Progress Report
Third Quarter 2008 – July 1st to September 30th
Midwest Roadside Safety Facility
Midwest State's Regional Pooled Fund Program
September 15, 2008

Pooled Fund Projects with Bogie Testing or Full-Scale Crash Testing in Past Quarter

Development of a TL-4, Four-Cable, High-Tension, Barrier System for 4:1 V-Ditch Applications

In 2008, the Pooled Fund Member States voted to use the 10000S funds to re-run the 1100C vehicle test into a modified high-tension, four-cable, barrier system placed in a 4:1 V-ditch using a slightly firmer median soil condition within the impact region. The bottom cable was positioned 13.5 in. above the ground with the remaining cables spaced upward using 10.5 in. between cables. On August 25, 2008, the 1100C re-test was performed in a 46-ft ditch with the cable barrier placed 4 ft up the back slope. Shortly after impact, the vehicle was captured by the lower cable. Following the crash test, it was apparent that the roof and upper A-pillar region had been crushed downward by one of the high-tension cables. The roof crush exceeded the limits provided in MASH-08, thus resulting in a small car test failure. An investigation was performed to determine the cause for the unfavorable outcome. From inspection, the cable brackets detached as designed, thus leaving only the bolts in the post flanges. However, the exposed bolt heads were sufficient to prevent upward cable movement at some post locations, thus not allowing the translation of certain tensioned cables up and over the small car. Refinements to this cable attachment bracket, or the implementation of a new bracket, should prevent this unfavorable outcome. Documentation and reporting of this research and testing program will continue into the fourth quarter as all funded tests have been performed.

Continuation funding for the high-tension, cable barrier system was included in the Year 19 Pooled Fund Program.

Standardizing Posts and Hardware for MGS Transition

At the April 2008 Pooled Fund meeting, the States voted to re-test Design K using a 2270P vehicle. Funds for this re-test were added to the Year 19 Pooled Fund program. The 2270P re-test had been planned for May or June 2008 but was delayed due to wet weather conditions. However, the crash test was successfully performed on July 7, 2008. Now, an 1100C crash test is required on the same design in order to complete the test matrix. Once the small car test has been completed on a simplified, alternative steel-post transition, the wood post alternative will be developed using dynamic bogie testing.

Pooled Fund Projects with Pending Bogie Testing or Full-Scale Crash Testing

Phase I Development of a TL-3 MGS Bridge Rail

In the second quarter, a third railing concept was developed and included the use of weak S3x5.7 posts spaced on 3 ft - 1.5 in. centers. In the fourth quarter, this weak-post concept will be attached to a reinforced concrete deck section and subjected to dynamic bogie testing. BARRIER VII computer simulation modeling was performed on the weak post variation of the MGS and will continue in the fourth quarter. A literature search was continued on weak-post, W-beam guardrail systems in order to evaluate post to rail attachments. Several concepts for attaching weak posts to standard reinforced concrete bridge decks were also refined in the third quarter. Three static tests were performed to determine bolt release loads for various rail attachments to weak posts. Following the bogie testing program, the preferred post concept and railing design will be submitted to the Pooled Fund members for review and comment.

Testing of Cable Terminal for High Tension Cable (1100C & 2270P)

Work on this project will commence after crash testing has been completed on the high-tension, four cable barrier system. It is planned to adapt the breakaway cable lever arm technology, developed during the low tension testing, into the high-tension barrier system. Partial project funding is available in this program.

Performance Limits for a 6-in. High, AASHTO Type B Curb Placed in Advance of the MGS

An analysis of the curb test results revealed critical placement of the MGS to be 4 ft and 8 ft for the standard 31-in. and modified 37-in. designs, respectively. For the standard MGS, testing would likely be needed using a 2270P vehicle at the 4-ft offset. For the 37-in. tall, MGS, testing would likely be needed using a 2270P vehicle at the 8-ft offset. In addition, the 37-in. tall MGS may also require 1100C vehicle testing at the 4-ft offset location. Originally, the 31-in. tall MGS was constructed 4-ft behind a 6-in. tall, AASHTO Type B curb. Testing with a 2270P vehicle was delayed due to rain and wet soil conditions. The project was then discussed at the April 2008 Pooled Fund meeting. Later, the curb system was removed.

Following a survey of the Pooled Fund member states at the meeting and through emails, the majority voted for the 4 to 8 ft range of use for the MGS. Later, MwRSF researchers presented this information, along with a revised research plan, to the states in writing. From this request, the member states decided to crash test the MGS using an 8-ft curb offset. The first 2270P crash test is now planned for the fourth quarter. If additional crash testing is required, future funds will be needed to complete this effort.

Paper Studies

Cost-Effective Measures for Roadside Design on Low-Volume Roads

An analysis of culvert treatments was completed in the third quarter, indicating a strong need to remove any rigid or semi-rigid railing structures located on bridges which are not properly transitioned to guardrail nor shielded. A field trip was conducted in two Nebraska counties during the third quarter to evaluate the need for treatment of roadside slopes. Detailed documentation of slope profiles was conducted, analyzed, and treatment possibilities were considered. On many low-volume roadways, slope rates near the roadsides are steeper than 2:1 and more than 10 ft deep, often leveling out near trees or fields. Both cable guardrail and W-beam guardrail systems were selected as treatment options where appropriate, and an RSAP benefit-to-cost analysis was performed on these options. Results of the study should be completed in the fourth quarter. Safety treatments for roadside trees were also analyzed during the third quarter. Many different hazard arrangements were considered, including small clusters of trees, scattered occurrences, and dense rows of trees located adjacent to the roadway. Results of the tree study were completed and indicated a very strong need to remove many trees located up to 10 ft from the side of the roadway for tree diameters greater than 6 in. Additionally, treatment of a fourth hazard category was initialized, consisting of treatment of low-volume bridges and bridge rails. Results of the bridge rail analysis should be completed in the fourth quarter.

Submission of Pooled Fund Guardrail Developments to AASHTO TF-13 Hardware Guide

To date, 19 systems have been submitted to TF-13 for review and approval. Ten systems were approved for the Guide at the September 2007 meeting. Nine systems more were reviewed in May 2008. MwRSF is implementing edits into several CAD details in preparation for the upcoming AASHTO Task Force 13 meeting to be held on September 29-30, 2008.

Development of Warrants for Median Barrier System

The project is nearing completion. A draft final report has been prepared and will be submitted by the end of the third quarter.

Cost-Effective Upgrading of Existing Guardrail System

The literature review of historical W-beam accident studies has been completed. A listing of W-beam guardrail installations has been obtained from Kansas for use in the RSAP study. These sites will soon be surveyed to document selected guardrail installations. The field investigation process is now planned for the fourth quarter of 2008.

Projects Funded by Individual State DOTs and Routed Through NDOR and/or Pooled Fund Program

Iowa RSAP Analysis of Culvert Treatments (Iowa Department of Transportation)

The RSAP analysis of safety treatments for cross drainage culverts has been completed. The analysis examined the safety performance of untreated culverts, extending the culvert out of the clear zone, installing safety grates, and shielding the hazard with W-beam guardrail. The variability in construction costs for extending culvert grates forced this study to focus on identifying accident costs associated with each treatment alternative. Accident costs for each alternative were tabulated for a wide variety of roadway and roadside characteristics. Highway designers can use these tabulated accident costs to calculate benefit-to-cost ratios for each of the safety treatments studied. The analysis appeared to indicate that the use of culvert safety grates was most appropriate for low and medium volume roadways, while culvert extension appeared to provide the most cost beneficial alternative for some high volume facilities. Review of the draft final report was recently completed by the Iowa DOT. MwRSF will complete report edits in the fourth quarter.

Development of a New, TL-4 Precast Concrete Bridge Railing System (Nebraska Department of Roads)

The original project objective was to develop a TL-4, aesthetic, open concrete bridge railing for use on cast-in-place decks as well as precast deck panels. At a July meeting with the sponsor, the grouted joint detail was selected for implementation into the CAD details so that the single rail section could be fabricated. Dynamic impact testing of a single rail section attached to a short section of cast-in-place deck was planned to occur in the third quarter of 2008. Construction of the bridge support beam had been constructed previously. The short deck section was planned to be cast in August 2008. Fabrication of the 16-ft long, single rail section was planned for August 2008. Once the single rail test is completed, MwRSF will make plans for the construction and full-scale testing of the new bridge railing and deck system. In the third quarter, documentation and reporting of the Phase I R&D program was continued.

The research team also had discussions with the sponsor regarding project delays and funding expenditures exceeding the budget estimate. From this discussion, it was noted that the sponsor had no additional funds for this project. As such, the research team noted that they would investigate other external sources for funding and in-kind contributions. The research team investigated the FHWA Highways for Life program as a potential funding source. However, MwRSF was unable to find the required industrial partner to champion the research proposal. Other potential options include the NCHRP IDEA program for the spring 2009 funding cycle or query other State DOTs whom may have interest in the use of precast concrete railing systems. Construction of the bridge deck and rail sections as well as the completion of the crash testing program will commence after new project funding is obtained.

Qualification of Type II and Type I End Terminals for Box Beam (New York DOT)

In 2007, three 1100C full-scale vehicle crash tests were performed on two NYSDOT box beam terminal systems. A draft report documenting the test results has been prepared and submitted to NYSDOT for review and comment. Preliminary comment has been obtained and report edits are completed. Additional test results are planned for this report.

In 2008, a continuation project was approved to provide new funding for an additional crash testing program. Two 2270P crash tests were performed in the third quarter – one on July 11th and another on July 31st. The next planned crash test involves a TL-2 1100C vehicle impacting the box beam terminal after traveling through a ditch section. Construction of the ditch section as well as the box beam guardrail and terminal systems have begun. However, subsurface debris and rain have slowed progress considerably. A TL-3 2270P crash test is to follow the TL-2 1100 C test. The two crash tests are planned for completion in late September or early October.

Universal Breakaway Steel Post for Guardrail (Minnesota DOT)

Several breakaway concepts have been brainstormed and designed, with CAD details prepared. Thirteen dynamic bogie tests have been performed on the initial and modified concepts, along with data analysis. Concept refinement and new CAD details were also completed. In June 2008, six additional bogie tests were performed on MGS CRT posts placed in soil and impacted at varying impact orientations – 0, 45, and 90 degrees. Data analysis, documentation, and reporting of the breakaway post testing occurred in the third quarter. CAD details for the preferred post concept were submitted to the MnDOT for review and comment. Two full-scale vehicle crash tests are planned for the fourth quarter of 2008. Curved and slotted thrie beam bullnose sections have been acquired. Construction planning will begin once approval is obtained from the sponsor.

Development of a Test Level 1 Timber Curb-Type Railing for Use on Transverse, Timber, Nail-Laminated Deck Bridges (West Virginia DOT)

The project consisted of adapting and modifying a crashworthy TL-1 timber bridge railing system for use on nail-laminated, transverse timber deck bridges, while using the proposed MASH 08 guidelines. A sloped end section was also developed for rural, low-speed, low-volume applications. Five static tests and one 2270P crash test were completed in the third quarter. Construction of the timber bridge deck and timber railing and transition systems were also completed in July. Concrete deck surfacing was applied to the deck as well. The static test results demonstrated that the use of shear plates or split rings were not required for the wood interfaces between the rails, scupper blocks, and timber deck. On July 18th, a 2270P full-scale crash test was successfully performed. Documentation and reporting of the research project was initiated. Completion of the draft report is expected in the fourth quarter.

Development of a Test Level 2 Steel Bridge Railing and Transition for Use on Transverse, Timber, Nail-Laminated Deck Bridges (West Virginia DOT)

The project consisted of adapting and modifying a crashworthy TL-2 steel bridge railing system for use on nail-laminated, transverse timber deck bridges, while using the proposed MASH 08 guidelines. An approved TL-2 approach guardrail transition was implemented into the design. Four dynamic bogie tests on posts attached to nail-laminated timber deck bridges were successfully performed on August 26, 2008. Dynamic bogie testing demonstrated that the alternative deck system could withstand the post loading without damaging the deck. Draft design details were submitted to and approved by the sponsor. The bridge deck system used in the TL-1 project was also used for this study. Mr. Glenn Lough of the West Virginia DOT traveled to Lincoln and witnessed the bogie testing program. Following the tests, steel post hardware and sections of the timber deck were shipped to the sponsor for review and to assist in the implementation of the railing system. Complete CAD details for the bridge railing and transition systems will be completed in the fourth quarter. Documentation and reporting of the research project was initiated. Completion of the draft report is expected in the fourth quarter.

Awaiting Reporting

Development of a Temporary Concrete Barrier Transition

Two pickup truck crash tests were successfully performed on a transition between temporary concrete barrier and permanent concrete median barrier. The evaluation was performed using the MASH-08 guidelines. In the third quarter of 2008, significant progress was made toward the completion of the draft research and test report. A draft report should be submitted to the Pooled Fund members for review and comment in the fourth quarter.

Approach Slopes for W-Beam Guardrail Systems

At the conclusion of this testing program, the MGS guardrail system can now safely be located at a 5-ft offset distance from the edge of the traveled way on slopes of 8:1 or flatter. A draft report documenting this research remains under internal review.

Midwest Guardrail System Placed at the Breakpoint of a 2:1 Slope

An MGS system utilizing 9-ft long, W6X9 steel posts spaced at 6-ft 3-in. centers was successfully crash tested utilizing a 2270P Dodge Quad Cab vehicle. The vehicle was safely redirected. A draft report has been prepared and remains under internal review. A TRB paper was presented at the 2008 Annual Meeting of the Transportation Research Board and accepted for publication.

Termination of Temporary Concrete Barrier

An anchor system utilizing two driven steel posts and soil plates from the existing cable anchorage system was tested with a 2270P impacting 4 ft - 3.6 in. upstream of the joint between barriers 1 and 2. The crash test met all salient test criteria. A test report documenting the results has been prepared and remains under internal review.

Draft Pooled Fund Reports Completed

Polivka, K.A., Sicking, D.L., Reid, J.D., Bielenberg, R.W., Faller, R.K., and Rohde, J.R., *Performance Evaluation of Safety Grates for Cross-Drainage Culverts*, Draft Report to the Midwest State's Regional Pooled Fund Program, Transportation Research Report No. TRP-03-196-08, Project No.: SPR-3(017), Project Code: RPFP-04-02 and RPFP-05-07 – Years 14 and 15, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, September 3, 2008.

Final Pooled Fund Reports Completed

Hitz, R.A., Molacek, K.J., Stolle, C.S., Polivka, K.A., Faller, R.K., Rohde, J.R., Sicking, D.L., Reid, J.D., and Bielenberg, R.W., *Design and Evaluation of a Low-Tension Cable Guardrail End Terminal System*, Final Report to the Midwest States' Regional Pooled Fund Program, Transportation Research Report No. TRP-03-131-08, Project No.: SPR-3(017), Project Code: RPFP-01-03, RPFP-04-07, and RPFP-05-03 – Years 11, 14, and 15, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, July 15, 2008.

Stolle, C.S., Polivka, K.A., Reid, J.D., Faller, R.K., Sicking, D.L., Bielenberg, R.W., and Rohde, J.R., *Evaluation of Critical Flare Rates for the Midwest Guardrail System (MGS)*, Final Report to the Midwest States' Regional Pooled Fund Program, Transportation Research Report No. TRP-03-191-08, Project No.: SPR-3(017), Project Code: RPFP-04-03 and RPFP-05-05 – Years 14 and 15, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, July 15, 2008.

Draft Reports – Projects Funded by Individual State DOT and Routed Through NDOR and/or Pooled Fund Program

Not applicable in this quarter.

Final Reports – Projects Funded by Individual State DOT and Routed Through NDOR and/or Pooled Fund Program

Not applicable in this quarter.

<u>Pooled Fund Consulting Summary</u>

Midwest Roadside Safety Facility
July 2008– October 2008

This is a brief summary of the consulting problems presented to the Midwest Roadside Safety Facility over the past quarter and the solutions we have proposed.

Problem # 1 –Thrie Beam Bridge Rail Termination

State Question:

Dear MwRSF,

Attached is a photo of a thrie beam bridge rail attached to a box culvert.

I was told that because this is a bridge rail, it does not need to have an end treatment on the downstream end to transfer the force of an impact to the ground. Is this correct? Or should it be treated with a transition, a section of guard rail and then a terminal that has a cable attached to some ground tubes?

MwRSF Response:

Erik:

My first question is whether this bridge rail can be impacted by reverse direction traffic? If yes, then the bridge end requires treatment, likely occurring from an approach rail and guardrail end terminal. If not, then we can move on to the next question.

If this bridge rail was designed to redirect errant vehicles through membrane action (tensile capacity), moment transfer through rail/posts, and shear transfer, then downstream anchorage would be need to be provided. If hazard shielding is required within a few spans from the end of the barrier system but without anchorage, the rail system may not be capable of performing in an acceptable manner.
For rigid bridge railing systems that provide additional strength near the ends, downstream approaches may not necessarily be needed but would need to be determined based on the individual design.

Ron

Figure 1. Thrie Beam Bridge Rail

Problem # 2 – Barrier Design for Off Road Applications

State Question:

Dear MwRSF,

I was doing some digging around in our library and found MwRSF's report on "Development of a Temporary Barrier System for Off Road applications".

Some questions:

1. Will the ski design attach to the current MwRSF barrier design?
2. What modifications are necessary for the barrier to accept the ski design?
3. What slopes would the ski design be acceptable to be installed on?
4. What is the working width of the barrier with the skis on it?

Erik Emerson P.E.
Wisconsin Department of Transportation
Standards Development Engineer
4802 Sheboygan Ave Room 651
P.O. Box 7916
Madison, WI 53707-7916

MwRSF Response:

With regards to the ski attachment, this system has a strong potential for being used with the standardized TCB, now referred to as the KS/FL F-shape TCB. To be sure, we will need to evaluate the changes made to the barrier over the last decade. I will ask Karla, Bob, or Scott to review these changes for you next week and report back to you.

The TCB ski system was tested and evaluated on level terrain. Since the system is designed to resist barrier rotation, it may be reasonable to assume that it would perform in an acceptable manner when placed on roadside slopes on 1:12, or possibly 1:10. However, it would be recommended that the barrier be propped up to be perpendicular to level terrain by using the screw-jack system that is part of the ski hardware.

Since working width was likely not reported when this research was conducted, we would need to evaluate the overhead film data to determine the working width. Recall that working width is measured from the front face the barrier (toe) to the farther part of the barrier or vehicle during redirection. The back of the ski hardware would likely be this point. I will have our staff also determine this value.

Ron

Ronald K. Faller, Ph.D., P.E.
Research Assistant Professor

Problem # 3 – F-Shape in Front of Curb Adjacent to MSE Wall

State Question:

Dear MwRSF,

Hey guys 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?

MwRSF Response:

Rod:

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.

Please let us know what you think!

Ron

Problem # 4 – CALTRANS Concrete Barrier

State Question:

Dear MwRSF,

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.

MwRSF Response:

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.

Ron

Problem # 5 – CALTRANS Concrete Barrier- Part II

State Question:

Dr. Faller,

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).

MwRSF Response:

Hello Erik!

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 be appropriate to require at least a 30 ft barrier length before one changes reinforcement patterns.

Ron

Problem # 6 – MGS Median Barrier

State Question:

Dean Foecke of the Ohio DOT called MwRSF and asked our thoughts on using the MGS system in a median barrier configuration with rail on both sides.

MwRSF Response:

Hi Dean,

Earlier today, you and I discussed the use of the MGS system as a median barrier. 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.

I have copied Nick Artimovich on this email. Nick if you have any comments or concerns, please let us know.

Problem # 7 – PCB Treatments

State Question:

Dr. Faller,

I received this email from our Cleveland district - and Dennis 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?

Thanks,

Dean Focke, P.E.
Roadway Standards Engineer
Office of Roadway Engineering
Ohio Dept. of Transportation

MwRSF Response:

Dean:

Sorry for the delayed response on the TCB question noted below.

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.

Please let me know if you have any questions or comments on the information provided above. Thanks!

Ron

Ronald K. Faller, Ph.D., P.E.
Research Assistant Professor

Problem # 8 – MwRSF PCB

State Question:

Ron/Bob: Not sure who to contact regarding some questions I have on your MwRSF PCB, so I'm trying both of you. I did try and call earlier and decided to not leave a lengthy voice mail, and did not want to bother Dean, thus the e-mail.

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.

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

Mark Ayton, P. Eng.
Senior Engineer, Highway Design
Design & Contract Standards Office
Ontario Ministry of Transportation
Garden City Tower, 2nd Floor North
301 St. Paul Street
St. Catharines, Ontario
L2R 7R4

MwRSF Response:

Hi Mark,

I have responded to you questions below. Please call or email if you have further questions.

Thanks

Bob Bielenberg, MSME, EIT
Research Associate Engineer
Midwest Roadside Safety Facility
527 Nebraska Hall

Lincoln NE, 68588-0529
402-472-9064
rbielenberg2@unl.edu

Ron/Bob: Not sure who to contact regarding some questions I have on your MwRSF PCB, so I'm trying both of you. I did try and call earlier and decided to not leave a lengthy voice mail, and did not want to bother Dean, thus the e-mail.

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 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.

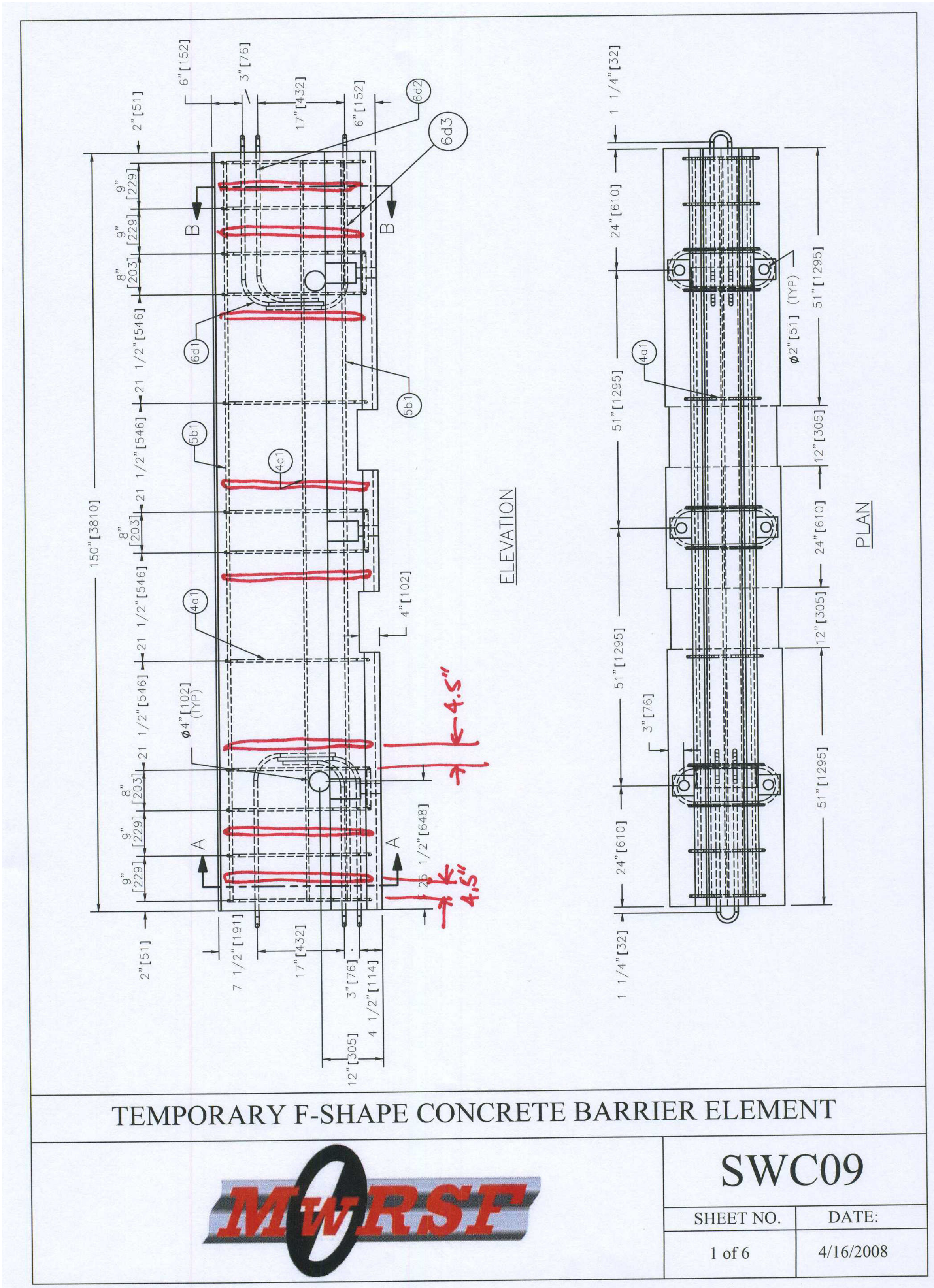


Figure 2. MwRSF PCB Metric Reinforcement Schematic

Problem # 9 – Installation of Concrete Barrier on Super Elevation

State Question:

Dear MwRSF Pool fund members,

When contractors are slipforming concrete barrier on a super elevated section of road, are the contractors installing the concrete barrier perpendicular to the super elevation, or are the 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.

Sincerely,

Erik Emerson P.E.
Wisconsin Department of Transportation
Standards Development Engineer

MwRSF Response:

When a barrier is installed on the inside of a super elevated section, as shown in Erik's drawing, 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.

Thanks,

Dean

Problem # 10 – Temporary Concrete Barrier Tie-Down to Concrete with Overlay

State Question:

Dear MwRSF,

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?

Your help as always is greatly appreciated.

Erik Emerson P.E.
Wisconsin Department of Transportation
Standards Development Engineer

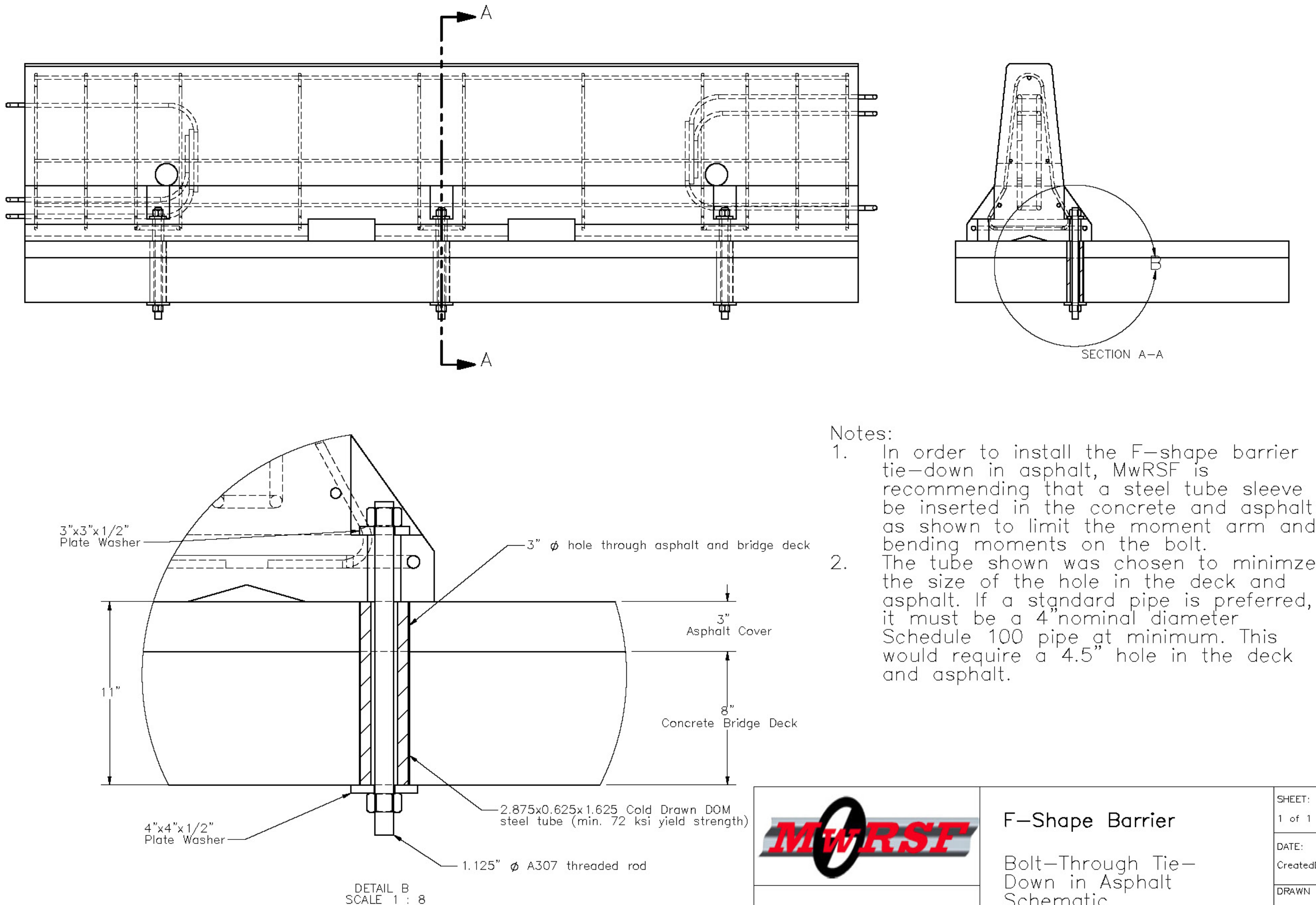
MwRSF Response:

Hi Eric,

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.

Contact me with any questions or concerns.

Thanks



Notes:

1. In order to install the F-shape barrier tie-down in asphalt, MwRSF is recommending that a steel tube sleeve be inserted in the concrete and asphalt as shown to limit the moment arm and bending moments on the bolt.
2. The tube shown was chosen to minimze the size of the hole in the deck and asphalt. If a standard pipe is preferred, it must be a 4"nominal diameter Schedule 100 pipe at minimum. This would require a 4.5" hole in the deck and asphalt.

MwRSF Midwest Roadside Safety Facility	F-Shape Barrier Bolt-Through Tie-Down in Asphalt Schematic		SHEET: 1 of 1 DATE: CreatedDate DRAWN BY: Your Initals
	DWG. NAME. Tie-down_through_asphalt_concept	SCALE: None UNITS: Inches	REV. BY: Your Initials

Figure 3. Tie-Down Through Asphalt Overlay Schematic

Problem # 11 – Cracking and Climate Effects on CALTRANS Concrete Barrier

State Question:

Dear MwRSF,

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.

Sincerely,

Erik Emerson P.E.
Wisconsin Department of Transportation
Standards Development Engineer

MwRSF Response:

Erik:

Thanks for the email inquiry on concrete barriers.

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.

Please let me know if you have any other questions or comments, and I look forward to hearing from you in the near future.

Ron

Ronald K. Faller, Ph.D., P.E.
Research Assistant Professor

Figure 4. Cracking of Concrete Barrier

Problem # 12 – Alternative Bolt-Through Tie-Down Anchors

State Question:

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.

MwRSF Response:

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.

Table 1. Structural Capacities of Threaded Rods

Nominal Threaded Rod Diameter (in.)	Gross Area (in.2)	Tensile Area (in.2)	Gross Elastic Section Modulus (in.3)	Calculated Rod Diameter (in.)	Net Elastic Sec Modulus (in.3)	Steel Grade or Specification	Yield Strength (ksi)	Ultimate Strength (ksi)	Maximum Shear Strength (ksi)	Net Yield Force (kips)	Net Ultimate Force (kips)	Net Elastic Bending Capacity (kip-in.)	Net Shear Force (kips)	
1.000	0.7854	0.606	0.0982	0.8784	0.0665	A307	36	60	20.7846	21.8160	36.3600	2.3954	12.5955	
1.000	0.7854	0.606	0.0982	0.8784	0.0665	A325	92	120	53.1162	55.7520	72.7200	6.1216	32.1884	
1.125	0.9940	0.763	0.1398	0.9856	0.0940	A307	36	60	20.7846	27.4680	45.7800	3.3842	15.8587	
1.125	0.9940	0.763	0.1398	0.9856	0.0940	A325	81	105	46.7654	61.8030	80.1150	7.6144	35.6820	
1.250	1.2272	0.969	0.1917	1.1108	0.1345	A307	36	60	20.7846	34.8840	58.1400	4.8434	20.1403	
1.250	1.2272	0.969	0.1917	1.1108	0.1345	A325	81	105	46.7654	78.4890	101.7450	10.8977	45.3156	
Ronald K. Faller														
January 8, 2006														
1.000	0.7854	0.541	0.0982	0.8300	0.0561	B-12 Coil	120	140	69.2820	64.9200	75.7400	6.7351	37.4816	Only Root Area Provided By Manufacturer
Ronald K. Faller														
September 2, 2008														

Figure 5. Anchor Comparisons