# Midwest States Pooled Fund Program Consulting Quarterly Summary

## Midwest Roadside Safety Facility

**07-01-2008 to 10-01-2008**

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## F-Shape in Front of Curb Adjacent to MSE Wall

### Question
State: KS
Date: 07-21-2008

I need some advice from you on a project currently being constructed.
This project is located in Topeka and involves a new road on new alignment that is adjacent to an active RR. The road was designed using a 40 mph design speed with a projected traffic volume of 13,250 vpd, with 3% Trucks.
The roadway is elevated relative to the RR tracks. A Mechanically Stabilized Earth Wall (MSEW) is being used. Currently the roadway is designed with a type B curb with guard rail located about 4'-5' from the back of the curb. The problem is that the guardrail posts get into the MSEW earth reinforcement straps and the separation fabric that is utilized to keep chloride laden runoff from coming in contact with the reinforcement straps.
These issues as well as future maintenance concerns with the guard fence design is pushing the need for an alternative design. The current proposal is to install a 32" tall F-shape barrier about 3' from the face of the barrier to the back of the Type B Curb. The barrier will be doweled into a 10" thick PCCP slab. In most cases there will be about 3 feet behind the back of the barrier to the 3:1 slope or the MSEW. My initial thought is that this option will be ok based on the site specific issues, such as the 40 mph design speed. Note that we may change to a laydown curb (slope faced similar to AASHTO Type G however our curb height is only about 1 ½" flowline of curb) instead of the Type B curb. My thought was that if it is ok for Type B curb then it should be OK for a laydown curb. Do you agree?

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### Response
Date: 07-22-2008
You bring up a difficult problem. I am not aware of any crash test data used to determine the safety performance of vehicles launched over curbs and impacting 32-in. (813 mm) tall, safety shape barriers. Upon review of Chuck Plaxico's curb testing project, NCHRP Report No. 537, it was apparent that the bumper trajectory did not exceed 730 mm for a 2000P pickup truck vehicle impacting an AASHTO Type B curb at 70 kph and angles of 5, 15, and 25 degrees when placed within 1 m of the curb face. As such, it would seem unlikely that a 2000P (or 820C) vehicle would override a safety shape barrier with a top height of 38 in. (965 mm) above the roadway surface.

For us, another concern would be the effect that the forward placed curb would have on vehicle stability during vehicle redirection with a safety shape barrier. In lieu of this concern and due to the lack of sufficient test data, we cannot recommend placing a safety shape barrier behind a 6-in. tall Type B curb, even if located on a roadway with a 40 mph design speed. Instead, we would rather that you consider using a vertical, or near vertical, shape barrier for this application. If that option cannot be achieved, the next best alternative would be to utilize the single slope barrier, which offers improved safety performance over safety shape barriers.

# CALTRANS Concrete Barrier

## Question

State: WI
Date: 07-28-2008

WisDOT is considering switching to the Caltrans single slope barrier design. However, WisDOT has some concerns, if MwRSF could provide some input it would be greatly appreciated.

1. Would MwRSF have a recommendation on the use of expansion joints with the Caltrans barrier? My reading of the Caltrans details indicates that the addition of an expansion joint is need when there is some change in continuity (e.g. next to a bridge parapet, over an expansion joint in the pavement...).
2. WisDOT is thinking that a shrinkage joint (i.e. a tooled in joint, steel will not be cut) would be needed to control shrinkage cracking. WisDOT is thinking of installing the shrinkage joint every 20 feet. Would MwRSF have a recommendation on the use of a shrinkage joint and how often to use a shrinkage joint?
3. WisDOT is wondering what the minimum length of barrier (including the anchors) could be installed with the design indicated on the drawings. I know that anchor sections for the Caltrans barrier are 10 feet long, but I do have concerns that two anchors and 10' of barrier may not have enough capacity to withstand an impact. If MwRSF has an opinion, on this topic please provide comment.

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## Response

Date: 08-01-2008
I will do my best to answer your questions and provide comment below those sections.
WisDOT is considering switching to the Caltrans single slope barrier design. However, WisDOT has some concerns, if MwRSF could provide some input it would be greatly appreciated.

1. Would MwRSF have a recommendation on the use of expansion joints with the Caltrans barrier? My reading of the Caltrans details indicates that the addition of an expansion joint is need when there is some change in continuity (e.g. next to a bridge parapet, over an expansion joint in the pavement...).

** The Wisconsin DOT is examining CAD details for possible implementation of the CALTRANS single-slope concrete barrier and associated design variations. Thus, I recommend that someone from WsDOT contact CALTRANS to obtain feedback on their concerns and experiences with using this design, including accident experience, discussion on expansion/contraction joints, cast-in-place versus slip-formed construction experience, maintenance and repair experience, barrier durability and cracking, etc.
** In general, MwRSF has stated previously that expansion joints are not necessary as long as adequate structural reinforcing steel in provided to meet temperature and shrinkage requirements. Also, structural steel is needed to resist the vehicular impact loading.
** If the barrier is attached to a rigid pavement surface that requires an expansion joint, it would seem appropriate to match barrier joints with those already placed in rigid pavements. However, I recommend that further discussion be made with CALTRANS officials to investigate their recommendations to ensure proper barrier performance and longer barrier life. As noted previously, increased steel reinforcement and anchorage is needed at barrier end sections as well as at expansion joint locations.

2. WisDOT is thinking that a shrinkage joint (i.e. a tooled in joint, steel will not be cut) would be needed to control shrinkage cracking. WisDOT is thinking of installing the shrinkage joint every 20 feet. Would MwRSF have a recommendation on the use of a shrinkage joint and how often to use a shrinkage joint?

   ** From what you note, I assume that the WsDOT desires to place vertical grooves in the barrier surface and to the depth of the outer rebar at 20-ft increments so that cracking will be limited to these locations. However, I really not sure why this is desired. Concrete always will have minor cracking in it. It is the steel reinforcement that holds it together as concrete is weak in tension and strong in compression. A surface crack can go into compression when loaded and will not cause a problem. A gap in the concrete will require greater deformation in the concrete before the crack is closed and compression strength of concrete is realized. Imagine a reinforced concrete beam that has a 0.5" crack placed in the outer 3" of the compression face. In this case, the beam is reduced until the crack is closed. If adequate temperature and shrinkage steel is provided, I am not too concerned of small barrier cracks as long as the concrete barrier is not spalling due to poor concrete materials.
3. WisDOT is wondering what the minimum length of barrier (including the anchors) could be installed with the design indicated on the drawings. I know that anchor sections for the Caltrans barrier are 10 feet long, but I do have concerns that two anchors and 10' of barrier may not have enough capacity to withstand an impact. If MwRSF has an opinion, on this topic please provide comment.

   ** I thought the barrier was slip-formed or cast-in-place as a monolithic section. Please clarify what you mean by 10-ft barrier sections.

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**Response**

Date: 08-04-2008

Thank you for providing comments. We have tried to contact Caltrans, but I wanted to get an additional opinion.

You are correct that the Caltrans barrier is a slip formed barrier; however, our staff will have situations where the total length of the barrier needed for a location may be too small to absorb the impact of a vehicle.

For an example, a designer needs to install 25' of the Caltrans barrier. Using the measurements on the Caltrans drawings the two anchors sections are 20' long. This would leave only 5' of "normal" barrier to absorb the impact of a vehicle.

I was thinking that small of section of the Caltrans barrier details would not be sufficiently strong enough to withstand an impact. I was contemplating that a different reinforcement design would be needed after a certain minimum length of barrier was installed (e.g. less that 30 of total barrier length, designer should switch the reinforcement of the Caltrans barrier to look more like the reinforcement used in crash tests for the thrie beam transitions from temporary barrier to permanent barrier).

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**Response**

Date: 08-04-2008

Once again, I will place my comments following your questions below.

You are correct that the Caltrans barrier is a slip formed barrier; however, our staff will have situations where the total length of the barrier needed for a location may be too small to absorb the impact of a vehicle.

** Thus, I assume that you will be installing a cast-in-place concrete barrier system. If properly anchored, a short length of a reinforced concrete parapet should be able to redirect impacting vehicles.

For an example, a designer needs to install 25' of the Caltrans barrier. Using the measurements on the Caltrans drawings the two anchors sections are 20' long. This would leave only 5' of "normal" barrier to absorb the impact of a vehicle.

** The end sections are typically designed with increased reinforcement in the parapet as well as a stronger anchor/foundation below the end sections as compared to interior sections. If the end are designed appropriately, they should also be capable of redirecting the impacting vehicles, say at the TL-3 impact conditions. Recall that approach guardrail transitions are connected to the parapets ends and also are TL-3 compliant. As such, you should be able to count the entire barrier length for redirection, assuming that was the design intent. Thus, I recommend that you discuss this issue with the appropriate CALTRANS officials.

I was thinking that small of section of the Caltrans barrier details would not be sufficiently strong enough to withstand an impact. I was contemplating that a different reinforcement design would be needed after a certain minimum length of barrier was installed (e.g. less that 30 of total barrier length, designer should switch the reinforcement of the Caltrans barrier to look more like the reinforcement used in crash tests for the thrie beam transitions from temporary barrier to permanent barrier).

** If the ends have been designed to be 20 ft in length (10 ft per end). As you noted, the change in reinforcement would only be applicable for 5 ft in the middle of the installation. Thus, it would not be reasonable to switch the reinforcement pattern for only 5 ft. It may beappropriate to require at least a 30 ft barrier length before one changes reinforcement patterns.

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# Weld Detail for Guardrail Attached to the Top Slab of a Culvert

## Question

State: IL
Date: 07-29-2008

After review of the crash testing report, and discussions between our Bureaus of Design and Bridges and Structures, here is what we have come up with for a weld detail between the post and plate for this application. Does this appear appropriate?
Attachment: https://mwrsf-qa.unl.edu/attachments/e166559533b480358875bd561a658bf2.jpg

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## Response

Date: 08-15-2008
During testing of the W-beam system attached to the top slab of a culvert, it was found that a 3 pass weld was necessary on the front and back sides of the front flange of the post. If this 3 pass weld was not completed, the post tore away from the base plate which compromised the performance of the system. Therefore, you would need to change your weld detail on the front flange of the post to be a 5/16" 3 pass weld on the front and back sides of the front flange. It is fine to increase the weld sizes for the back flange and the web of the post and these only need to be a single pass.
We cannot recommend using only a single pass weld on the front flange unless it had been proven to work with testing. Therefore, this would require funding a bogie test in order to prove or disprove its performance capabilities.

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# Alternative Bolt-Through Tie-Down Anchors

## Question

State: NE
Date: 08-01-2008

Several states have made inquiries regarding alternative anchors for the F-shape barrier bolt-through tied-down. The original design was tested with 1.125" diameter A307 threaded rod that was epoxied 12" into the concrete apron at MwRSF in order to develop the full-capacity of the rod.

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## Response

Date: 08-01-2008
Figure 5 shows a list of anchors that were compared with the anchor used in the original bolt-through tie-down design. The listed anchors compare different diameters and grades with the highlighted original anchor. The criteria for selecting an alternative anchor would be that any alternative anchor must have equal or greater bending and shear capacities that the original anchor. In addition, MwRSF would recommend that the alternative anchor have similar or better ductility to Grade 5 threaded rod.
It should also be noted that the use of alternative anchors is only allowable if the anchor capacity can be sufficiently developed. For configurations where the state bolts the anchors through the deck, this should not be an issue. However, states that wish to epoxy the threaded rods into the concrete surface must insure that the epoxy and embedment depth are sufficient to develop loads equivalent or higher than the capacity of the original tested anchor.

Attachment: https://mwrsf-qa.unl.edu/attachments/dfddbab8bdb2e704dbc55b24da0af79c.jpg

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# PCB Treatments

## Question
State: OH
Date: 08-08-2008

I received this email from our Cleveland district - and a colleague has a good question about the inevitable PCB gap we seem to have after a couple of barrier moves caused by construction phases.
The overlapping method is the method done the most here in Ohio, but I worry about the ends not being anchored. Personally, I've never seen the "shoe" but if it was sufficiently secured to the PCB without any potential snag points on the traffic side, it might be the better solution.
What is your insight on a better way to handle this recurring construction problem?
Email Received:
Question on PCB.

When the contractor is switching phases he generally just moves the PCB run over several ft. He does not load it on a truck and place each piece.

The problem is the runs don't come out the same and there are gaps.

It seems like every contractor has a method to handle the gaps.

Here are two such methods.

1. Places a steel plate over the gap, see attachment.
2. over lap the PCB.

Question is which one do you think is safer?

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## Response
Date: 08-08-2008
Historically, we have recommended the overlapping method in situations where TCBs are to be placed in front of a rigid end of a concrete parapet. This recommendation was given prior to the development of several in-line attachments between freestanding and permanent concrete barriers. For the overlapped option, we stated to use 8 barrier sections beyond the end of the permanent barrier with a 2-ft gap between the freestanding and permanent barriers in order to reduce the propensity for vehicle pocketing and snag on the upstream barrier end. For overlapping TCBs, it would seem reasonable to use an overlap of at least 8 or 9 barrier segments for each run " front and back. However, I believe that the gap between both barrier runs could be reduced to 6 to 12 in. or so due to both barrier systems being freestanding, thus reducing the propensity for vehicle snag/pocketing. If limited space exists at the roadside edge for the overlapped option, one may consider the slight flaring of the rearward (shielded) TCB system in order to save space near the shoulder.

You noted another alternative where large steel shoes are placed over the gap produced when two barrier cannot connect to one another in line. For this system, it would be important for the shoe to not cause the vehicle to snag on raised components " screw handles, plates edges, or other structural features. Also, it would be important for the shoe to be able to transfer the necessary loads to allow the TCB system to perform in a safe manner, thus capable of transferring tensile, shear, and/or bending loads across the joint. However, it may be preferred to have tensile capacity in this type of connection using anchors into the barrier. KsDOT bridge engineers explored this option for bolted down, F-shape sections where a gap was needed in the TCBs. There may be other considerations when the shoe system is used for freestanding applications that have not yet come to mind. However, you may want to contact Rod Lacy and Scott King in Kansas to explore their current attachment and anchorage options. Other options may include using nested thrie beam on the front and rear faces, and combinations of other steel elements, and anchored into the TCB faces using common anchors. Gap lengths would need to be considered in the design and limited to a specified range. Finally, one could develop an adjustable F-shape section that could fit between barrier ends and serve to transfer the necessary loads across the gap.

# MGS Median Barrier

## Question

State: OH
Date: 08-08-2008

Ohio DOT wants MwRSF's thoughts on using the MGS system in a median barrier configuration with rail on both sides.

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## Response

Date: 08-08-2008
Looking into this application further, we believe that the MGS can be safely used with W-beam and blockouts on both sides in a median barrier configuration. This is based on the following:

1. Several of the existing 31" high guardrail designs have been successfully tested in median configurations. These include the NUCOR NU-GUARD 31 and the Gregory GMS 31. Both of these systems did not use blockouts and the Gregory system was tested with the splices at the posts. An MGS median system would be specified with splices away from the posts and 12" blockouts as used on the standard roadside system. This should increase the rail capacity and reduce snag as compared to the existing tested 31" high median guardrail systems.
2. While the stiffness of the MGS guardrail system would increase due to the use of front and backside w-beam rails, we do not believe that this is cause for concern. The MGS was successfully tested with ¼ post spacing which is would be much stiffer and have much lower deflections than an MGS median system with the additional w-beam rail. Thus, the additional stiffness of the system is not a concern.
3. You noted in our discussion that the system would be installed on the edge of shoulder and not in the 6:1 median ditch. As such, there should be no concerns with vehicle compatibility.

Based on the above statements, we believe that the MGS can safely be used in a median installation. MwRSF will also seek formal FHWA approval of a median MGS system if Ohio or the other Midwest Pooled Fund States so desire.

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# MwRSF PCB

## Question

Date: 08-13-2008

We're about to finally implement in Ontario your PCB as a non proprietary "Type M" temporary concrete barrier for work zones.

My first set of questions is with respect to rebar. In Canada we use metric rebar which is different from US metric or US customary. Our standard rebar is Grade 400 rebar which has a min yield strength of 400MPa and a min tensile strength of 600MPa. I understand an ASTM A615M Grade 420 bar has a minimum yield strength of 420MPa. Will this be a concern for the PCB?

The next metric problem I have is our sizes are different. Your #4, #5 and #6 bars have nominal diameters of 12.7mm, 15.875mm and 19.05mm respectively. Our standard 10M, 15M and 20M metric bars have nominal diameters of 11.3mm, 16.0mm and 19.5mm respectively. Originally we were going to specify 15M bars throughout, including the loop bars which is discussed below. For the #4 stirrups, we would like to now consider using the slightly smaller 10M bars unless you have a real concern.

For the loop bars, you have specified different steel " #6 smooth A706 Grade 420 steel. What is the rationale for using the larger diameter low alloy non-deformed steel bars for the loops ? Is it for welding or other reasons? Would you have a concern with our Grade 400 deformed 15M bars for the loops, or will we have to specify low alloy 20M smooth bars or Grade 400 20M smooth bars?

The second issue I would like to discuss is anchoring to concrete bridge decks with 90mm thick asphalt overlays, which is standard in Ontario. We have been reviewing the capacity of the specified Red Head anchors against other anchoring systems with significantly higher capacities to try and accommodate the 90mm standoff. I note Florida DOT allows a 1" to 2" asphalt overlay for their anchors into concrete.

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## Response

Date: 08-13-2008

Attachment: https://mwrsf-qa.unl.edu/attachments/c3ebb66add542ac4ef6853687ccba5ce.pdf

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## Response

Date: 08-14-2008

I have responded to you questions below inred.

My first set of questions is with respect to rebar. In Canada we use metric rebar which is different from US metric or US customary. Our standard rebar is Grade 400 rebar which has a min yield strength of 400MPa and a min tensile strength of 600MPa. I understand an ASTM A615M Grade 420 bar has a minimum yield strength of 420MPa. Will this be a concern for the PCB?

The US rebar standard is Grade 60 which has a 60 ksi yield. This converts to a 414 MPA. The difference in strength is negligible, so I would not be concerned about the grade.

The next metric problem I have is our sizes are different. Your #4, #5 and #6 bars have nominal diameters of 12.7mm, 15.875mm and 19.05mm respectively. Our standard 10M, 15M and 20M metric bars have nominal diameters of 11.3mm, 16.0mm and 19.5mm respectively. Originally we were going to specify 15M bars throughout, including the loop bars which is discussed below. For the #4 stirrups, we would like to now consider using the slightly smaller 10M bars unless you have a real concern.

With regard to the bar size, it appears that your 15M and 20M bars have larger diameters than our No.5 and No. 6 bars. However, the US No. 4 bar has a diameter of 0.5" while your 10M bars have a 0.445" diameter. That is a 21% reduction in area. We would not recommend using bars with that much of a reduction in area. Thus, we would recommend that you substitute your 15M bars in locations where you are currently using the

10M bars. We believe this is necessary based on the amount of damage we have observed these barriers having during full-scale testing. We believe that the current barrier reinforcement is approaching its safe minimum capacity, so we have been holding the line and making alternative designs be equally as stronger or stronger than the tested barrier configuration.

We would recommend that you use the 15M bars for the stirrups as well, but that may not be practical. If you have to use the 10M bars for the stirrups, we would recommend that you install additional 10M stirrups 4.5" from the each stirrup adjacent to the tie-down anchor pockets as well as an additional stirrup between the two stirrups on the end of each rail. See the attached sketch.

For the loop bars, you have specified different steel " #6 smooth A706 Grade 420 steel. What is the rationale for using the larger diameter low alloy non-deformed steel bars for the loops ? Is it for welding or other reasons? Would you have a concern with our Grade 400 deformed 15M bars for the loops, or will we have to specify low alloy 20M smooth bars or Grade 400 20M smooth bars?

The loop bar steel is a different spec because we have found that the small bend diameter can cause reduced ductility and toughness in some grades of steel which compromises the impact strength of the loop. As such, we current specify that the loop steel must have a minimum yield strength of 60ksi, a minimum tensile strength of 80 ksi or 1.25 times the yield strength " whichever is higher, and a minimum % elongation of 14%. A706 and A709 steel both meet that spec. Others may as well. The bars can be deformed or smooth as long as the steel is within spec. Some of our states prefer smooth, so it is on the drawings that way. Again, we would recommend that you use the 20M bar because of the area difference between the No. 6 bar and the 15M bar (30% difference in area)

The second issue I would like to discuss is anchoring to concrete bridge decks with 90mm thick asphalt overlays, which is standard in Ontario. We have been reviewing the capacity of the specified Red Head anchors against other anchoring systems with significantly higher capacities to try and accommodate the 90mm standoff. I note Florida DOT allows a 1" to 2" asphalt overlay for their anchors into concrete.

We do NOT recommend installing any of our concrete tie-downs with asphalt cover. Florida asked us about this as well. The issue is that the asphalt cover creates large bending moments in the anchors which cause them to fail at much lower loads than the designed and tested system. The original projects were for anchorage on concrete surface bridge decks, and we have never had a chance to develop an anchorage that works with concrete overlays. That said, Florida continues to use it because they have no other option. We cannot recommend this type of installation, but it is up to you if you want or need to use it. You may want to account for additional barrier deflection for tie-downs used with the asphalt overlay. This issue has been brought up several times by other states as well. We will try to submit this as a problem statement for next year's Midwest States Regional Pooled Fund.

Is there a convenient time on Thursday that I could call either of you to discuss the above? Thanks,

I should also note the concrete strength of the barrier should be f'c = 5000 psi. The barrier was designed based on this strength, but some plans have been found using 4,000 psi concrete.

# Temporary Concrete Barrier Tie-Down to Concrete with Overlay

## Question

State: WI
Date: 08-19-2008

I was ask by a construction engineer, if it O.K. to bolt the 12.5-foot temporary concrete barrier to a bridge deck that has an asphalt overlay.
Currently, our detail (see attached) does not allow this. Why does the barrier need to rest on concrete? If there is an asphalt overlay on the bridge deck, is there some other modification to the design that we should do?

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## Response

Date: 08-21-2008
Our concern with installation the bolt-through tie-down in asphalt is that the asphalt increases the moment arm on the bolt and the corresponding bending stresses. We have come up with a retrofit using a pipe sleeve in the asphalt and concrete that should eliminate this issue. See the attached schematic.

Attachment: https://mwrsf-qa.unl.edu/attachments/2db0856ceef544c4dfb20ccf4cc98b04.pdf

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# Installation of Concrete Barrier on Super Elevation

## Question

State: WI
Date: 08-20-2008

When contractors are slip-forming concrete barrier on a super elevated section of road, are the contractors installing the concrete barrier perpendicular to the super elevation, or are they installing the barrier plumb (i.e. perpendicular to center of the earth)?
I'm trying to get an idea of the state of practice in other states, so whatever assistance you could provide would be greatly appreciated.

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## Response

Date: 08-20-2008
When a barrier is installed on the inside of a super elevated section, constructing the barrier vertically is probably the safest alternative. The problem with using a vertical barrier only develops when the barrier is installed on the outside of the super. In this situation, the vehicle has a significant upward velocity as it travels up the superelevated cross-section. When a vehicle that is already traveling upward strikes a safety shape, the propensity for climbing is dramatically increased. Hence, there is concern about using a vertical safety shape on the outside of a steeply superelevated curve.

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# Cracking and Climate Effects on CALTRANS Concrete Barrier

## Question

State: WI
Date: 08-25-2008

As you are probably well aware, WisDOT has been in the process of redeveloping our concrete barrier design. MwRSF has been a highly valued resource in this process. Recently, I've been ask "How well does the Caltrans single slope design would perform in a climate similar to ours"
After some internal discussions, I believe that the question is twofold:

1. When compared to other concrete barrier designs, has a Caltrans experienced problem with the accident performance of the barrier system in colder/snowier climates.
2. Does the barrier have cracking problems similar to the attached pictures. The pictures depict two different scenarios. The photo labeled as impact, is from accident impact on one of our freeway sections. The photo labeled construction cracking is develops typically within 48 hours of construction near a tooled in joint.

Does MwRSF have any insight into these matters? I've also attached a file with the Caltrans design.

Attachment: https://mwrsf-qa.unl.edu/attachments/5e8fce574f470c950e5a11453d2cfc15.JPG

Attachment: https://mwrsf-qa.unl.edu/attachments/d1ca926fa8c1c4a7969b6dec3275c6c7.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/c90a0dc766787934e214ac249ac2fa5b.pdf

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## Response

Date: 08-25-2008
For question no. 1, MwRSF does not have information on the real-world crash performance of the CALTRANS TYPE 60 barrier systems.
Your question no. 2 pertains to WsDOT's experience with cracks occurring near tooled-in joints shortly after construction. To date, I have not seen nor heard of this cracking pattern in other states. In the provided construction photograph, cracks appear to have occurred at the man-made joints in both barriers. When and how was the tooled-in joint made? Did the cracks occur after the man-made joint was placed. I assume the answer is yes. If yes, then this cracking is likely due to either the joint type, or its fabrication process, the reinforcement layout used within the barrier, the footing design, or combinations thereof.
Based on everything you have described previously, WsDOT has limited longitudinal rebar in the barrier and no/limited vertical steel as interior locations. It is unclear as to how the barrier is anchored. If the steel is not distributed well in the cross section, I question whether the concrete may want to shrink at one elevation where no steel exists, but it may not shrink as much at other elevations if a greater steel percentage exists in that region, thus causing the diagonal cracking pattern shown in the photograph. At this time, this is only an untested hypothesis.
I suspect that this cracking pattern would not occur if you would use a higher level of longitudinal and vertical reinforcement. But, I must ask why this type of joint is placed when rebar and concrete remain intact within the barrier. If a joint is desired, it would seem more reasonable to provide a through-joint in the barrier where increased reinforcement is used adjacent to the expansion joint.

In the accident photograph, two damage locations are depicted. The noted concrete damage resulted from a motor vehicle impact into the barrier, although the conditions are unknown. What is shown is excessive damage that has occurred due to inadequate longitudinal steel reinforcement and no shear reinforcement. Moderate changes in the design reinforcement would greatly improve impact performance and reduce maintenance requirements resulting from this crash as well as from other environmental influences.

# Bridge Pier Protection

## Question

State: KS
Date: 09-17-2008

We have a project in the KC area that involves reconstruction of two high-speed (65 mph) major highways; however, the existing bridge columns are to remain in place. These columns will not accommodate the AASHTO LRFD requirement for impact load (1800 kN or about 400 kips). We plan to protect these median columns with a 51" or 54" barrier. The attached detail provides details for an application that was previously done for a TL-3 design; we used 32" barrier for that application.
On this project we plan utilize a barrier that will meet TL-5 criteria. We propose to use a tall wall and construction diaphragms between the barrier to isolate the columns if a TL-5 level impact is experienced. The thought is that if the columns are not isolated then the impact load will be transmitted through the barrier and aggregate backfill to the column. Is the last statement true, if not is there an approximation on the amount of load that gets transmitted to the column.
Can we meet TL-5 by tying the barrier into the concrete shoulder and having a granular backfill with diaphragms near the column area?
Can we go vertical for the entire height? If not any recommendations on the shape to address head slap?

Attachment: [https://mwrsf-qa.unl.edu/attachments/a8607139fb8e810220a5e4bcee9ffb58.PDF](https://mwrsf-qa.unl.edu/attachments/a8607139fb8e810220a5e4bcee9ffb58.PDF)

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## Response

Date: 09-18-2008
You note below that you desire to use either 51" or 54" tall barriers to shield the bridge piers. Currently, there are several 42" tall, reinforced concrete barriers that meet the TL-5 impact conditions of NCHRP Report No. 350. Years ago, MwRSF also developed details for tall, TL-5 single-face, safety shape barriers for the WsDOT. The use of 42" high, concrete barriers placed in front the bridge piers would prevent a head-on collision into the piers when a truck leaves the roadway at 15 degrees or less. Using an adequate length of need upstream of the piers would reduce the tendency for the truck to get behind the barrier. A gradual flaring of the barrier toward the median (i.e., away from the roadway) would reduce the required barrier length. The available, 42" reinforced concrete median barriers can be placed on a aggregate base with asphalt placed on both sides or doweled into a RC slab or footing. For the 50+" single-face, TL-5 barriers, anchorage options have consisted of RC slabs or footings.

For these barrier and anchorage options, the RC barriers resist the heavy truck lateral impact loads. With 42" tall barriers, a portion of the tractor-trailer vehicle extends over the top of the barriers. For this truck and trailer-box lean over the barrier top, this vehicle portion could potentially impact the exposed portion of the pier above the top of the parapet. However, the truck and trailer box would not be expected to provide a significant impact event against the stout bridge pier columns. To further protect against this truck-pier impact event, RC diaphragms could be installed between the piers (i.e., parallel to the roadway) in order to stiffen and strengthen the piers to resist truck, trailer lean and subsequent pier impact.

Do you plan to use TL-5 barriers placed forward from the piers? If yes, can you use any of the existing double-face or single-face designs? Do you also want to mitigate pier impact with those vehicle components that lean over 42" tall parapets? Or, do you want to use 50+" parapets to reduce the lean over lower height parapets.

If you want to use a vertical shape parapet, you certainly could do so. If you want to mitigate any tendencies for head ejection and head slap against taller parapets for passenger vehicle impacts, modifications to the new TL-5 barrier could be made. However, the basic top geometry (setback) should be followed and as published in the research report.

With regard to placing traverse diaphragms or compacted fill between the parapets, I do not think either is necessary as long as the appropriate barrier is selected for use.

Please provide any clarifications, comments, and/or questions regarding the information provided above. Once we receive that information, we will continue brainstorming solutions for your situation.

# Cut Slopes

## Question

State: IL
Date: 09-17-2008

What I see from the 1996 RDG is in Chapter 6, Subsection 6.4.1.9 Earth Berm (P. 6-8,9), which says that "slope rates should not exceed 1:2, although steeper slopes can be used if they are smooth and liberally rounded at the base."

But I don't see any such information in the 2002 RDG, so it apparently got removed in that revision. Also don't see that there are any references identified for this information in the 1996 RDG.
Attachment: [https://mwrsf-qa.unl.edu/attachments/5600117d8badc8b0d3233d667f19a0f3.pdf](https://mwrsf-qa.unl.edu/attachments/5600117d8badc8b0d3233d667f19a0f3.pdf)

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## Response

Date: 09-17-2008
I have reviewed the results presented in NCHRP Report No. 158 which was also discussed at the spring Pooled Fund meeting. I have also reviewed the guidance in prior RDGs. Basically, the NCHRP authors do not recommend using slopes beyond a 2:1 back slope when the foreslope is flat. Front end bumper/vehicle snag into the slope was a noted concern. Dean and I are also concerned with a 1:1 slope as it would be the worst situation for causing vehicle rollover, especially for higher center of mass vehicles found on the roads today and as compared to the test vehicles used in the early 70s.
Therefore, we recommend treating the 1:1 back slope situation by one of the following options. First, as you mentioned, a reinforced concrete parapet could be installed close to the base of the back slope but actually cut into it to match the wall height slightly above the soil grade. A vertical parapet would be preferred, although single slope or other approved shapes could be used. Alternatively, a smooth MSE or block type wall could be constructed at the same cut back location, thus producing a smooth vertical parapet for redirecting vehicles. Both of the barrier options would be backed up (i.e., supported) with soil over most of the vertical height.

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# Concrete barrier questions-ZOI for SSB and use of taller or vertical walls

## Question

State: WI
Date: 09-18-2008

As Part of WisDOTs process to develop a new set of standard detail drawings for concrete barrier, WisDOT requires some additional assistance developing some details.
Issue 1: ZOI for various heights of Single Slope Concrete Barrier:
Reviewing the crash test report on the Single Slope Caltrans barrier did not indicate a ZOI. Discussions with Caltrans indicated that they have performed some crash test, but the report is not final. Not knowing when Caltrans will release the report, WisDOT is in proposing to do the following:
1. Because the slope of the Single slope barrier wall is constant, a vehicle should "ride up" the barrier regardless of wall height the same amount (e.g. if the truck rides up 3" during an impact to a 32" single slope barrier it should ride up 3" during an impact on a 56" barrier). This would allow someone to interpolate working width between two known working widths. In fact taller walls may have less deflection because vehicle contacts the upper part of the barrier, prevents the vehicle from lean on top of the wall.
2. Use the 27" working width of ZOI-2 crash test from MwRSF Research Report No. TRP-03-151-07 as the working width for the Caltrans 32" concrete barrier. Although there are differences between the 32" Caltrans barrier and the barrier used in the ZOI-2 test (ZOI-2 barrier has a narrower top and a flatter front face), WisDOT believes that these differences are small considering the variability of real world crash test.
3. It appears that in the Caltran's crash test of a 56" wall that the pickup truck did not lean over the barrier (i.e. 0 working width).
4. Therefore if one were to use a linear interpolation between working width of the 32" ZOI-2 crash test and the 56" Caltrans, WisDOT could calculate working widths for intermediate barrier heights of 36, 42 and 51.
WisDOT understands that crash testing would be the preferred method to determine working width. However, given MwRSF's experience in crash testing does this procedure sound reasonable? Or, does MwRSF have an alternative suggestion on how WisDOT can determine working width for the various barrier heights?
Issue 2 Use of taller walls or vertical barrier in confined locations:
There are going to be situations were designers, have limited space to install a barrier wall (typically near structures). In these locations, designer cannot get the require working width for a given barrier height or shape. Currently WisDOT allows designers to either install a taller barrier wall or vertical barrier.
Given MwRFS's experience in crash testing, is there a preferred alternative (e.g. install the vertical wall of same height, install a taller single slope wall, install a taller vertical wall...). If there is no general preferred alternative, what other factors should a designer consider when selection a barrier wall in these situations.

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## Response

Date: 09-19-2009
My responses to your questions are provided below inred.

Issue 1: ZOI for various heights of Single Slope Concrete Barrier:
Reviewing the crash test report on the Single Slope Caltrans barrier did not indicate a ZOI. Discussions with Caltrans indicated that they have performed some crash test, but the report is not final. Not knowing when Caltrans will release the report, WisDOT is in proposing to do the following:

** CALTRANS has several published reports on various crash testing of single-slope concrete and steel barriers using different test levels.

1. Because the slope of the Single slope barrier wall is constant, a vehicle should "ride up" the barrier regardless of wall height the same amount (e.g. if the truck rides up 3" during an impact to a 32" single slope barrier it should ride up 3" during an impact on a 56" barrier). This would allow someone to interpolate working width between two known working widths. In fact taller walls may have less deflection because vehicle contacts the upper part of the barrier, prevents the vehicle from lean on top of the wall.

** Two terms have been noted above " zone of intrusion (ZOI) and working width. The ZOI was termed by MwRSF researchers in a Pooled Fund study and refers to the maximum vehicle extent behind the top front corner of the barrier. ZOI was reported to vary by barrier shape/type and Test Level. Working width is the lateral distance from the original front face/toe of barrier to the greatest of vehicle extent, barrier deflection, or barrier width.

** Using the same test level and identical barriers, the taller barriers have the potential for reduced ZOI. However, the height where this reduction occurs may not be known. Even though the maximum vehicle extent may be reduced with increases in barrier height, the working width could technically increase since the barrier base would be wider.

2. Use the 27" working width of ZOI-2 crash test from MwRSF Research Report No. TRP-03-151-07 as the working width for the Caltrans 32" concrete barrier. Although there are differences between the 32" Caltrans barrier and the barrier used in the ZOI-2 test (ZOI-2 barrier has a narrower top and a flatter front face), WisDOT believes that these differences are small considering the variability of real world crash test.

** The working width for the single-face, single-slope, concrete barrier was approximately 27", as observed for Test ZOI-2 by MwRSF. Recall that this measurement was taken from the front toe of the barrier. Thus, an effective TL-3 ZOI measurement would have been about 21" for this test. The published ZOI value for the prior TL-3 impacts into 32" tall, sloped-face barriers is 18", while 24" was provided for vertical-face barriers. Actually, the single-slope barrier would likely fall between the two noted ZOI values, thus substantiating the 21" measurement from test no. ZOI-2.

3. It appears that in the Caltran's crash test of a 56" wall that the pickup truck did not lean over the barrier (i.e. 0 working width).

** If no truck lean over the top-front corner of the barrier was observed for pickup truck vehicle, then the ZOI would be zero. However, the working width would be the base width of the rigid parapet.

4. Therefore if one were to use a linear interpolation between working width of the 32" ZOI-2 crash test and the 56" Caltrans, WisDOT could calculate working widths for intermediate barrier heights of 36, 42 and 51.

** I am not sure whether you are seeking working widths or ZOIs for the varying height, single-slope barriers. Also, ZOI may not vary linearly as a sudden change may occur at a height sufficient to prevent vehicle extent over the top of the barrier. We would need to review all of the CALTRANS single-slope barrier tests to determine (estimate) the ZOI for each test and then provide a ZOI guide value for a given test level. In addition, MwRSF has shown how crash testing was used to demonstrate that fixed objects could be allowed within the ZOI. Please note that the original ZOI guidance was conservative and based on the premise that fixed objects would not be contacted if outside of the ZOI. However, fixed objects could be placed within the ZOI if proven to not cause undue risk to occupants or pedestrians nearby. Also, it should be noted that the ZOI concept has not been adopted by AASHTO but serves as a could guide to use to improve motorist safety.

** Are you seeking TL-3 or TL-4 ZOI values?

** Does the WsDOT desire to keep all fixed objects outside of the ZOI?

WisDOT understands that crash testing would be the preferred method to determine working width. However, given MwRSF's experience in crash testing does this procedure sound reasonable? Or, does MwRSF have an alternative suggestion on how WisDOT can determine working width for the various barrier heights?

** A preferred procedure would be to first review the single-slope, crash testing reports published by CALTRANS to determine whether the linear approach is reasonable. If it is, then no additional work would be needed. However, it there are concerns with this approach, then MwRSF would need to acquire film/video from CALTRANS to determine more accurate ZOI and/or working width values. This secondary effort may require considerable resources, that of which may quickly utilize a moderate portion of the Year 19 Pooled Fund consulting funding. If this level of effort is required, then we would first need to obtain Pooled Fund approval to proceed.

Issue 2 Use of taller walls or vertical barrier in confined locations:

There are going to be situations were designers, have limited space to install a barrier wall (typically near structures). In these locations, designer cannot get the require working width for a given barrier height or shape. Currently WisDOT allows designers to either install a taller barrier wall or vertical barrier.

Given MwRFS's experience in crash testing, is there a preferred alternative (e.g. install the vertical wall of same height, install a taller single slope wall, install a taller vertical wall...). If there is no general preferred alternative, what other factors should a designer consider when selection a barrier wall in these situations.

** The ZOI and working width measures for a rigid barrier system are generally of little concern when used in medians to prevent cross-median crashes. However, values for ZOI and working width may be more of concern when placed very close to rigid, fixed objects. Before I can answer the question above, it would be helpful to understand the type of hazards that are anticipated to be shielded by the family of single-slope barrier systems. In addition, it is imperative to know what test level is being considered for each barrier variation. Thanks!

** Of course, other factors that warrant consideration when placing these barriers include: end and interior anchorage for the barrier base, safety treatment for the barrier ends, propensity for head ejection out of side windows and head slap against rigid parapets and objected mounted on top or close behind.

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# Plastic Block-Outs for W-beam Guardrail

## Question

Date: 09-24-2008

In our role as General Engineering Consultant to the Illinois Tollway, we are tasked with keeping the Tollway Standards up to date. The Tollway has adopted the Illinois Department of Transportation's Highway Standard for standard w-beam guardrail. IDOT recently revised their standard to use the Midwest Guardrail System, which has the higher rail and 12" block-outs. IDOT allows the use of plastic block-outs and considers them equivalent and interchangeable with wood. They also allow plastic and wood to be intermixed within a run of guardrail.

The FHWA also considers the "crashworthy" plastic block-outs to be interchangeable with wood. That means they need a FHWA acceptance letter before they can be used.

The Tollway does not have tort immunity, and therefore is very cautious about safety devices. Currently, the Tollway does not mix and match and does not allow plastic block-outs. The Tollway's policy is to only replace in kind when guardrail is in need of repair. There is concern with UV deterioration and stocking of many types of blocks because all maintenance is done by Tollway forces.

Has there been any concern over UV deterioration for plastic blocks?

The FHWA sent me an approval letter for the "Monroeville 12 inch MGS Composite Offset Block" dated April 16, 2008. To your knowledge has any other product been approved or crash-tested using the larger 12" plastic block-out?

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## Response

Date: 09-24-2008

Thank you for your inquiry regarding the use of plastic blockouts. For many years now, several companies have developed various compositions of recycled blockouts for use in strong-post, W-beam guardrail systems. Based on my understanding of these blockouts, most would contain additives that inhibit UV degradation, such as the use of carbon black in tires. However, if you were concerned with UV degradation, it would be recommended that you ask a potential supplier/manufacturer whether resistance to UV deterioration is provided.

With regard to the use of plastic and wood blockouts in standard, W-beam guardrail systems, I am not concerned with using multiple, approved blockouts within the same longitudinal barrier system. Although this type of configuration may be rare in new construction, it may be more common during guardrail maintenance operations over time. In any event, the safety performance of the barrier system should not be affected with the use of different blockout materials that have been approved for use in the barrier system.

I am familiar with the research report which documents the crash testing of the MGS using a 12" deep composite blockout. Unfortunately, I am not aware of any other 12" deep plastic blockout that has been tested, evaluated, and approved for use with the MGS. However, there is new variation of the MGS which utilizes round wood posts with 12" deep wood blocks adapted to the round posts.

Finally, other 31" tall W-beam guardrail systems have been developed to date and have included options for use with and without blockouts. If further information is required on these systems, I recommend that you review the FHWA website for approved crashworthy hardware as well as the corporate websites for NUCOR-Marion, Trinity Industries, and Gregory Industries.

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# IaDOT Questions

## Question

State: IA
Date: 09-30-2008

1. Do you have a recommendation regarding the partial use of an existing w-beam bullnose installation? Specifically, we have a situation where an existing bullnose is protecting the open area in the median between two bridges. One of the bridges is being replaced and traffic will be head-to-head on the remaining bridge. So some of the bullnose will need to be removed in order for the contractor to replace the bridge. Question: how much, if any, of the remaining bullnose installation can we use as a standalone guardrail installation for the head-to-head traffic during the construction period? This would not be used long-term following construction.
2. I have attached a drawing of a bracket used to connect high tension cable to a concrete barrier or bridge end. Our intent is to use these brackets on the trailing end of bridges, instead of a ground anchor, to provide continuous protection for errant vehicles. For continuous median installations, this bracket would be located beyond the clear zone for opposing traffic. The manufacturer has informed me that the bracket is the same one that is used on a ground anchor, and it is almost identical to a low-tension bracket used by South Dakota (detail attached), but with larger bolt holes. This cable attachment portion of the bracket would be located behind the bridge rail where it cannot be hit. However, part of the bracket is exposed on the front side of the bridge rail and could possibly be a snag point. I was hoping you could give me your opinion on this design in general. Also, I would like to know whether you feel this setup would require additional crash testing. The manufacturer has told me that since this is an anchor, and not a terminal, that crash testing is not required.
3. I have also attached a drawing of what we call our "Permanent Road Closure Barricade." The design is based on that of a Type III Barricade. However, this design has more than two posts and the rails extend entirely across the width of the road. We are in the process of updating the drawing, and I am questioning the crashworthiness of this design. Specifically, I was unable to find any crashworthy Type III Barricades that were wider than 8 feet. Or is this barrier not subject to that restriction since it is not used in a work zone? Additionally, the change we are making to the drawing is a result of the reflective sheeting peeling off some of the installations. To combat this, we are proposing to have the sheeting applied to thin aluminum sign stock and that, in turn, would be bolted to the rails of the barricade. Would this change the structural integrity of the barricade and if so, the requirement that it be crash tested?

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## Response

Date: 10-03-2008

1. Please send us details of the W-beam bullnose system along with a description of how much of the system is desired to be removed and/or detached from the bridge. Once we have this information, we will review the design and test reports to determine if your proposed changes would affect the results of those observed in testing.

2. The cable bracket is depicted on the front face of the parapet near the downstream end. This type of bracket would appear to provide concerns for vehicle snag on the bracket and anchored cables. You noted that it would be on the back-side face even though it is shown on the front face. It would be cleaner to attach the bracket on the back side. You note that the manufacturer stated the bracket is nearly identical to the SD DOT bracket used on low-tension designs and identical to their own ground anchor for HT or LT designs? The manufacturer plans to use this on high-tension designs as part of a system. If it already has been evaluated in HT cable barrier testing, then I do not see an issue with structural capacity. If it has not, then the manufacturer would need to provide some assurance that it would also work with HT systems through calculations, components tests, full-scale test, etc. The downstream cable anchorage scenario would not likely need to be re-tested as it is a rigid barrier transitioning into a flexible barrier.
3. I am not aware of any crash testing on such a design but will have a staff member review the designs and then get back to you.

Attachment: https://mwrsf-qa.unl.edu/attachments/512a4a6f3f8e42093dfe3297b33d72b6.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/e3b119a5bf2c50171fcedfd9395e4a14.pdf

Attachment: https://mwrsf-qa.unl.edu/attachments/69bf9f10342f92330803d4fc24915c8e.pdf

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## Response

Date: 10-07-2008

I have attached details of our W-Beam bullnose system. We would like to be able to utilize half of the existing installation (shown as the "T" distance on RE-67) when traffic is head-to-head on the side of the roadway near the top of the page. I am unsure whether we would need to leave the 5-foot radius end section attached, or if we would need to replace that with some other type of end terminal.

Attachment: https://mwrsf-qa.unl.edu/attachments/6bd130bac52e2894ea2266d28617fa2e.jpg

Attachment: https://mwrsf-qa.unl.edu/attachments/533035b032cf6780c82f7b1f2623439d.jpg

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## Response

Date: 10-30-2008

I have reviewed the materials that you have provided. Based on this review, I offer the following comments. First, I understand that two-way traffic will be utilizing the lanes provided at the top of the page (page RE-67), while the bridge at the bottom of the page is being replaced. As such, the hazard between the twin bridges still requires shielding. As I see it, you have two basic options.

Option 1 consists of removing the bullnose buffer end and guardail sections that connect the bullnose barrier to the lower bridge end. Once that material is removed, a crashworthy guardrail end terminal and anchorage system would be connected to the guardrail system shown at the top of the page. The flared guardrail length would be selected such that the system provides adequate shielding of the median hazard.

Option 2 consists of removing a portion of the lower approach guardrail transition (i.e., the section connecting the bullnose guardrail to the lower bridge end). The "STS" segment, measuring approximately 18.75' in length, could be removed to allow construction of the new bridge. Then, in the first two spans of remaining rail, an approved anchorage system could be installed such that anchorage is provided in both directions, thus simulating a rigid attachment to the bridge end. If this option is desirable, we could assist with this detail.

The basic design consists of a standard foundation tube with soil plate at the last two wood BCT posts. A standard steel channel strut is connected between the two posts/steel sleeves. No impact head would be needed in this region as no crashes would be expected at this far end and since the bridge/road is closed. Standard anchor cable hardware would be placed between the two wood posts and in both directions (reverse cables in first span). You would need to drill an extra set of holes to place the second cable anchor bracket on the rail close to the top of post 1, similar to that near the top of post 2. Now the rail would be anchored in both directions " tension and compression, thus simulating a rigid attachment to the bridge end. We have done this in our thrie beam bullnose testing as well as in recent box beam testing when we were unsure with load direction would occur.

I am enclosing CAD details for a new MnDOT bullnose R&D project currently within MwRSF. For this effort, we have placed both the standard and reverse direction cable anchorages on the downstream end. In prior bullnose testing efforts, we switched the direction of the single cable anchorage from one test to another. In future testing, we will place two anchor cables on the downstream end " one in each direction.

It should be noted that this double anchorage should be used on any bullnose design that incorporates a free end that requires anchorage.

Our CAD details show the use of a 6-ft long tube without a soil plate but with a channel strut between two tubes. An alternative would be to use the shorter BCT tubes that incorporate soil plates and use the channel strut.

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# Caltrans Barrier Request

## Question

State: WI
Date: 09-30-2008

Has MwRSF looked into :
1. TL-3 Working widths for various heights of caltrans barrier?
2. A recommendation on what to do near fixed objects (e.g. go with taller barrier, go with vertical barrier, go with taller and vertical barrier) in areas where working width is reduced?

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## Response

Date: 10-02-2008

MwRSF has reviewed available information on the working widths and zone-of-intrusion (ZOI) values for the single-slope, concrete barriers evaluated by MwRSF and CALTRANS. Based on this information, the working width and zone-of-intrusion measures for the 32-in. tall, single-slope barrier are approximately 27 and 21 in., respectively. The published ZOI value for the prior TL-3 impacts into 32" tall, sloped-face barriers is 18", while 24" was provided for vertical-face barriers. Actually, the single-slope barrier would likely fall between the two noted ZOI values, thus substantiating the 21" measurement from test no. ZOI-2. The maximum effective height for this lateral extent was nearly 42 in. As such, a conservative approximation would be to use the 27-in. working width value from 32 to 42 in. Beyond the 42-in. barrier height, the ZOI would be approximately zero for TL-3 conditions and valid through the 56-in. barrier height. Thus, the working width for 42 to 56-in. tall, single-slope barriers would be the base barrier width or 24 in.

Please note that these ZOI values provide conservative results for the placement of fixed objects on and nearby the barrier system. Actually, a fixed object could be placed closer to the top-front corner of the barrier as long as acceptable crash performance was obtained.

Previously, MwRSF provided guidance for selecting the top geometry of taller vertical, or near vertical barriers, in an effort to prevent head contact with tall barriers. A head ejection envelope was provided. However, guidance was not provided for sloped-face barriers in terms of the recommended top-barrier geometry or head ejection envelope.

In summary, here are my suggestions:

SS Barrier Height WW ZOI
32" 27 21
36" 27 (assumed) ?? (measured from top front corner but unknown)
42" ( 27 (assumed) ?? (measured from top front corner but unknown)
42" (>) 24 (barrier width) 0
51" 24 (barrier width) 0
56" 24 (barrier width) 0

Vertical Barrier Height
32" 24 24 (vertical barrier has toe and front corner at same position)

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