Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

04-01-2012 to 07-01-2012

Transition from Temporary to Permanent Barrier

Question

State: WI
Date: 03-26-2012

Bob,
I know that we talked about this a long time ago (This project keeps getting put on the back burner because of other issues. So, I lose my notes about it.).
What tension and shear values should I put on the anchors located on the attached drawing?
I know that the system was crash tested using asphalt stakes anchoring down the temporary barrier. I'm assuming that it is OK to use the same pattern for anchoring the barrier down on concrete as well.

Attachment: https://mwrsf-qa.unl.edu/attachments/7aef7f5187372aa1f2d0f3f7ae39ee10.pdf

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Response

Date: 03-27-2012
There are two types of mechanical anchors specified in the attached detail.

For the 3/4" dia. x 6" long Powers Fasteners mechanical anchors, we would recommend that the manufacturers listed ultimate strengths be followed as guidance for any alternative anchor. Powers currently lists the ultimate shear and tensile capacities of those anchors in 4,000 psi concrete as 17.9 kips and 21.96 kips, respectively.

http://www.powers.com/pdfs/mechanical/07246BT.pdf

For the Red Head Multi-Set II RL drop-in anchor, the manufacturer lists the ultimate tensile loads of 9.48 kips for 4,000 psi concrete and ultimate shear loads of 10.48 kips for concrete strengths over 2,000 psi. We have tested these anchors dynamically to significantly higher loads, but we would stick with the manufacturers listed strengths if you are going to include them on your details.

http://www.itwredhead.com/pdfs/submittals/multi-set.pdf

We do allow the transition design to be used with the bolt-through tie-down option when applied to transitions on concrete surfaces using the configuration shown. The asphalt pin and bolt-through tie-down systems are believed to possess similar lateral restraint and thus can be interchanged in the transition design as needed.

Response

Date: 04-02-2012

Bob,

As I was reviewing the drawing, I did find another mechanical anchor that I would like the strenght requirements for. The mechanical anchors are 5/8"x 4" power fasteners wedge-bolt anchors that attach the top of the cap to the permanent and temporary barrier.

Response

Date: 04-02-2012

The ultimate shear and tensile capacities for the 3/4" dia. x 4" long Powers Fasteners mechanical anchors in 4,000 pis concrete are 12.14 kips and 17.5 kips, respectively.

Transitioning 31

Question

State: DC
Date: 06-24-2009

I got the question below about transitions
between existing 27 inch high w-beam barriers and repairs made using 31 inch high versions. The specific question below also asks about using a 27 inch high terminal when the owner wants to use it to terminate a new 31 inch high LON barrier.

1) Based on your MGS research and your vast experience with 27 inch high barriers, is there an appropriate transition length over which you may "safely" raise or lower w-beam rail by 3 inches? 1a) Assume all other features are equal. 1b) Assume known differences like G4-1(S) transitioning to MGS or vice-versa.

2) If an owner thinks they can save money by specifying a 27-inch high w-beam terminal when using 31 inch high barriers, what is the "safe" place to begin the 3-inch rise?

These questions assume the owner is basically satisfied with the performance of 27-inch high barriers and or terminals but wants to begin moving to 31 inch systems whenever possible. If these are questions that should not be trifled with unless crash tested, that, too, is an answer.

Response

Date: 04-04-2012
See response below in red.

I got the question below about transitions
between existing 27 inch high w-beam barriers and repairs made using 31 inch high versions. The specific question below also asks about using a 27 inch high terminal when the owner wants to use it to terminate a new 31 inch high LON barrier.

1) Based on your MGS research and your vast experience with 27 inch high barriers, is there an appropriate transition length over which you may "safely" raise or lower w-beam rail by 3 inches? 1a) Assume all other features are equal. 1b) Assume known differences like G4-1(S) transitioning to MGS or vice-versa.

** For comparison purposes, I assume that you are referring to metric height guardrail placed with a 27.75" top rail height. My best recommendation would be to transition the 3.25" height differences over approximately 50 ft or two 25-ft long sections of W-beam guardrail. With this reasonable grade change, I am confident that there would not be any complications nor degrading effects on the barrier's safety performance. Only increasing the rail slope to 3.7 degrees should also allow for a relatively easy placement of the eight splice bolts within the lapped joints.

2) If an owner thinks they can save money by specifying a 27-inch high w-beam terminal when using 31 inch high barriers, what is the "safe" place to begin the 3-inch rise?
** To be reasonable sure that the safety performance is not degraded and without testing, I would "conservatively" recommend that the height difference not begin until the end of the downstream end of the terminal is reached.

These questions assume the owner is basically satisfied with the performance of 27-inch high barriers and or terminals but wants to begin moving to 31 inch systems whenever possible. If these are questions that should not be trifled with unless crash tested, that, too, is an answer.

Anchoring Temporary Barriers over Bridge Expansion Joints

Question

State: MN
Date: 04-07-2012

The Minnesota DOT is looking to implement standards for anchoring temporary barrier over bridge expansion joints. Early discussions have led us to reviewing 2 standards used by the Kansas DOT (attached). The first drawing is for situations where the anticipated movement at the expansion joint is less than 1.5". We like this standard and are wondering if you know of its history or use by other states? Do you have any recommendations or suggestions regarding anchoring barriers over expansion joints or know of any other examples that have been crash tested?

The second attachment was developed by John Jones at the Kansas DOT. Even though the barrier is only rated to TL-3, John designed the wire rope that runs through the barriers to take a TL-4 load, independent of any other connections. He also designed the 1" dowel between the barriers to take a TL-4 load without any other support. Lastly, he designed the steel cover plate to take a TL-4 load without any support from the wire rope or 1" dowel. Hence, this design should be very conservative as each "link" was independently designed for TL-4. He indicated the system has NOT been crash tested. Are you aware of any other state details or crash tested systems that can accommodate more than 1.5"? John thought this system could be used over expansion joints with movement up to 4-5 inches.

Any thoughts, recommendations, or references to other state standards or details would be greatly appreciated.

Attachment: https://mwrsf-qa.unl.edu/attachments/502eea604b2d331ebd4eeae01c469643.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/806c59db15e36727507f0cfb80fdfcee.pdf

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Response

Date: 04-13-2012

We are aware that the KsDOT developed a design detail for providing temporary positive protection across an expansion joint utilizing a pinned, F-shape TCB in combination with dowel bars, interior cables, and a steel cap plate. We were involved with some of the initial discussions but did not conduct any type of analysis. Instead, it was my understanding that the KsDOT Bridge Division conducted an FEA analysis to verify the structural capacity and investigate plate deformations within the cap plate. I believe that the KsDOT has implemented this system in a couple of locations over the last several years but have not seen photographs of the system nor feedback on how it worked.

A couple of years ago, the MoDOT also investigated the KsDOT system for use in a particular application. Joe Jones of MoDOT was involved in this discussion. Since I do not know what transpired from this situation, you may want to contact Joe directly to further investigate whether the steel cap plate and other KsDOT details were ever implemented. I have provided an electronic pdf copy of the prior email correspondence on this issue that occurred between MwRSF, KsDOT, and MoDOT. Additional details on gaps and lengths were provided by KsDOT.

Over the years, the Florida DOT has implemented significant details for the F-shape concrete barrier segments into their state standards. I would suggest that you review their many details which may provide additional insight for spanning expansion joints with pinned TCBs. MwRSF has provided review and comment on many of their TCB details when asked.

http://mwrsf-qa.unl.edu/view.php?id=285

Finally, I am not aware of any other details or anchored temporary barrier systems that have specifically accommodated expansion joints/gaps.

Please let us know if you have any further questions or comments regarding the limited information provided above. I am sorry that we could not be of more help on this matter.

Attachment: https://mwrsf-qa.unl.edu/attachments/7274a4f88367c75a41ec5dab5bac272f.pdf

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MWTSP with vertical-faced bridge rail

Question

State: IA
Date: 04-11-2012

What would it take to evaluate the attachment of the steel post Midwest transition to a vertical-faced concrete bridge rail (no tubes/channels connecting the bridge rail to the first guardrail post)? I'm specifically referring to the bottom drawing of Figure 96 on page 167 of TRP-03-210-10. I am interested in attaching this directly to a vertical-shape bridge end post with perhaps some sort of chamfer at the end to reduce snagging potential. Would this be something you might be able to recommend based on some investigation and simulation, or would it require a crash test (or two)?

Response

Date: 04-12-2012
In general, I believe that acceptable safety performance would be obtained with the attachment of the AGT (bottom – Figure 96) to a vertical concrete buttress with the minor flared back end. We know that this detail has been successfully evaluated when attached to steel post - steel beam bridge railing systems with an additional tube or channel member carried from the bridge to the first few transition posts. We believe that the removal of the backside beams and attachment to a concrete parapet would allow slightly more thrie beam deflection relative to the more rigid bridge system. However, we do not believe that wheel snag, excessive barrier deflections, or vehicle pocketing would cause any concerns.

It may be possible to compare this design to other wood post AGTs which use the half-post spacing to help determine maximum barrier deflections. If available, this information would be used to make comparisons to this design and hopeful show that similar or lower barrier deflections would be obtained with the larger steel posts at half-post spacing. Comparisons would then be made to the original Iowa transition which was accepted for use with safety shape ends and flared vertical ends. If needed, BARRIER VII computer simulations could be used to estimate the increased barrier deflections between original steel-post AGT designs with tubes/channels and modified AGT with additional hardware removed. Once those results were obtained, a comparison of simulation results would be made between Figure 96 and Iowa transition as well as an evaluation of wheel snag.

If needed, the last resort would be an actual 2270P test on a flared vertical concrete buttress.

Does this make sense?

Response

Date: 04-12-2012
Yes, it makes sense. In your opinion, would this design require a curb at the point of attachment to the bridge (or might FHWA require such)?

And on a related note, do you have a minimum recommendation for the design of the chamfer (i.e. flare or turnout) to be used at the approach end of a vertical bridge rail? I know we have several different designs that we use: one flares back 2 inches in 12 inches while another flares back 6 inches in 24 inches…

Response

Date: 04-12-2012

In my opinion, I would not think that a curb would be required. I believe that a 2-in. minimum flare back would be required. I have enclosed a few samples that have been included in FHWA acceptance letters for which I am familiar.

Attachment: https://mwrsf-qa.unl.edu/attachments/afab5d00ae04d2532a20c17963322700.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/afab5d00ae04d2532a20c17963322700.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/afab5d00ae04d2532a20c17963322700.pdf

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Low Tension Cable Slopes

Question

State: NE
Date: 02-28-2012

Low Tension Cable Guardrail

We are implementing the 30" high low tension cable guardrail with 27" & 24" lower cables.

The proposed standard installation for given slopes:

- S3X5.7 " 5'3" with 4' post spacing at 4' from a slope**less than 1V:1.5H** " tested to 25 degrees 62 mph by MwRSF.

- S3X5.7 " 5'3" with 8' post spacing at 3' from a slope**1V:1.5H to 1V:2H** " transition between practice & tested.

- S3X5.7 " 5'3" with 16' post spacing at 2' from a**1V:2H** or flatter standard practice & TL-2 tested in the 60's & 70's (possibly with 27" high top cable).

The explanation behind this implementation is the failed test which used 16' spacing 1' from a 1V:1.5H, where the front tire did not contact the soil before flipping, the tire is in contact with the soil on a flatter slope.

My thought is that the tested; flat concrete to 1V:1.5H slope, is different from the real world 2% lane, 4% shoulder, 1V:10H surfacing under guardrail to a 1V:2H slope, that this is enough difference to keep the front tire on the slope in testing conditions & that this force on the vehicle will keep it from flipping over the top of the cable.

Can the 2' from a**1V:2H** be modeled with the slope rounding which Ken Opiela spoke of at TRB?

Response

Date: 04-12-2012
Over the past couple of years, the placement of low-tension cable barriers near steep slopes has garnered much discussion. Some of this discussion has been archived in the following Pooled Fund Consulting Summaries:

http://mwrsf-qa.unl.edu/view.php?id=233

http://mwrsf-qa.unl.edu/view.php?id=234

http://mwrsf-qa.unl.edu/view.php?id=328

The recent questions found below pertain to the potential transitioning of the crashworthy low-tension, cable barrier system to variations in the configuration when placed near alternatives fill slopes.

Previously, a low-tension, cable barrier system with three cables at 30, 27, and 24 in. provided acceptable TL-3 safety performance under NCHRP Report No. 350 when placed 4 ft away from the slope break point of a 1½:1 fill slope. In this test, maximum dynamic barrier deflection reached approximately 125 in. (10.4 ft) with a vehicle traveling on level terrain prior to impacting the barrier system. Using this information, the 2000P vehicle extended approximately 77 in. (6.4 ft) beyond the slope break point. For a 77 in. (6.4 ft) lateral offset on a 1½:1 fill slope, the corresponding vertical drop from a horizontal datum is **51.3 in. (4.3 ft)**.

Previously, TTI researchers crash tested a low-tension, 3-cable barrier on level terrain with a 16-ft post spacing at TL-3 of NCHRP Report No. 350. This 2000P crash test resulted in a 3.4 m (134 in.) maximum dynamic barrier deflection, which was slightly larger than the barrier deflection observed by MwRSF in the ditch test.

If a 16-ft post-spacing design were placed 4-ft away from the slope break point of a 1½:1 fill slope, the 2000P vehicle would likely extend 86 in. (7.2 ft) over the slope terrain. For a 86 in. (7.2 ft) lateral offset on a 1½:1 fill slope, the corresponding vertical drop from a horizontal datum is **57.3 in. (4.8 ft)**. If a 16-ft post-spacing design were placed 4-ft away from the slope break point of a 2:1 fill slope, the 2000P vehicle would likely extend 86 in. (7.2 ft) over the slope terrain. For a 86 in. (7.2 ft) lateral offset on a 2:1 fill slope, the corresponding vertical drop from a horizontal datum is **43 in. (3.6 ft)**. >From this simple investigation and comparison, one may infer that the 16-ft post spacing design at a 4-ft lateral offset may result in slightly reduced safety performance as compared to the 4-ft post spacing design at a 4-ft lateral offset for 1½:1 fill slopes. For 2:1 fill slopes, one may infer that similar performance would likely be obtained. However, another factor may influence whether or not the 16-ft post-spacing design would adequately perform when placed at a 4-ft lateral offset. Recall that the 4-ft post spacing design has many more cable hook bolts and posts for which to support and contain the three cables. As a result, the 16-ft post spacing design may allow for increased cable drop and vehicle roll motion with reduced cable support from decreased hook bolts and posts. Thus, it still may be difficult to infer whether comparable vehicle and barrier performance would be obtained with the 16-ft post spacing design.

If a 16-ft post-spacing design were placed 3-ft away from the slope break point of a 1½:1 fill slope, the 2000P vehicle would likely extend 98 in. (8.2 ft) over the slope terrain. For a 98 in. (8.2 ft) lateral offset on a 1½:1 fill slope, the corresponding vertical drop from a horizontal datum is **65.3 in. (5.4 ft)**. If a 16-ft post-spacing design were placed 3-ft away from the slope break point of a 2:1 fill slope, the 2000P vehicle would likely extend 98 in. (8.2 ft) over the slope terrain. For a 98 in. (8.2 ft) lateral offset on a 2:1 fill slope, the corresponding vertical drop from a horizontal datum is **49 in. (4.1 ft)**. >From this simple investigation and comparison, one may infer that the 16-ft post spacing design at a 3-ft lateral offset may result in moderately reduced safety performance as compared to the 4-ft post spacing design at a 3-ft lateral offset for 1½:1 fill slopes. For 2:1 fill slopes, one may infer that similar performance would likely be obtained. Once again, differences in system design may influence whether the 16-ft post-spacing configuration adequately performs when placed at a 3-ft lateral offset. As noted above, the 4-ft post spacing design has many more cable hook bolts and posts for which to support and contain the three cables. Thus, the 16-ft post spacing design may allow for increased cable drop and vehicle roll motion, thus making it difficult for the 16-ft post spacing design to provide similar vehicle and barrier performance.

Similar comparison could be made for a 16-ft post spacing design at a 2-ft lateral offset with either slope condition. However, the results noted above would lead one to believe that further degraded performance may be obtained as compared to the 4-ft and 3-ft lateral offsets. In summary, it is difficult to know to the degree at which barrier performance is potentially degraded with increased post spacing and reduced lateral offset from steep fill slopes.

As noted above, the physical crash testing programs were performed with impacting vehicles traveling on level terrain prior to striking the cable barrier systems. You noted that in real-world applications, highways are typically constructed with cross slopes, shoulder slopes, slight grades or rounding near barrier and at slope break point in advance of the steel fill slopes. However, these geometric conditions may not exist at the outside of curves which protect steel slopes. In any event, it was theorized that these deviations from level terrain testing may potentially mitigate some of the degrading effects noted above. As a result, you inquired as to whether transitions to an 8-ft post spacing at a 3-ft lateral offset and then a 16-ft post spacing at a 2-ft lateral offset may be considered in advance of steep fill slopes. Based on our review of test results, simple investigation and comparisons above, and engineering judgment, we feel that it is difficult to support such transitions without additional R&D. This R&D could begin with computer simulation with LS-DYNA or other vehicle dynamics simulation codes, but it may eventually require compliance testing to verify that such real-world conditions can mitigate other degrading effects and prevent pickup truck rollovers.

Guardrail Need paper by Wolford and Sicking

Question

State: KS
Date: 04-23-2012

I have been reading Transportation Research Record 1599,*Guardrail Need, Embankments and Culverts* which was written by Dan Wolford and Dean Sicking and have found it extremely useful. I am interested in running a similar analysis using updated accident costs and RSAP to put together similar charts for use by our local County and City partners. Would it be possible to get the matrix of site situations used for this report? We would like to use it as starting point for our analysis and add to it.

Please do not hesitate to call if you have any questions.

Thank you in advance for any assistance you can provide.

Response

Date: 04-24-2012

I have dug into the materials associated with that paper. The original research report has more detail and is attached. Let me know if that helps any.

I have not been able to locate the actual matrix of sites used in the modeling yet. I will have a student do some digging and let you know if we find anything.

Thanks

Attachment: https://mwrsf-qa.unl.edu/attachments/1469dcbde727dc70856af76dbc386122.pdf

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Asphalt Width under TCB Transition to Bridge

Question

State: IA
Date: 04-25-2012

The approach transition between free-standing F-shape PCB and a rigid barrier was tested at MwRSF using a asphalt pad that extended for approximately 36" behind the back side of the PCB segments.

We do believe that it is possible to reduce the amount of asphalt behind the barrier, but there are concerns. Reducing the amount of asphalt behind the barrier too much could lead to a disengagement or shear punch out of a significant section of the asphalt. This would increase pin and barrier deflections which could adversely affect the safety performance of the system. As such, we would recommend a minimum of 2 ft of aphalt behind the back side of the PCB segment in the region of the transition. This should provide sufficient resistance for the anchors used in the transition.

Increasing the depth of the asphalt could potentially decrease the amount of asphalt needed, but component testing of the anchor pins in different asphalt thicknesses was conducted during the project and the difference between 2" and 6" of asphalt on the anchor loads was minimal. Thus, we don't believe that thicker asphalt would be effective unless it was much thicker.

One other item to note with respect to paved surfaces and PCB's is that we generally recommend that paving extend underneath and behind free-standing barrier segments as well. As a general rule, we recommend that the paved surface extend a minimum of 4 ft behind the back side of free-standing PCB segments to allowed for controlled deflection of the barrier on an even, consistent surface. As the barrier deflects, the paving helps provide consistent barrier to ground friction and prevents the back of the barrier from sliding off the pavement or tripping on uneven ground while engaged with the impacting vehicle.

Thus, for a total installation we would recommend 2 ft of asphalt behind the transition section and 4 ft of asphalt behind the free-standing barriers.

Response

Date: 04-25-2012
For the roadside TCB transition to bridge, a 2-inch asphalt surface is shown extending 12 inches behind the back face of the TCB. Is it possible to reduce this to 6 inches? If so, would this require a greater depth of asphalt?

Response

Date: 04-25-2012

Due to width constraints, we would be interested in a thickness recommendation for an asphalt pad that extends only 12 inches behind the TCB transition.

Response

Date: 04-26-2012

As noted previously, the original development of the asphalt pin tie-down evaluated various pin diameters in asphalt thicknesses varying from 2"-6". The testing was limited, but there was little to no variation in the peak lateral loads on the pins when using 2", 4" or 6" of asphalt. Thus, we don't expect
increased
load capacity due to the increased thickness of asphalt.
In addition, this component testing did not evaluate short widths of asphalt behind the pin.

The addition of more asphalt thickness may prevent the shear breakout of reduced asphalt width behind the PCB segments. Thus, other potential combinations of asphalt thickness and width could potentially restrain the tie-down pins. We have conservatively recommended 2' of asphalt width behind the PCB segments mentioned previously in order to be conservative. Other combinations may work and could be confirmed through future component testing or full-scale testing.

Another option would be to replace the asphalt in the area of the PCB approach transition with a reinforced concrete slab. The reinforced slab should constrain the pins as well or better than the asphalt and should elminate the potential for shear breakout of the pins. We believe that a 6" thick concrete slab with with a single mat of no. 4 rebar would work. The longitudinal and transverse bar spacing should be 12" on centers. The concrete slab would only need to extend 6" behind the back side of the PCB segments.

Clear Zone for Roadways with Design Speed of 70 mph

Question

State: WI
Date: 04-25-2012

We are looking to increase our posted speed of the rural freeways to 70 mph. We have a project that is looking to use a design speed of 75 mph. I was asked what should be the clear zone for a 75 mph design speed.

I believe that some of the work Dr. Sicking put together for the NCHRP Report 665 may be able to provide guidance.

Response

Date: 09-28-2012

The clear zone adjacent to high speed roadways was originally determined from lateral encroachment data collected adjacent to high speed test tracks at General Motors. Every ran-off-roaad event was identified and investigated to determine the vehicle trajectory after leaving the roadway. The distribution of lateral travel distances was developed from these accident investigations and the national clear zone distance for high speed highways was set equal to the 70^{th} percentile lateral encroachment distance.

This same approach can be used to estimate the appropriate clear zone distances for high speed highways using data from NCHRP Report 665. This study collected more than 800 vehicle trajectories from single-vehicle, ran-off-road crashes on high speed roadways. Further, the crash sampling method produced a large bias toward severe crashes. Thus, even though approximately half of these crashes involved impacts with fixed objects which may tend to shorten the lateral travel distances, the large bias toward more serious crashes should produce the opposite effect. Thus, the data from NCHRP Project 665 is believed to be the best source of vehicle trajectory data currently available.

Unfortunately, the number of crashes collected from 75 mph highways was somewhat limited. When lateral encroachment data from controlled access highways with 70 & 75 mph speed limits is examined, the 70^{th} percentile lateral encroachment was found to be 10.5 m or 34.5 ft. This value closely matches the Roadside Design Guide recommendation of 32-35 ft for 70 mph highways. Historically, encroachment data has been extrapolated to higher speed facilities by incorporating 80^{th} percentile encroachment distances. The 80^{th} percentile encroachment distance from the curve below was found to be 13 m or approximately 43 ft.

The appropriateness of using this approach to extrapolate encroachment distances to higher speed limit facilities was then evaluated by using data from 65 mph highways to estimate the appropriate clear zone at 70 mph. As shown in the figure below, the estimated clear zone width for 65 and 70 mph roadways was found to be 8.3 and 10.4 m respectively. The close correlation between the two estimates for 70 mph roadways and the correlation with the RDG provide strong support for the method used to estimate appropriate clear zone for 75 mph highways.

Attachment: https://mwrsf-qa.unl.edu/attachments/4f9bc375ddab920ba77c232736c4158a.pdf

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Median width

Question

State: WI
Date: 04-25-2012

Dear MwRSF,

We have a reconstruction project of a freeway that has had CMC crash history and has had cable barrier installed in the median. The project team indicated that they would like to reconstruct the median wide enough to not need cable barrier.

The WisDOT thresholds for CMC is the same as the Caltran's threshold.

Can MwRSF provide a recommendation on median width?

Response

Date: 05-02-2012

Appropriate median width for cable barriers is not a fixed distance. Rather the width is related to the traffic volume or ADT. previous research has shown that cross median crash rate (CMC) had a second order relationship with respect to ADT. As such, selection of appropriate median widths for use of cable barrier depends on the traffic volume and the benefit cost ratio of the addtion of the cable median barrier.

MwRSF conducted previous research into cross median crashes based on Kansas data. The research was detailed in TRP-03-206-08. For very high ADT roads, MwRSF would recommend large median widths of 100' or more in order to compensate for the increased propensity for CMC events due to the high traffic volumes. If median widths of this magnitude are not permissable, data from the Kansas study found that a median width of 70 ft or more were not recommended for cable median barrier unless ADT was extremely high or accident data determined a need for median barrier in a specific area.

Further details on guidance for cable median barriers and median width can be found in the attached report.

Attachment: https://mwrsf-qa.unl.edu/attachments/f369be5eff2910b2d76def8f8dfb1f8f.pdf

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TL-2 permanent concrete barrier

Question

State: WI
Date: 04-25-2012

Could you review these drawings? My concerns are:

Can the footing be used with or without soil behind it?

Where is the LON point on the sloped end treatment?

What rate should the sloped end treatment be flared away fromt he roadway?

I know that the curb near the slope end treatment needs to be removed.

Attachment: https://mwrsf-qa.unl.edu/attachments/079d821959c3e7810ea8d49206e31b25.pdf

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Response

Date: 04-25-2012

Can the footing be used with or without soil behind it?

I am assuming that "footing" refers to the 8" slab you have underneath the barrier, and that "soil behind it" refers to the soil fill on the backside of the rail which brings the groundline to the same height as the top of the slab on the front of the barrier.

For the barrier itself, this soil backfill is unnecessary. Remember, this TL-2 barrier was tested as a bridge rail. Thus, it was assumed that there was no fill behind the barrier at all.

For the end treatment, the soil backfill is necessary. Note, all testing of this end treatment was conducted with the system attached to a flat / level surface. If the ground surface on the backside of the barrier were significantly lower, roll would be introduced to the vehicle as it was impacting the upstream end of the system and the vehicle may become unstable (rollover)

Where is the LON point on the sloped end treatment?

The Length of Need for the low profile barrier system was defined as the joint location (or connection point) between the 20" high barrier and the sloped end treatment.

What rate should the sloped end treatment be flared away from the roadway?

TTI originally designed and tested this end treatment without flaring the system at all " it remained parallel to the roadway and inline with the barrier. As such, the drawings included in chapter 8 of MwRSF's report no. TRP-03-109-02 "Development of a low-profile bridge rail for test level 2 applications" also illustrated a parallel end treatment. The end treatment was never evaluated as a flared system.

Additionally, the height of your end treatment at its far upstream end is 6". This height should be limited to a maximum of 4" (as was tested by TTI and recommended in the MwRSF report noted in the previous paragraph)

Steel backed timber transition to rigid barrier

Question
State: WI
Date: 04-25-2012

Does MwRSF have access to the crash test of the Steel Backed Timber Guardrail transtion to a straight stone masonary guardwall parapat- TL-3?

This transition is listed in FHWA memorandum HSA-10/B-64D2.

Also in the same memo they show a 70 foot radius installation of the steel-backed timber guardrail circular curves 70-ft radius and below. Considering how the short radius system had problems is this design viable?

Does MwRSF have an option about the steel backed timber guardrail transition?

Response
Date: 05-03-2012

I have found a few reports documenting the full-scale testing on the steel-backed timber guardrail and transition designs (tested at TII). Please see the attached files. Note, only hte succesfull tests are contained in the reports. Thus, the TL-3 test of the transition to masonry wall is not contained in these reports. I'm not sure that a formal report was ever created for that test.

Further, I cannot find any documented analysis or testing that illustrates the crashworthyness of the steel-backed timber guardrail installed with a tight radius. As such, I
would
need
further evaluation of the noted short radius system (as small as 70 ft) before
I could recommend installing it.

Attachment: https://mwrsf-qa.unl.edu/attachments/51f04abfc0b0c7665d8222019abb9e4d.pdf

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MGS Bolted to Intake Top?

Question

State: IA
Date: 05-01-2012

Where an intake/catch basin cannot be avoided within a run of guardrail, would it be feasible to bolt a steel post to the intake top, similar to the method of bolting to a low-fill box culvert? If so, should half-post spacing be utilized?

Response

Date: 05-03-2012

MwRSF has previously answered a similar question were a water drainage structure was obstructing the placement of a post. For w-beam systems (MGS) not in a transition regions, it was recommended to simply omit a single post at the obstruction location. The system would act as a 12.5 ft long span system - recall, MGS long-span was crash tested to 25 ft span without the need for nesting the rail.

If a drainage structure is obstructing more than one post installation, then bolting the posts to the top of the intake would be a better solution.

Response

Date: 05-03-2012

When you say "simply omit a post," I assume you mean omit a post, but install the CRT posts upstream and downstream of omitted post, correct? Or is simply omitting a post where there is not a culvert opening/dropoff allowable?

If this particular run of guardrail was installed flush with a curb, would that change the recommendation at all?

And finally, should half-post spacing be utilized when bolting to the top of an intake - especially where two or more consecutive posts cannot be installed?

NOTE: feel free to call me if you'd like to discuss this.

Response

Date: 05-08-2012

Yes, we recommend using CRT posts on the upstream and downstream ends of the region where posts will be omitted. The MGS Long Span system was designed with 3 CRT posts on both the US and DS ends and can be used to span lengths up to 25 ft (the equivalent of 3 missing posts). Although the required number of CRT posts may go down when omitting only 1 or 2 posts, until we have testing to illustrate the crashworthyness of the modified system, MwRSF will continue to recommend using 3 CRTs on each side to prevent pocketing and snag.

I do not anticipate a major issue with the guardrail being installed flush with a curb. The MGS was successfully tested with a 6" offset from a curb. Installing the system flush with the curb should ensure the vehicle and guardrail interlock, even if the system is missing a post or two.

The MGS long span system can be installed with span lengths of up to 25 ft, same as 3 omitted posts. If the drainage structure you are installing guardrail over requires more than 3 omitted posts, the 1/2 post spacing guardrail system designed for culverts would be a possible solution. However, you would need to be carefull in attaching the posts to ensure (1) the drainage structure can handle the post-anchor loads, and (2) the posts are installed with the correct length - recall these posts extended 9" below groundline before attaching to the top of the culvert slab.

51-inch Integral Barrier

Question

State: WI
Date: 06-11-2012

Dear MwRSF,

Our structure's department is working on a standard detail drawing for providing vehicle protection for columns that are close to the roadway (i.e. designed for TL-3 impact loads). Structures would like MwRSF to review the attached drawings and provide comment.

Attachment: https://mwrsf-qa.unl.edu/attachments/9f52be2aaaf38f883434914c368290cf.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/c696828d6def6f7f926f66fc95806e77.pdf

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Response

Date: 06-11-2012

The barrier detailed in your previous e-mail provides more than enough strength for a TL-3 / TL-4 barrier.

Thrie beam/Barrier plate attachment

Question

State: WI
Date: 06-19-2012

We have some structures people retrofitting thrie beam to sloped end treatments. Prior to reading the report on retrofitting thrie beam transitions, I was instructing them to put the a connecting plate (see drawing) to get the rail vertical closer to vertical. I did this based on some crash tests done at TTI.

Based on the retrofit research, should they be installing the plates?

Is having the plate above the sloped transition an issue (see email and associated attachment below)?

Given that our sloped end section and the blunt end section can allow interaction with a vehicle's wheel, should we place something near the bottom of the barrier to prevent this interaction?

Attachment: https://mwrsf-qa.unl.edu/attachments/b3a398c4ba6d8247f317cadf82b575fe.pdf

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Response

Date: 06-19-2012

Using the attachment plates to keep the thrie beam vertical is important " we recommend you continue to use them on you installations. That being said, The attachment plate was not designed for use on sloped end barriers. Your concerns for snag on that upper corner are justified as previous testing on transitions has shown even small amounts of snag can negatively affect a systems crashworthiness. My 1st thought to mitigate potential snag issues is to cut the anchor plate to match the slope of the concrete barrier (the 4 5/8" x 7 ½" section detailed in your drawing).

I am still looking for the TTI test report of transition hit with the end shoe on the outside of the guardrail " resulting in snag and vehicle rollover.

Median Barrier Anchoring Options

Question

State: IA
Date: 06-22-2012

We have a project on I-80 coming up where we will be installing the "head ejection" median barrier. I am requesting your assistance in developing a few options for anchoring the barrier into existing pavement.

As shown in the attached PDF, the barrier will be installed on three slightly different median pavement configurations. In all cases, the existing unreinforced PCC slab is 10 feet wide and 12 inches thick. Note that the barrier may shift left or right within the slab, but should not get any closer than 1 foot from the edge of the slab.

It would be preferable to use the same (or very similar) anchoring details for all three configurations. The final pavement elevation must match existing.

Please let me know if you have any questions.

Thanks!

Response

Date: 06-25-2012

Dr. Faller has asked me to help you with the anchoring of the TL-5 median barrier to an existing concrete slab. From your comments below, I'm assuming that you are wanting to dowel/epoxy into the existing median slab and not use the asphalt keyway of the original (as tested) design. If so, this could be accomplished in a couple of different ways: 1) the stirrups could be modified to be open at the bottom, extended in length, and placed into the slab " this option would resemble the stirrups used in the end section configuration. 2) The stirrups could remain the same and #5 dowel bars would be placed at 18" intervals to match up with and anchor the stirrups. Note " the #4 dowel bars shown in the original report and design drawings for the TL-5 median barrier were used to anchor the rebar cage during casting. These bars were not considered in the strength analysis of the barrier.

Let me know if either of these two options sounds like what you had envisioned... or if I'm completely off base.

Response

Date: 06-25-2012

You got it. Either of those options (or some version of those options) would be much more agreeable to us instead of removing a bunch of concrete and pouring a new, separate footer. And you're right " we are not interested in using the asphalt keyway on this project.

Do I need to pick between the two options, or were you planning on evaluating both of them?

Response

Date: 06-25-2012

The two options are directly related since they would both include utilizing epoxy to anchor #5 bars to the slab (same spacing/intervals too). Thus, the embedment depth and location of the holes/anchors would be identical. As such, I can sketch up both options and you can choose between the two based on cost and constructability.

For both options, the spacing would always remain consistent at 18 inches " both front and back sides of the barrier. However, the embedment depth would be a function of the epoxy strength and concrete strength. You would have to use the manufacturers technical manual / recommendations on embedment to obtain full capacity of the rebar. Does Iowa have a preferred epoxy, or is this open?

Recently, MwRSF has been utilizing the HIT-RE 500 epoxy from Hilti (1,800 psi bond strength) for our anchorage designs. This product coupled with a concrete f'c of 4,000 psi would require only 6 inches of embedment to ensure full capacity of a #5 bar.

Also, are you planning on casting the barrier with the 1/18 face slope or with a vertical face (design option discussed in report). I only ask because if the 1/18 slope is being utilized, the dowels will be bent to match the slope of the stirrups " anchoring at an angle is usually not desired.

Response

Date: 06-25-2012

I would be interested in seeing a sketch of both options. This would be extremely helpful when used to explain the options to others.

I'm not sure that we have a preferred epoxy, but we do have a list of approved sources. I found the following passage in ourMaterials IM 491.11. Appendix C is the only place I found a listing for Hilti RE-500. Can you take a look and see if these are the types of systems we should be employing in this situation, or if we should limit the systems listed in Appendix C, or if we should provide a separate list of approved sources for this particular project?

Appendix C contains polymer grouts for dowel bar installation. Either an encapsulated chemical

anchor system or a pressure-injectable system with mechanical proportioning and mixing shall be

required to blend the material to uniform consistency.

To obtain approval for products under Appendix C, the laboratory evaluation will consist of bonding

a No. 5 reinforcing bar in a 4-inch deep 3/4-inch diameter hole in a concrete specimen and

performing a pullout load test. The test specimen shall develop a 40-pound minimum pullout load in

one hour and a 24-hour pullout load at a minimum of 10,000 pounds. The specimen will be kept at laboratory temperature. Two specimens are needed to obtain the average of each pullout load.

Products meeting the requirements for Appendix C will also be placed on Appendixes A and B.

Manufacturers whose products require special equipment such as an injection or mixing equipment

shall recommend which equipment can be used with their product.

We are planning on having the barrier slipformed, so we will probably be using the 1/18 slope on the barrier face. Would this require an additional bend in the stirrups in order to avoid drilling/anchoring at an angle?

Response

Date: 06-26-2012

Please the attached PDFs for the two epoxy anchorage options previously discussed. Please note that the embedment dimension of 6" is based on Hilte's HIT-RE 500 epoxy (bond strength of 1,800 psi). If another epoxy is desired, then the embedment depth may need to be altered to ensure ultimate tensile capacity can be obtained. Also, this epoxy anchorage retrofit design assumes the concrete slab that you are anchoring too has sufficient size and strength as to prevent movement, rotation, and damage to the slab. In your case of a 10 ft by 12 in deep slab, this should not be an issue.

Option 1 is divided into an Option 1a and 1b. 1a keeps the stirrup angled to follow the barrier before being bent to vertical 2 inches from the base of the barrier. 1b has the sides of the stirrup being bent to vertical near the top of the barrier, thus eliminating the need for the small bend near the base.

For Option 2, I recommend doweling in using straight bars during the epoxy/anchoring stage, letting the epoxy set, and then bending the tops of the dowels inward to match and tie to the angled sides of the barrier stirrups. Stirrups would remain identical to original design.

Option 1 will save on material cost as the amount of steel is reduced, but Option 2, may be easier to construct during installation

Attachment: https://mwrsf-qa.unl.edu/attachments/1e28ada6df46ebb12fc52a0af8ef8e94.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/dbbee6f4ab38455e17de61da7e5c5fb1.pdf

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Response

Date: 07-05-2012

Does anything change if we will be using epoxy-coated rebar for the cage and the dowels?

Response

Date: 07-05-2012

In a recent study on epoxy anchors, MwRSF did dynamic testing on both black rebar and epoxy coated rebar dowels. The testing has been completed, but the report is still being put together. Bob is the one finishing the report, so he would know the conclusions of this study better than I. Unfortunately, he is out of the office until next week. From my recollection, I believe there is a 5-10 percent decrease in anchorage strength expected for epoxy coated bars. If this is correct, then the embedment depth would need to be extended slightly (1/2" to 1") in order to ensure full capacity of the rebar can be developed.

The Hilti design manual lists an embedment depth of 5¾" is required to obtain ultimate capacity of a #5 bar " I rounded this up to 6" to be a little conservative and to get a nice even number. The Hilti manual does not mention anything for the effects of epoxy coated rebar. Thus, if my above statements are correct (Bob help me out here) then the embedment depth should be increased to minimum of 6.5".

Don't take any of this as accurate until Bob confirms this...

Response
Date: 07-05-2012

Scott is correct that our recent dynamic testing of epoxy coated rods with chemical adhesives showed approximately a 10% reduction when compared to black steel. Thus the increased embedment of 6.5" indicated by Scott is warranted and should provide the necessary strength.

Response
Date: 07-08-2012

The other point of concern I had was with the splice length required between the cage and the dowels being epoxied into the pavement - whether using epoxy-coated bars increases the required lap length.

Response
Date: 07-16-2012

The required lap splice length for an epoxy coated #5 bar according to the ACI Code is 28 in. The original sketches I sent you showed 30" long dowels embedded 6 inches into the slab and extended 24 inches into the barrier (for uncoated rebar). Using 6.5 inches of embedment and extending 28 inches into the barrier, epoxy coated dowels would need to be 35 inches in length.

Nested vs 12 gauge thrie beam

Question

State: IA
Date: 06-26-2012

When it comes to approach guardrail transitions, is nested 12-gauge thrie-beam considered equivalent to a single 10-gauge thrie-beam (and vice-versa)? How about nested w-beam?

Response

Date: 07-13-2012

Nested 12-gauge thrie beam would provide greater overall bending and tensile strength than that provided by a single 10-gauge thrie beam. Most crash tests on thrie beam bridge railing and approach guardrail transition systems likely have utilized nested 12-gauge thrie when additional strength was needed or desired. However, MwRSF has conducted limited crash testing on thrie beam bridge railing and approach guardrail transitions where only one single 10-gauge thrie beam was utilized. Historically, many of us have been comfortable with allowing both thrie beam alternatives (i.e., nested 12-gauge T and single 10-gauge T) in situations where additional strength has been desired. Some of the complaints often pertain to the need to stock and differentiate between 10 and 12 gauge thrie beam sections. However, others may not necessarily hold the same opinion.

With regards to W-beam sections, nested 12-gauge beams would once again be stronger than a single 10-gauge beam in terms of bending and tensile capacity. To date, MwRSF has not conducted any research on strong-post W-beam guardrail systems where rupture concerns were fixed with a single 10-gauge rail instead of nested 12-gauge rails. However, MwRSF had proposed this option as one of many solutions for the original rupture observed in testing the Nebraska W-beam guardrail over a 4" tall concrete curb. In the end, two nested 12-gauge rails were used and provided successful performance. I would suspect that a single 10-gauge W-beam may also have worked to mitigate rupture concerns.

If single 10-gauge rails are desired as a replacement for all systems which use nested 12-gauge rails, it may be necessary to further investigate some of the more critical impact scenarios and systems with computer simulation and/or dynamic testing.

Please let me know if you have any further questions or comments on this matter. Thanks!

Short-Radius Guardrail

Question

State: IA
Date: 06-26-2012

I've got a few questions for you on the short-radius guardrail system that TTI successfully tested at TL-2:

1.1.It's my understanding that the maximum radius that can be used is 8 feet. Isthat correct?

2.We may have intersection angles that are less than or greater than 90 degrees. Can the 12.5-foot rail section be bent to angles other than 90 degrees? If so, should the number of CRT posts going around the curve remain unchanged? If bend angles other than 90 degrees are not allowed, how might you suggest we deal with such situations?

3.I see that the original Yuma County design incorporated a flare on the primary road side, but the TTI-tested version did not. Do you see any problems using a flare on the primary road side?

4.I understand that the rail height as tested was 27 inches. Do you see any issues with raising the rail to the FHWA-recommended minimum of 29 inches? How about to 31 inches? If so, do the holes in the CRT posts need to be shifted lower by 2 inches (or 4 inches)?

5.Any reason why we couldn't install this system with mid-span splices? How about with 12-inch blockouts?

Most of our rural sideroad intersections have radii in the 25- to 30-foot range. Do you have any other suggestions on how we might run guardrail around the corner in a manner that more closely matches the existing radius?

Thanks for your help.

Response

Date: 07-17-2012

Ron forwarded me your email to address your short-radius questions. However, before I can address them, I need to clear up which design we are referencing.

TTI developed and tested two short-radius designs. One was tested under the TL-3 criteria for NCHRP 230 and one was tested under the TL-3 criteria for NCHRP 350. Neither of these systems met the safety requirements or were implemented.

Recently, TTI sought TL-2 approval of the Yuma County short-radius design that was tested in 1988 at SWRI based on their engineering analysis. This system was tested under the PL-1 criteria of the AASHTO Bridge Specifications.

No current system has been successfully tested to TL-2. If you can identify which system you are referring to, I will take a shot at answering your questions.

Thanks

Response

Date: 07-17-2012

I was referring to the Yuma County design that " correct me if I'm wrong " has been accepted at NCHRP 350 TL-2.

Response

Date: 07-18-2012

I have looked over your short-radius questions and have comments below in red.

I've got a few questions for you on the short-radius guardrail system that TTI successfully tested at TL-2:

1. It's my understanding that the maximum radius that can be used is 8 feet. Is that correct?

   The Yuma County system was tested at SwRI with the 8' radius that TTI shows in their details. The performance of larger radii is not fully understood for this particular system as it only underwent limited testing. MwRSF has generally stated that smaller radii are more critical for short-radius designs. A smaller radius size will result in a stiffer curved section, while larger radii will tend to decrease the stiffness of the curved section. Based on the previous research, the use of smaller radii seems to demonstrate more promise for short radius designs. No one has successfully tested any short-radius system radii larger than 16' to either the NCHRP 230 or 350 criteria. As such, we cannot recommend increasing the size of the Yuma County system without further analysis.

   FHWA Technical Advisory T5040.32 recommends the use of a short-radius guardrail that was developed by the State of Washington. This design was tested under the impact requirements set forth in NCHRP Report No. 230. The crash testing demonstrated that the system could contain a 1,800-lb small car and a 4,500-lb sedan. However, the testing program was not complete, and the results were marginal in some cases. Guidance for installing the short-radius guardrail is given for systems with radii ranging between 8.5 and 35 ft. The technical memorandum also notes that testing conducted on a 35-ft radius Washington State design did not perform adequately when impacted at 60 mph by a large vehicle (4740 lbs). Satisfactory results were obtained for the 35-ft radius system when a test was performed at a reduced speed of 50 mph with the large vehicle.

   We currently have a project with Wisconsin DOT to evaluate the use of the Washington system with larger radii. This work is currently underway and should provide some guidance as to the use of larger radii with short-radius systems.

2. We may have intersection angles that are less than or greater than 90 degrees. Can the 12.5-foot rail section be bent to angles other than 90 degrees? If so, should the number of CRT posts going around the curve remain unchanged? If bend angles other than 90 degrees are not allowed, how might you suggest we deal with such situations?

It is very difficult for us to make recommendation on the Yuma County system regarding intersection angles other than 90 deg. Small variation in the bend angle should not affect the performance of the system greatly, but it is difficult to define what the magnitude of the acceptable angles would be. The angle of the sides of the system affects performance as the smaller the angle, the stiffer and more energy the system absorbs when vehicles impact on the nose due to the angle that the guardrail is bent during impact. Obviously, as the angles vary a great deal from 90 deg. we begin to approach either a general curved guardrail system or a bullnose system. Thus, it would be possible to employ a bullnose design with flared sides on the very small interior angles or to follow guidance for curved guardrail on very large angles. However, specific guidance on intermediate angles is hard to give without further study, especially on a system where we have only limited test data.

3. I see that the original Yuma County design incorporated a flare on the primary road side, but the TTI-tested version did not. Do you see any problems using a flare on the primary road side?

The use of the flare in the system should be acceptable. We actually employed a parabolic flare in the MASH short-radius system we partially developed. The use of the flare helped reduce the potential for the vehicle to be impaled by the guardrail rail if the vehicle impacted directly along one of the sides of the system.

4. I understand that the rail height as tested was 27 inches. Do you see any issues with raising the rail to the FHWA-recommended minimum of 29 inches? How about to 31 inches? If so, do the holes in the CRT posts need to be shifted lower by 2 inches (or 4 inches)?

We would not recommend changing the rail height of the system. Our experience with testing the small car vehicles with the bullnose and short-radius systems has shown that the small car would be very likely to underride the system if the guardrail height were increased. If you desire to attach the system to a run of 31" high MGS, you can employ a height transition. In the past, our recommendation has been to transition the 3.25" height difference over approximately 50 ft or two 25-ft long sections of W-beam guardrail.

5. Any reason why we couldn't install this system with mid-span splices? How about with 12-inch blockouts?

We see no issues with using midspan splices or 12" deep blockouts in the system.

Most of our rural sideroad intersections have radii in the 25- to 30-foot range. Do you have any other suggestions on how we might run guardrail around the corner in a manner that more closely matches the existing radius?

As noted in the discussion of larger radii above, there is only limited testing of larger radius systems and that was mostly done under the NCHRP 230 or PL-1 guidance. Thus, we are leery of increasing the radius of the Yuma County system. The best guidance at this time is the FHWA memo noted above. In addition, you may want to contact Roger Bligh at TTI and see if they investigated the use of larger radii with the Yuma County design. Finally, the work we are doing with WisDOT should shed some light on the subject as well.

Attachment: https://mwrsf-qa.unl.edu/attachments/b82bbf8953b9b18f96febea7d6c821f8.pdf

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Response

Date: 07-18-2012

Thanks, Bob. In your answer to my #2 question, you mention following "guidance for curved guardrail." May I ask to what guidance you are referring?

Response
Date: 07-18-2012

I saw that coming as soon as I wrote that comment.

Currently, the guidance on curved guardrail systems is pretty limited. . One research study regarding vehicle accidents or curved roadways and testing of W-beam guardrail on curves was conducted by ENSCO and sponsored by FHWA in 1989 through 1991. This research study involved the testing and evaluation of strong-post, W-beam guardrail systems that were located on the outer edge of a horizontal curve with a 1,192-ft radius. For this study, the successful safety performance of the curved W-beam barrier system was observed on flat ground for an 1,800-lb small car and a 5,400-lb pickup truck impacting at 60 mph and 20 degrees using flat roadway conditions. However, three subsequent pickup crash tests were unsuccessful (i.e., each resulted in vehicle rollover) when the W-beam guardrail system was installed in combination with a super-elevated, curved roadway. These crash tests were performed using the impact safety standards found in NCHRP Report No. 230 and the AASHTO 1989 Guide Specifications for Bridge Railings. As such, no strong-post, W-beam guardrail systems have been successfully tested for use on super-elevated, curved roadways according to NCHRP Report No. 350 safety performance guidelines or the current Manual for Assessing Safety Hardware (MASH). Because the ENSCO research study is the only available testing of beam guardrail on curved roadways, designers are limited to guidance on the installation of W-beam guardrail on curves based on limited tests of curved guardrail on flat ground and the use of engineering judgment.

At this time, NCAC has an NCHRP project, NCHRP 22-29 Performance of Longitudinal Barriers on Curved, Superelevated Roadway Sections, to further investigate those installations. This together with the WisDOT study we are doing should hopefully further our understanding of guardrail on curves.