Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

04-01-2010 to 07-01-2010

Illinois Temporary Concrete Barrier

Question

State: WI
Date: 03-31-2010

Please review the attached details from Illinois. Would this barrier be acceptable in Wisconsin? I presume it is approved for use by FHWA.
The Illinois barrier does not meet the requirements of WisDOT S.D.D.s. I did a cursory review of the details and compared the details for both states. There are some differences as I outline below:
Overall dimensions are the same.
The location of the loop bars are different vertically.
Anchor locations are different.
Anchor hole size is different.
Dimensions and shape of the connecting loop bars are different.
Steel anchor stakes are different is size and shape.
Illinois shows no provision for anchoring to a bridge deck or pavement.
These are a few of the issues I spotted quickly.
If the Illinois barrier is acceptable, the field staff would be required to write a CCO in order to incorporate the details into their contracts. The two barriers could not be intermixed.
Please provide some guidance as to the use of the Illinois concrete barrier. The staff on the USH 41 projects are trying to be proactive in case this barrier does show up in this area. I don't know how the N-S Freeway is handling this situation.

Attachment: http://mwrsf-qa.unl.edu/attachments/4781589ce82736cc8f5aad7a7f0603f7.pdf

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Response

Date: 04-02-2010
I have reviewed the Illinois barrier detail you sent. We believe that the design will be okay for free standing applications. The design is basically the MwRSF F-shape with the Oregon connection. A few additional comments:

1. Oregon barrier and connection loops and pin were tested to 350, thus the connection should not be an issue.

2. There is a small difference in barrier connection gap between the Illinois detail and the Oregon design (1" for Oregon vs. 2" for Illinois), but this should not be a big issue. It may produce larger barrier deflections than the tested Oregon design, but would be comparable with the MwRSF F-shape. The Oregon barrier achieves its low dynamic deflections largely due to the reduction of the barrier gap.
3. Oregon barrier has more moment capacity (more longitudinal steel, farther to outside), but MwRSF barrier has met MASH with current reinforcement.
4. Longitudinal steel is placed farther out in the toe of barrier in Oregon design, but again should not be an issue. It may lead to higher deflections if toes fracture. This hasn't been a problem with MwRSF F-shape barrier testing.
5. The tie-downs systems developed to pass through the toe of the barrier will not fit in current Illinois barrier design. Thus, we do not recommend using the tie-down systems with the Illinois barrier.

FHWA approved a Colorado barrier that was very similar to the Illinois barrier design. The only real difference was the steel reinforcement which was setup to match the Oregon detail rather than the MwRSF barrier. See attached.

The difference in reinforcing steel is not a big issue as the MwRSF barrier has met MASH with current reinforcement.

The tie-down anchorage for this is very different than the MwRSF barrier.

Again, we would be hesitant to apply the MwRSF tie-down anchor systems with the Illinois barrier as detailed, but believe it should be acceptable in a free-standing configuration.

W-Beam Guardrail Near Steep Slopes w/ Wood Plank Soil Containment System

Question

State: WI
Date: 04-09-2010

We are developing plans for the mill and
resurfacing of STH 145 from STH 100 to STH 167 in Waukesha and Washington Counties.

We have a question for you regarding the beam guard on the NB approach to B-67-217. The beam guard in question is between Sta 42+12 to 45+12, Rt. The existing beam guard has 10' posts tightly spaced and has some timber planking along the inside face more or less serving as a short retaining wall. The beam guard, posts and planking are in reasonably good condition. Our original concept was to replace the beam guard and 10' posts and saw off the old posts at the top of the planking. During the review process it was questioned whether this beam guard would be considered crash worthy? We would like your opinion as to the best way to resolve this issue.

I am attaching the following for your review:
PD01 - Plan/profile showing the proposed beam guard (Sta 38+50 to 45+00)

DT03 - Steel Plate Beam Guard Special detail
0321 - photo
Special Beam Guard Detail - from the as-built plans

Response

Date: 05-05-2010
First, there is concern with the placement of a ground-mounted, wood rub-rail system under the guardrail that may cause an impacting vehicle to vault upward as a wheel contacts the timber member, thus increasing the propensity for a vehicle to override the guardrail or become unstable during redirection. This result would especially be of concern when the wood member vertically extends greater than 4 inches above the ground-line for standard 27" tall guardrail systems.
The use of 6"x8" by 10' long wood posts in the noted situations may also result in premature post fracture and reduced energy dissipation when the drop in back slope in minimized (i.e., embedment maximized). However, the use of a 1/2-post spacing could garner back some of the reduced capacity if premature post fracture occurs. For cases where the soil drop is maximized, the soil may yield and allow post rotation prior to reaching a wood post fracture condition. To reduce concerns for wood post fracture in these special situations, it may be preferred to utilize long steel posts which would remain intact and dissipate energy during displacement of the barrier system.
As such, there are safety concerns with using a 27" tall, W-beam guardrail system when coupled with the exposed wood plank, complicated steep slopes, wood posts, and TL-3 impact conditions with higher C.G. passenger vehicles.

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."

Barrier Protection in Median Crossovers

Question

State: WI
Date: 04-23-2010

You will recall the question I asked about the need for barrier protection at median crossovers during our discussion Wed afternoon. As I mentioned, this question has been raised at the director level, and I will need to report back to them within the next 2 weeks. You both provided some excellent insight and perspectives that Erik, Bill and I were very satisfied with; however, I would like to pose this question again as a consultant request for a more formal response.
Like many states, we build median crossovers during construction to shift traffic from one set of pavements to another. Occasionally, we decide to leave these crossovers in-place rather than remove them at the end of the project. A question has been raised as to whether these median crossovers should have some sort of barrier protection or just delineated. It has been suggested that we install end protected temporary concrete barrier at these crossovers as soon as possible. Others have suggested we investigate installing cable barrier at these locations. I believe that some think having a paved surface connecting the two roadways poses a greater risk for a CMC; and therefore protection is warranted. I don't believe this question is directed providing a safe, proper design for the crossovers. I think that is a separate issue.
I've been asked to report on this issue in about 2 weeks at our May Safety Engineering Executive Group meeting that is comprised mostly of directors within our Division.

Response

Date: 05-03-2010
During your recent visit in Lincoln, Dr. Sicking, Mr. Bielenberg, and myself met with Mr. Emerson, Mr. Bremer, and yourself to discuss the status of the Wisconsin DOT safety research projects. Toward the end of our meeting, you later sought comment on a potential issue involving the long-term presence of temporary and/or permanent median crossover roads between divided highways.

Median crossover roads are often used to transfer motor-vehicle traffic to the opposing vehicle lanes when construction and/or maintenance operations require the closure of selected traffic lanes found ahead. The majority of these crossover roads are typically used for short-term operations. Typically, these temporary roads are removed; however, some of these roads are occasionally allowed or intended to remain in place within the median region even though their use is discontinued. Crossover roads often remain in place due to their future use in maintenance operations or due to the high cost to remove them. Under these circumstances, questions have been raised as to whether there exists a significant risk or opportunity for motorists to utilize these crossover roads, thus potentially resulting in crossover median crashes. In addition, these questions have led to further discussions on whether the remaining crossover roads need to be protected with median barrier systems, thus preventing vehicles from traveling on the closed roads and into opposing traffic lanes.

If crossover roads must remain in place, several options should be considered for reducing or eliminating concerns for their non-approved use by motorists.

First, it may be possible to cover and/or camouflage the crossover roads with soil and vegetation to eliminate concerns for their use by motorists. Crossover roads that are covered and more closely resemble the natural median conditions should be provide no greater risk of accidental vehicle crossover than the adjacent upstream and downstream median regions. If the roads are ever needed in the future, the soil and vegetation could be removed to expose the paved road surface.

Second, if complete coverage of the crossover roads is not feasible, then partial removal of the crossover road surfaces could be considered for the first 4 to 6 ft laterally away from the outer edges of the paved median shoulders. With a 4 to 6 ft width of soil region (or other width yet to be determined), grass vegetation could be used to visually close off the roads to deter their potential use by motorists.

Third, it may be reasonable and economical to line the center region of the crossover road system with a row of closely-spaced traffic delineator posts in a pattern that runs parallel to the divided highways. A row of traffic delineators posts would be highly visible during the day as well as the night and would denote to the motorists that the roads are not for public use. If truly deemed necessary, traffic warning signs could also be strategically located to further inform motorists that the crossover roads are not for public use.

Some have suggested that median barriers should be used to close off the crossover roads in order to prevent motorists from intentionally (i.e., for turning around) or accidentally (i.e., due to driver inattention) traveling on these roads. However, the use of new containment barriers at these locations would result in new risks to motorists in errant vehicles as compared to their non-use. If barriers are not used in the median regions adjacent to the crossover roads, it would also seem inappropriate to locate barriers only across the region of the crossover roads. In addition, you noted that no accident data currently pointed to concerns at these crossover road locations. Thus, I would not recommend the use of short median barriers to cover crossover roads unless future research studies reveal that it is cost-beneficial to shield median crossover roads.

Single Slope Barrier Question

Question

State: OH
Date: 04-29-2010

On page 2 of 2 in the pdf named rm43_jan07 there is a requirement to construct a reinforced end anchorages at all expansion joints. When the barrier wall abuts a inlet that requires a expansion joint on each end (I-2.1_jul05_v8.pdf) an end anchorage would be required on both sides. Our previous drawing rm-4.3_4-18-03.pdf did not require an reinforced end anchorage but the note on page 1 of 2 labeled End Anchorage required "all horizontal rebar through a permissible construction joint to continuously reinforce abutting barrier".
Do you have any information or testing on why this change would have been made?
If 57" single slope barrier wall is being constructed with no rebar or foundation and abutting reinforced inlet with foundation separated with a .75" expansion joint; what would be your opinions on performance, TL, snagging potential, etc. This barrier is TL-5 along a continuous run but what would be the rating if ending the barrier run with no foundation or rebar?

Attachment: http://mwrsf-qa.unl.edu/attachments/7b2e6711d72e6948f5cd5cb9951d3708.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/f759fba996033ff8252fee1d3a226c40.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/9579305d29e3e0871a65d89f8b21e77a.pdf

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Response

Date: 04-30-2010
Thanks for the email inquiry regarding single slope concrete barriers. I have briefly reviewed your three sets of CAD details and will try to answer your questions and provide additional comments. My remarks will be provided below in **RED**.

On page 2 of 2 in the pdf named rm43_jan07 there is a requirement to construct a reinforced end anchorages at all expansion joints. When the barrier wall abuts a inlet that requires a expansion joint on each end (I-2.1_jul05_v8.pdf) an end anchorage would be required on both sides. Our previous drawing rm-4.3_4-18-03.pdf did not require an reinforced end anchorage but the note on page 1 of 2 labeled End Anchorage required "all horizontal rebar through a permissible construction joint to continuously reinforce abutting barrier".

Expansion Joints
**** Per detail RM-4.3, January 2007, it is clearly and correctly stated that the reinforced concrete end foundation anchorages are required when an expansion joint or gap is to be placed within the single slope barrier system or when terminating the barrier at its end. When such barrier discontinuities are used in rigid barriers, the redirective capacity of the barrier can be greatly reduced due to its inability to form multiple yield lines throughout the section. As a result, a common practice has been to increase the size/quantity for the vertical and longitudinal steel reinforcement at the end sections as well as to increase the embedment depth of the concrete section, such as to integrate a grade beam or footing into the end section. At expansion joints, a narrow gap is often placed completely through the entire cross section, say 2 in. Per detail RM-4.3, April 2003, the OH DOT treated expansion joints in a similar manner to that now shown in the 2007 detail. However, the length of the embedded end anchorage has increased from 10 ft in 2003 to 15 ft in 2007, which is a reasonable modification.**

Construction Joints

**** Per detail RM-4.3, April 2003, it is clearly and correctly stated that the end longitudinal steel reinforcement is to be carried across the construction joint in a continuous manner. This treatment is often handled by leaving exposed rebar segments of sufficient length out of the end of the cast-in-place or slip-formed barrier section. When the concrete construction is eventually continued, new longitudinal rebar are tied to the exposed bars and covered with concrete. Full continuity is provided at these locations when sufficient lap length is provided to ensure moment transfer across construction joint locations.**

**** In the OH DOT 2007 detail RM-4.3, it appears as though the longitudinal bars are now ended within the first concrete pour, and then ¾-in. diameter by 18" long dowel bars are used to connect the two abutting vertical surfaces to one another. In this configuration, less than 9" of bar overlap would occur. It also may be difficult to place the dowel bars reasonable close to the existing bars and ensure continuity. The required overlap to ensure moment continuity would certainly exceed 9". If full moment continuity is expected across the construction joint, then it would be recommended to extend the longitudinal reinforcement of appropriate length through the joint. Then, the new longitudinal steel would splice to the exposed steel when the concrete placement operations were continued. If the dowel joint detail is still desired, it would be necessary to ensure that moment continuity and adequate bar development is provided across the joint.**

Do you have any information or testing on why this change would have been made?

**** Unfortunately, I do not recall any prior discussions with Dean Focke, OH DOT regarding the change in details for the construction joints between 2003 to 2007. In addition, I am unaware of any test results regarding this issue. I went back to look at the original CALTRANS CAD details for the Type 60 family of barriers. When construction joints were shown, the footnote stated "Reinforcing steel shall extend continuous through construction joints." In addition, we assume full barrier capacity through construction joints since the steel is continuous (i.e., adequately lapped and developed) and concrete fills the gap. As such, no end anchorages and footings are needed at these construction joint locations.**

If 57" single slope barrier wall is being constructed with no rebar or foundation and abutting reinforced inlet with foundation separated with a .75" expansion joint; what would be your opinions on performance, TL, snagging potential, etc. This barrier is TL-5 along a continuous run but what would be the rating if ending the barrier run with no foundation or rebar?

**** I am not a proponent for non-reinforced concrete barriers even though the Ontario tall wall previously demonstrated the ability to meet TL-5 when placed within a shallow asphalt concrete pad on the front and back sides. Non-reinforced barriers would likely crack over time, even to the point where visual gaps would exist throughout the cross section. In this scenario, no rail continuity would exist, and vehicle redirection would be a dependant on a combination of several factors, including the inertial resistance of the thick concrete barrier, any bond between the barrier and support surface, and the limited structural capacity of the concrete cross section (shear, tension, torsion, bending, etc.) away from the gap location. It would be helpful to review a rough sketch or CAD detail for the configuration noted above as I am having difficulty picturing this scenario. Would it be possible to obtain such a sketch?**

NY Cable Terminal LON

Question

State: WI
Date: 05-01-2010

IF we are using the NYDOT cable barrier terminal, what is the estimated length of need for this system for impacts on both the upstream and downstream ends of the system?

Response

Date: 05-01-2010
I reviewed the NY cable anchor report to help in the determination of the beginning of length of need for impacts on your cable terminal.
For the downstream impacts, test nos. 98 and 104 are applicable.
1. Test 98 was a 1800 lb small car impacting 39.4' upstream of the downstream anchor at 55.8 mph and 11 degrees. Test no. 98 gated.
2. Test 104 was a 1800 lb small car impacting 43.5 upstream of the downstream anchor at 61.8 mph and 15 degrees. Test no. 104 gated.
3. Test 100 was a 4780 lb sedan impacting 99' upstream of the downstream anchor at 57.7 mph and 23 degrees. Test no. 100 redirected.
Based on these tests, it appears that the minimum length that you can assume for redirection for reverse direction impacts would be greater than 43.5' upstream of the anchor. However, the minimum length from the downstream anchor where redirection will occur cannot be accurately defined from the data in the report. Test no. 100 on middle of the system LON was conducted 99' upstream of the end anchor and redirected. This would indicate that the system is capable of redirection for vehicles impacting 99' upstream of the downstream anchor. This is likely a conservative estimate, but it is all we have for a basis.
For the upstream impacts, test no. 107 is applicable.
1. Test 107 was a 4850 lb sedan impacting 38' downstream of the upstream anchor at 56.5 mph and 25 degrees. Test no. 107 redirected.
The results from test no. 107 indicate that the system is capable of redirecting impacting vehicles 38' downstream of the anchorage. The IS value for this test is approximately 9% less than the NCHRP 350. This test had a 38% higher IS value than NCHRP 350 test 3-35, so it is safe to assume that the system can safely redirect vehicles impacting a minimum of 38' downstream of the anchorage.

Guardrail Over Box Culvert

Question

State: WI
Date: 05-05-2010

A project has to install a beam guard over a box culvert. Two options would be to use the long span guard rail or attach to the box culvert using a plate. Although both alternatives are crash tested, it appears that some modification will be needed to fit the given location.
If the long span detail is used, they will not be able to get the 2' grading behind the post, Is it possible to use longer post with the long span detail?
If the beam guard post are attached to the box, the post will have to be longer than what was crash tested (see PDF). At what depth, top of finished surface to top of box, can the plate detail be used?
At the pooled fund meeting there was some discussion about changing the welding the post to the plate. Could you forward me an updated detail?
I assume at a certain point if there is adequate soil mass behind the post, could shorter post with decreased post spacing be used?

Attachment: http://mwrsf-qa.unl.edu/attachments/132977f6f576818a1a2bf741e6a75ca5.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/9e4895e3277fd72e3c93306c958f15de.pdf

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Response

Date: 06-11-2010
MwRSF has successfully developed and crash tested two W-beam guardrail systems to span across long concrete box culverts, such as those measuring up to 24 ft in length. For the first system, the metric-height W-beam guardrail was configured with a 27-3/4-in. top mounting height, while the Midwest Guardrail System (MGS) was utilized for the second configuration with a 31-in. top mounting height. For both designs, three 6-in. x 8-in. by 6-ft long wood CRT posts were placed adjacent to the long span using the 6-ft 3-in. post spacing. Beyond the CRT wood posts, the guardrail system was transitioned into a steel post, wood block, semi-rigid barrier system which also used 6-ft long posts and a 6-ft 3-in. post spacing. For both crash-tested systems, a region of level, or relatively flat, soil fill was provided behind the CRT wood posts.
For some situations, you noted that it may be difficult to provide 2 ft of level, or mostly level, soil grading behind the wood CRT posts. As such, your inquired as to whether the wood CRT posts could be lengthened to account for the reduction in soil resistance resulting from an increased soil grade behind these six posts, especially when placed at the slope break point of a 2:1 fill slope.
MGS
Recently, MwRSF performed limited research to determine an acceptable MGS post length for a 6-in. x 8-in. solid wood post installed on 2:1 fill slopes. Although unpublished at this time, MwRSF determined that 7.5-ft long wood posts are an acceptable alternative to W6x9 by 9-ft long steel posts when considering the 31-in. tall MGS placed on a 2:1 fill slope using a 6-ft 3-in. post spacing.
The MGS Long Span system utilizes six CRT wood posts. A CRT post's moment capacity about its strong axis of bending is approximately 81 percent of that provided by the standard wood post. In the absence of dynamic component test results, it is believed that the six CRT wood posts could also be fabricated with the 7.5-ft length when used in the MGS Long Span system. If the steep fill slopes continue beyond the location of the CRT posts, then the guardrail would transition to the MGS for 2:1 Fill Slopes using either 6-in. x 8-in. by 7.5-ft long wood posts or W6x9 by 9-ft long steel posts.

Metric-Height W-beam

For the metric-height, W-beam guardrail system configured for long-span culverts, it would seem reasonable to utilize three 7-ft long wood CRT posts adjacent to each end of the box culvert if 2:1 fill slopes are present in this region. If the steep fill slopes continue beyond the location of the CRT posts, then the guardrail would transition to the metric-height, W-beam guardrail system for 2:1 fill slopes using W6x9 by 7-ft long steel posts spaced on 3-ft 1-1/2-in. centers. However, this half-post spacing system resulted in slightly decreased lateral barrier deflections as compared to those observed for standard W-beam barriers with 6-ft 3-in. post spacing. Thus, it would also seem appropriate to provide two 7-ft long W6x9 steel posts at 6-ft 3-in. spacing (i.e., 12 ft - 6 in.) between the last 7-ft long wood CRT post and the start of the half-post spacing. Therefore, all posts beyond the last wood CRT post would be configured as 7-ft long W6x9 steel posts placed at the slope break point of 2:1 fill slopes.

It should be noted that this guidance is provided using our best engineering judgment in the absence of full-scale crash testing, computer simulation, dynamic component testing, or combination thereof. If new information becomes available, MwRSF may deem it necessary to revise this guidance.

Based on the success of MGS Long-Span system, MwRSF now believes that the 1.5 m lateral offset requirement for the Metric-Height, Long-Span, W-beam Guardrail System is overly conservative for culvert slabs covered by mostly level soil fill. As such, it is MwRSF's opinion that the minimum lateral offset between the back side of the CRT wood posts and the front face of the headwall can be reduced from 35 in. to 24 in. while providing comparable safety performance. With this adjustment, the minimum recommended lateral offset between the back side of the rail and the front face of the headwall would be approximately 48 in. or 1.22 m. for the metric-height variation. In addition, it is MwRSF's opinion that the Metric-Height, Long-Span, W-beam Guardrail System has the potential to be placed even closer to the front face of the culvert headwall. However, further reductions in the minimum lateral offset could only be evaluated through full-scale crash testing.

Recently, you requested information regarding the soil fill depth where one would switch from the culvert-mounted, W-beam guardrail system to the standard W-beam guardrail system with posts embedded in soil without special anchorage.

Several years ago, MwRSF developed a metric-height, W-beam guardrail system for attachment to the top slab of a concrete box culvert when structure lengths exceeded 25 ft. The new design utilized an anchored post spaced on 3 ft " 1-1/2 in. centers. Each post was also configured with a welded base plate capable of absorbing energy upon impact. The testing program used a "practical" minimum soil depth of 9 in. Upon completion of the successful testing program according to TL-3 of NCHRP Report No. 350, it was recommended that the back of the posts be placed a minimum of 10 in. from the front face of the culvert headwall. A dynamic barrier deflection of approximately 16.5 in. was observed.

The noted crash testing demonstrated that the barrier system performed in an acceptable manner with 9 in. of soil fill. The researchers also believe that the barrier system would have performed in an acceptable manner with soil fill depths of approximately 43 in., thus replicating the expected safety performance of half-spaced posts used in combination with metric-height, W-beam guardrail systems. For soil fill depths of 43 in. on culvert slabs, it would seem unnecessary to utilize a barrier system with anchored posts or a half-post spacing. Instead, a standard, full-spacing, metric-height, W-beam guardrail system would be used on a culvert if adequate soil fill depth is provided along with a minimum of 2 ft of level (or mostly level) terrain behind the posts.

If the soil fill depth is less than 43 in. but greater than 3 ft, it would seem both desirable and reasonable to use a barrier system that does not require attachment to the culvert slab. Unfortunately, no research has been performed to determine the minimum post length/embedment depth for metric-height, W-beam barriers to meet the TL-3 safety performance guidelines. However, we believe that a W-beam guardrail system with a slightly reduced post length and reduced post spacing would have a high probability for meeting current impact safety standards, especially if configured with the 31-in. top mounting height. However, satisfactory barrier performance can only be determined with the use of full-scale vehicle crash testing.

Barrier Flare Rates

Question

State: WI
Date: 05-06-2010

I have a project where project staff may have to flare beam guard at a greater rate than what is listed in table 5.7 of the RDG. What research was used to develop this table? Does MwRSF have any guidance on this topic?
Just yesterday, I was asked about flare rates for concrete barrier. Does MwRSF have some research on this topic?

Response

Date: 05-19-2010
I am attaching a pdf copy of a MwRSF research report regarding flare rates. As noted in the MwRSF Report No. 157, the guardrail flare rates were determined by James Hatton, FHWA. Unfortunately, I do not believe that these flare rates were based on actual full-scale vehicle crash testing.
Later, MwRSF conducted a flare rate study involving the MGS which shown that flare rates as steep as 5:1 were acceptable. I have provided a link to download this report if you cannot find it. Of course, our guidance in this report pertains only to the MGS.
The file 'TRP-03-191-08.pdf' (86.7 MB) is available for download at
for the next 7 days.
It will be removed after Wednesday, May 26, 2010.
Finally, I have included a pdf copy of a journal paper covering the flare rate topic that may also provide more refined conclusions and guidance. In Section 8 of the paper and based on computer simulations, the authors note that the modified G41s may not perform effectively when installed with flare rates steeper than 15:1 under TL-3 impacts under NCHRP Report 350.
Years ago and while at TTI, Dean prepared guidelines for temporary concrete barrier. I believe those guidelines are published in NCHRP Report 358.

Attachment: http://mwrsf-qa.unl.edu/attachments/ee28ba55f7a2c1fe3586b0a7b0e36bdc.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/0625dccc7ba784614b9b48d906e15cf0.pdf

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MGS Spacing Guidelines

Question

Date: 05-06-2010

Please review the attached excerpt from the draft Tollway Traffic Barrier Guidelines manual that we are working on.
What I am proposing does not use special posts and does not eliminate any posts, but limits the maximum and minimum post spacing to try to control the changing rigidity. Let me know what you think. The max and min values are just numbers I made up for discussion purposes. They could be more or less if you are comfortable with the idea. Maybe my idea works up to a certain size drainage structure and then we go to CRT posts???
Is the purpose of the CRT to prevent pocketing?

Attachment: http://mwrsf-qa.unl.edu/attachments/1fbbc2cb22183993c6f3fadbcc04d8f4.docx

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Response

Date: 07-01-2010
I believe that establishing a maximum and minimum post spacing for these special applications without the need for CRT post (or other specialized posts) has merit. The minimum spacing you proposed is very near a ½-post spacing (3' vs. 3'-1.5"), thus it seems reasonable. Along the same lines, I would consider 150% of standard post spacing as an acceptable maximum spacing limit. Your proposed limit is very close to this value (9'-6" vs. 9'-4.5"), and thus also seems reasonable.
I also agree with and encourage your statement to keep the post spacing as uniform as possible in these situations in order to prevent large variations in stiffness that cause pocketing.
One important thing to note here, these post spacing variations should only be applied to standard segments of the guardrail system. Guardrail transitions and terminals are carefully designed to accommodate increases in stiffness along the system. Therefore, these general rules do not apply and any variations to a transition or terminal need to be individually analyzed.
To answer your question about the use of CRT posts in the Long-span system " They reduce the affects of vehicle snag on the posts. CRT posts are designed to maintain bending strength about the string axis (laterally) but are substantially weaker about the longitudinal direction. Thus, when a vehicle contacts a CRT post, it will fracture or break away and consequences of vehicle snag are minimized.
I like both transitions (to ½ post and to ¼ post spacing) and only have a few comments.
First, the distances shown from the hazard/obstruction to the beginning of the transition segments should be shown as minimums. There hazards may not line up as nicely as shown in your drawings, making these exact distances not possible. By stating the minimum lengths required you cover all situations.
Second, in circumstances where the hazard/rail is susceptible to impacts from vehicle traveling in the opposing traffic direction, you would want to make the transition symmetric about the hazard (i.e., change the downstream portion to match the upstream.
Third, the length of ¼ post spacing prior to the hazard must be greater than 7.5 ft (as determined from the full-scale test NPG-6 as the distance from contact to maximum deflection). You currently have 12.5 ft listed for this distance, thus it could be shortened slightly. I would recommend a minimum of 9 ft of ¼ post spacing prior to the hazard. However, 12'-6" is conservative and could still be used.

Approach Transition Post Alternatives

Question

State: NE
Date: 05-07-2010

Please review and comment.
Latest changes:
Is a w8x21 " 7' a quality substitute for the w6x25 " 7' for posts 1-3?
These match the 2000 Texas Transportation Institute TTI testing to NCHRP 350 - Contract No. DTFH61-97-C-00039 as well as letter from Ron faller " Sept 26 2002.
Posts at the end of the nested thrie-beam and through the 6'-3" single thrie-beam and 6'-3" transition to w-beam follow testing performed by MwRSF Research Report No. TRP-03-210-10 "Design K".
Testing also known as:
Midwest States' Regional Pooled Fund Research Program
Fiscal Years 2008-2009 (Years 18-19)
Research Project Number SPR-3 (017)
NDOR Sponsoring Agency Code RPFP-08-05
The need for using two tests to justify the bridge approach section comes from the later test using a thrie beam bridge rail instead of transitioning to a concrete rail.

Response

Date: 05-24-2010
A W8x21 post has a flange width of 5.25 in. while the tested W6x25 had a flange width of 6 in. This could cause significantly differences to force-deflection characteristics of these posts. This coupled with the 10% strength difference between the posts raises too many unknowns for me to say that the posts can be used interchangeably.

With your alternate assembly design when you are specifying W6x25 posts, hopefully you are putting these larger posts after the W6x15 posts. The W6x25s cannot simply replace the W6x15s (at least without component testing and analysis to show there isn't any snap problems)

Finally, one minor error I see in the drawings " the post 3'-1.5" post spacing is labeled as between post nos. 7 and 10. It should be 7 " 11. It's drawn correctly, just labeled incorrectly.

ZOI

Question

State: KS
Date: 05-16-2010

I have a question for you about ZOI research that you guys did several years ago.
On the summary diagrams for TL-3 barriers, you show a 24" lateral distance for vertical and only 18" for a safety shape. Why is the safety shape less? I would have thought a 32" vertical would have been less.

Response

Date: 05-17-2010
For permanent, sloped-face, concrete barriers, the front-impact-side wheel will begin to climb the barrier face and both result in both vertical rise and roll away from the barrier. For most vertical-face barriers, the engine hood and crushed quarter panel will have a maximum lateral extent over the parapet due to reduced vehicle climb as compared to sloped-face barriers.

MGS Lower Height Tolerance

Question

State: KS
Date: 05-18-2010

With the improvements in the MGS, how much down tolerance do we have? Will it pass at 27" or lower?
I understand that in light of the G41S tests that got raised to 27 ¾" by virtue of metric rounding for English units. So how far down do you think the MGS could really go since it is a much better system then the G41S. Is it worth doing some modeling or testing. The reason I ask is in regards to maintenance activities, leave in place on future 3R projects, etc.

Response

Date: 05-18-2010
We have been recommending a downside tolerance of 27¾ in.
My opinion is as follows:
MASH " maybe 27" to 27½"
NCHRP 350 " maybe 27½" to 27¾"

TL-2 Guardrail Installation Height

Question

State: KS
Date: 05-20-2010

Do you know what the lowest installation height for wbeam guardrail (G41S)would be for a TL-2 application?
The reason I asked that we may be able to use this as guidance for leave in place guardrail for routine maintenance. I recall that Dean was doing research on approipriate test levels for roadways that was based on a B/C type analysis. In other words a TL-2 device could be
justified on a 55+ mph roadway in lieu of TL-3. Does this sound
familiar to you and is that done?
The tolerance for routine maintenance is still needed. Requiring DOTs to upgrade all guardrail that is less then 27.75" except on 3R/4R or new construction is not practical. As you already know there are many
other improvements that would have much better B/C ratio. Such as
horizontal curves, sight distance, and access control projects to name a few.

Response

Date: 05-20-2010
We have investigated the guardrail height issue. The findings indicate that 27"
should be able to work at TL-2 impact conditions. We even went as far as investigating 25" and the results were inconclusive if it had a chance of working at TL-2 impact conditions.
In addition, as written in the pre-proposal for the TL-2 MGS Bridge Rail that Kansas submitted during the past pooled fund meeting, previous testing of standard W-beam guardrail and the MGS with rail height of 27" and 27-3/4"
indicate that a 27" tall MGS Bridge Rail is plausible if limited to TL-2 applications.

27â€• W-beam Guardrail Testing

Question

State: KS
Date: 05-26-2010

Do you guys know of any successful tests on 27" W-beam per TL-3? The reason I ask the RDG makes reference to these and others are saying it has never passed at 27". If so can you provide me some basic information, i.e. date, TL, w-beam height, testing facility, etc.?
The RDG will specify 27.75" minimum with emphasis to use taller w-beam.

Response

Date: 06-10-2010
Here is the information on prior crash testing of 27" W-beam guardrail that you requested.

Attachment: http://mwrsf-qa.unl.edu/attachments/650cd9d0f849c5fbfb1abd705aeab931.docx

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Cable Barrier Anchor Post

Question

State: NE
Date: 06-02-2010

1.) Can the soil; plate be bolted to the post after being driven?
The installer would like to drive the post without the soil plate attached.
What size bolts and how many could be used?
2.) Post #2 " the Cable Bracket Detail shows a cut to allow a ¼" x 1.5" piece of metal cut to fold down and hold the cables in place. These break easily sometimes during installation and regularly in a crash.
Are these mainly for installation convenience?
Do these need to be in place for the guardrail to perform properly?
If they are missing does the post need to be replaced?
Is there other non proprietary/ approved methods of hooding these cables in place without being too strong?

Attachment: http://mwrsf-qa.unl.edu/attachments/e6681a0541a4fb22c62b0fe8348fe5cc.jpg

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Response

Date: 06-14-2010
Answers to your questions...
1) Yes the soil plate can be added after the post is driven into the ground. You will have to dig a hole large enough to properly place the plate on the post " note it is 2 ft wide and extends 2.5 ft below the surface. Also, you will need to compact the soil around the post/plate when the hole is filled. Uncompacted soil will not generate the necessary resistance for the anchor. We use a pneumatic tamper and install soil in 8 inch lifts when we compact soil at our test site.
You can attach the plate to the downstream post flange using four 3/8 in. diameter bolts " two near the top and two near the bottom of the plate.
2) Leaving the slots open shouldn't negatively affect the safety performance of the system. I believe they were originally designed for maintenance issues. I am not aware of any additional approved methods of holding the cables in the slots.

Cable Barrier Adjacent to Slope

Question

State: NE
Date: 06-04-2010

Could we simulate a cable guardrail 2' from the edge of a 2:1 slope?

Was this simulated a few years ago with the NCHRP 350 vehicles when the testing was performed on the flat to a 1.5:1 slope?

What effect will using the MASH 09 vehicles have on these previous simulations?

I would like to keep the front tire on the slope by simulating our typical cross section a 4% shoulder slope in front of the guardrail. The 2' behind the cable we normally break to a 6:1, then the 2:1 slope.

When we place cable guardrail 2' from a 2:1 our plan specifies S 3 x 5.7 x 7' posts with soil plates on 16" max. spacing.

The new inline end section is what we would use in the future to anchor this " if this makes a difference.

What would help this placement?

Closer post spacing?

Response

Date: 06-13-2010

I will try to address some of your questions below with my responses in red.

Could we simulate a cable guardrail 2' from the edge of a 2:1 slope?

> Yes, we can simulate the cable guardrail 2' from the edge of a 2:1 slope. We proposed similar research regarding a low-tension version of the 4 cable median barrier at this year's Pooled Fund meeting. The cost for this kind of analysis would be in the $35K range to do the analysis. If you want to formally address this, we can develop a proposal and budget. The simulation analysis will provide guidance on this issue, but full-scale testing will likely be required in order to fully address this issue.

Was this simulated a few years ago with the NCHRP 350 vehicles when the testing was performed on the flat to a 1.5:1 slope?

The simulation effort for the 1.5:1 slope was done using Barrier VII and would not address some of the slope changes you are proposing.
The previous analysis looked solely at the effect of reducing the post spacing on barrier deflection and did not address the interaction of the vehicle and slope.

What effect will using the MASH 09 vehicles have on these previous simulations?

Using the 2270P vehicle would likely result in additional barrier deflection. In addition, the higher CG for the 2270P vehicle could potentially adversely affect the capture of the vehicle we saw in our previous testing of the 2000P adjacent to the 1.5:1 slope.

I would like to keep the front tire on the slope by simulating our typical cross section a 4% shoulder slope in front of the guardrail. The 2' behind the cable we normally break to a 6:1, then the 2:1 slope.
When we place cable guardrail 2' from a 2:1 our plan specifies S 3 x 5.7 x 7' posts with soil plates on 16"max. spacing.

This type of installation could be modeled. However, based on our previous experience with the 2000P testing on 1.5:1 slope, the cable barrier might require reduced post spacing to effectively capture the vehicle.

The new inline end section is what we would use in the future to anchor this " if this makes a difference.

What would help this placement?

Lots of factors including, cable tension, post spacing, cable spacing, and post offset could all have effects on this type of installation.

Vertical Barrier

Question

State: KS
Date: 06-07-2010

Do you have some details of a 32" vertical barrier with thrie beam attachments? I appreciate it.

Response

Date: 06-08-2010
I have attached a few items for you. MwRSF had previously taken a thrie beam to safety shape transition and converted it to a vertical shaped wall. The design was submitted to FWHA for acceptance - both the drawing and the acceptance letters are attached. Also, TTI conducted a test on a thrie beam to vertical wall transition. The Ohio DOT drawing and the acceptance letter are attached.
Attachment: http://mwrsf-qa.unl.edu/attachments/eceb039e3896b12d27658363214a67e6.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/2c21982c20bdff22a89b0e29397a6ff4.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/295068b56e132e5dd8647a3fedff4833.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/e3804475e2effe1874d04e61593cb5a2.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/608fcf8bd10f4ec746ea1b2957ef8098.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/63c409b5eec7cab85f89ff035f6b7c83.pdf

Attachment: http://mwrsf-qa.unl.edu/attachments/3301d2a9476faad4b31dcf5764002760.pdf

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Motorcycles and Pavement Drop-Offs

Question

State: IL
Date: 06-11-2010

Can you recommend good research for us to review regarding edge drops and motorcycles? Of special interest is the milled edge at lane or edge locations.

Response

Date: 06-14-2010
I do not know of any specific studies regarding motorcycles. I sent an email to Dr. Clay Gabler at VT who has done a large amount of motorcycle accident research. If anyone knows about a motorcycle accident study in this area, it would be him.
His reply was as follows.
"I can see how motorcycle and a pavement edge dropoff could be a tough problem. Probably even tougher than a car which attempts to steer too quickly back onto the highway after going off the pavement edge. However, we have not come across any studies to date on the motorcycle crashes involving pavement dropoff. Likewise, none of the crashes that we have investigated to date have involved this as a crash causation mechanism. "
At this time, it doesn't appear that there is much guidance or study in this area. I will keep a lookout for studies and let you know if I find any.

FLDOT Median Barrier

Question

State: FL
Date: 06-11-2010

Please take a look at the lightly reinforced concrete median barrier shown in the attached PDF file. Have you ever seen anything like this one? What test level criteria do you think this would meet?

It is difficult to determine from the drawing, but there are three #4 longitudinal bars (one near the top and two near the bottom) shown in "Section A", which is most likely intended to resist lifting and handling stresses. Since the drawing is not to scale, the aspect ratio of the elevation view may be misleading. There are only four anchors per unit (anchors paired transversely, then spaced at +/- 10.5' o.c. longitudinally). Would this affect your opinion?
Attachment: http://mwrsf-qa.unl.edu/attachments/47b8d16c0c02f7a9c277a9cb3d173891.pdf

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Response

Date: 06-14-2010
I have looked over the median barrier detail you sent and I have some concerns with it.

1. The barrier has no longitudinal steel that I can identify. As such, I the capacity of the barrier sections are very limited and you could expect a large amount of fracture and cracking in any impact.
2. It appears that the barrier sections are in segments with no connection between them. This has been show to be very detrimental to impact performance. When the a vehicle impacts one segment, you get a large shear displacement of the unconnected segment relative to the next segment downstream. This leads to snag on the downstream barrier which can cause excessive decelerations and vehicle instability. In addition, the lack of continuity between barrier segments increases the load on any one section that is impacted. Thus, with the lack of reinforcement in this barrier, you can expect an even higher level of barrier fracture and damage.
3. The anchorage capacity used for the sections is difficult to judge. The anchors do not appear to have a lot of capacity, but they are closely spaced. However, these anchors will do very little to address the two points above.

I see the longitudinal steel now. This is still a very low amount of reinforcement for a TL-3 barrier and I would still expect the damage to the concrete sections to be very high. I can only see two anchor per unit, and they appear to be 1' apart longitudinally. Additional anchorage may help on some level, but the if the barrier is constrained more rigidly, the loads in the sections will increase. Thus, the lack of reinforcement becomes a more significant issue. For example, we tested the F-shape PCB section in as similar bolted down configuration at TL-3 with the 2000P vehicle , we observed cracking and fracture of the barrier section completely through the mid span of the barrier. Subsequent testing of the F-shape barrier with the 2270P vehicle has shown similar levels of damage in less constrained configurations. This PCB section has much more reinforcement and an joint connection to help distribute loads and it appears to be at or near its peak capacity. Thus I would assume that this section will not fare as well.
The concerns for lack of connection and continuity still hold true regardless. Thus, I would still be skeptical of the barriers TL-3 performance, but there may be potential for TL-2.
I have not seen a detail like this before. I would not expect this system to pass TL-3 of NCHRP 350 or MASH. There may be some chance of it passing at TL-2, but more analysis would been needed.

Chamfer Allowed On Temporary Concrete Barrier

Question

State: WI
Date: 06-15-2010

Attached is the SDD for our 12' -6" temp conc barrier. While we've had this standard for awhile, we were also phasing out our former 10'-0" barrier, so our experience is relatively recent.

The primary feedback we've gotten is that the new barrier is much more susceptible to breaking/spalling along the edges.

I received another phone call today that after only 2 or 3 uses they are seeing 10 -20% of the barriers showing these problems; whereas, with our 10 ft barrier they'd typically get 10 " 15 uses before seeing this kind of damage. So, they are asking if vendors can add a ¾-in chamfer to:

- The front and back bottom edges
- The vertical edges on the ends

You'll see we allow ¾-in chamfer on the bottom edge on the ends now.

Attachment: http://mwrsf-qa.unl.edu/attachments/3ca86923db530becfbb3bdc2338421f8.pdf

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Response

Date: 06-15-2010
We have no issues with allowing the ¾" chamfer you are requesting on the front and back bottom edges and the vertical edges on the ends. Florida has been using similar chamfers for some time now to reduce that kind of damage.

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.

Curved Rail W-beam Transition and Exposed Concrete Edge

Question

State: MO
Date: 06-25-2010

During your move I had asked whether or not you viewed the curved parapet treatment (shown on photos that I sent you) to be crashworthy. You mentioned some research that had been done by TTI, but as you were mid-move, you were unable to put your hands on the data. Have you been able to locate it and do you have an opinion?
Since then, another issue has come up. We are replacing hundreds of bridges around the state in the coming few years, and in an effort to make the undertaking as cost-effective as possible, we have adopted a 350-approved bridge end connection that employs two double-nested w-beams for stiffness at the end. This arrangement bears the federal approval number HMHS-B65 if you wish to view its test summary and drawings. The FHWA approval letter cautions practitioners to taper the parapet wall to the top of the rail to avoid snagging potential.
Unfortunately, we've recently realized that about 30 bridges have been built without the taper (as shown below). In fact, the condition in the field has the parapet height about 2 inches above the rail. The wall does have a 3/4 inch chamfer around its top and sides.
My questions are these. Do you see a snag potential in the as-built condition? If so, is it drastic enough for us to fix it? Grinding the concrete taper is not an option so the only remedy would be to raise the rail: an exercise we don't want to undertake. Of course we will if there is a danger to the public.

Attachment: http://mwrsf-qa.unl.edu/attachments/f2f1bc0ff69ec560721ff5f0c10577df.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/0eaf2fceacf6cd7ca5b70b6c1a976355.jpg

Attachment: http://mwrsf-qa.unl.edu/attachments/361c3a8a63e5c1536cf9b10d098479f0.jpg

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Response

Date: 06-25-2010
I have reviewed a few prior development and crash testing efforts regarding the exposure of concrete buttresses above the rail element in approach guardrail transitions, more specifically two systems that were developed with funding from the Pooled Fund Program.
For the first barrier system, a thrie beam approach guardrail transition was developed and crash tested for use with a half-section New Jersey shape concrete barrier. The thrie beam's top mounting height was 31 in., while the top height of the concrete parapet was 32 in. The top horizontal edge of the concrete parapet's upstream end was not chamfered. As such, there was 1 in. of exposed concrete above the 31-in. tall thrie beam. For this design, the 1-in. of exposed concrete did not result in excessive vehicle snag on the upstream end.
For the second barrier system, a thrie beam approach guardrail transition was developed and crash tested for use with a single-slope concrete median barrier. The thrie beam's top mounting height was 31 in., while the top height of the end concrete parapet was 31 in. but with the concrete surface sloping upward at an 8:1. The top horizontal edge of the concrete parapet's upstream end was not chamfered. As such, there was no exposed concrete above the 31-in. tall thrie beam except for the upper 8:1 sloped surface.

For the approach guardrail transition system shown below, there is approximately 2 in. of exposed concrete above the upper W-beam guardrail. The original crash-tested transition system (link provided below) was configured with the concrete end flush with the top of the W-beam rail, thus mitigating any concerns for the engine hood and front quarter panel to snag on the concrete. Due to the lower W-beam rail height as compared to existing thrie beam transitions and the 2-in. exposed height, it is recommended that the transition system be retrofitted to mitigate the potential for vehicle snag on the upper concrete edge.
http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/pdf/b-65.pdf
MwRSF would be willing to discuss and brainstorm potential options for safely retrofitting the transition system to mitigate the vehicle snag concerns. I look forward to hearing from you in the near future.

Attachment: http://mwrsf-qa.unl.edu/attachments/43ce716b7cf36f247595877b63e822f0.docx

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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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