, it would be acceptable to use the SRT 31 and its associated flare with the MGS long span as long as the minimum system length is still met.

With respect to tangent terminals, the length of the terminal is included in the 62.5 feet required adjacent to the CRT posts in the MGS long span. The SKT system uses a similar anchorage to the FLEAT system that was tested to NCHRP 350 with the MGS. No terminal system has repeated their beginning of LON tests under the MASH criteria. Thus, we have confidence in their ability to anchor systems under NCHRP 350 loading and they potentially can develop the loads for MASH impacts. However, we cannot definitively determine their anchor capacities with respect to MASH loading at this time.

For a tangent terminal with a 50:1 flare, we do not expect to see a large difference in the loading of the terminal anchor. This flare is approximately a 1.15 degree angle and would not have a large effect on anchorage of the MGS long span system.

FHWA has also had discussion regarding the flaring of tangent terminals in a general sense based on NCHRP 350 testing of terminals. They, along with industry and researchers at MwRSF and TTI, determined that 15:1 flares were appropriate for general tangent terminal applications (see attached). However, the use of these more aggressive flares with shorter system lengths or special applications like the long span would likely require further analysis and study.

Finally, with respect to your last question, we have given guidance previously based on reduced system lengths for the MGS long span system with shorter unsupported span lengths. This guidance was based on recent research we had done on the minimum system length of the MGS system, collection of data regarding the capacity of the generic end anchorage we use in our evaluation testing, and the assumption of tangent guardrail. The addition of a flared terminal such as the FLEAT could create potential issues when applied with reduced system lengths. Specifically, there are some concerns that shorter systems would place the CRT's and unsupported span significantly closer to the end terminal. Placement of two critical regions of the barrier in such close proximity could have additional consequences and affect the performance of the barrier system. Thus, we believe more study would be required to evaluate the flared terminals with the reduced system lengths that we developed for Connecticut.

Attachment: http://mwrsf-qa.unl.edu/attachments/3cdc352ae738398fcbb064165893faae.pdf

Welded Vs Continuous Stamped Asymmetric Section

Question

State: WY
Date: 09-11-2014

One of our major suppliers of guardrail components, Universal Industrial Supply out of Utah provide the asymmetric transition pieces as shown in the attached drawings. As you can see, they have cut a thrie beam element and welded a rear flange to the bottom of the asymmetric section. During development of the “transition to the transition," I know Midwest was using a welding section and then found a manufacturer who could make stamped sections. Is this welded section acceptable? Are other states having problems getting the continuous stamped asymmetric section?

Attachment: http://mwrsf-qa.unl.edu/attachments/7196b09c000d1f60179b699cba8b8e50.docx

Attachment: http://mwrsf-qa.unl.edu/attachments/bfedaa857e8e7789e58f0bc4e717ef22.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/39423e454e86e78b6162635667b36465.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/a35e065d4425882f95494368b87909d5.jpg

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Response

Date: 09-13-2014

The asymmetric W-to-thrie beam transition piece shown in the attached photos corresponds to a design variation that was evaluated during the development of the MGS upstream stiffness transition for approach guardrail. In test no. MWT-4, a 2000P vehicle impacted upstream of this W-to-thrie beam transition piece at a speed of 98 .1 km/h (61.0 mph) and at an angle of 25.3 degrees. As the truck progressed into the guardrail during test no. MWT-4, vehicle redirection continued until the front bumper contacted the point of the flat plate extension on the bottom of the transition element. The start of the weldment proved to be a stress concentrator that produced a tear in the W -beam to thrie beam transition section. The tear quickly propagated through the entire segment and ruptured the rail system. Thereafter, all redirection stopped, and the test vehicle moved forward into the end of the stiff transition section where it was brought to an abrupt stop. Thus, this test showed that the current design was unsafe and should not be used on high-speed roadways.

http://mwrsf.unl.edu/researchhub/files/Report188/TRP-03-94-00.pdf

Following test no. MWT-4, MwRSF designed a revised W-to-thrie beam transition piece that is recommended for use when creating approach guardrail transitions with the MGS. This design is attached and has been successfully crash tested to the MASH criteria.

Attachment: http://mwrsf-qa.unl.edu/attachments/99cd4312bfc7ee4adb43f22bb12318c9.pdf

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Guardrail Placement in a Cut Area

Question

State: IL
Date: 10-04-2010

Here is a question that has been posed to me by a designer in one of our Districts. They want to minimize cut of the existing back slope (virtually not touch it), while squeezing in a curb and guardrail along the roadway. This results in the earth slope rising steeply behind the curb and within the deflection space of the guardrail system.

I am suggesting to them to use a concrete barrier, if a barrier is needed here.

However, can you see any way the guardrail might work? I think this probably is not possible because the increased embedment of the posts would lead us to a shorter post in order to compensate for the increased fill. However, there is not room for deflection before encountering the back slope and the deflecting system will be interfered with by that slope. Also, the vehicle itself will encroach into the back slope area, contributing lifting and/or snagging potential.

Attachment: http://mwrsf-qa.unl.edu/attachments/fa18d95533e003dd793697416c139003.jpg

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Response

Date: 10-05-2010
Both the 1:1 and 1.5:1 cut or back-slopes on the upper side of the road would be potentially hazardous and provide an increased propensity for impacting vehicles to climb the unprotected slope and result in vehicle rollover. As such, your group has accurately identified the need to shield the hazard if it cannot be removed, flattened, etc. assuming traffic volumes, speeds, other factors, etc. warrant shielding it.

Placing a standard MGS directly in front of the steep slope would result in the impacted vehicles contacting the slope under the rail as the barrier deformed backward. The guardrail system would likely be more stiff as the built-up soil would provide increased soil resistance for the steel posts in addition to that already provided by the increased fill height located behind the curb section. The back side of the guardrail system would also likely make contact with the back-slope as it deformed during the high-energy impact event.

Although there would exist the possibility for this system to perform in an acceptable manner, full-scale testing would likely be needed to demonstrate satisfactory performance for the MGS with a back-slope starting under the rail and at the post locations. If 12 in. of clear and level terrain (33 in. from rail face) were provided behind the posts, I think the system would likely perform in an acceptable manner with the adjacent 1:1 back-slope shown in the plans.

Unfortunately, it does not appear as though the clear and level terrain can be provided behind the guardrail system. For such situations, it may be necessary to utilize a more rigid barrier system at the base of the back-slope.

Response

Date: 06-11-2013
Could you provide guidance for flatter backslopes? Would the same 12-inch offset behind the posts be

recommended for 3:1, 4:1, 6:1, 8:1 backslopes? Or at some degree of slope, could the toe of the slope be located closer to the post or face of rail?

Response

Date: 09-12-2014

Strut to Median Barrier Slope Question

Question

State: MN
Date: 09-16-2014

Attached is a sketch for a 72" pier protection strut going down to meet a 54" glare-screen median barrier. We are wondering what slope should be used to transition from the 72" strut down to the 54" median barrier? We also are wondering if there needs to be anything special done to go from a 36" wide vertical pier protection strut transitioning to the 54" tall “F" style median barrier.
Attachment: http://mwrsf-qa.unl.edu/attachments/555b57fd15ea26501056e69b55801459.pdf

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Response

Date: 09-17-2014
In general, we have recommended the use of 8H:1V height transitions for permanent concrete barriers in order to reduce the propensity for vehicle gouging and snag on the upper concrete surface. We also noted this vertical slope in a prior Pooled Fund consulting inquiry found below.

http://mwrsf-qa.unl.edu/view.php?id=265

In terms of the location where the 54" tall barrier transitions down to 36" barrier, I might suggest the use of the same 100 ft longitudinal distance recommended in the weblink above and for 54" tall barrier in advance of the multi-pier configuration.

Please let me know if you have any further questions. Thanks!

Guardrail Post Bolt Hole Size

Question

State: WI
Date: 09-17-2014

My question is about placing a slotted hole in the post flange that is larger than what was used in the crash test. This is shown on the attachment. Thanks.

Attachment: http://mwrsf-qa.unl.edu/attachments/bf0bd590095b74ffaa098d69ab25faa1.pdf

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Response

Date: 09-19-2014
In general, it is not a good idea to change a critical element of the design from its crash tested variation. An increased hole size in the critical front flange would reduce the capacity of the steel post to resist bending. Horizontal slots would exaggerate this reduction of post capacity, especially for those located closest to the base plate when moments are maximized. Of course, the overall effect that the slot versus hole has on a barrier system safety performance would depend on how conservative was the design initially. At this point, we have not been provided much information in terms of design/test level, construction details, and actual crash testing results. Further details and information would be needed to further explore this issue.

Alternatively, it may be possible to integrate slots into the rail at splice and/or post locations to improve constructability without decreasing capacity.

Thanks!

Slots in Steel Post and Rail Bridge Rail

Question

State: WI
Date: 09-24-2014

My question is about placing a slotted hole in the post flange that is larger than what was used in the crash test. This is shown on the attachment. Thanks.

Attachment: http://mwrsf-qa.unl.edu/attachments/1a1596e57d67184244a636bafde703c2.pdf

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Response

Date: 09-24-2014

In general, it is not a good idea to change a critical element of the design from its crash tested variation. An increased hole size in the front flange would reduce the capacity of the steel post to resist bending. Horizontal slots would exaggerate this reduction of post capacity, especially for those located closest to the base plate when moments are maximized. Of course, the overall effect that the slot versus hole has on a barrier system safety performance would depend on how conservative was the design initially. At this point, we have not been provided much information in terms of design/test level, construction details, and actual crash testing results. Further details and information would be needed to further explore this issue.

Alternatively, it may be possible to integrate slots into the rail at splice and/or post locations to improve constructability without decreasing capacity. As such, We would recommend utilizing horizontal slots in the tubular rail sections to obtain the desired construction tolerances.

NH 5-3(103)129; Karrow to Mountainside; CN 2017001 - Two Tube Bridge rail to parapet detail

Question

State: WY
Date: 09-25-2014

Would you have any suggestions for this gentleman (Chris) from TDH engineering? He would like to construct a concrete parapet at the end of Wyoming's TL-4 Twin Steel Tube Railing for a project in Montana. It posses an interesting question since our rail cantilevers beyond the post. I thought I did see another state that used some kind of parapet at the end of the steel tube railing, but I don't know if it was secured to the railing. I am interested in the response as well.

Details at: http://www.dot.state.wy.us/home/engineering_technical_programs/bridge/standard_details.html

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I am working on the above MDT project where we are using the 2 tube TL-4 rail (MDT calls it W-830 rail) on a structure, and the road designers are calling for an impact attenuator instead of a bridge approach section due to space constraints off the end of the structure. My thought was that since attenuators can be backed up on concrete parapets, if there is a way to tie the 2 tube rail to a short concrete parapet, that might be a solution in this case. If you have any information about situations where 2 tube rail has been transitioned to concrete rail or parapet I would appreciate it if you could pass those on so we can review. If you have any other experience about utilizing an impact attenuator in conjunction with 2 tube rail (without an intervening bridge approach rail section) that would be great to hear about also. Thanks for any input you can provide.

Response

Date: 09-25-2014
We looked at a somewhat similar problem a while back for Iowa regarding transitioning of the BR27C bridge rail to a concrete parapet. See the discussion and solution at the link below.

http://mwrsf-qa.unl.edu/view.php?id=691

For the Wyoming TL-4 rail you have shown, we would propose a similar solution that attaches the rail to a flared parapet be cutting the tube to match the flare. Because the Wyoming bridge rail uses the tube rails to provide the majority of the redirective capacity of the barrier, unlike the Iowa BR27C rail which has a concrete parapet, we would recommend that base plates be attached to the flare cut tubes at the parapet to allow for anchoring of the tubes to the parapet. You may also want to consider keeping a bridge rail post relatively close to the parapet to limit the loading of the tubes and anchorage where they attach to the parapet. Note that the parapet design would need to consider impact loading from a vehicle as well as sufficient capacity to anchor the tube railing.

Let me know if that gets you going in the right direction or if you need more guidance.

Response

Date: 09-25-2014

I am not quite sure I understand what to do with the bottom railing.

I think they were thinking more in terms of a parapet shown in the following bridge rail end, although in this case, the concrete parapet would be downstream.

http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&railing=129

Response
Date: 09-25-2014
The example I sent was for a combination bridge rail with only one tube and a concrete base, but the concept would be the same for a two tube design. Essentially, we recommend overlapping the parapet with the tubes. We recommend tapering the parapet and then cutting the tubes to match the taper. This is done to ensure that the impacting vehicle doesn't snag on the parapet end.

We have not recommended systems like the one in the link below for the downstream end due to concerns for vehicle snag on the end if the parapet. For upstream ends, the design shown may work because the vehicle is stepping down for the parapet to the tubes. However, on a downstream end, the vehicle would have a tendency to redirect along the front face of the tubes with some components of the vehicle protruding past the front face of the tubes. This could creat vehicle snag as it reaches the parapet end.

Let me know if that clears things up.

Response
Date: 09-26-2014

I was thinking of something like this.

Attachment: http://mwrsf-qa.unl.edu/attachments/0687e78a2d47db52775f5dba635d8420.jpg

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Response
Date: 09-26-2014

I like this idea very much!

Thanks for your help!
