Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

07-01-2010 to 10-01-2010

Tie-Down Strap for Temporary Barrier

Question
State: WI
Date: 07-02-2010

I was reviewing MwRSF's detail for the temporary barrier tie-down strap. I have a question about a plate installed near the bottom of the connection pin (see attached drawings).
How is this plate installed? If it is welded to the connection pin, I don't think the connecting pin can be installed. I don't think that the plate can be installed after the tie-down strap and the barriers are installed (not enough room to work).
Looking at Iowa's concrete barrier detail, they don't show the plate on the connection pin.
If the plate is needed what is its' purpose? I don't believe that it is needed for the double shear connection (i.e. the double shear is provided by the second set of loops).
Any information you can provide would be greatly appreciated.

Attachment: http://mwrsf-qa.unl.edu/attachments/c615f8a23e79526dec1e46733f22721e.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/a269732e3cc2bca7256c57b38272308f.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/017ecc98bc6e7a6a1ee7f6633e3323b7.pdf

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Response
Date: 07-02-2010
The plate on the bottom of the connection pin is not welded. It slides onto the bottom of the connection pin and is held in place by the bolt.
In order to install it with the strap tie-down, you have to lift the strap up, put the plate and bolt on the connection pin, and then lower the strap back down. The strap can then be bolted to the drop-in anchors or secured with wedge bolts.
The system was tested with the bolt and plate in place. Because this system relies on loading of the connection pin to restrain the barrier, we believe that the retention plate and bolt are necessary to prevent the pin from pulling out of the loops. I don't believe that the bolt has sufficient capacity to prevent the pullout of the pin under high loads. Thus, the plate is likely necessary.

ZOI

Question

Date: 04-21-2010

I was wondering if you'd be able to send me your report (Guidelines for Attachments to Bridge Rails and Median Barriers: regarding the ZOI) for consideration in my review of a recent submittal for a continuous CRB median barrier that tapers up to cast-in-place 1350mm high (with a vertical face) near the location of bridge piers behind the median. I am no longer with Equilibrium and am now working on a major bridge project reviewing engineer's submittals for a different project.
The divided highway is a 90 kM/hr high use one, and I have personally never seen a Vertical face barrier of 1350 high with a 453 minimum clearance (measured from traffic side to face of pier behind) ZOI behind it (610 is noted as being preferred).
In general cases, should the geometry of the vertical 1350 height face beyond the physical obstructions and the taper zone back to the typical CRB height be defined on drawings? Is 453mm an acceptable minimum ZOI?
If you can send the document by PDF, it'd be appreciated. Let me know if you have any questions, or if the above is unclear.

Response

Date: 04-27-2010
I have enclosed a copy of the requested report. Please note that the ZOI information mostly pertained to test levels 3 and 4. Information for TL-5 was not determined nor provided therein. However, as barrier height is increased, the ZOI would decrease for TL-3 and 4 conditions.
Various height for rigid parapets have been used across the U.S. For TL-5 barriers, it is common to use 42" tall parapets. In addition, it is not uncommon for States to use 51 to 54" tall parapets when shielding objects or for additional glare screen protection.

Response

Date: 07-07-2010
My responses to your recent email are inred

Some questions related to ZOI and traffic barriers;

1. Treatment of CRB placed up against MSE (concrete panel) walls parallel to traveled highways (I.e. Are barriers even needed, should the MSE wall be designed for Impact, or just be designed for repair, panel replacement)

**Does CRB stand for a permanent or temporary concrete barrier " either precast or cast-in-place? Regardless, MSE walls would not need to be shielded unless done so to: (1) prevent vehicular impacts into MSE walls located within clear zone if the crash results in serious safety risks to motorists; (2) prevent significant repair costs to MSE wall panels, if found to occur; or (3) prevent structural damage to highway/roadway infrastructure located above as well as to surrounding motorists
- adjacent and above.
**It should be noted that TTI researchers are currently conducting a research study pertaining to vehicular impact into MSE walls. I do not have any results from this study but would recommend that you contact Dr. Roger Bligh at TTI for further details.

2. Some of our drawings show a 1.0m sliding distance for divided highway precast CRB's. If the sliding distance is reduced at overpass columns in the centre, should there be a transition detail from free to fixed? (Current details seem to show a rising of the height to vertical 1300mm high barriers).

**If temporary or portable concrete barrier are installed in a free-standing manner, then the location of discrete fixed objects on the back side could have serious consequences. Free-standing. portable concrete barriers move laterally when impacted. Vehicle redirection occurs as a result of the inertial resistance of the barrier, the axial tension developed throughout the long, inter-connected barrier system, and the friction developed between the barrier base and the support surface. If barrier movement is restricted at discrete locations, vehicle could pocket into the barrier, snag on barrier components, override the barrier, become unstable upon redirection, etc. Depending on the location of the fixed object, transitioning of the barrier system from free-standing to fixed may be required. Some barrier systems may have options for transitioning the lateral barrier stiffness, others may not.
**I am not sure how the rise in barrier height corresponds to the placement of hazards and free-standing and rigid barriers. Can you provide further details regarding the situation to which you refer?

3. Do you have any information on the California 60G barrier Design, and what levels of Crash testing it meets (I.e. CAN/CSA-S6-06)?

**CALTRANS has conducted significant research on a family of single-slope concrete barriers. The research results from these crash testing programs are contained on two different locations of their website. Actual research reports and crash videos are available. I will ask that one of my colleagues sends to you the links if you are unable to locate them.
http://www.dot.ca.gov/research/researchreports/dri_reports.htm
http://www.dot.ca.gov/research/operations/roadsidesafety/index.htm
**Scott " do you have any additional information on the Type 60G barrier?

4. Are you familiar with the ZOI TL-4 of 230mm from Keller, Sicking, Polivka, and Rohde, feb 26-2003 document: do any of your findings disagree with this?

**I do not understand your question. MwRSF prepared a TL-4 ZOI chart for concrete parapets based on a review of research findings available at that time. No new study has been performed to review and/or update the prior findings. As such, they stand as prepared until further research is funded.

5. Is it normal practise to reduce shoulder widths at underpass column support locations on divided highways (>80kM/hr): what is the absolute unsafe minimum that should be accommodated in these types of situations.

**Unfortunately, I do not have an answer to this question and must defer to any guidance provided within the AASHTO document entitled, "A Policy on Geometric Design of Highways and Streets."

Cable to W-beam Transition

Question

State: IA
Date: 06-29-2010

I would like your opinion regarding a construction issue with Iowa's cable guardrail to w-beam transition (which is now voided). The standard drawing for this transition (http://www.iowadot.gov/erl/archives/2009/april/RS/content_eng/re84.pdf) is based on the South Dakota design.
"â€¹Ã...â€œCase A' on the drawing allows one of the transition brackets to be placed on the w-beam end of the w-to-thrie transition piece. As is clear from the drawing, especially in the plan view on sheet 2, this configuration has proven very difficult to construct; the downstream post and blockout interfere substantially with the path of the cables as they travel from the transition bracket to the end anchor.
In your opinion, should we allow "â€¹Ã...â€œkinks' in the cables as they travel around the post and blockout? If not, would we be able to adjust the location of the transition bracket and the end anchor to provide a straight line of travel for the cables?

Response

Date: 07-08-2010
We have looked at your details and have a few comments and responses to your questions. We looked at several options for Case A in your details.
A total of four solutions were investigated. The first solution consisted of allowing the wire rope to bend around the post at the midpoint between posts at standard spacing. However, analysis of the degree of bending of the wire rope around the posts, in combination with concern that the wire ropes will either lose tension during post deflection or be pulled from the terminal, indicates that this alternative is likely not an acceptable solution without crash testing to prove crashworthiness.
The second solution proposed by the Iowa DOT was to shift the downstream transition bracket further upstream which would decrease the effective angle to the anchor bracket and allowing the cables to bypass bend locations around the post. While this design would help alleviate cable interference with the post, it is not known what the effect of shortening the overlapped cable length would have on the design. Changing the position of the transition bracket would changed the angle of the cables to the ground anchor. One of the concerns in the original design of this system was the potential for snag of the vehicle in the area where the cables angle down towards the ground anchor. Thus, I am leery of changing the transitioning of the cables or the location of the anchorage without further analysis.
An additional option proposed was to drill a hole in the blockout of the post which interfered with the cables. This design option has several advantages, in that the positioning of the bracket and the W-beam do not change relative to each other, minimizing the potential for snagging, pocketing, and loss of cable tension. However, the required size of the hole required to pass the cable through the blockout would be very large, which could lead to lower compressive strength of the blockout, greater propensity for twisting, and the cables would be subject to post rotation or fracture in the soil. Damage to the post at the point of cable routing could interfere with the cable's tension and could potentially cause catastrophic release of the cable from the end terminal. Furthermore, the additional labor required for field drilling holes in the blockout and the potential to cause unexpected damage are high; therefore this is not an optimal solution.

The final design option is to add an additional 12-6" of guardrail between the the flared crashworthy end terminal and the approach transition. By introducing an additional span of guardrail, transition bracket interference issues, cable tension concerns, and field operations are maintained. In addition, this options allows the cable transition to be completed before the approach transition to the bridge rail begins. Though this may be the be slightly more expensive option, it is nonetheless the most crashworthy from a design standpoint, and will most likely result in acceptable performance of the transition design.

An additional issue which was brought to my attention was the standard plan design of the cable anchor. This cable anchor, a 4" x 4" anchor angle, does not have sufficient strength to maintain the loads from the cables during a crash event. Cable loads on anchors can, in TL-3 crash conditions on low-tension cable guardrail systems, rise as high as 60 kips with peak loads from a single cable as high as 25 kips. It is conceivable that higher-energy impacts may cause tension increases in excess of this number. The angle bracket anchor shown in your detail will most likely not be sufficient to maintain these loads without a large degree of deformation, which may compromise the performance of the anchorage. It is recommended that Iowa adopt the design tested in the test report prepared for the South Dakota Department of Transportation entitled, "Crash Testing of South Dakota's Cable Guardrail to W-beam Transition", by Faller, Sicking, Rohde, Holloway, Keller, and Reid, MwRSF Research Report No. TRP-03-80-98. Anchor bracket design details tested in the report are attached. This design uses a gusseted anchor plate that is significantly stronger.

Attachment: http://mwrsf-qa.unl.edu/attachments/317b0be6df04499b8435df6c07010b63.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/46f344d8a8cddce89c91f080cab58956.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/4477943bfe48882d283d6755630ce943.jpg

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Type 2 Downstream End Terminal - Illinois DOT

Question

State: IL
Date: 07-13-2010

I received the following message from a colleague in my department

"> Have you ever placed 2 Type 2's at the end of SPBGR Type D? See the attachment. There is a ramp merging into the mainline which is one way, so the Type D is on the departing end on both sides. Could I use this or use something like a CAT 350, even though it would never be hit head on."
I'm not finding a record of sending this to you before, as I'd promised my colleague in our District 2.

The Type 2 we refer to is our downstream anchor terminal, and we're wondering about applying it on the downstream end of double face guardrail.
It's our Standard 631011-06. http://www.dot.il.gov/desenv/hwystds/rmpdf211.html

We'd welcome any opinion or observations on using this for anchoring the downstream end of guardrails. Per one of our earlier discussions with you or Karla, we do plan to correct the length of the soil tube to 6' from the 7' shown.

Attachment: http://mwrsf-qa.unl.edu/attachments/ad4b5bebeebf99ef37f96a9149b5645b.pdf

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Response

Date: 08-24-2010
I am enclosing a pdf file which compares your downstream anchor hardware to that currently used by MwRSF. Within the file, corrections are noted that show which dimensions are actually used within our CAD details. For our general guardrail testing programs, we utilize a standard end anchorage system on both the upstream and downstream ends of our 175-ft long W-beam guardrail installations when terminals are not being evaluated. These anchorages were adapted from the modified BCT, also consider the MGS rail height and cable anchor increased rise, and include a channel strut, two lengthened foundation tubes, and a common cable anchor with bolted attachment plate and bearing plate on each end.
In addition, you inquired as to whether the trailing end hardware noted above could be utilized in a double rail or median-type configuration where reverse-direction impacts could not be achieved on the end spoons. In such installations, we believe that this trailing end terminal hardware in combination with the MGS would likely provide sufficient capacity to successfully contain and redirect most passenger vehicles impacting at high speeds and angles. Unfortunately, no full-scale crash testing programs have yet been performed on most trailing end terminal systems.
Currently, there have been concerns with many different non-crash tested trailing end terminals that the small car vehicles could become snagged or wedged under the anchor cable on the downstream end if the end post is not fractured or does not release in a timely manner.

Two prior crashworthy box beam guardrail end terminals have utilized a post breaker system to ensure post fracture and cable release prior to snagging the small car vehicle. However, the current generation of energy-absorbing and flared W-beam guardrail end terminals do not utilize post breaker features for releasing the cable anchor end located near the groundline of post no. 1. As such, there could be an argument for not utilizing post breakers in trailing end guardrail terminals if similarly configured to current W-beam terminals in terms of anchorage. This opinion would be based on the design, prior crash testing performance, and in-service experience of most crashworthy W-beam guardrail end terminals.

Using engineering judgment and in the absence of crash testing, we believe that the downstream trailing end terminal hardware, similar to that used at MwRSF and shown herein, could be utilized in a double rail, median-type configuration. However, full-scale crash testing is the only true way to determine the safety performance of the downstream trailing end terminal system. In addition, it should be noted that future testing may provide a basis for modifying our opinions on this issue.

If one were to have significant concerns regarding the potential for small car snag or wedging under the cable anchor, then a slight design change may be considered. First, it may be advantageous to incorporate blockouts with the end posts in foundation tubes, thus allowing a 8-in. lateral shift of the post, strut, and anchor cable. Such a design modification would likely require a longitudinal stagger of the anchor posts combined with a single post installed between the two blockouts. Unfortunately, there are also concerns with this design variation, such as little or no experimental experience, lack of prepared design details, unique loading on anchor posts and foundation tube, and potential for inadequate cable length with the 8-in. lateral shift.

At this time, MwRSF has received research funding from the WisDOT to examine, test, and evaluate a standardized downstream anchorage system for the MGS. With this project, I am hopeful that we will be able to provide design guidance for both roadside and median applications, including for double, median-type W-beam guardrail systems.

Attachment: http://mwrsf-qa.unl.edu/attachments/2c558f6e62d35a7574f787f7ff4a1f4e.pdf

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Steel Bridge Railing Question

Question
State: IA
Date: 06-03-2009

What would be your opinion of installing a steel bridge railing (Illinois 2399 curb-mount) at standard post spacing (6'-3" as tested), but increasing the post spacing at four locations on the bridge in order to accommodate some structural members? Our consultant feels they can limit the maximum post spacing at these locations to 7'-6". Do you think allowing the larger post spacing at these locations would be feasible without additional testing, or should we be investigating other options?

There would be only 1 spacing of 7'-6" at each of the four locations on the bridge.

Response
Date: 06-15-2009
MwRSF feels that increasing the post spacing from 6'-3" to 7'6" in only a few non-adjacent spans is a possible task. However, the bridge rail must be stronger to accommodate the 20% increase in moment due to the elongated post spacing. As such, we recommend the following:
Replace the 4"x4" bottom tube with another 8"x4" tube (the top tube). Thus, the bridge rail would consist of 2 8"x4" tubes. Assuming the top and bottom rail carry equal loads (which it really doesn't " top takes more load), this small change would provide a 30% increase in rail strength - enough to accommodate the 20% increase in moment.
This rail combination should be used throughout the bridge to ensure rail continuity and prevent snag points
Also, keep the bottom of the lower tube at 14" above the roadway. Thus the top of the lower tube is 22" above the roadway (2" gap between rails). This will allow the lower rail to better interact with an impacting vehicle and absorb more of the impact load.

Response
Date: 06-16-2009
We have an additional question to follow up the attached email which recommended that an 8" x 4" tube be used on the bottom rail throughout the bridge.

Since this bridge is relatively long, using an 8" x 4" tube for the bottom rail over the entire length would result in a significant increase in the steel quantity and cost. (The length of bridge to receive new rail is about 3,000 feet and the weight difference between a 8 x 4 x 5/16 tube and a 4 x 4 x 1/4 tube is 11.14 pounds per foot. Thus there would be an increase in steel of about 2 x 3,000 feet x 11.14 lb./ft. = 66,840 pounds.) Also, we would like to minimize the additional total dead load that is added to the bridge since the weight capacity of the bridge is an issue. (We are even planning to use lightweight concrete for the curbs on this project.)

In view of this, would it be possible to strengthen the rail at only the few areas where the span would exceed 6' 3"? In order to accomplish this, could the rail be strengthened at just those longer rail spans and any necessary adjacent spans, while using a 4 x 4 x 1/4 tube for the bottom rail throughout the rest of the bridge? The following are some ideas for your consideration to accomplish this:

Increase the wall thickness of the standard top and bottom rails in order to get a 20 % or greater increase in the section modulus (S) for bending. This would result in no change in the outside railing geometry.

Install a tubular member inside of the standard top and bottom rails in order to get a 20 % or greater increase in the section modulus for bending. For example, a 4 x 4 x 1/4 tube has a S of 3.90 inches^3. If a 3 x 3 x 3/16 tube (S = 1.64 inches ^3) were inserted inside of the 4 x 4 tube, the total S for the bottom rail would be increased by 42 %. This would result in no change in the outside railing geometry.

Add another 4 x 4 x 1/4 tube directly above the standand 4 x 4 x 1/4 bottom rail to increase the bending strength. In order to avoid a snag point, this section would need special fabrication at the ends for a transition down to the typical bottom rail.

Replace the bottom rail with a 8 x 4 x 5/16 tube as recommended in the attached email, except fabricate a special transition down to a 4 x 4 x 1/4 tube at the ends in order to avoid a snag point.

Please let us know if any of the above concepts would be acceptable, and if so, we will ask the consultant to investigate further.

Response

Date: 06-24-2009

We do feel that we can strengthen the rail in the areas surrounding the extended post spacing only. With a 3,000 ft bridge, using the increased rail size for the entire system would be wasteful. Comments on the proposed solutions are discussed below.

(1). Using a thicker / stronger rail in certain areas will result in abrupt stiffness transition points at the connections between the two rail types. These stiffness transitions could lead to vehicle instabilities or snagging.

(3) & (4) Altering the shape of the rail in these locations can lead to more vehicle interaction problems (snagging, instabilities, wedging, etc...). As such, we do not favor the option of transitioning between different rail geometries without testing these transitions.

(2) MwRSF does like the tube-in-a-tube idea for strengthening the rail. The inserted tube should fit relatively snug inside the original tubes, so that the smaller tube develops load before the rail suffers larger deformations. The 3x3 tube inside of the lower rail (4x4x1/4) tube is a good fit. However the upper rail should also be reinforced. The same 3x3 tube could be used if its position could be centered inside the 8x4 (perhaps resting it between the attachment bolts, bolting through the 3x3 tube, or using spacers to position the 3x3 tube inside the 8x4 tube.

The inserted reinforcement tubes should be extended out from elongated spacing, though the adjacent spacing of 6'-3", and to the nearest ¼ spacing. The 1/4 points of the rail are recommended for the stiffness transition to prevent the tube end from occupying a point of maximum deflection / deformation (midspan) or a stress concentration point (at the posts). Thus, the inner tubes should be extended 94 inches past the posts of the longer spacing (6'-3" plus 19"). Total length of the inner tubes would then be 188 inches plus the length of the longer post spacing (approximately 7'-6" from your previous e-mail.

Response

Date: 07-14-2010

I've got one more (hopefully the last) request for you regarding our I-74 bridge rail replacement. Apparently our consultant, rather than incorporating your previous advice, has developed an alternate method for spanning the wide expansion joints on the I-74 bridge. This method places specially-designed posts on either side of the joint, spaced 5 feet apart.

Could you please review and comment on the attached drawings showing the proposed design? Just as before, this will be used at a total of four locations on the bridge - on both sides of the road at each of the two suspension towers.

The post spacing varies in order to avoid the vertical stringers located just beyond the edges of the bridge deck.

The consultant felt that he needed to space the corbels (and therefore the posts) in order to avoid the vertical trusses due to the tight tolerances (see the attached picture of the current bridge). The vertical trusses are located approximately 1'-5" behind the face of rail. Would you agree that even if a post were placed at a truss location, that the truss would lie outside the working width of the barrier?

The proposed spacings have not been analyzed. Do you feel the abrupt changes in post spacing throughout the bridge is concerning enough to warrant a possible redesign? If we could somehow reduce the depth of the corbels, perhaps that would allow them to be installed at truss locations?

Response

Date: 07-15-2010

The full scale testing on the original Illinois steel tube bridge shows a maximum dynamic deflection less than 3 inches. Also, although the working width of the system was not specified in the summary pages, the vehicle does not appear to extend more than 12 inches past the face of the rail. Thus, the 1'-15" of clear space between the face of rail and the vertical trusses provides enough room to minimize the risk of vehicle snag on the truss members. Further, the 17 inches of space matches that of the recommended offset from the head ejection envelope developed in TRP-03-194-07 for the 95th percentile passenger (14 in. + 3 in. = 17 in.).

With a maximum dynamic rail deflection

Two Loop PCB Connection

Question

State: NE
Date: 06-19-2010

Has MwRSF tested a 12.5' Concrete Protection Barrier with two loops?
I'm thinking this is from the 2001-2003 era.
Have we tested two loops in the end of a concrete bridge rail or median rail?
I thought we tested this with the Kansas style steaked-down with 3 stakes on the traffic side.
Then 2 barriers staked down with 2 stakes each, then 1 or 2 staked down with 1 stake.
What was the name of this research study?
Would the tied down barrier move less than the free standing barrier and put less force on the loops?

Response

Date: 07-20-2010
The original 350 testing utilized 2 loops per end. Later, we added the third loop to get double shear " top and bottom.
The only 2-loop TCB system that was crash tested and evaluated while anchored corresponded to the steel tie-down strap system. And, this TCB used a version where each loop was configured with 3 small bent rebar. All of tied-down systems and transitions used the Kansas version with 3 rebar loops per barrier end.
Without detailed analysis, I believe that an anchored TCB would encounter reduced tension within the loops as compared to a free-standing TCB. However, the loops would potentially experience increased shear and moment at the concrete interface if one barrier shifts relative to the other. This shifting has been observed in the anchored barrier testing. Please note that no directed study has been made for comparing the various loop configurations under free-standing and anchored installations.

Bridge Rail Retrofit

Question

State: WI
Date: 07-28-2010

WisDOT has a project where an existing thrie beam bridge rail was installed too low. Regional staff has asked if:
1. The existing longitudinal channel on top of the bridge rail could be removed.
2. A small box beam or steel tube could be bolted to the existing post (i.e. to get the correct rail height)
3. Existing longitudinal channel is reinstalled.
From what I understand, the existing deck bolts and nuts are very rusty and difficult to remove. This will make it difficult to remove the existing post and replace with new taller posts. In addition the taller post are more expensive to fabricate than the smaller box beam/steel tubes.
An example of the retro fit is attached (W Rail Retrofit.pdf)
An example of our current thrie beam retro fit is also attached (3002.pdf).
The rail used on the existing bridge is also attached.

Attachment: http://mwrsf-qa.unl.edu/attachments/28512cd0f5bfc72c5969b343d44ea512.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/3a51aff78a5d81ef3821b41b90e5d86e.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/87462e84fb39314b81880a3bf71c895a.pdf

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Response

Date: 08-02-2010
The Wisconsin design is similar to the Missouri thrie beam and channel bridge rail tested by TTI. However, the bridge rail plans show the system as 4 inches shorter than it was tested at previously. Thus, the addition of the 4 in. tall spacer block to the top of the shorter post allows the rail to be installed at the correct height. The use of four 5/8 in. diameter bolts to connect the block to the post should provide more than enough strength to prevent shear failure during an impact.
The anchorage for the Wisconsin bridge rail seems to be a modification of the tested system as well. Tested used three 1 in. diameter A307 bolts, while the current drawings show 4 7/8 in. diameter A449 bolts. Noting that A449 provides a 20-50% increase in strength (depending on grade), the Wisconsin bridge rail design should provide equal or greater anchorage strength.
Therefore, the proposed bridge rail design appears to be of comparable strength and geometry to that of the tested Missouri thrie beam and channel system.

Cable Guardrail Questions

Question

State: NE
Date: 07-30-2010

The new in-line cable end treatment requires post 3 through 7 to be spaced @ 16'. What is the offset to a fixed object in this area?
When we design a long run of guardrail in the past we have used an intermediate anchorage section. Is this still necessary?
If so, is there a design for the new in-line intermediate anchorage section?
The spacing in front of a 1.5:1 slope requires 4' post spacing. Is it acceptable to have 16' post spacing then 4' spacing?
Or, is there a suggested length of transition of 8' post spacing?
Have you been able to run a simulation when our slope is 2:1, with a 2% lane and 4% shoulder slopes? I think this will keep the front tire on the slope and not require the 4' post spacing.

Response

Date: 08-25-2010
The new in-line cable end treatment requires post 3 through 7 to be spaced @ 16'. What is the offset to a fixed object in this area?
**A 2000P pickup truck was crash tested at the length-of-need of the end terminal at the TL-3 conditions of NCHRP Report No. 350. The vehicle impacted post no. 3 which was 15 ft downstream from the upstream steel anchor post. For this crash test, the working width was reported to be approximately 84 in. when using a 254-ft long installation.
**Please note that the target impact angle for this test was 20 degrees, as required by NCHRP Report No. 350. The new MASH guidelines now utilize an impact angle of 25 degrees. With higher impact angles, one would expect higher angle loading and slight increases in anchor movement, thus resulting in greater barrier deflection and working width near the system ends.
When we design a long run of guardrail in the past we have used an intermediate anchorage section. Is this still necessary?
**As noted above, the test installation was 254 ft long. For longer test installations than denoted above, dynamic barrier deflections and working widths would be expected to increase.
**A prior Pooled Fund R&D program resulted in the successful development, testing, and evaluation of three alternative anchor systems in lieu of the large cast-in-place reinforced concrete anchor blocks. However, the R&D program did not evaluate changes in anchor spacing. As such, we would recommend that NDOR continues to utilize an anchor spacing equal to or smaller than that currently specified, especially since barrier deflections and working widths could be greater with the use of the alternative anchor options.
If so, is there a design for the new in-line intermediate anchorage section?
**The alternative anchor options were developed for terminating and anchoring the ends of the three cables. I am unclear as to the difference between end anchor hardware and the anchor hardware used at intermediate anchor sections. Please forward those details to us for review as I am unaware of prior crash tests performed to evaluate the safety performance of the overlapped cables with two intermediate anchor sections crossed in opposite directions.
The spacing in front of a 1.5:1 slope requires 4' post spacing. Is it acceptable to have 16' post spacing then 4' spacing?

**The SdDOT three-cable guardrail to W-beam transition utilizes a cable barrier with 16-ft post spacing that transitions into a cable barrier with 4-ft post spacing in advance of the BCT W-beam terminal. No intermediate post spacing was integrated into this original SdDOT design. More than 60 ft of cable barrier with 4-ft spaced posts was used to prevent pocketing near the BCT end. No testing was performed upstream of the 4-ft post spacing design. However, I do not believe that the reduction in post spacing would create a significant pocketing concern for large vehicles or penetration concern for small cars when used in combination with the standard cable hook bolt.

**For the three-cable barrier with 4-ft post spacing in front of a 1.5:1 fill slope, MwRSF performed a 2000P crash test according to the TL-3 conditions of NCHRP 350. An 820C small car test was not performed nor deemed necessary by the MwRSF team. The successful 2000P crash test resulted in nearly 125 in. of dynamic deflection when placed 4 ft from the slope break point, thus resulting in the vehicle extending nearly 6 ft off of the slope. The vehicle's lateral extension off of the slope further accentuated the barrier deflections observed in the 2000P test.

**TTI crash tested a 3-cable barrier on level terrain with a 16-ft post spacing at TL-3 of NCHRP 350. This testing resulted in 3.4 m (134 in.) of dynamic deflection, which was slightly larger than the deflection observed above in the ditch. Since it is uncertain where the 4-ft post spacing will end w.r.t. the ditch start/finish, it would be reasonable to expect the 4-ft spacing to overlap regions of level terrain. When the 4-ft post spacing is installed on level terrain, dynamic deflections would likely be reduced below 125 in.

**Although it would not be deemed necessary at this time, one may consider the use of 4 or 5 spans with posts spaced on 8 ft centers prior to reaching the 16-ft post spacing region.

Or, is there a suggested length of transition of 8' post spacing?

**See comments noted above.

Have you been able to run a simulation when our slope is 2:1, with a 2% lane and 4% shoulder slopes? I think this will keep the front tire on the slope and not require the 4' post spacing.

**No work on this project has been performed. This work was included in a Pooled Fund study that was not funded in the Year 21 final program. I will copy this request to John Reid and Bob Bielenberg to determine what level of effort would be required to conduct this specific request.

Temporary Sand Barrel Arrays

Question

State: WI
Date: 08-09-2010

I'm looking into providing additional guidance for our staff on the use of temporary crash cushions and sand barrel arrays.
During my reviews, I found NCHRP Report 358 Recommended Practices for Use of Traffic Barriers and Control Treatments for Restricted Work Zone (see attached). I have the following questions:

1. The sand barrel arrays were designed for NCHRP 230 impacts. How would the layouts change for a MASH vehicle (e.g. offsets, barrel layouts...).
2. The guidance on what treatment to use to protect the blunt end of the temporary barrier was based on the barrier being installed for 1 year or less. If a project will have temporary barrier installed for more than a year what steps should be taken by a designer?
3. Would the charts (figures 4.17-4.24) have significant changes to the break points between different end treatments because of the new MASH vehicles? Or do these charts represent the most current state of the art for temporary barrier end treatment protection?

Response

Date: 08-16-2010
I have responded to your questions inred
below.

1. The sand barrel arrays were designed for NCHRP 230 impacts. How would the layouts change for a MASH vehicle (e.g. offsets, barrel layouts...).

   With regards to the sand barrel layouts, MwRSF could look at the barrel arrays and attempt to adjust them. However, we believe that it would be more appropriate for you to contact the sand barrel manufacturers in order to get their recommendations for the barrels arrays with the MASH vehicles.

2. The guidance on what treatment to use to protect the blunt end of the temporary barrier was based on the barrier being installed for 1 year or less. If a project will have temporary barrier installed for more than a year what steps should be taken by a designer?

   The end treatment guidance in NCHRP 358 was based on benefit/cost analysis. Thus, the longer the sand barrel array was installed, the more likely that a more robust, long term attenuator would be worth installing. That said, we do not believe that leaving the sand barrels in place for a period over one year hugely problematic. If the barrel array is installed for a much longer time than one year, then you may want to rethink which type of system you use.

3. Would the charts (figures 4.17-4.24) have significant changes to the break points between different end treatments because of the new MASH vehicles? Or do these charts represent the most current state of the art for temporary barrier end treatment protection?

   The charts mentioned in NCHRP 358 are currently the best guidance for barrier flare rates n the work zone. No further analysis has been done to update those tables with more recent accident data or to make considerations for MASH.

Structural Analysis of Approach Transitions

Question

State: WI
Date: 08-13-2010

I have a major project team that is challenging my requirement that they provide structural analysis fortheir transitions. They indicate the following:

A. "AASHTO LRFD defines analytical procedures for structural design of barrier (aka bridge parapet) connection to bridge decks. The intent is to ensure that the connection to the deck, and the deck itself, offers greater resistance than the barrier (i.e., make sure the bridge deck is not the weak link). As far as we know, AASHTO does not establish analytical procedures for barrier design for purposes of load classification (e.g. TL-3) and physical crash testing is required. There is however some history of FHWA accepting analytical procedures (structural calculations) used to demonstrate that a customized bridge parapet will perform at least as well as a similar crash-tested version."

B. If it is desired and/or required to adopt an analytical procedure for designing barrier transitions, what will the basis of those procedures be? From a structural engineering perspective, behavior of reinforced concrete barrier under static loads is predictable enough. Behavior of the foundation (structure interaction with subgrade below and pavement adjacent) is more difficult to predict and normally involves assumptions which are quite conservative. Structure response to dynamic loading (vehicular crash) is very complex and difficult to predict even when materials and construction are well controlled. Because of this complex behavior and variability in conditions, as well as unknowns associated with the crash vehicle itself, a purely analytical method to assess barrier performance may necessarily be very conservative. The adjacent pavement and subgrade would offer substantial resistance to overturning, but this is proven with confidence empirically (crash test) and not so easy to demonstrate analytically (as mentioned above). Could barriers be treated similar to gravity retaining walls, using the TL-3 equivalent static loading forces from the AASHTO LRFD.

C. What precedents exist for either analytical methods or empirical methods for designing barrier and barrier transitions? I think the team would benefit from a historical perspective, and also perhaps a wider geographic (national) perspective, as well as local precedent.

In your opinion there is little difference between designing a roadside barrier and a bridge parapet(i.e. the impact forces and how to deal with them are about the same). The fact that one is in the soil, verse connected to a deck, may allow for different methods to handle overturning moments (e.g. a roadway barrier could be wedged between lifts of asphalt or tied into a footing).

Response

Date: 08-13-2010
Please see my comments inred!
I have a major project team that is challenging my requirement that they provide structural analysis fortheir transitions. They indicate the following:

A. "AASHTO LRFD defines analytical procedures for structural design of barrier (aka bridge parapet) connection to bridge decks. The intent is to ensure that the connection to the deck, and the deck itself, offers greater resistance than the barrier (i.e., make sure the bridge deck is not the weak link). As far as we know, AASHTO does not establish analytical procedures for barrier design for purposes of load classification (e.g. TL-3) and physical crash testing is required. There is however some history of FHWA accepting analytical procedures (structural calculations) used to demonstrate that a customized bridge parapet will perform at least as well as a similar crash-tested version."

**The AASHTO LRFD Bridge Design Specifications provides guidance for designing bridge railings for use on bridge decks as well as those attached to bridge approach slabs. This guidance is intended to help engineers properly configure bridge railings as well as their attachment to reinforced concrete decks. Both solid and open concrete parapets can be configured as well as metallic beam and post systems. Combination concrete and metal systems are also addressed. Limited discussion is provided for timber railings. Yield-line analysis procedures have been provided for addressing the design of reinforced concrete parapets and railings. Inelastic design procedures are available for most metal systems. These rail design procedures were developed and/or documented in a 1978 study report by TTI researchers and have been consistently used for a large share of railing systems. Upon design, it has been common practice for the design to be verified through the use of full-scale crash testing. Actually, full-scale crash testing has also been used for demonstrating the system's structural adequacy and safety even when the prior noted design procedures were not used. If crash testing has been shown to corroborate a design based on the noted procedures, then these procedures have also been used to modify other parapets as long that they provided equivalent or greater strength and did not pose increased risk for vehicle snag, rollover, or override.
For reinforced concrete parapets, the noted design procedures have also been to ensure that sufficient strength is provided at critical locations within the barrier, such as at barrier ends and at expansion joints. At such locations, the number of yield lines that can be developed is much reduced, thus potentially lower the redirective strength of the parapet. Therefore, it is imperative that these equations be utilized to modify a barrier's capacity to ensure that an impacting vehicle can be safely contained and redirected along the entire barrier length. Basically, the entire barrier must act as though it is continuous even though weakened sections may exist therein. End buttresses that are used to anchored approach guardrail section must also provide adequate structural strength so as to not allow for vehicles to penetrate directly behind the bridge railing if the entire length plus AGT must shield the hazard.

B. If it is desired and/or required to adopt an analytical procedure for designing barrier transitions, what will the basis of those procedures be? From a structural engineering perspective, behavior of reinforced concrete barrier under static loads is predictable enough. Behavior of the foundation (structure interaction with subgrade below and pavement adjacent) is more difficult to predict and normally involves assumptions which are quite conservative. Structure response to dynamic loading (vehicular crash) is very complex and difficult to predict even when materials and construction are well controlled. Because of this complex behavior and variability in conditions, as well as unknowns associated with the crash vehicle itself, a purely analytical method to assess barrier performance may necessarily be very conservative. The adjacent pavement and subgrade would offer substantial resistance to overturning, but this is proven with confidence empirically (crash test) and not so easy to demonstrate analytically (as mentioned above). Could barriers be treated similar to gravity retaining walls, using the TL-3 equivalent static loading forces from the AASHTO LRFD.

**As noted above, the yield-line and inelastic design procedures are appropriate for designing the barrier systems that are anchored to both the bridge decks and approach slabs. These procedures have also been used for designing similar parapets to soil grade beams. In most cases, full-scale crash testing has demonstrated that the procedures are effective. However, when we use such procedures, we use a load factor of 1 using our MwRSF loads and not necessarily the loads noted in AASHTO. In addition, we would use the appropriate reduction factor for determining the various capacities, such as bending of reinforced concrete. These equations may not always work in every case due the various types of anchorage or support. In such cases, approximations are sometimes made for certain parameters based on experience and historical crash testing results under review. In some special cases, the published dynamic design loads have also resulted in overdesigned moment slabs for concrete parapets placed on MSE walls when used in static overturn analysis and design.

C. What precedents exist for either analytical methods or empirical methods for designing barrier and barrier transitions? I think the team would benefit from a historical perspective, and also perhaps a wider geographic (national) perspective, as well as local precedent.

**Both analytical methods, computer simulation, and full-scale crash testing have to be used by themselves, or in combination, when developing and verifying the safety performance of guardrails, transitions, and bridge railings/median barriers. In most cases, crash testing was used but not in all. After researchers, designers, and engineers have become familiar with these methods, the more experienced personnel know when to apply one or more than one method to ensure that a system is properly configured.

In your opinion there is little difference between designing a roadside barrier and a bridge parapet(i.e. the impact forces and how to deal with them are about the same). The fact that one is in the soil, verse connected to a deck, may allow for different methods to handle overturning moments (e.g. a roadway barrier could be wedged between lifts of asphalt or tied into a footing).

The procedures are generally the same. The foundation systems could vary between roadside and bridge applications.

Response

Date: 08-16-2010

I have to summarize your response to me about yield-line analysis. Am I on the mark with this comment? I want to say:

An errant vehicle imparts the same amount of force into roadside barrier or bridge parapet. Yield-line analysis has been used to develop both roadside and bridge parapets. Crash testing has proven yield-line analysis can provide a structural adequate roadside barrier or parapet. Some of these crash tests may have had failing crash test results because of the roadside barrier or parapet was not functionally adequate.

This design methodology provides that the barrier itself has:

- Sufficient reinforcement so that the force of vehicle impact can be withstood by the barrier or transition (i.e. the barrier does not shatter and allow the vehicle to pass through the barrier)

- Sufficient reinforcement and footing to prevent the barrier from shifting during (e.g. provide a snag point or pocket) or pivoting during an impact (e.g. if the barrier tips over during an impact or provides a ramp to launch a vehicle in to the air has it done its' job?).

How forces get absorb by a deck, footing or soil may be different. However, a structural design engineer should have the necessary skill set to develop a design.

Yield-line analysis is only required at special transitions and unique situations (e.g. sign bridge integrated into barrier...). A "normal section" of single slope barrier with end anchorages does not need to be analyzed. However, it does need sufficient longitudinal steel to prevent shrinkage cracking.

Is this correct? I'm having difficulties defending this topic because I'm not a structural engineer. An I know that structural engineers will be present at my meeting. So I want to run this past someone who knows more about barrier design than I do.

Response

Date: 08-16-2010
See my comments below inred!

An errant vehicle imparts the same amount of force into roadside barrier or bridge parapet.(This would be true if both barriers were rigid. If one barrier is allowed to displace, then the impact load would likely be reduced.) Yield-line analysis has been used to develop both roadside and bridge parapets.(If configured with reinforced concrete.) Crash testing has proven yield-line analysis can provide a structural adequate roadside barrier or parapet.(Yes.) Some of these crash tests may have had failing crash test results because of the roadside barrier or parapet was not functionally adequate.

This design methodology provides that the barrier itself has:

- Sufficient reinforcement so that the force of vehicle impact can be withstood by the barrier or transition (i.e. the barrier does not shatter and allow the vehicle to pass through the barrier)

- Sufficient reinforcement and footing to prevent the barrier from shifting during (e.g. provide a snag point or pocket) or pivoting during an impact (e.g. if the barrier tips over during an impact or provides a ramp to launch a vehicle in to the air has it done its' job?).(Do not allow vehicle override or rollover for passenger vehicles.)

How forces get absorb by a deck, footing or soil may be different. However, a structural design engineer should have the necessary skill set to develop a design.

Yield-line analysis is only required at special transitions and unique situations (e.g. sign bridge integrated into barrier...). A "normal section" of single slope barrier with end anchorages does not need to be analyzed. However, it does need sufficient longitudinal steel to prevent shrinkage cracking.(Yield-line analysis is used at all locations, including interior regions, ends, gaps, special shape transitions, etc. However, experience may help determine if one really needs to perform the analysis at each location. The use of different types of footings may require that the certain terms in the yield-line analysis equations be neglected or minimized. Prior crash testing results may be used to support those changes.)

MASH Temporary Barrier Deflection Vs NCHRP 350 Deflection

Question

State: WI
Date: 08-16-2010

I believe that MwRSF indicated that the use of MASH crash test vehicles is increasing barriers deflection distances. How much has the temporary barrier deflection increased using MASH vehicles compared to NCHRP 350 vehicles?

Response

Date: 08-16-2010
We have seen an increase in the deflected of the F-shape PCB when impacted with MASH vehicles.

Free-standing TCB deflections were significantly higher when testing was conducted with the 2270P vehicle under the MASH criteria as opposed to testing conducted with the 2000P vehicle under the NCHRP Report No. 350 criteria. TCB deflections increased 25 to 76 percent when the F-shape TCB was tested under MASH impact criteria. See attached table.

Test No.	Vehicle	Mass (kg)	Speed (km/h)	Angle	IS (kJ)	Dynamic Delfection (m)	Static Deflection (
ITMP-2	2000P	2005	100.3	27.1	161.5	1.15	1.14
TB-1	2270P	2268	99.5	25.7	162.9	1.44	1.44
TB-2	2270P	2268	99.7	25.4	160.0	2.023	1.854

This increase in deflection is due to a couple of factors

1. Higher mass = more inertia transfer and higher load
2. Higher vehicle stability encourages less climb and vehicle rotation which allows the vehicle to directly load the barrier longer.

In addition, the photos
(Figure 1 and Figure 2 attached)
show the damage from the 2270P testing. In this tests vertical cracks were observed completely through the barrier section. This amount of barrier damage was not observed in the 2000P testing and again suggests that our impact loads have increased.