Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

01-01-2013 to 04-01-2013

Tie Down Straps on thin HMA Pavements

Question

State: MO
Date: 12-14-2012

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

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

Questions:

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

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

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

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

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Response

Date: 12-17-2012
1. There are several unknowns with this type of installation of the tie-down strap. using the straps with a mill out as you have shown would prevent loading of the anchors through a layer of asphalt and provide similar anchor capacity to the tested design. However, doing so would require lengthening the sides of the strap. This change in geometry would likely affect the force-deflection and energy absorption of the strap to some degree

and modify the loading of the anchors. The effect of these changes cannot be quantified without further study, but could potentially increase barrier deflections and the anchor capacity as compared to he tested system. Thus, while the potential exists for this type of modification to work, we cannot accurately quantify the impact performance without further study.

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

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

Response

Date: 12-27-2012

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

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

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

Response

Date: 01-02-2013

Responses in Red.

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

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

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

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

We would not be as confident in the second option to mill underneath the barriers and effectively lower the height 3 3/4". There has been very little research done on reduced height barriers with sloped faces for TL-2.

The majority of the reduced height sections for TL-2 have been vertical face designs. As such, we would be more wary of this option, especially when considering high CG vehicles and the potential for barrier climb. That is not to say that it cannot work, but that our confidence is lower than the first option.

Retrofitting brush curb

Question

State: WI
Date: 11-16-2012

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

Are there other concrete retrofit designs?

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

Response

Date: 01-03-2013

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

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

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crash cushion layouts for various situations

Question

State: WI
Date: 01-23-2013

Dear MwRSF, I know that people are probably cringing when I email a question about crash cushions but here I go. I have been digging around trying to figure out how to properly layout crash cushions for point hazards (e.g. bridge piers, overhead signs...) So I sat down and started drawing some situation out and would like MwRSF to review them. I believe that I will need to add this guidance into our design manual because we are using more and more crash cushions. I've included a couple of PDFs for layouts. "Hazard layouts without concrete transitions.pdf" is a layout for smaller hazards that do not need a concrete transition. I'm using this to set up a size limit (i.e. After a certain size fixed hazard a designer needs to add a concrete transition). "Hazard layouts with concrete transitions.pdf" is a set of drawings showing how to layout a hazard relative to concrete transition section. This PDF also has layouts for median and side of the road. "Roadside layout steps.pdf" is showing the process of layout driveway curb limits for a roadside barrier installation. I know I asked a similar question about what angles to use. I believe that MwRSF indicated that a flatter approach angle on the nose of the crash cushion should be used. The problem I have with using a flatter angle is that in opens the median up for u turns and could cause drainage issues at the roadside. If MwRSF could give review and provide comments it would be appreciated.
Attachment: http://mwrsf-qa.unl.edu/attachments/a7fa755328ff4eb89511b80637c90b9b.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/84cdfd6260fe596dfdf725a3b562d4f6.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/e5852a149552c9812a8e4ac437be6bbf.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/9d65978b65e8ee4e6fc5721ff22e44c5.pdf

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Response

Date: 05-10-2013

There are many separate issues that are raised here involving the proper installation of crash cushions across a wide variety of installations. The scope of this inquiry is likely outside what can be answered in this forum.

In order to better address this issue, a problem statement was submitted by WisDOT for the 2013 Midwest Pooled Fund meeting.

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post proximity to underground obstruction

Question

State: WI
Date: 01-24-2013

How close can an underground obstruction be to a MGS post before the obstruction influences post performance?
Attachment: http://mwrsf-qa.unl.edu/attachments/c1b0eaf217b1152042861f2f4863d7c7.pdf

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Response

Date: 02-04-2013
The effect of underground structures on post performance is related to several factors. These factors would include the size and type of the obstruction, the location of the obstruction relative to the post, the orientation of the obstruction relative to the post, and the embedment depth obstruction relative to the post.

For the culvert pipe installation shown on your detail, the effects should be less than if the pipe ran in front of or behind the posts. We would recommend a minimum offset of 12 inches from the post for this type of installation.

TL-5 Median barrier

Question

Date: 01-28-2013

Please accept my compliments on a very interesting and detailed new concept concrete barrier design.

Manitoba Infrastructure and Transportation is investigating options to replace an existing F Shape (TL4) concrete median barrier with a TL5 system. The cross section has yet to be finalised but the Vertical-Faced, Concrete Median Barrier Incorporating Head Ejection Criteria design that MwRSF developed has been part of our assessments. Construction of the new barrier is tentatively scheduled for summer 2013.

To help us with the new system for our Department, I am seeking your assistance to quantify and/or qualify a number of conditions and criteria for the TL5 concrete barrier as it relates to your barrier.

1) In the report on this barrier, it appears that an L4000 concrete mix (28 MPa) was used for both the foundation as well as the barrier itself. a. What is the complete mix design for the L4000 mix? (weight, volume, cement content, water content, aggregate composition, air content, slump, etc.)? b. What would be the recommended concrete mix design for a slip formed installation of this design?

2) What would be the consequences if the bottom of the barrier were not imbedded 75 mm below the finished grade of the adjacent asphalt pavement? I.e., the base of the concrete barrier is at finished grade (compacted aggregate or concrete pad). What size (diameter and length) of, and how many, vertical connecting pins (dowels) would be required to suitably attach the barrier to a concrete pad?

3) We have a number of overhead bridge sign structure supports that are located in a narrow median (approximately 900 mm wide). These supports cannot be removed and consist of approximately two - 220 mm diameter aluminium poles centred in the existing F Shape concrete median (see photo below). What would be the suggested means to provide some protection to vehicles and occupants from these installation using this barrier. E.g. increase the height of the barrier (to what height; at what slope should the top rise e.g.. 10:1, 20:1; would the cross section remain or would it change at the top)? An end anchor system would be required on either side of the structure. The opening could/would be treated in some form using standard guardrail components and hardware such as thrie beam.

4) Are there any special considerations needed for this barrier design to address bridge piers located in the median. These considerations would primarily related to a possible change in the cross section from a symmetrical cross section to an asymmetrical system (possible vertical back face)? A taller system or a system located further from the pier could be considered as possible design options to address vehicle roll. (See the photo below; the median shoulder is significantly wider at the piers.) We do not yet know what access is required to the bridge piers and/or footings.

5) Our local contractors have been contacted regarding this proposed slip form barrier construction. They have indicated to us that a minimum cover over the reinforcing steel should be 100 mm (4 inches). What would be the consequences, or benefits, to this barrier if the stirrups and longitudinal reinforcing were modified to accommodate the suggested 100 mm of cover as opposed to this barrier's design of 65 mm? I appreciate you may not be in a position to answer these questions without detailed analysis and possibly testing but your learned opinion would be appreciated in this regard. Please do not hesitate to contact me at your convenience if you need any clarification. Thank you in advance for your time and consideration of my questions.

Attachment: http://mwrsf-qa.unl.edu/attachments/d9f1111289e15ff28ed962a8bd6a23e9.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/d7358e905a840b42c554f5565006d0c1.jpg

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Response

Date: 01-30-2013

To help us with the new system for our Department, I am seeking your assistance to quantify and/or qualify a number of conditions and criteria for the TL5 concrete barrier as it relates to your barrier.

1) In the report on this barrier, it appears that an L4000 concrete mix (28 MPa) was used for both the foundation as well as the barrier itself.
    a. What is the complete mix design for the L4000 mix? (weight, volume, cement content, water content, aggregate composition, air content, slump, etc.)?
    b. What would be the recommended concrete mix design for a slip formed installation of this design?

The barrier was designed with a minimum concrete compressive strength (f'c) of 4,000 psi (28 Mpa). Cylinder testing indicated that we received a concrete mix with an f'c over 5,000 psi (35 MPa). However, this increase in concrete strength would only result in a 2% increase in barrier strength. Therefore, a minimum f'c of 4,000 psi (28 MPa) is still recommended.

Since the barrier was only 200 ft long, we did not slipform the test installation. Slipforming would have increased the test costs greatly for such a short segment of barrier. As such, the exact mixture we used would not be ideal for slipforming. I would recommend discussing the mixture ratios with contractor / slipformers as they know much more about concrete mixtures than I do. The only important thing about the mixture is that it results in a minimum f'c of 4,000 psi (28 MPa).

2) What would be the consequences if the bottom of the barrier were not imbedded 75 mm below the finished grade of the adjacent asphalt pavement? I.e., the base of the concrete barrier is at finished grade (compacted aggregate or concrete pad). What size (diameter and length) of, and how many, vertical connecting pins (dowels) would be required to suitably attach the barrier to a concrete pad?

We have had discussions like this previously with various State DOTs. I will refer you to our company's Q&A website in which this topic has been discussed and options were sketched. Please visit http://mwrsf-qa.unl.edu and search for Question ID No. 629 from June of 2012.

3) We have a number of overhead bridge sign structure supports that are located in a narrow median (approximately 900 mm wide). These supports cannot be removed and consist of approximately two - 220 mm diameter aluminium poles centred in the existing F Shape concrete median (see photo below). What would be the suggested means to provide some protection to vehicles and occupants from these installation using this barrier. E.g. increase the height of the barrier (to what height; at what slope should the top rise e.g.. 10:1, 20:1; would the cross section remain or would it change at the top)? An end anchor system would be required on either side of the structure. The opening could/would be treated in some form using standard guardrail components and hardware such as thrie beam.

I'm assuming that the concrete barriers are going to be replaced with the TL-5 barrier being discussed here. Yes, end section would need to be present on both sides of the structure. Increasing the barrier height may limit trailer-box roll into the barrier, thus minimizing the possibility of box impacts to the support poles. See below for common practices on height increases. However, the barrier would have to get wider to accomplish an increase in height.

I see two options for shielding this support:

(1) Transition the barriers to single faced sections (again see below). The barrier would need to be brought in front of the poles and may terminate on the downstream side of the poles. This would incorporate two independent barrier segments that are not connected. Each free end would face downstream and be protected from end on hits by the opposite barrier. We refer to this as a “fish scale" scheme as the protection is continuous upstream but open downstream of the hazard. For this option, you may need a wider median as it requires a barrier on each side of the support poles, so a minimum of 9" + 20" + 20" = 49 inches (54" by the time you include space between the poles and the barriers). This option would provide TL-5 protection.

(2) You could use thrie beam elements or structural tubes on both sides of the median to bridge the gap between each barrier end. Of course, the rail elements would need to be tapered (end shoe design for thrie beam) to prevent vehicle snag. This design would not likely meet TL-5 and may not meet TL-4 depending on the rail strength and height location. However, it would provide protection for passenger vehcles.

4) Are there any special considerations needed for this barrier design to address bridge piers located in the median. These considerations would primarily related to a possible change in the cross section from a symmetrical cross section to an asymmetrical system (possible vertical back face)? A taller system or a system located further from the pier could be considered as possible design options to address vehicle roll. (See the photo below; the median shoulder is significantly wider at the piers.) We do not yet know what access is required to the bridge piers and/or footings.

I have seen similar designs to what you are proposing here. Single sided versions of this barrier have been developed by a few State DOTs (sorry I can't find the drawings right now) for uses in pier protection, slope separation, and sign bridge protection (like #3 above with a wider median). Vertical back faces were used in most of these situations. To accomplish this while still keeping the strength, the flat top portion of the barrier was extended backward. Thus, the barrier width remained at 20 in. (508 mm).

Taller systems to reduce trailer-box roll over the barrier have also been configured for pier protection installations. This was accomplished by extending the top sloped portion of the barrier upward. Holding this slope constant ensures the head ejection envelope is not violated. I do not have a grasp on the quantity of trailer-box extent behind the barrier as a function of barrier height, only the general idea that taller barriers reduce the roll and extent of the box. I have heard of State DOTs extending up to as high as 54 in. (1,372 mm).

5) Our local contractors have been contacted regarding this proposed slip form barrier construction. They have indicated to us that a minimum cover over the reinforcing steel should be 100 mm (4 inches). What would be the consequences, or benefits, to this barrier if the stirrups and longitudinal reinforcing were modified to accommodate the suggested 100 mm of cover as opposed to this barrier's design of 65 mm?

If an increase in clear cover is desired, the internal steel would likely need to be increased (larger bars or more bars) to account for the loss in depth. A new reinforcement design would need to be configured such that the barrier strength remained the same as the original, as tested, version.

Termination of the Texas HT steel bridge railing

Question

State: IA
Date: 07-18-2011

Scott and I like the options which flare back onto the parapet at the same elevation. Option 5 may be easier to deal with considering it may use a smaller anchor plate on the back wall. To mitigate concerns for snag increased snag on the posts, it may be necessary to use a minimum tangent length of the tube prior to bending it back to the parapet. Let me know if you have additional questions regarding this matter. Thanks!

Attachment: http://mwrsf-qa.unl.edu/attachments/21c438bcafa06ef856b8e038d0aecc16.pdf

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Response

Date: 07-19-2011

Thanks, Ron. Just one point of clarification: the termination of the elliptical tube is a free end; it is not attached to the concrete. Does this have any impact on your recommendation?

Do you know if the end that TXDOT uses was ever crash tested? (it's a free end as well)

Response

Date: 07-25-2011

I am not aware of any passenger vehicle crash tests being performed on the Texas HT bridge rail. I have contacted my colleagues at TTI to also inquire about any passenger vehicle crash tests on the original system as well as a similar rail on a vertical parapet where a lower rail was added. Based on this inquiry, both TTI and TxDOT have stated that no passenger vehicle crash tests were performed.

Attachment: http://mwrsf-qa.unl.edu/attachments/0f63009ff7a0e10de41554f3fa7aa31f.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/f909276a25c7d6fd1516a51c09fdbdf8.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/c6b07abd72d3265acc8986204a819c82.pdf

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Response

Date: 01-29-2013
Could you share your opinion on the preferred treatment at the trailing end of a BR-27C bridge railing? See both pages of the attachment. Note that in some cases, the trailing end could lie within the clear zone of opposing traffic.

My personal preference is the non-flared version. With this version, would it be further advisable to attach the end of the rail to the concrete with a plated connection? Or perhaps to use a reduced post spacing near the end

Attachment: http://mwrsf-qa.unl.edu/attachments/200e97b1ace0861a96b95b25297af70c.pdf

Response

Date: 01-30-2013

To begin with, I will assume that both the combination bridge rail and the concrete parapet are crashworthy. The tapered/flared concrete end (proposed) is more desirable for downstream end impacts as the non-flared end (original) could potentially cause snag issues during vehicle redirection. However, the flared end (proposed) is drawn such that it leaves the end of the rail open for potential snag issues for reverse direction hits. As such, we recommend using the flared concrete end, but extending the rail to flared concrete end. Thus, snag potential will be minimized for both directions of travel.

A few notes on the design:

(1) The concrete taper/flare should begin on the same plane as the face of the posts (or further back for a more conservative design.) This flare depth will best minimize snag potential.
(2) The flare/taper angle should be gradual / shallow enough to minimized snag. (i.e., 3:1 – 4:1)
(3) There are benefits to bolting the rail to the concrete parapet to ensure stiffness and full lateral capacity. However, if the concrete end is flared and the rail is extended to the flared and cut to match the slope of the flare, the free end of the rail would be supported laterally by the sloped face of the concrete parapet. Thus, attaching the rail to the concrete is not necessary and the rail can remain free.

Response

Date: 04-09-2013

I realize I'm reviving an older question, but could you take a minute and review page 2 of the attached drawing? Specifically, I'm curious if you would consider this design acceptable for both directions of travel (forward- and reverse-direction impacts). The tube railing is cut to match the flare of the concrete with an approximate 1-inch gap between the two.

Please feel free to suggest any enhancements as well.

Attachment: http://mwrsf-qa.unl.edu/attachments/beb5830cb30a14ff9087f12058235e90.pdf

Response

Date: 04-10-2013

Here's another option for you to consider.

Attachment: http://mwrsf-qa.unl.edu/attachments/9705e5670d307c41dd465f6e162a2bad.pdf

Response

Date: 04-26-2013

Both rail termination details you have included should perform well during impacts from either direction. Having the free end of the rail cut with a flare to match the taper of the concrete parapet should minimize vehicle snag during impacts. Also, the flared cut allows the rail to utilize the tapered concrete parapet as a lateral support to insure rail strength at this termination location (after a small lateral deflection the rail would be pressing against the concrete wall). The short distance between the free end of the rail and the first post also helps insure strength at the end of the rail.

If I was to pick one design over the other, I would go with the straight rail section. Thus, end section rail fabrication would be simple as a standard rail segment would just have to be cut to correct flare. Also, the straight rail design forms a more continuous barrier face near the top of the bridge rail. Although the change in barrier profile is rather minimal for the second design you sent me (with the rail bent backward away from traffic), continuity always helps create a smooth, stable redirection.

Response

Date: 04-29-2013

I wanted to get your opinions regarding the termination of the metal tube portion of the Texas HT railing (TXDOT drawing attached). We have a project where we will be using the HT railing on a bridge and using a 44" tall F-shape concrete barrier off both ends of the bridge, and we are developing ideas on how to transition between the two.

Please see the attached PDF. It presents 5 different options for transitioning from the Texas HT barrier (on the bridge) to Iowa's 44-inch concrete barrier (off the bridge). Please provide your comments and/or recommendations regarding the use of each of the 5 options. Note that the top width of our 44-inch barrier is 8-1/2 inches.

Attachment: http://mwrsf-qa.unl.edu/attachments/21c438bcafa06ef856b8e038d0aecc16.pdf

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Deflection for MGS - Distance from the Front Face of Rail to a Fixed Object Hazard

Question

State: WY
Date: 01-30-2013

To all Pooled Fund Members and MwRSF: We are completing our details to implement MGS and I have a question about the distance to fixed object hazards which is very confusing in the roadside design guide. In the old days prior to the definition of working width, we specified the distance from the back of the guardrail system to the hazard as at least the dynamic deflection distance. The new Roadside Design Guide has several values for deflection of MGS. It also has values for working width, which is typically the width of a system (in the case of MGS, approximately 23 inches from the rail face to the back of the posts) plus the dynamic deflection. However the working widths fluctuate all over the board. What I would like to as, is what should we be using as the clear distance from the front face of the rail to a fixed object hazard behind the rail so the fixed object hazard is not struck. I would like to have values for the standard post spacing, half post spacing and quarter post spacing. I don't recall if we talked about deflection in our past review of various state standards for MGS. I would have normally posed the question on the Pooled Fund Website, but I am interested in both MwRSF take on this as well as what other states are using.

Response

Date: 01-31-2013

The deflections and working widths listed in the RDG for the MGS do fluctuate, even for the steel post version with standard 6'-3" post spacing. This fluctuation in the working widths is a reflection of several factors.

1. First, there has been a transition in the soil resistive forces that we use in our full-scale crash tests under MASH. Thus, the original crash testing of the MGS with the 2270P vehicle under 22-14 would have likely used a soil foundation that was less stiff than the soil recommendations that were eventually incorporated into MASH. Thus, there will be some variation of deflection and working width based on the change in the foundation conditions.

2. Second, the RDG presents tests with both the 2000P and 2270P vehicle types. Again the MGS was developed and tested during the transition between NCHRP 350 and MASH. Thus, the change in pickup truck vehicles represents an approximately 13.5% increase in kinetic energy. This change in impact conditions also accounts for some of the variation you are observing between the working widths and deflections in the full-scale testing.

3. Third, the RDG shows deflections for a wide range of MGS systems, including wood and steel post versions as well as several special applications. Thus, the use of different post types, post spacing, slopes, flares, etc… affect the working width numbers.

4. Finally, full-scale crash tests are not an exact science. We have tried over the years to develop test procedures to make crash test results more consistent and repeatable. The current soil standard in MASH is one part of that effort. However, even with these efforts, there is a certain degree of variation from test-to-test that is difficult to

avoid. Thus, full-scale crash tests of two identical MGS systems may result in deflections that vary. This is simply difficult to avoid given all of the potential variation in materials, environmental conditions, soils, and other factors.

While it is clear that deflection and working width data taken from full-scale crash tests can vary for several reasons, we have still not answered your questions on what values you need to consider for your installations. Our advice here would be to review the available data from the crash tests of most similar systems and error on the side of being conservative. For example, if you have an MGS system installed on a 2:1 slope, then we would recommend using the working width guidance from the full-scale crash test of the 2:1 slope. For standard, steel post installations, we may suggest considering a working width of 60 in. The 60-in. working width corresponds with the upper end of the values observed in the full-scale testing and also allows for some tolerance if the soil for your real world installations in not as stiff as the soil currently specified in MASH. For the wood post versions of the standard MGS system, we would recommend that you refer to the crash tests of the specific wood post system and use those working widths if they are increased over the 60-in. For the ½ post and ¼; post spacing versions of the system, we would recommend using the tested working widths listed in the RDG.

Let me know if this addresses your concerns and if you have further questions.

Response

Date: 02-01-2013

Ohio introduced MGS to our standards just a couple of weeks ago. This is the table for the deflection values for our designers to accommodate. The minimum barrier clearance values are from face of rail to the hazard.

Links to our drawings if anyone wants to see what we've got so far:

“MGS" drawings
http://www.dot.state.oh.us/Divisions/Engineering/Roadway/DesignStandards/roadway/Pages/StandardConstructionDra

Attachment: http://mwrsf-qa.unl.edu/attachments/13a0478d8f8d89a86b7d397ecbc7b8b1.jpg

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Response

Date: 02-01-2013

Wisconsin uses working with. For MGS we measure from face of rail.

Most designers just draw a line on the plan for beam guard. So they don't have a good idea where the post are.

Response

Date: 02-01-2013

So what values do you use then for each post spacing I mention?

Response

Date: 02-02-2013

Because we use weaker wood I had to use bigger working with values than what MwRSF recommended. The chart below is on the backside guidance of our standard drawings.

Attachment: http://mwrsf-qa.unl.edu/attachments/b7fbea420d833d8d880ce03d8465c771.jpg

Ballasting Work Zone Devices

Question

State: MO
Date: 01-30-2013

MoDOT encounters a wide variety of ballasts across a number of work zone devices and isn't always sure that they're acceptable. The Roadside Design Guide (RDG) does not have a great deal to say on the issue of ballast for work zone devices. It speaks only of channelizing devices and says, “The ballast should not present an obstruction if the cone is struck" and then goes on to list double-stacking, heavier devices, sandbags or recycled tire rings as acceptable solutions. The only other mention is that rocks and chunks of concrete are not acceptable.

Questions:
Are there any general rules of thumb (material? Height? Weight?, etc.) for ballast that would allow an agency to conduct a visual inspection and deem an installation appropriate?
Is the RDG guidance enough?
Do the rules apply to other devices such as moveable barricades and X-base portable sign supports?
Are the ballasts shown in the attached .pdf acceptable?

Attachment: http://mwrsf-qa.unl.edu/attachments/aa3791f5a18fd3d65637d46a851b338b.pdf

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Response

Date: 02-28-2013
We are not aware of any formal guidance on ballasts for channelizing devices, moveable barricades, nor X-stand portable sign supports. Based on the guidance in the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, the maximum stub height is 4 inches. Therefore, the height to the top of the ballast should not be more than 4 inches above the roadway surface. In addition, the ballast should not be elevated nor should it interfere with the breakaway features of the device. In addition to the RDG guidance, ballasting should be deployed in a similar manner to either the tested configuration or the manufacturer's recommendation for ballasting

Common ballasts we have seen are sandbags and recycled rings as you have shown in the attached pictures. I would have concerns with placing recycled rings around the center of an X-stand as shown on the second page of the attached photos. This ballast placement could potentially affect the performance of the breakaway mechanism when impacted.

These general rules could be applied to all work zone devices, such as channelizing devices, moveable barricades, and X-stand portable sign supports.

Barrier Scupper

Question

State: WI
Date: 01-30-2013

Dear MwRSF, Could you take a look at these drawings. I'm not quite familiar interpreting the chart that LRFD manual on snag. I found it hard to believe that opening did on influence the roll over (see FHWA letter). ____ Subject: Barrier Scupper Please see attached details for US 41 typical cross section for the Duck Creek to Lineville mainline. Barrier will be placed along the median and the outside adjacent to the existing frontage roads. We are proposing scuppers to be placed as curb flankers at the sag curves along the outside barriers. We would be proposing one scupper about 30 to 50 feet upstream from the low points. Therefore, there would be four scuppers at each low point. The scuppers are being proposed as a “failsafe" measure to drain storm water in case the roadway inlets at the sag condition get clogged or can't drain out. Inlets and storm sewer is designed for the barrier-contained roadway. We do not plan to add scuppers along the continuous grade upstream of the sags. We would ideally be looking at a 4-inch x 18" scupper at the base of the 42-inch SSCB. Due to construction means and methods, we could consider a 3" high scupper x say 24" length. We do not believe the 3-inch or 4-inch scupper height introduce a snagging issue. Please advise if you have any concerns or require further justification. We are trying to include with the February 1 PS & E. Thanks for your attention. I look forward to hearing from you.
Attachment: http://mwrsf-qa.unl.edu/attachments/32f6ce338489eafa83590dc8070c37d9.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/ab2a28d9cbe52b7096865eb9e63f2702.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/7e675ace1558ffb4e0817d2fd6d315b3.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/903a66d673d079dbe81d90d4e81ccf37.pdf

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Response

Date: 01-31-2013

From Pooled Fund Consulting Summary Nos. 228, 263, and 439, it would appear that your hole heights of 3 or 4 in. would fall below our recommended upper bounds. As such, concerns related with wheel interaction with hole would likely be very limited.

Ron

MGS Post Leave-out Size in Mow Strips and Other Pavements

Question

State: WY
Date: 02-07-2013

TTI did some testing regarding "leave outs" for conventional w-beam guardrail (27 3/4" Height). The recommendation was to provide at least 7" free on the back side of the post. Are these recommendations valid with MGS or should the dimension be increased somewhat until further testing is completed. Likewise, the Pooled fund performed testing on posts in rocky conditions. Are these recommendations valid for MGS? Attached is a copy of a 2004 FHWA Memo regarding the subject.
Attachment: http://mwrsf-qa.unl.edu/attachments/f969f8932ccbebd03b67b5805632902e.pdf

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Response

Date: 02-07-2013
I believe that this issue has been dealt with previously in Question 171 on the website.

If this does not fully address your question, let me know.

http://mwrsf-qa.unl.edu/view.php?id=171

Thanks

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modifying TTI TL-2 31" guardrail

Question

State: WI
Date: 02-08-2013

I noticed TTI has a TL-2 beam guard transition to rigid barrier. I was wondering if it is possible to modify the transition to use nested rails verse 10 gauge thrie beam and using 12 inch deep blocks? I assume that the transition is the area in the square. Do you think that there is a minimum length of beam guard prior to the transition like the MGS TL-3 transition? report number FHWA/TX-12/9-1002-8
Attachment: http://mwrsf-qa.unl.edu/attachments/9950ec41bfae0f6be9e92e4502db92f1.pdf

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Response

Date: 02-11-2013

I do not forsee any issues with extending the blockout to 12" from the tested 8" blocks. This would only help minimize vehicle snaging potential.

Going from a 10 ga. thrie beam section to a nested 12 ga. section has been considered allowable in previous designs. Nested 12 ga. thrie beam has about a 50% increase in the cross section area, elastic section modlus, and plastic section modulus in comparison to 10 ga. thrie beam. Thus, it is a stronger section. The only issues that I can see arise from this switch would be increased pocketing at the connection between the w-to-thrie segement and the nested thrie. However, due to the short length of the thrie beam (only connects the bridge parapet to the 1st post) and the fact that this is a TL-2 system, I do not have much concern for an increase in pocketing.

To answer how much w-beam is necessary upstream of the transition, I will refer to and amend the 3 criteria listed on page 154 of the MwRSF TL-3 transition (TRP-03-210-10).

1. the first required a 12.5 ft seperation from the assemetrical w-to-thire transition segment and the downstream stroke of the energy absorbing terminal. this requirement will stay the same. However, the stroke of a TL-2 energy absorbing terminal will be less than that of a TL-3 terminal. Thus the total distance upstream of the transition segment will be less.

2. the second requirement called for a minimum of 46'-10.5" upstream of the assemetrical w-to-thrie transition segment. This distance was to ensure proper anchorage of the system. With the drop down to TL-2, this distance can be conservatively reduced by 12.5' to a distance of 34'-4.5".

3. the third requirement called for a 25 ft distance between the assemetrical w-to-thrie transition segment and the start of a flared guardrail terminal. Due to the reduction in impact energy with a TL-2 system (compared to TL-3), this distance can be conservatively reduced by 6'-3". Thus, a distance of 18'-9" should be seperating the transition segment and the start of a flare.

Guardrail connection to low fill culverts

Question

State: NE
Date: 02-14-2013

TRP-03-114-02

This research shows the guardrail connection to Low fill culverts.
The posts connected to the culvert are half-spaced.

The diagram of tested guardrail shows 6 – half-spaced posts prior to and beyond the culvert.

Is this half-spacing required for this full distance? Or can we start the half-spacing at the culvert?

Response

Date: 06-04-2013

I had a significant amount of discussion on this issue with the Kansas DOT and have been providing feedback on their CAD details. When the KsDOT details are complete, I had planned to send them to the Pooled Fund members for discussion. There is a lot of history on the half-post spacing issue for metric-height rail. The KsDOT details were extended to MGS where half-post spacing is typically used.

I have enclosed some of our prior correspondence between MwRSF and KsDOT on the MGS attached to culvert slabs. I also accessed the KsDOT standard details and attached the two most current plans on this matter. Let me know if you have further questions after reviewing the prior email discussion. Thanks!

Attachment: http://mwrsf-qa.unl.edu/attachments/8642fdd6760804bd3042d79aa7ff2df4.zip

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MGS Thrie Beam Curb

Question

State: KS
Date: 02-26-2013

KDOT has a project where we are installing corral rail on a low fill culvert. We will be attaching MGS guardrail to the corral rail and have some questions concerning the 4" edge curb we would typically install with our bridges when we attach guardrail to the corral rail. Please see the attached standard drawing for the curb location. The project is scheduled to go to letting in the near future and I would like to discuss a few alternatives we are considering with you. Please give me a call when you have an opportunity.

Response

Date: 02-26-2013

Over the last few months, we have been discussing KsDOT Drawing No. RD613A which pertains to a thrie beam approach guardrail transition with the new, simplified, steel-post MGS stiffness transition. This combined system is depicted with a lower 4" wedge curb under the first 12' 6" segment of nested thrie beam rail and adjacent to the concrete corral rail. An additional drawing, No. RD611A, provides details for the MGS.

The region adjacent to the bridge end is configured with half-post spacing of three W6x15 steel posts. Our original MWT test series utilized similar posts in this region but with the thrie beam attached to a thrie beam bridge railing that also utilized an upper channel rail. The upper channel rail was carried onto the first post or two and tapered down to shield an exposed end. When this approach guardrail transition is anchored to a concrete end, the upper channel rail would not exist, thus slightly reducing the lateral stiffness of the first W6x15 post or two.

Over the years, thrie beam AGTs with half-post spacing near the bridge end have been successfully crash tested and evaluated (or even grandfathered based on other prior testing) when attached to: (1) thrie beam bridge railings with additional upper tube or channel rails as well as W6x15 posts (Missouri and FPL systems), (2) concrete parapet with tapered end sections, a backup thrie beam, as well as large wood posts (CALTRANS system), and (3) NDOR concrete bridge end with good tapered geometry and larger steel posts at half-post spacing and one large span with simulated post via a horizontal tube and offset blockout. Per my recollection, these systems did not utilize a lower concrete curb and met NCHRP Report No. 350 impact safety standards. Although the upper channel/tube is not used with first two W6x15 steel posts, I believe that this half-post spacing system would be crashworthy with or without a lower concrete curb as long as a reasonably tapered concrete end were utilized to mitigate concerns for wheel contact and snag as well as any subsequent vehicular instabilities that lead to rollover.

In the late 90s, an Iowa thrie beam approach guardrail transition system with steel posts at quarter-post spacing near bridge end and with a lower 4" concrete curb was successfully crash tested under NCHRP Report No. 350. Later, this same thrie beam AGT with curb was successfully crash tested under MASH. Unfortunately, a similar version of this transition did not provide satisfactory safety performance when evaluated without the curb under NCHRP Report No. 350. More recently, another MASH crash test was performed on the identical system to the original Iowa transition but without the curb, and again the test results were unsatisfactory. As such, most quarter-post spacing designs adjacent to bridge end may likely require the use of a lower concrete curb. We do have a successful test on a thrie beam transition with quarter post spacing and without the curb. However, this system attached to a single-slope median end and used thrie beam on each side of the parapet.

I have also attached our comments that were previously provided to Ohio and Iowa on a similar issue. After you review the attached information, please feel free to call me to further discuss. We can set up a time for the call as well.

Attachment: http://mwrsf-qa.unl.edu/attachments/a302a0089237b8f098a604216a232c3e.zip

Attachment: http://mwrsf-qa.unl.edu/attachments/69dbcf3908aef5e4ce7bb5e1b459869b.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/e2b6150235ca2dc944fa0105b09aa739.pdf

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Design Forces for Traffic Railings

Question

State: MN
Date: 03-01-2013

It was mentioned during the meeting that the new recommended minimum barrier height for TL-4 barriers is 36". Below is shown a table found in the AASHTO LRFD Bridge Design Specification. Table A13.2-1 - Design Forces for Traffic Railings Is there a similar table which shows equivalent values for MASH? If not, is there a corresponding equivalent load for the 36" height that is different from the current 54 kip?
Attachment: http://mwrsf-qa.unl.edu/attachments/6026a67e5bb6ec9b19f8b18624b40ece.jpg

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Response

Date: 03-02-2013
The short answer to your question is… no, there is not a complete bridge rail design table for MASH similar to the one found in AASHTO's Bridge Guide. However, there are multiple pieces of this table to be found in various testing reports and journal papers. For example, TTI published a journal paper through the Transportation Research Board that summarizes recent MASH TL-4 testing into 32" and 36" tall parapets. Within this paper, they recommend a minimum height of 36" for TL-4 rails and a lateral force capacity of 75 kips. A reference to this paper is below:

Sheikh, N. M., Bligh, R. P., and Holt, J. M.,*Minimum Rail Height and Design Impact Load for Longitudinal Barriers that Meet Test Level 4 of MASH*, Transportation Research Record, Journal of the Transportation Research Board, No. 2309, January 2012.

MwRSF agrees with the 36" minimum rail height as our own crash simulations indicated the minimum height to be 35"-36". Additionally, both MwRSF and TTI have conducted MASH TL-4 impacts on 32" tall F-shape / Jersey shaped barriers that resulted in the 10000S vehicle rolling over the barrier. However, MwRSF feels that the 75 kips design load may be on the load end of the range. We have been designing MASH TL-4 barriers to a capacity of 90-100 kips based on numerical analysis, simulation, and crash testing results. Now, clearly, TTI's barrier with an 80 kip capacity successfully redirected the vehicle, but the calculated 80 kips value may vary depending on who is running the analysis and the methods applied. As we discussed over the phone, there are multiple ways/assumptions to calculate concrete bending strength and barrier strength. For example, including the compression steel in the bending strength calculations (TTI has historically only included the tensile steel, so I would assume this to be true of their 80 barrier design). Thus, their 80 kip design strength may calculate out to 85-90 kips if both layers of reinforcement were included in the design. Additional varying ways/assumptions made during calculations may include reduction factors, concrete strength, and barrier depth for irregular / non rectangular barrier cross sections. So, to sum up this paragraph, we are not saying that 75 kips is incorrect, but rather that it is on the low end of the MASH TL-4 design strength range and MwRSF is more comfortable with a more conservative 90-100 kip design strength… unless of course the barrier has been crash tested and shown adequate.

Similar to the above discussion, MwRSF has long felt that the design capacities listed in by AASHTO are on the low side. A complete analysis of TL-5 loads was previously conducted and documented in the attached report. From this analysis, the TL-5 design load was determined to be 210 – 225 kips. Note, there was no change between NCHRP report 350 and MASH concerning TL-5 testing criteria. Of course, it would follow that AASHTO's TL-6 design load 175 kips was also low if the TL-5 design load is over 200 kips already.

MwRSF also feels the AASHTO TL-3 design load was on the low end of the range. For NCHRP Report 350 criteria, we had regularly witnessed TL-3 impact loads between 55 and 70 kips. Under MASH, the pickup truck increased in mass and the impact angle was also increased from 20 to 25 degrees. Thus, TL-3 impacts have an increased Impact Severity and would be expect to impart higher loads to the bridge rail. Subsequently, MwRSF has used a 70-80 kip range when designing MASH TL-3 bridge rails.

Hope this helps. Let me know if you have any questions.

Attachment: http://mwrsf-qa.unl.edu/attachments/bb0ff0def3b2d17efa7df24b6e4f0f92.pdf

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4" Edge Curb (MGS Guardrail Approach Transition)

Question

State: KS
Date: 03-01-2013

After our meeting on Wednesday I checked the Nebraska and CALTRANS transition design and found the following information (See attachments for more detail). See page 3 in the Caltrans attachment and pages 3 and 4 in the Nebraska attachment. 3'-1.5" - KDOT distance from beginning of bevel to center of 1st post ? - Nebraska – Unclear from details, but appears to be similar to Caltrans (i.e. less than 3'-1.5") 33" (953 mm) – Caltrans distance from beginning of bevel to center of 1st post 3" KDOT – distance from end of bevel to back of rail 14" (350 mm) Nebraska – distance from end of bevel to back of rail 5" (125 mm) Caltrans – distance from end of bevel to back of rail From this information it appears KDOT's 1st post is offset a greater distance from the bevel than the other two designs and the end of the bevel is located closer to the back of the thrie-beam than the other two designs. According to our discussion on Wednesday it's my understanding KDOT should not remove the curb in place with the transition design we currently use. Is this correct?

Response

Date: 03-01-2013

The Nebraska AGT detail uses a distance of approximately 37.5" based on center of splice to first simulated post via tube rail and blockout. The slope and lateral offset of tapered concrete end has worked well. An earlier version with less slope and offset was tried but resulted in excessive wheel snag and floorboard deformation.

Thus, it would be good for KDOT longitudinal distance to be similar or less than others.

It would be beneficial to have longer flatter flare with more offset that short steep flare with minimal offset. A larger lateral offset reduces tendency for wheel to impact on end of concrete. A flatter flare reduces scrubbing forces along the tapered section as well as helps to prevent a rapid wheel redirection which can contribute to vehicle roll during redirection. In other words, a wheel snag on the front face of a short taper may behave like a wheel contacting the upstream end.

For half-post spacing design, I believe that it could be installed with or without curb if end section contact is reduced, taper design is conservative, and distance to first post within range of those noted. Knowledge of crash tests of several systems gets us to this general opinion.

We also noted that standardization of concrete ends with half- and quarter-post spacing designs and w/ and w/o curbs would be desirable in future. Some MASH testing with 1100C and 2270P vehicles may possibly be necessary.

Concrete Barrier Design Guidance with End Sections

Question

State: MN
Date: 03-06-2013

As we discussed in our meeting last Thursday, attached are two barrier details for you review. As a reminder, we would like to know what your design methodology is (assumptions and process) and what recommendations you might have for both the interior and end regions of the barrier.

Each half of the split median barrier (Fig 5-397.131) is treated as TL-4.

I also have a question:

Could I get a PDF copy of the*Guidelines for Attachments to Bridge Rails and Median Barriers (February 26,2003)*?

If you need any more information, please let me know.

Thanks again for the help,

Attachment: http://mwrsf-qa.unl.edu/attachments/e1045fde7b54495d76c3fc30b20e4a18.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/46ebd8b599bc95244869b802fc7841ce.pdf

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Response

Date: 03-18-2013

I conducted a full Yield Line analysis on one of the bridge rails you sent, and I tried to list / show all of the assumptions that I made. I attempted to be as detailed as possible in order to show every step. Take a look at the attached file. Is this what you were looking for in regards to methodology of calculating barrier strength?

Let me know what questions you have.

Attachment: http://mwrsf-qa.unl.edu/attachments/28908854d224057a4714f6b74d30c426.pdf

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Response

Date: 04-19-2013

Sorry it has taken so long to get back to you. We wanted to arrange a phone conference to discuss the barrier design, but because of the schedule of the others involved with the barrier design question, we've decided to try and resolve our questions using e-mail. If a phone conference become necessary, we can try and arrange one.

Again, thank you for helping us with our barrier design. We appreciate your time and expertise in this matter.

After reviewing the design you provided, I had some questions concerning some of your assumptions.

First, the design assumes that the rebar in the barrier yields both longitudinally and vertically, front and back. Because the bars are so close to the compression face, shouldn't a yield check be performed to make sure the bars yield as assumed?

Second, the hook on the dowel into the deck does not appear to be fully developed, should this be accounted for when calculating M_C for both the interior and exterior regions?

Third, a ϕ of 0.90 was used to modify the barrier resistance, where does this value come from? As I read AASHTO ϕ is equal to 1.0 for extreme event cases.

Last, We have always assumed that the longitudinal bars need to be fully developed on both sides of the yield line. Some of the longitudinal bars in the barrier are underdeveloped for the end region by this assumption, should this be considered when calculating the capacity of the end region?

Attached is the original e-mail and pdf of the calculations you sent me for your reference.

Thanks again for your help,

Response

Date: 04-29-2013

I have answered your questions in RED below.

Sorry it has taken so long to get back to you. We wanted to arrange a phone conference to discuss the barrier design, but because of the schedule of the others involved with the barrier design question, we've decided to try and resolve our questions using e-mail. If a phone conference become necessary, we can try and arrange one.

Again, thank you for helping us with our barrier design. We appreciate your time and expertise in this matter.

After reviewing the design you provided, I had some questions concerning some of your assumptions.

First, the design assumes that the rebar in the barrier yields both longitudinally and vertically, front and back. Because the bars are so close to the compression face, shouldn't a yield check be performed to make sure the bars yield as assumed?
I use an Excel spreadsheet program to calculate the bending strength of reinforced concrete cross sections. It calculates the strain distribution throughout the entire cross section and relates that to the estimated stress in the steel (assumes elastic, perfectly plastic behavior). Thus, the program should have checked for yielding and calculated all stresses according to strain at each depth.

Second, the hook on the dowel into the deck does not appear to be fully developed, should this be accounted for when calculating M_c for both the interior and exterior regions?
I assumed adequate anchorage to develop the yield strength of each bar. Yield Line Theory requires that the barrier deflects/bends/yields to absorb energy and balance out the energy of the impact. If no yielding occurs, the analysis procedure would not be valid.
If a dowel will not develop full yield strength, I would recommend altering the bar / hook details. Of course, the development lengths found in ACI 318 are conservative in nature and designed for static loading. Under dynamic loading, failure stresses are typically increased. Thus, often times we can rely on field proven or crash tested embedment/anchorage designs.

Third, a ϕ of 0.90 was used to modify the barrier resistance, where does this value come from? As I read AASHTO ϕ is equal to 1.0 for extreme event cases.
The 0.9 factor comes from ACI 318 for bending strength. We typically use it to give some safety factor to designs, but you may elect not to.

Last, We have always assumed that the longitudinal bars need to be fully developed on both sides of the yield line. Some of the longitudinal bars in the barrier are underdeveloped for the end region by this assumption, should this be considered when calculating the capacity of the end region?
My answer here will mirror what was said above… yield line requires the full yield strength of the reinforcement. Thus, it's easier to just extend longitudinal bars to obtain the proper development length. When designing end section reinforcement, we specify longitudinal bar lengths that span the critical length, an additional foot or two for conservatism, and the required development length (or splice length if being splice to interior section reinforcement).

Attached is the original e-mail and pdf of the calculations you sent me for your reference.

Coring concrete barrier

Question

Date: 03-06-2013

We have a project that did not cast enough concrete cylinders to meet our QA/QC requirements.

I was asked if it is O.K. to take a 6" core sample out of our single slope barrier for strength testing.

My initial thought was this is O.K. provided that:

The sample did not hit the same rebar.
The core hole would be refilled.
If possible, sample from the non-traffic side.

I wanted to check with you to see if there was something I was forgetting.

Response

Date: 03-07-2013

I am good with your approach as long as you follow your guidelines.

Cadd Drawing for WSDOT General Missing Installation Detail

Question

State: WI
Date: 03-14-2013

Can I get all the CADD drawings for General Missing Post Retrofit Drawing from TRP-03-266-12? I'm looking for the drawings on figuires 69 through 72 Thanks

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Response

Date: 03-19-2013
I have attached the General Missing Post Installation Detail drawings (Figures 69-72 from report TRP-03-266-12)
Attachment: http://mwrsf-qa.unl.edu/attachments/ff22e4dc6f0f109c9e4041258ffa97d7.pdf

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Response

Date: 03-21-2013
Here are the actual AutoCAD details.

Attachment: http://mwrsf-qa.unl.edu/attachments/78c5e5e742baddec70b42a3dfe1b0746.dwg

Attachment: http://mwrsf-qa.unl.edu/attachments/12c402e9b58b937f654d0927191de606.DWG

Attachment: http://mwrsf-qa.unl.edu/attachments/4aaa1191ab9db44cf63d28c8b089153e.DWG

Attachment: http://mwrsf-qa.unl.edu/attachments/3684e1f5391c8a9f4d50bb3b50e8f1f4.DWG
