MIDWEST POOLED FUND PROGRAM

Progress Report - Third Quarter 2010
July 1st to September 30th
Midwest Roadside Safety Facility
Nebraska Transportation Center
University of Nebraska-Lincoln

September 27, 2010

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

Maximum MGS Guardrail Height – Program Year 20

On June 29, 2010, MwRSF conducted one small car crash test (test no. MGSMRH-1) into a 34-in. tall Midwest Guardrail System (MGS) using an 1100-kg Kia Rio according to the TL-3 safety performance guidelines of MASH. The small car was successfully contained and redirected. Photographs for this test are shown below.

Based on the results of the first small car test, the MGS top mounting height was raised to 36 in. On September 9, 2010, a second small car test (test no. MGSMRH-2) was conducted into a 36-in. tall Midwest Guardrail System (MGS) using an 1100-kg Kia Rio according to the TL-3 MASH safety performance guidelines. Again, the small car was successfully contained and redirected. Photographs for this test are shown below.

Universal Breakaway Steel Post for Thrie-Beam Bullnose
Program Year 21 [TPF-5(193) – Supp. #35]

This research project provides continuation funding for the development and testing of a universal breakaway steel post for the thrie beam bullnose barrier system. The initial development and crash testing was performed under a recent MnDOT research study using the NCHRP Report No. 350 safety performance guidelines.

Two full-scale vehicle crash tests are planned under this supplemental project. The first crash test (test designation no. 3-30) was performed on September 13, 2010 using an 820C small car vehicle impacting at the target conditions of 100 kph and 0 degrees on the nose of the barrier system and offset using the ¼-point aligned with the centerline of the device. During the test, the vehicle was safely contained within the bullnose median barrier system, and all of the occupant risk measures were met. Photographs for this test are shown below.

The second crash test (test designation no. 3-31) is planned for late September 2010 using a 2000P pickup truck vehicle impacting at the target conditions of 100 kph and 0 degrees. This test will be used to evaluate the penetration distance into the system.

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

Standardizing Posts and Hardware for MGS Transition – Program Years 18 and 19

A draft report was prepared for the simplified, steel-post, approach guardrail transition system attached to the MGS. The draft report was sent to the member states for review and comment on August 9, 2010. Comments were due on August 30, 2010. A final report will be prepared in late September.

A BARRIER VII computer simulation effort is nearly completed to evaluate the dynamic barrier performance when using wood posts with both an upper and lower bound for post-soil behavior. Initially, an 8-in. x 10-in. wood post was being considered as a replacement for W6x15 steel posts used in approach guardrail transitions. However, dynamic bogie testing was re-initiated to explore the impact performance of 6-in. x 10-in. wood posts embedded in soil. During the testing of two 6-in. x 10-in. posts, inconclusive results were obtained as one post fractured and another provided desirable results. Upon examination, it was realized that the fracture occurred at a knot located within the critical region. As a result, two more dynamic bogie tests are planned. Once post testing is completed, the BARRIER VII simulation effort may require a few modifications and subsequent analysis for verification. A second report will contain the results of the wood-post transition system, which is now planned for completion in the Fourth Quarter of 2010 while awaiting the results from two dynamic post tests.

Development of a TL-4, Four-Cable, High-Tension, Barrier System for 4:1 V-Ditch Applications – Program Years 12, 14, 18, 19, and 20

In the Second Quarter of 2010, MwRSF planned to reconstruct the high-tension, four-cable median barrier system in the 4:1 V ditch and 4 ft up the back slope. Subsequently, an 1100C small car re-test was to be performed using the TL-3 safety performance guidelines of MASH. Unfortunately, significant spring and fall rain was observed in the Lincoln area during these periods, thus resulting in a water-filled ditch for long rain periods. Water pumping was utilized during the 2nd and 3rd quarters. When the ditch no longer had standing water, soil coring was started in an effort to place posts. However, the constant high water table did not allow for soil removal. As a result, barrier construction has not commenced in the bottom of the V-ditch and the construction and crash testing program has continued to be delayed.

At this time, MwRSF personnel are exploring the option for burying large polymer tanks at each post location found in the ditch bottom to prevent the water table inflow from saturating the compacted soil surrounding the installed posts.

Impact Evaluation of Free-Cutting Brass Breakaway Couplings – Program Year 20

Following discussions with FHWA and the Illinois Department of Transportation, two low-speed, crushable-nose, pendulum tests were required on various luminaire poles in order to investigate the impact performance of a new, free-cutting, breakaway, brass coupling. The brass coupling is planned for use as replacement to existing, higher-cost couplers.

In 2009, two low-speed pendulum tests were performed. The maximum allowable change in velocity was exceeded in both pendulum tests. The Illinois DOT modified the design of the brass couplers. In 2010, two low-speed pendulum tests were performed on the modified brass couplers. The first pendulum test was unsuccessful with a 50-ft tall, heavy steel pole, while the second test was successful with a 30-ft tall, aluminum pole. A third test was performed on the currently-available coupler in combination with the tall, heavy steel pole. For the low-speed test on the currently-available coupler, the change in velocity was below the limit of 5 m/s. However, the high-speed extrapolation for the change in velocity exceeded the limit when considering the critical pole configuration.

In the Third Quarter of 2010, the ILDOT further modified the brass coupler and conducted static component testing to evaluate design changes. At the time of this progress report, information was unavailable as to the redesign and test results.

MwRSF will conduct additional pendulum testing with available funds in October 2010 in an effort to document and report the testing program by December 2010. To date, funding for four pendulum tests has been budgeted, while five tests have been performed.

Phase I and II – Guidelines for Post-Socket Foundations for Four-Cable, High-Tension, Barrier Systems – Program Years 19 and 20

Previously, four dynamic component tests were performed on prototype post-socketed foundation systems placed in a weak soil condition (sand). Concrete fracture was observed in the two 5-ft long test specimens, while only concrete cracking of the shaft was observed in a 3-ft long specimen. Due to the rupture of several concrete shafts, the design criteria were re-evaluated and revised.

In the Third Quarter, the design criteria was modified to incorporate only the peak loading that could be imparted to the foundations from vehicles striking the S3x5.7 posts. Previously, a higher design loading was utilized based on the strongest post found in highway cable barrier systems. Subsequently, four new post-socket designs were configured. CAD details will be completed either in late Third Quarter or early Fourth Quarter. Construction and dynamic component testing of new post-socketed foundation systems will occur in the Fourth Quarter.

Testing of End Terminal for Four-Cable, High-Tension Barrier (1100C & 2270P) – Program Years 17 and 20

Previously, this project was delayed in order to complete the crash testing of the high-tension, four- cable barrier system placed in the V-ditch. However, work has begun to be ready for compliance testing in late 2010 or early 2011. The research objective includes the adaptation of a prior low-tension, cable barrier end terminal for use with high-tension cable barrier systems. The end terminal system incorporates a cable release lever technology at each end anchor foundation as well as steel breakaway support posts in the terminal region.

In the Second and Third Quarters, MwRSF reviewed and examined prior crash testing programs of cable barrier end terminals, reviewed existing terminal post configurations, and evaluated the potential for modifying the terminal posts and/or eliminating the breakaway slipbases. From this review, it is MwRSF’s opinion that: breakaway posts are beneficial for improving vehicle stability within the terminal region; releasable versus non-releasable cable ends reduce concerns for a centerline end-on impact resulting in a vehicle vaulting into the air with the undercarriage landing onto top of the steel terminal and line posts; the entire terminal geometry should be examined when selecting the critical lateral impact point of the terminal system and conducting the ¼-point offset, end-on small car test; and the cable barrier and end terminal systems should have sufficient length to adequately evaluate the potential for vehicular instabilities during end-on crash tests.

LS-DYNA computer simulations will be performed in the Fourth Quarter to predict and validate the small car behavior and dynamic barrier performance observed in test no. CT-4 (test designation no. 3-30) on the low-tension, three-cable end terminal system.

Wood Post MGS
Program Year 21 [TPF-5(193) – Supp. #31]

This research project provides funding for the crash testing and evaluation of the Midwest Guardrail System (MGS) installed with 6-in. by 8-in. Southern Yellow Pine (SYP) timber posts embedded in level terrain. Two full-scale vehicle crash tests are planned under this project using the Test Level 3 (TL-3) MASH safety performance guidelines – one with an 1100C small car and another with a 2270P pickup truck. CAD details have been started in the Third Quarter. Construction materials should be acquired in the Fourth Quarter. Construction and/or crash testing may be initiated in the Fourth Quarter

MGS Without Blockouts
Program Year 21 [TPF-5(193) – Supp. #33]

This research project provides funding for the crash testing and evaluation of the non-blocked, Midwest Guardrail System (MGS) installed W6x9 or W6x8.5 steel posts embedded in level terrain. Two full-scale vehicle crash tests are planned under this project using the Test Level 3 (TL-3) MASH safety performance guidelines – one with an 1100C small car and another with a 2270P pickup truck. CAD details have been started in the Third Quarter. Construction materials should be acquired early in the Fourth Quarter. Construction and/or crash testing may be initiated in the Fourth Quarter.

Paper Studies

Cost-Effective Measures for Roadside Design on Low-Volume Roads – Program Year 16

The analysis, evaluation, and documentation of treatment options for culverts, trees, bridges, and slopes/ditches found along low-volume roadways has been completed. A draft report has been prepared and is awaiting internal review. The draft and final reports will be completed in the Fourth Quarter.

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

To date, 15 components and 21 systems have been submitted to TF-13 for review and approval, and all have been approved for the Guide over the last 2 years. A small portion of supplemental funding was allocated in the Year 21 Pooled Fund Program.

Cost-Effective Upgrading of Existing Guardrail Systems – Program Year 17

In June 2009, an MwRSF field investigation team conducted a field survey of selected barrier installations throughout the State of Kansas. As part of this one week investigation, more than 60 specific sites were visited, measured, photographed, and documented. A review and compilation of the field survey information was completed in the Fourth Quarter of 2009. An analysis of the field data was initiated in the Fourth Quarter of 2009. Due to a shifting of staff priorities, work was greatly slowed in early 2010. However, analysis of field data will be completed in the Third Quarter of 2010. In the First and Second Quarters, a sensitivity study using RSAP was initiated to decrease the size of the analysis matrix. This analysis is also planned for completion in the Third Quarter. In addition, a containment level analysis to determine the appropriate severity indices was initiated and planned for completion during the Third Quarter or early Fourth Quarter.

Safety Performance Evaluation of Vertical and Safety Shaped Concrete Barriers – Program Year 16

In the Second Quarter of 2010, the research team requested assistance with the identification of bridge railing type for specific bridge accident sites in order to increase the number of accident records to be used in the analysis. These accident sites were all located at county roads. In the same quarter, the research team waited for the counties to gather the information on bridge railing type. In August 2010, the counties started sending pictures from the bridge railing sites. By the end of August 2010, only one third of the counties sent the necessary information, and only 20 percent of those bridge railings were concrete barriers. In late August, the research team decided to proceed with the project analysis using only those bridges located on State maintained highways. At this time, the research team is analyzing the accident data.

MGS Implementation – Program Year 18

In 2007, consulting funds were used to assist states with the MGS implementation effort. MwRSF began the effort with a review of CAD details from the Illinois and Washington DOTs. Project correspondence occurred via email with a pre-determined Technical Working group. To date, three subject areas were covered and are as follows: (1) Standard, Half, and Quarter Post Spacing; (2) MGS with Curbs and MGS on 2:1 Slopes; and (3) MGS with Culvert Applications. A fourth category, MGS Stiffness Transition, was delayed in order to await the completion of a simplified, steel-post and wood-post approach guardrail transition.

The draft reporting of the simplified, steel-post, approach guardrail transition system attached to the MGS was completed in the Third Quarter of 2010. The report was submitted to the State DOTs on August 9, 2010 with comments due on August 30, 2010. The final report will be completed in late September. The wood post R&D effort is awaiting two additional dynamic post tests with the bogie vehicle. Once those two tests are completed, the draft reporting of the wood post testing program is expected early in the Fourth Quarter.

The MGS implementation effort will likely commence in the Fourth Quarter of 2010 after both the simplified, steel- and wood-post transition reports are finalized.

LS-DYNA Modeling Enhancement Funding – Program Year 18

No work was performed on this project during the reporting period.

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

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

For this project, a TL-4, aesthetic, open concrete bridge railing was developed for use on cast-in-place decks as well as precast deck panels. Due to many factors, existing project funds were insufficient to complete the construction and crash testing phases of this research study. MwRSF-UNL researchers have sought funds from alternative sources including the NCHRP IDEA program and the 2009 Midwest States Pooled Fund Program. In 2010, MwRSF will seek funding from the FHWA Highways for Life Program. The draft report of the initial design work was initiated.

Awaiting Reporting

Midwest Guardrail System Placed at the Breakpoint of a 2:1 Slope – Bogie Testing Project Using Year 14 Contingency Funds

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. A draft report was sent to the States in the Fourth Quarter of 2009. A final report was completed in the First Quarter of 2010.

Previously, several member states noted a desire for a wood-post alternative for the MGS placed on a 2:1 slope. As such, a dynamic bogie testing program was conducted in order to determine the appropriate length of a 6-in. x 8-in. wood post for placement at the slope breakpoint of a 2:1 fill slope. A second draft report was initiated in the Fourth Quarter of 2009 which contains the results from the wood-post, component testing program as well as some additional steel post tests for comparison purposes. Work was continued on the draft report for the bogie testing program in the First and Second Quarters of 2010. A draft report has been prepared and is awaiting internal review.

Draft Reports - Pooled Fund

Thiele, J.C., Reid, J.D., Lechtenberg, K.A., Faller, R.K., Sicking, D.L., and Bielenberg, R.W., ***Performance Limits for 6-In. (152-mm) High Curbs Placed in Advance of the MGS Using MASH Vehicles – Part III: Full-Scale Crash Testing (TL-2)***, Draft Report to the Midwest State's Regional Pooled Fund Program, Transportation Research Report No. TRP-03-237-10, Project No.: SPR-3(017), Project Codes: RPFP-07-03 - Years 17, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, September 17, 2010.

Rosenbaugh, S.K., Lechtenberg, K.A., Faller, R.K., Sicking, D.L., Bielenberg, R.W., and Reid, J.D., ***Development of the MGS Approach Guardrail Transition Using Standardized Steel Posts***, Draft Report to the Midwest State's Regional Pooled Fund Program, Transportation Research Report No. TRP-03-210-10, Project No.: SPR-3(017), Project Codes: RPFP-08-05 and RPFP-09-03 - Years 18 and 19, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, July 26, 2010.

Final Reports - Pooled Fund

Stolle, C.S., Reid, J.D., and Lechtenberg, K.A., ***Update to Cable Barrier Literature Review***, Final Report to the Midwest State's Regional Pooled Fund Program, Transportation Research Report No. TRP-03-227-10, Project No.: TPF-5(193), Project Code: RPFP-09-01 - Year 19, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, August 12, 2010.

Thiele, J.C., Sicking, D.L., Faller, R.K., Bielenberg, R.W., Lechtenberg, K.A., Reid, J.D., and Rosenbaugh, S.K., ***Development of a Low-Cost, Energy-Absorbing Bridge Rail***, Final Report to the Midwest State's Regional Pooled Fund Program, Transportation Research Report No. TRP-03-226-10, Project No.: SPR-3(017) and TPF-5(193), Project Codes: RPFP-08-09 - Year 18 and RPFP-09-06 - Year 19, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, August 11, 2010.

Wiebelhaus, M.J., Terpsma, R.J., Lechtenberg, K.A., Reid, J.D., Faller, R.K., Bielenberg, R.W., Rohde, J.R., and Sicking, D.L., ***Development of a Temporary Concrete Barrier to Permanent Concrete Median Barrier Approach Transition***, Final Report to the Midwest State's Regional Pooled Fund Program, Transportation Research Report No. TRP-03-208-10, Project No.: SPR-3(017), Project Codes: RPFP-06-07 and RPFP-06-09 - Year 16, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, July 15, 2010.

Draft Reports - Individual State DOT and Routed Through NDOR/Pooled Fund

Not Applicable.

Final Reports - Individual State DOT and Routed Through NDOR/Pooled Fund

Schmidt, J.D., Sicking, D.L., Faller, R.K., Reid, J.D., Bielenberg, R.W., and Lechtenberg, K.A., ***Investigating the Use of a New Universal Breakaway Steel Post - Phase 2***, Final Report to the Minnesota Department of Transportation, Transportation Research Report No. TRP-03-230-10, Project No.: SPR-3(017), Supplement #39, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, August 9, 2010.

<u>Pooled Fund Consulting Summary</u>

Midwest Roadside Safety Facility
July 2010 – September 2010

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 – Cable Guardrail Questions

State Question:

Ron,

The new in-line cable end treatment requires post 3 through 7 to be spaced @ 16'. What is the offset to a fixed object in this area?

When we design a long run of guardrail in the past we have used an intermediate anchorage section. Is this still necessary?

If so, is there a design for the new in-line intermediate anchorage section?

The spacing in front of a 1.5:1 slope requires 4' post spacing. Is it acceptable to have 16' post spacing then 4' spacing?

Or, is there a suggested length of transition of 8' post spacing?
**See comments noted above.

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

Phil TenHulzen PE
Design Standards Engineer
Nebraska Dept. of Roads

MwRSF Response:

Phil:

I sent a packet to you through the mail today regarding the review of your current cable end terminal details. You requested that effort some time ago.

More recently, you requested additional comment on the low-tension, three-cable barrier system. Those comments are provided below in Red.

Ron

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

The new in-line cable end treatment requires post 3 through 7 to be spaced @ 16'. What is the offset to a fixed object in this area?

**A 2000P pickup truck was crash tested at the length-of-need of the end terminal at the TL-3 conditions of NCHRP Report No. 350. The vehicle impacted post no. 3 which was 15 ft downstream from the upstream steel anchor post. For this crash test, the working width was reported to be approximately 84 in. when using a 254-ft long installation.

**Please note that the target impact angle for this test was 20 degrees, as required by NCHRP Report No. 350. The new MASH guidelines now utilize an impact angle of 25 degrees. With higher impact angles, one would expect higher angle loading and slight increases in anchor movement, thus resulting in greater barrier deflection and working width near the system ends.

When we design a long run of guardrail in the past we have used an intermediate anchorage section. Is this still necessary?

**As noted above, the test installation was 254 ft long. For longer test installations than denoted above, dynamic barrier deflections and working widths would be expected to increase.

**A prior Pooled Fund R&D program resulted in the successful development, testing, and evaluation of three alternative anchor systems in lieu of the large cast-in-place reinforced concrete anchor blocks. However, the R&D program did not evaluate changes in anchor spacing. As such, we would recommend that NDOR continues to utilize an anchor spacing equal to or smaller than that currently specified, especially since barrier deflections and working widths could be greater with the use of the alternative anchor options.

If so, is there a design for the new in-line intermediate anchorage section?

**The alternative anchor options were developed for terminating and anchoring the ends of the three cables. I am unclear as to the difference between end anchor hardware and the anchor hardware used at intermediate anchor sections. Please forward those details to us for review as I am unaware of prior crash tests performed to evaluate the safety performance of the overlapped cables with two intermediate anchor sections crossed in opposite directions.

The spacing in front of a 1.5:1 slope requires 4' post spacing. Is it acceptable to have 16' post spacing then 4' spacing?

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

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

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

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

Or, is there a suggested length of transition of 8' post spacing?
**See comments noted above.

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

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

John/Bob:
Please review Phil's request in order to determine the level of effort that would be required to answer his specific question. If you have further questions for him, please email/call Phil to acquire clarifications and additional details. Thanks!

Problem # 2 – Structural Analysis of Approach Transitions

State Question:

Dear MwRSF,

I have a major project team that is challenging my requirement that they provide structural analysis for <u>their</u> transitions. They indicate the following:

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

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

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

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

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

MwRSF Response:

Erik:

Please see my comments below!

Ron

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

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

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

**The AASHTO LRFD Bridge Design Specifications provides guidance for designing bridge railings for use on bridge decks as well as those attached to bridge approach slabs. This guidance is intended to help engineers properly configure bridge railings as well as their attachment to reinforced concrete decks. Both solid and open concrete parapets can be configured as well as metallic beam and post systems. Combination concrete and metal systems are also addressed. Limited discussion is provided for timber railings. Yield-line analysis procedures have been provided for addressing the design of reinforced concrete parapets and railings. Inelastic design procedures are available for most metal systems. These rail design procedures were developed and/or documented in a 1978 study report by TTI researchers and have been consistently used for a large share of railing systems. Upon design, it has been common practice for the design to be verified through the use of full-scale crash testing. Actually, full-scale crash testing has also been used for demonstrating the system's structural adequacy and safety even when the prior noted design procedures were not used. If crash testing has been shown to corroborate a design based on the noted procedures, then these procedures have also been used to modify other parapets as long that they provided equivalent or greater strength and did not pose increased risk for vehicle snag, rollover, or override.

For reinforced concrete parapets, the noted design procedures have also been to ensure that sufficient strength is provided at critical locations within the barrier, such as at barrier ends and at expansion joints. At such locations, the number of yield lines that can be developed is much reduced, thus potentially lower the redirective strength of the parapet. Therefore, it is imperative that these equations be utilized to modify a barrier's capacity to ensure that an impacting vehicle can be safely contained and redirected along the entire barrier length. Basically, the entire barrier must act as though it is continuous even though weakened sections may exist therein. End buttresses that are used to anchored approach guardrail section must also provide adequate structural strength so as to not allow for vehicles to penetrate directly behind the bridge railing if the entire length plus AGT must shield the hazard.

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

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

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

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

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

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

Problem # 3 – Structural Analysis of Approach Transitions - II

State Question:

Dr. Faller

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

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

This design methodology provides that the barrier itself has:

- Sufficient reinforcement so that the force of vehicle impact can be withstood by the barrier or transition (i.e. the barrier does not shatter and allow the vehicle to pass through the barrier)
- Sufficient reinforcement and footing to prevent the barrier from shifting during (e.g. provide a snag point or pocket) or pivoting during an impact (e.g. if the barrier tips over during an impact or provides a ramp to launch a vehicle in to the air has it done its' job?).

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

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

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

Thanks for your help

Erik

MwRSF Response:

Erik:

See my comments below!

Ron

Dr. Faller

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

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

This design methodology provides that the barrier itself has:

- Sufficient reinforcement so that the force of vehicle impact can be withstood by the barrier or transition (i.e. the barrier does not shatter and allow the vehicle to pass through the barrier)
- Sufficient reinforcement and footing to prevent the barrier from shifting during (e.g. provide a snag point or pocket) or pivoting during an impact (e.g. if the barrier tips over during an impact or provides a ramp to launch a vehicle in to the air has it done its' job?). (Do not allow vehicle override or rollover for passenger vehicles.)

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

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

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

Thanks for your help

Erik

Problem # 4 – Bridge Rail Post Question

State Question:

Ron,

Iowa DOT is currently working with Illinois DOT in detailing a bridge rail for the new I-74 bridge. The bridge will accommodate a multi-use trail behind the rail on one side of the bridge. For this application, we are considering using the Pennsylvania HT rail with a supplemental “sidewalk rail tube” attachment, as detailed in Pennsylvania’s standard drawings (attached).

It is our understanding that the modified post used to support this rail has not been crash tested. Therefore, we request your assistance in assessing the effect, if any, the modified post and sidewalk rail tube would have on the crashworthiness of this rail configuration, and whether the presence of the sidewalk rail tube in a TL-5 impact could produce flying debris.

Note that we are interested solely in the use of the design shown in the “raised sidewalk section A-A” on sheet 1 of 3 of BD-615M. Also note that we will not be using the “sidewalk rail rod” or any of its associated hardware.

Please let me know if you have any questions about this request.

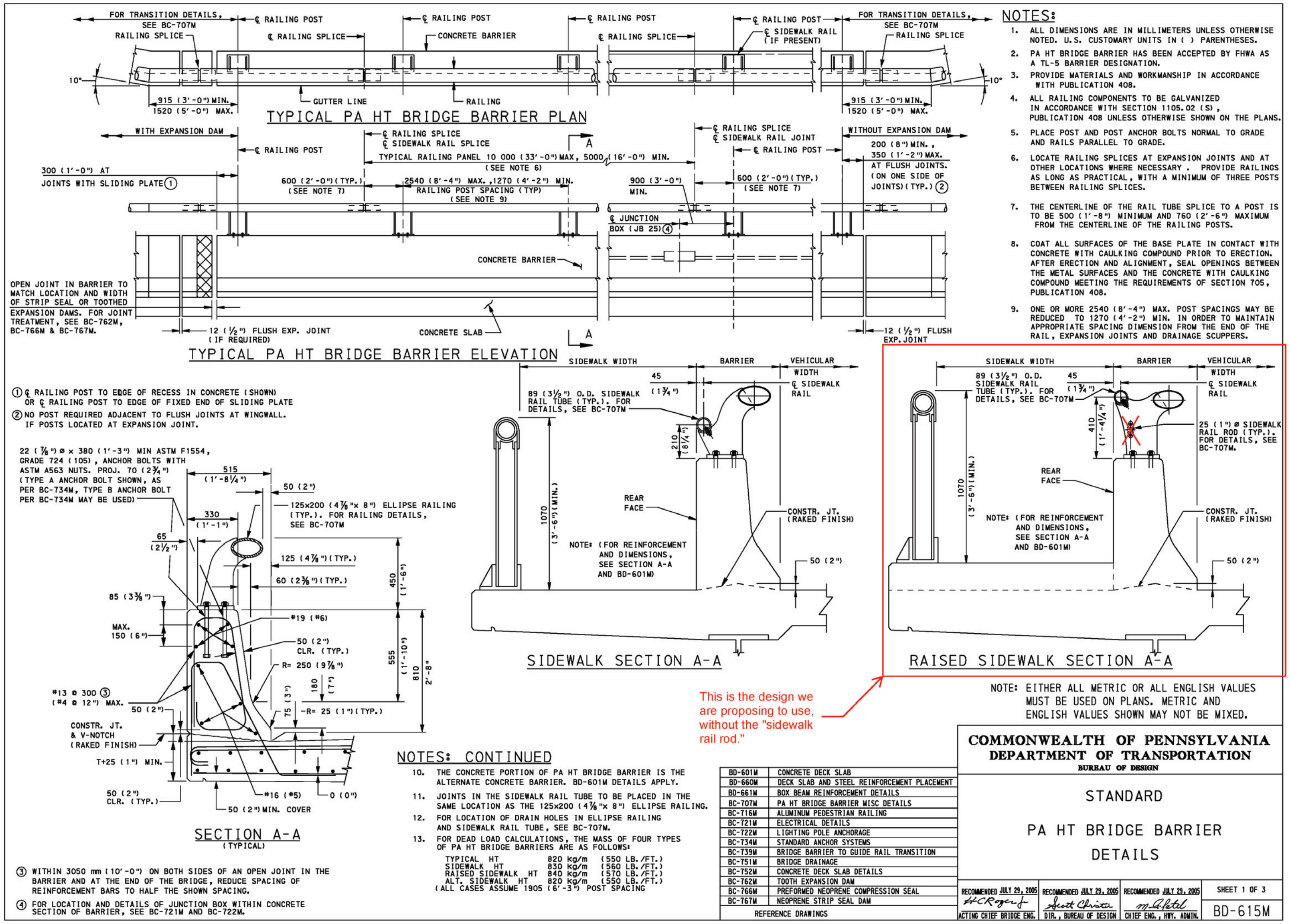
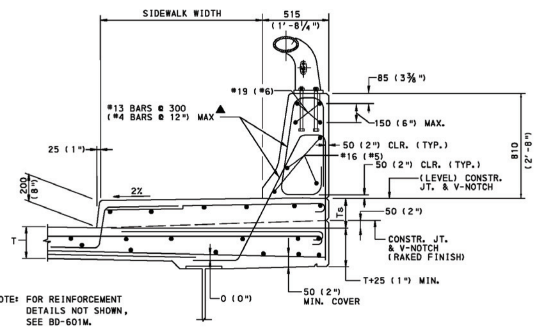


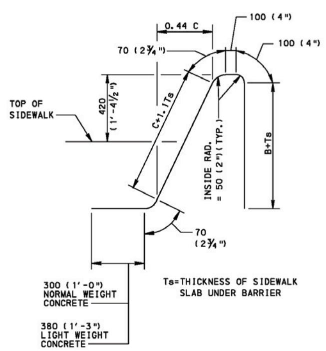
Figure 1. BD-615M



NOTE: FOR REINFORCEMENT DETAILS NOT SHOWN, SEE BD-601M.

ALTERNATE SIDEWALK DETAIL

▲ WITHIN 3050 mm (10'-0") ON BOTH SIDES OF AN EXPANSION JOINT IN THE BARRIER AND AT THE END OF THE BRIDGE, REDUCE SPACING OF REINFORCEMENT BARS TO HALF THE SHOWN SPACING.



PA HT BRIDGE BARRIER
(FOR DIMENSIONS "B" AND "C", SEE TABLE 1)

TABLE 1
METRIC UNITS
FOR PA HT BRIDGE BARRIER AT ALTERNATE SIDEWALK
B AND C DIMENSIONS

T	B	C
200	530	580
210	540	590
220	550	600
230	560	610
240	570	620
250	580	630
260	590	640
270	600	650
280	610	660
290	620	670

TABLE 1
U.S. CUSTOMARY UNITS
FOR PA HT BRIDGE BARRIER AT ALTERNATE SIDEWALK
B AND C DIMENSIONS

T	B	C
8.0"	1'-9"	1'-11"
8.5"	1'-9½"	1'-11½"
9.0"	1'-10"	2'-0"
9.5"	1'-10½"	2'-0½"
10.0"	1'-11"	2'-1"
10.5"	1'-11½"	2'-1½"
11.0"	2'-0"	2'-2"
11.5"	2'-0½"	2'-2½"

REINFORCEMENT DETAILS

REINFORCEMENT BAR NOTES

1. REINFORCEMENT BAR DIMENSIONS ARE OUT TO OUT OF BAR.
2. DIMENSIONS ALONG CURVED PORTIONS OF BAR ARE MEASURED ALONG THE OUTSIDE EDGE.

NOTES:

1. FOR NOTES, SEE SHEET 1.

NOTE: EITHER ALL METRIC OR ALL ENGLISH VALUES MUST BE USED ON PLANS. METRIC AND ENGLISH VALUES SHOWN MAY NOT BE MIXED.

BD-601M	CONCRETE DECK SLAB
BC-707M	PA HT BRIDGE BARRIER MISC DETAILS
REFERENCE DRAWINGS	

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
BUREAU OF DESIGN

STANDARD

PA HT BRIDGE BARRIER
ALTERNATE SIDEWALK DETAILS

RECOMMENDED JULY 29, 2005	RECOMMENDED JULY 29, 2005	RECOMMENDED JULY 29, 2005	SHEET 2 OF 3
H.C. Rogers Jr.	Scott Christie	M. Patel	BD-615M
ACTING CHIEF BRIDGE ENG.	DIR., BUREAU OF DESIGN	CHIEF ENG., HWY. ADMIN.	

Figure 2. BD-615M

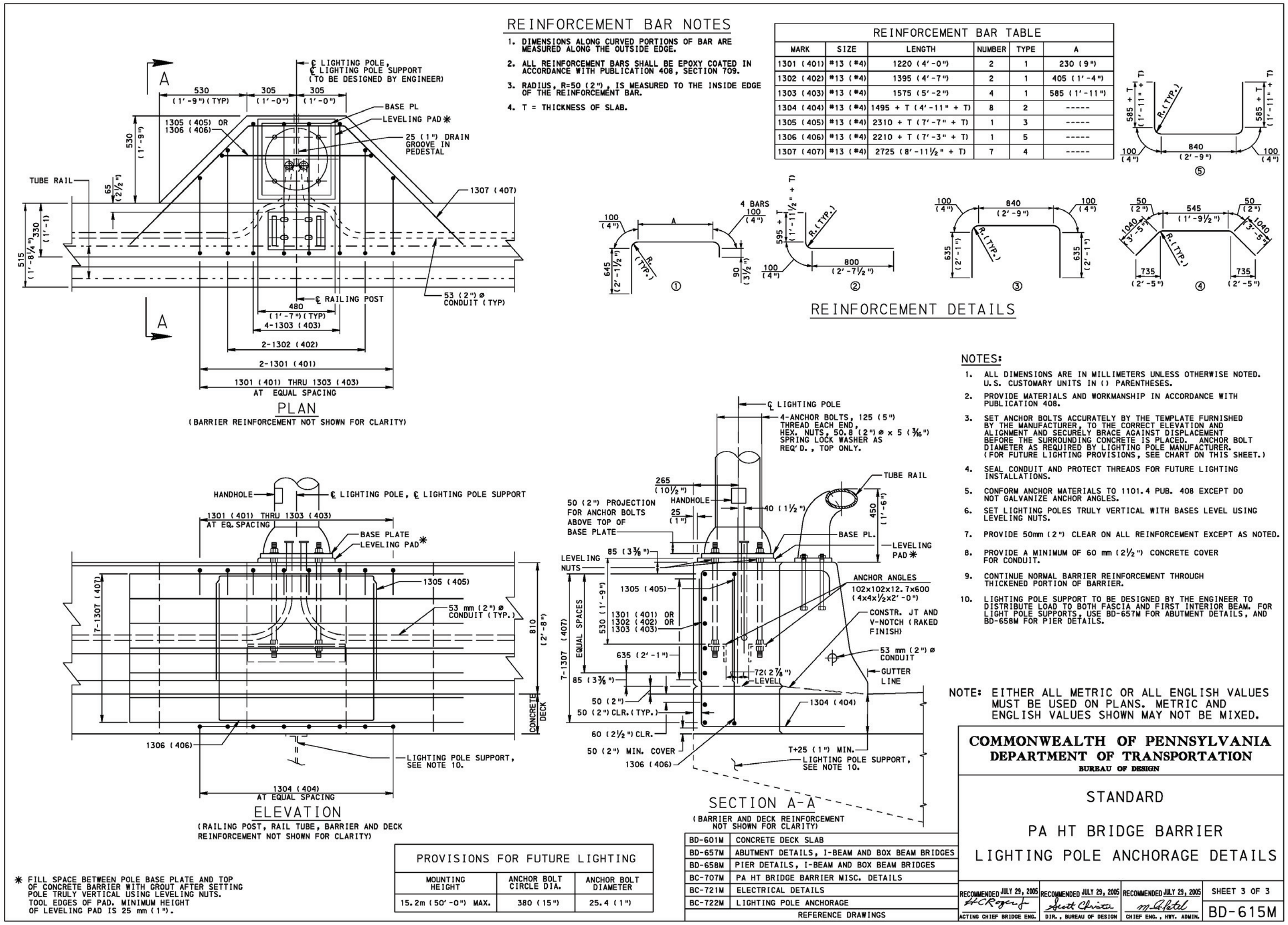


Figure 3. BD-615M

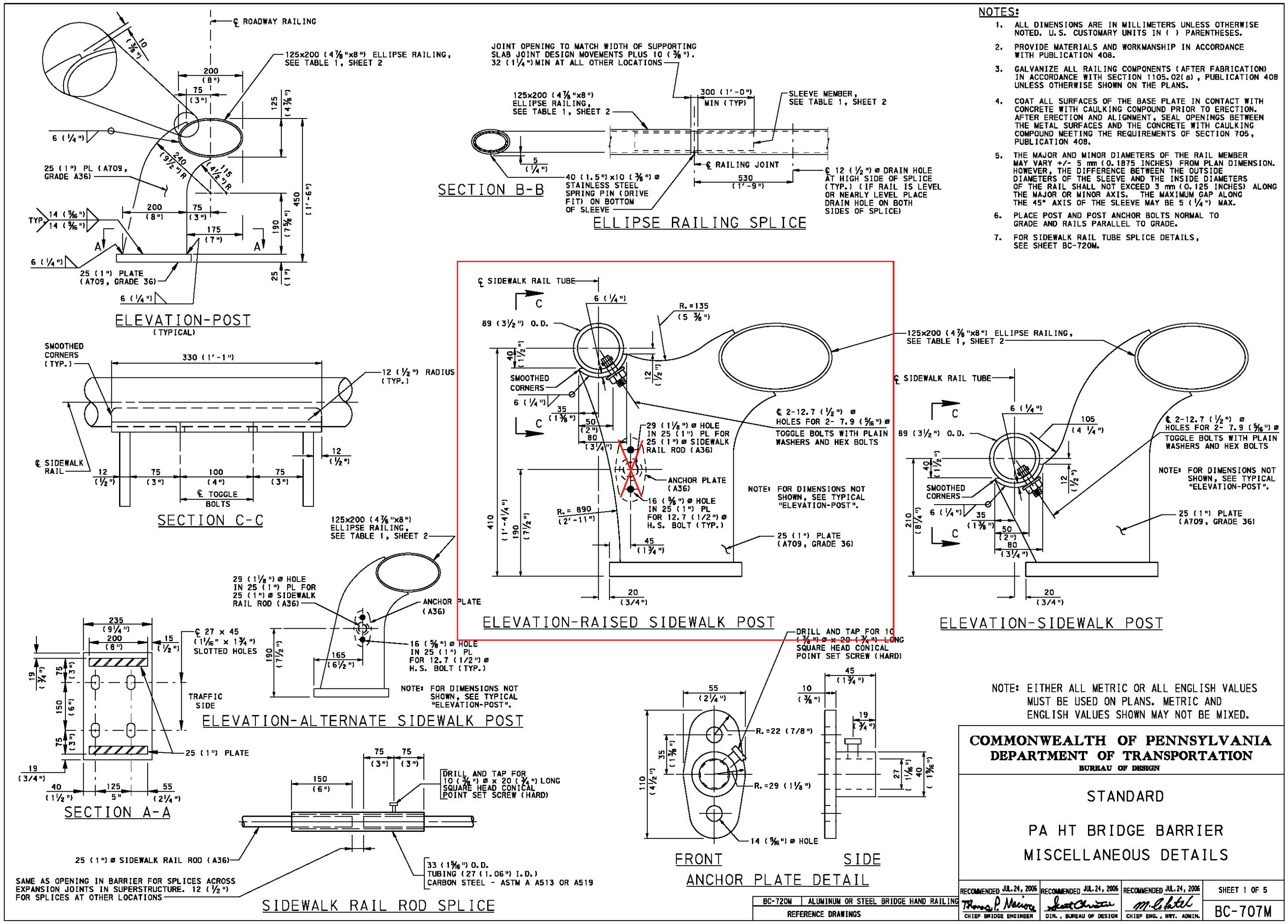


Figure 4. BD-707M

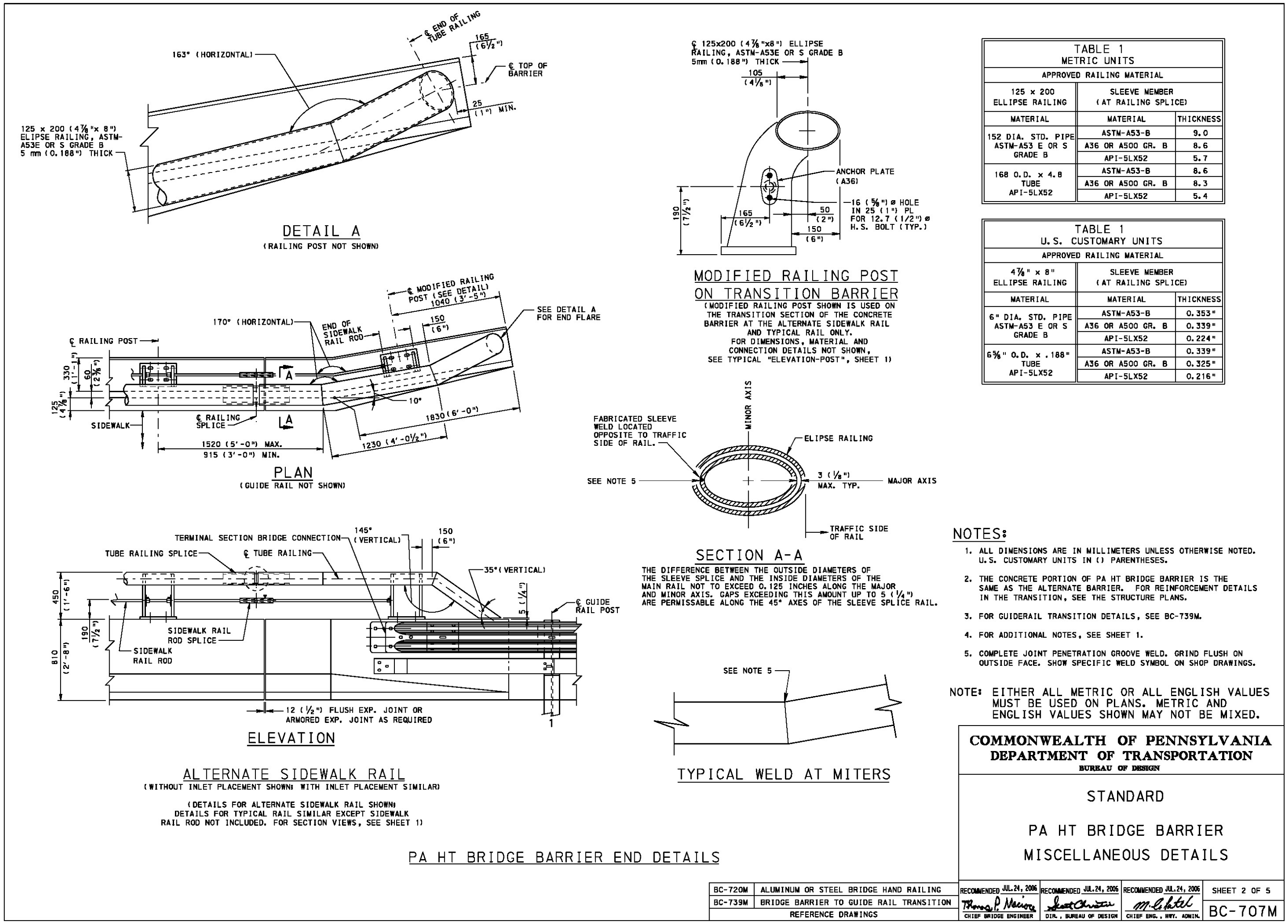


TABLE 1
METRIC UNITS
APPROVED RAILING MATERIAL

125 x 200 ELLIPSE RAILING	SLEEVE MEMBER (AT RAILING SPLICE)	
MATERIAL	MATERIAL	THICKNESS
152 DIA. STD. PIPE ASTM-A53 E OR S GRADE B	ASTM-A53-B	9.0
	A36 OR A500 GR. B	8.6
	API-5LX52	5.7
168 O.D. x 4.8 TUBE API-5LX52	ASTM-A53-B	8.6
	A36 OR A500 GR. B	8.3
	API-5LX52	5.4

TABLE 1
U.S. CUSTOMARY UNITS
APPROVED RAILING MATERIAL

4⅞" x 8" ELLIPSE RAILING	SLEEVE MEMBER (AT RAILING SPLICE)	
MATERIAL	MATERIAL	THICKNESS
6" DIA. STD. PIPE ASTM-A53 E OR S GRADE B	ASTM-A53-B	0.353"
	A36 OR A500 GR. B	0.339"
	API-5LX52	0.224"
6⅝" O.D. x .188" TUBE API-5LX52	ASTM-A53-B	0.339"
	A36 OR A500 GR. B	0.325"
	API-5LX52	0.216"

NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE NOTED. U.S. CUSTOMARY UNITS IN ( ) PARENTHESES.
2. THE CONCRETE PORTION OF PA HT BRIDGE BARRIER IS THE SAME AS THE ALTERNATE BARRIER. FOR REINFORCEMENT DETAILS IN THE TRANSITION, SEE THE STRUCTURE PLANS.
3. FOR GUIDERAIL TRANSITION DETAILS, SEE BC-739M.
4. FOR ADDITIONAL NOTES, SEE SHEET 1.
5. COMPLETE JOINT PENETRATION GROOVE WELD. GRIND FLUSH ON OUTSIDE FACE. SHOW SPECIFIC WELD SYMBOL ON SHOP DRAWINGS.

NOTE: EITHER ALL METRIC OR ALL ENGLISH VALUES MUST BE USED ON PLANS. METRIC AND ENGLISH VALUES SHOWN MAY NOT BE MIXED.

Figure 5. BD-707M

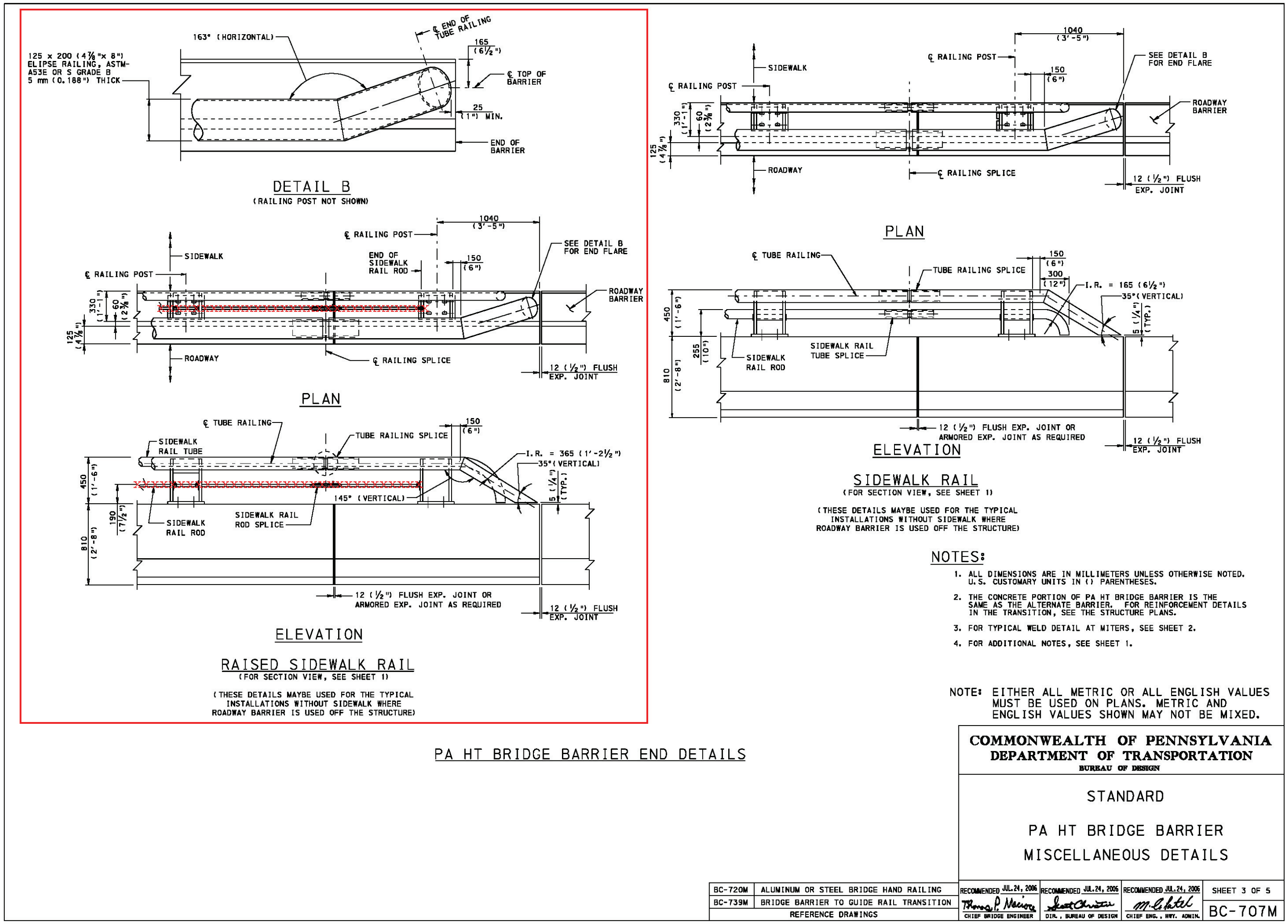


Figure 6. BD-707M

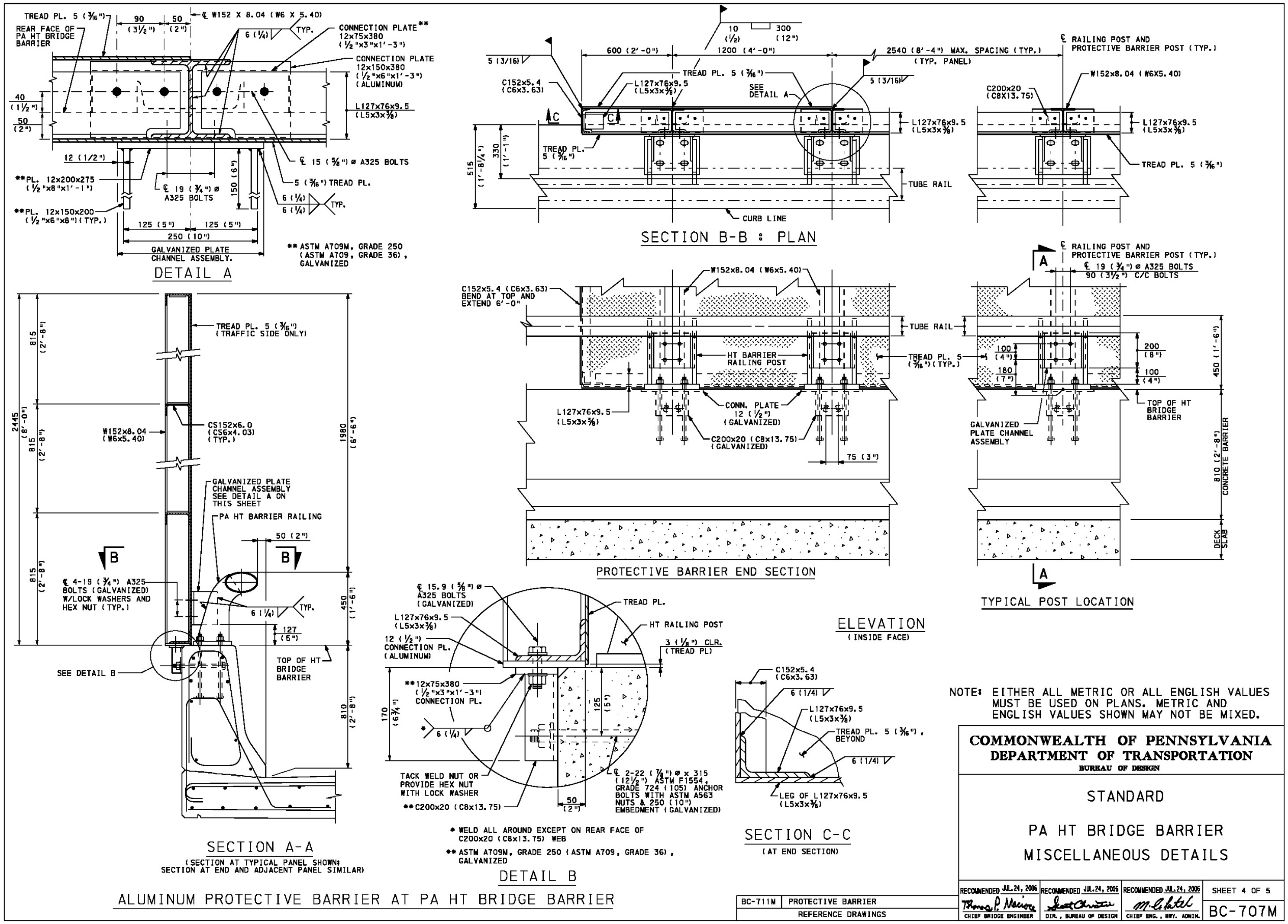


Figure 7. BD-707M

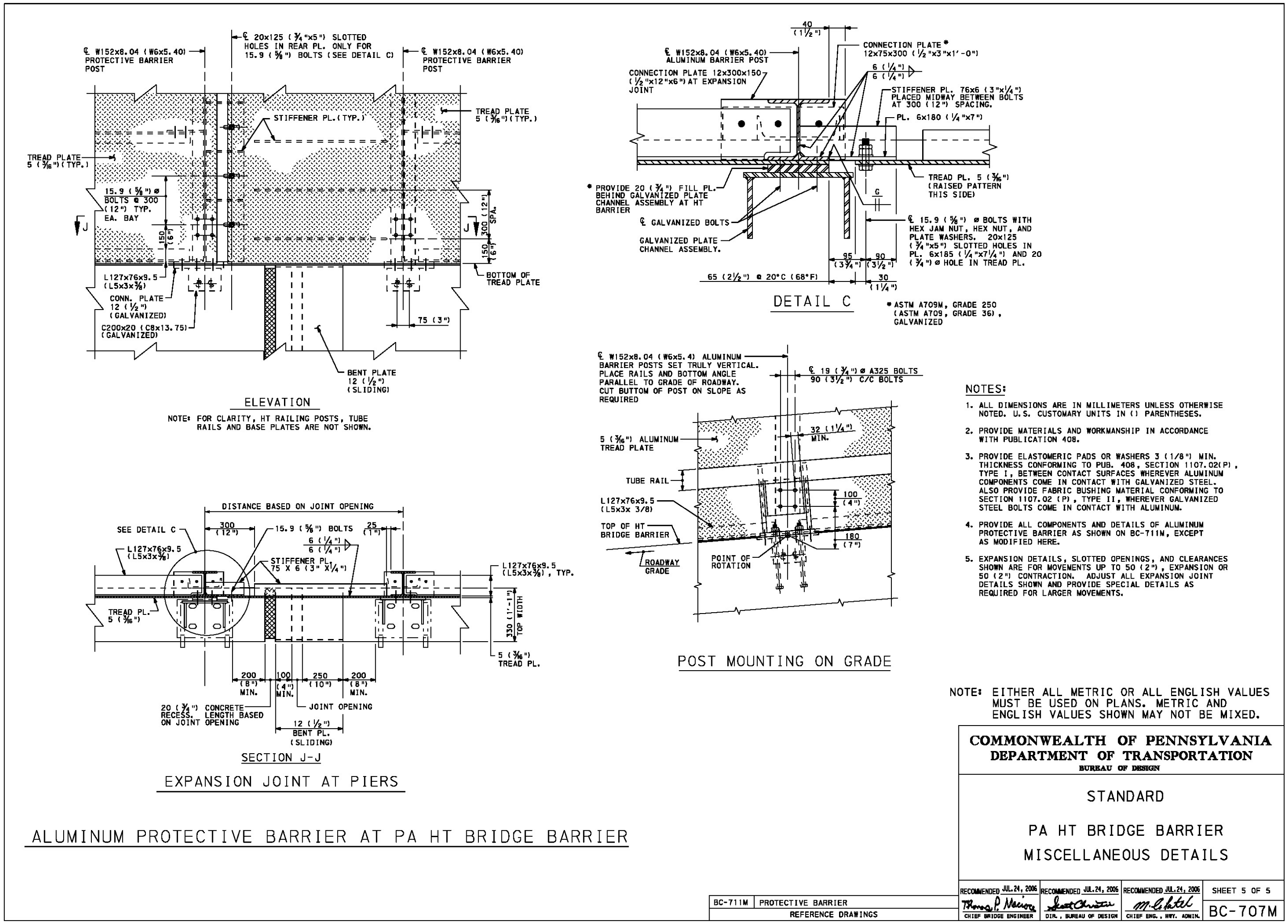


Figure 8. BD-707M

MwRSF Response:

Chris,

The TL-5, full-scale crash test of this barrier showed no evidence that the trailer would extend over the steel rail and contact either the back of the posts or the pedestrian rail. This is due in part to the rail height being 50 inches instead of the standard 42 inches for concrete barriers. Thus, the box leans on the rail, but the lower corner of the box never extends behind the rail.

Also, during the crash test, there was very little deformation of the rail. Therefore, I do not foresee any problems with the pedestrian rail being contacted and turned into flying debris.

The proposed system appears to maintain the crashworthiness of the original (tested) system.

Scott Rosenbaugh
Midwest Roadside Safety Facility (MwRSF)
University of Nebraska – Lincoln

Problem # 5 – ZOI, Thrie Beam, and MGS 2:1 Slope

State Question:

Hi Ron,

I've got a couple of questions for you:
1. Could you send me a picture or description of the current guidance for ZOI for a 42" F-shape concrete barrier at TL-3, TL-4, and TL-5 conditions (if available)?
2. Are you aware of any minimum and/or maximum allowable height guidance for thrie-beam guardrail?
3. Has an equivalent wood post design been determined for use with the MGS at the breakpoint of a 2:1 slope?

Thanks,

-Chris

MwRSF Response:

Chris:

Thanks for the inquiries! My incomplete comments are contained below. I will check out a few sources and get back to you.

Ron

Ronald K. Faller, Ph.D., P.E.

Research Assistant Professor

Midwest Roadside Safety Facility (MwRSF)

---

1. Could you send me a picture or description of the current guidance for ZOI for a 42” F-shape concrete barrier at TL-3, TL-4, and TL-5 conditions (if available)?
** I will do some digging on this one. I may need to review the past 26 consulting summaries to find the answer.

2. Are you aware of any minimum and/or maximum allowable height guidance for thrie-beam guardrail?
** I am not aware of the performance limits for this barrier in terms of maximum and minimum height. However, I will seek comment from my colleagues and review a prior TTI report to see if testing at variables heights was included.

3. Has an equivalent wood post design been determined for use with the MGS at the breakpoint of a 2:1 slope?
** Yes, the wood post equivalent is a 6-in. x 8-in. by 7.5-ft long SYP post. A draft research report has been initiated but remains incomplete at this time.

You may be a little surprised that the wood post equivalent to a 9-foot long W6x9 is a whole 1.5 feet shorter. However, the 9-ft and 8-ft W6x9 post lengths were fairly close when we selected the 9-ft length. I also believe post-soil forces are higher now than before – due to MASH installation procedures that were finalized in the latter years of the MASH document and performance specification for soil. Thus, the 7.5-ft wood post length is not that far off of the original two options. We re-conducted some steel post tests on a 2:1 slope so that we would have a comparison of the two material types with newer MASH installation procedures and soil conditions. Those comparisons are contained in the draft report.

Thanks,

-Chris

Problem # 6 – NU Rail

State Question:

Mr. Ron Faller,

NDOR has had a few impacts to our new (NU) rail that have resulted in the bottom of the slab falling off under the posts. We are currently considering the following changes to the post and slab reinforcing:

Move the horizontal leg of the back post bars down from just beneath the top layer of longitudinal slab steel to just above the bottom layer of longitudinal slab steel.

Move the horizontal leg of the front post bars up from just beneath the bottom layer of longitudinal slab steel to just above the bottom layer of longitudinal slab steel.

Flare the horizontal legs of both the front and back post bars from perpendicular to the post for the middle bars to 45° for the outside bars.

Extend the horizontal legs 1' for both the front and back post bars from 3' – 2" to 4' – 2".
Add additional longitudinal # 5 bars in the bottom of the overhang (from 12" spacing to 6").

Could you please review the attachment showing our existing design and these proposed changes? We would like your opinion on whether or not these changes would require new crash tests for acceptance and any other comments you would wish to share.

Thank you.

Scott Milliken, P.E.

Nebraska Department of Roads
Bridge Division

MwRSF Response:

Mr. Milliken,

We have reviewed your proposed changes to the deck reinforcement for the TL-4 version of the NDOR open concrete bridge rail. We have also reviewed photos from 3 different crash sites that were sent to MwRSF over the last 9 months or so. However, most of these photos show only the damage to the rail itself and do not illustrate the deck cracking that you are attempting to mitigate. Therefore, the following comments are based on general structural design and the understanding that cracking is occurring in the deck. They are not specific to damage at an individual crash site.

I am unclear as to the benefits of fanning the transverse steel. It may cover a greater area, but the lateral stress that each bar can take is reduced. Further, it may be a pain to layout/tie the steel in a consistent manner when all of the angles are changing. If you are wanting the transverse steel to cover a greater area and protect the deck at the post edge locations, I would recommend that you instead place an extra transverse (lateral) steel bar in the deck on both side of the post. These bars would be parallel to current legs and could extend 4'-2" into the deck to match the new proposed length of the legs.

The additional longitudinal steel (along the length of the bridge) near the bottom surface of the deck should provide additional resistance to bending and punching shear. These bars in combination with the additional transverse bars described above should help mitigate cracking.

Lowering the bag leg of the post reinforcement to bottom level of the deck may provide some additional steel near that surface to resist shear cracking, but it may also cause punching shear problems. Those back legs are carrying a compression load during impacts, and moving the bend to the bottom of the deck leaves the bottom of the deck susceptible to compression force punching through that surface – creating cracks and possible concrete spalling. Therefore, I would recommend leave these back legs in their current position.

I hope this answered your questions. Let me know if you have more (or I created more).

Scott Rosenbaugh
Midwest Roadside Safety Facility (MwRSF)
University of Nebraska – Lincoln

Problem # 7 – Type 2 Downstream End Terminal - Illinois DOT

State Question:

I'm not finding a record of sending this to you before, as I'd promised Bob Wagner in our District 2.

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

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

David L. Piper, P.E.
Safety Implementation Engineer
Bureau of Safety Engineering

MwRSF Response:

Dave:

I am enclosing a pdf file which compares your downstream anchor hardware to that currently used by MwRSF. Within the file, corrections are noted that show which dimensions are actually used within our CAD details. For our general guardrail testing programs, we utilize a standard end anchorage system on both the upstream and downstream ends of our 175-ft long W-beam guardrail installations when terminals are not being evaluated. These anchorages were adapted from the modified BCT, also consider the MGS rail height and cable anchor increased rise, and include a channel strut, two lengthened foundation tubes, and a common cable anchor with bolted attachment plate and bearing plate on each end.

In addition, you inquired as to whether the trailing end hardware noted above could be utilized in a double rail or median-type configuration where reverse-direction impacts could not be achieved on the end spoons. In such installations, we believe that this trailing end terminal hardware in combination with the MGS would likely provide sufficient capacity to successfully contain and redirect most passenger vehicles impacting at high speeds and angles. Unfortunately, no full-scale crash testing programs have yet been performed on most trailing end terminal systems.

Currently, there have been concerns with many different non-crash tested trailing end terminals that the small car vehicles could become snagged or wedged under the anchor cable on the downstream end if the end post is not fractured or does not release in a timely manner.

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

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

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

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

Respectfully,

Ron

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

Midwest Roadside Safety Facility (MwRSF)
University of Nebraska-Lincoln
130 Whittier Research Center

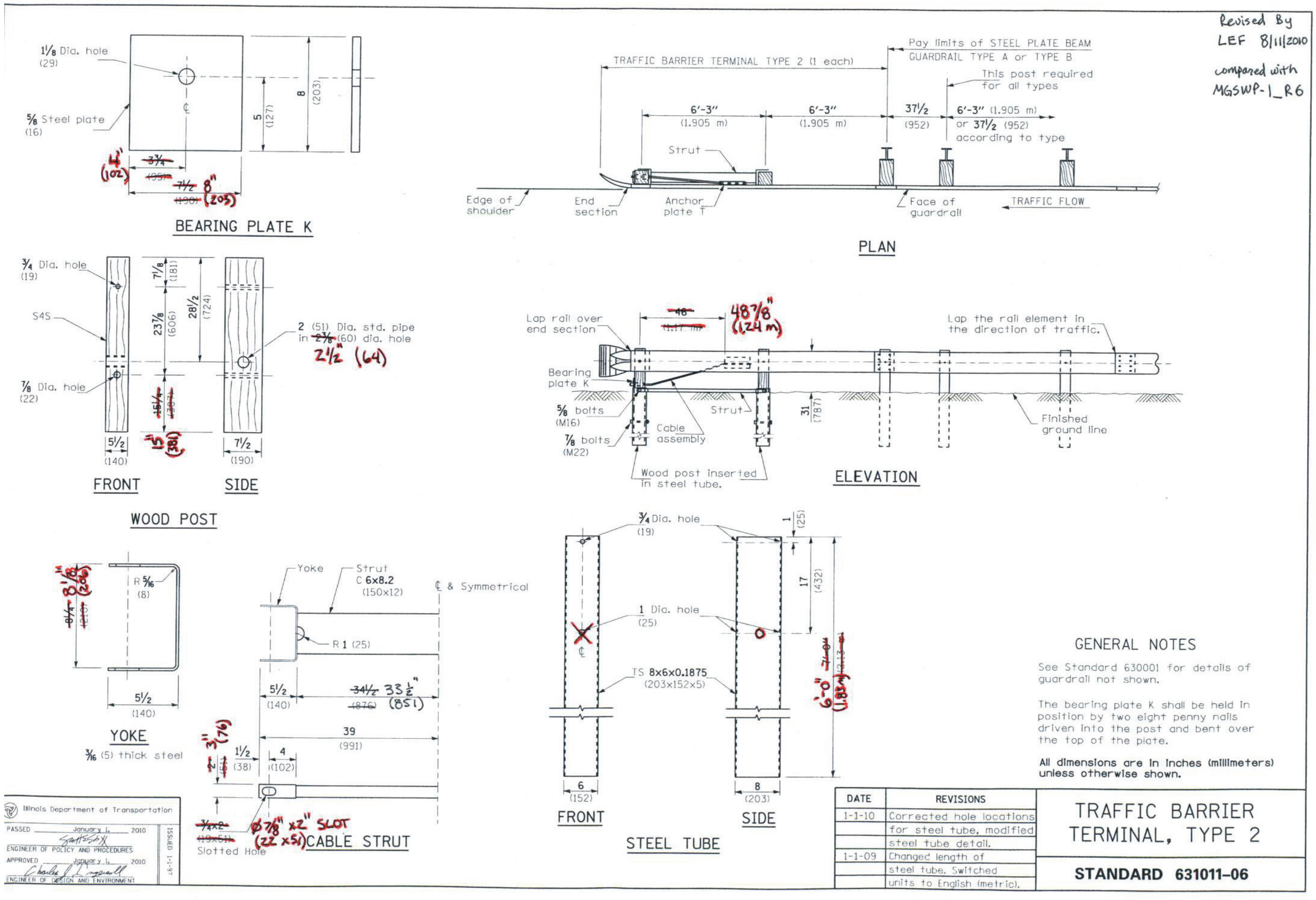

Revised By LEF 8/11/2010
compared with MGSWP-1_R6
1⅛ Dia. hole (29)
⅝ Steel plate (16)
BEARING PLATE K
¾ Dia. hole (19)
S4S
⅞ Dia. hole (22)
2 (51) Dia. std. pipe in 2⅜ (60) dia. hole
2½" (64)
FRONT
SIDE
WOOD POST
YOKE
3⁄16 (5) thick steel
Yoke
Strut C 6x8.2 (150x12)
℄ & Symmetrical
R 1 (25)
Slotted Hole
Ø⅞" x2" SLOT (22 x51)
CABLE STRUT
TRAFFIC BARRIER TERMINAL TYPE 2 (1 each)
Pay limits of STEEL PLATE BEAM GUARDRAIL TYPE A or TYPE B
This post required for all types
6'-3" (1.905 m)
37½ (952)
6'-3" (1.905 m) or 37½ (952) according to type
Strut
Edge of shoulder
End section
Anchor plate T
Face of guardrail
TRAFFIC FLOW
PLAN
Lap rail over end section
48⅞" (1.24 m)
Lap the rail element in the direction of traffic.
Bearing plate K
⅝ bolts (M16)
⅞ bolts (M22)
Cable assembly
Wood post inserted in steel tube.
Finished ground line
ELEVATION
¾ Dia. hole (19)
1 Dia. hole (25)
TS 8x6x0.1875 (203x152x5)
6'-0" (1.83 m)
FRONT
SIDE
STEEL TUBE
GENERAL NOTES
See Standard 630001 for details of guardrail not shown.
The bearing plate K shall be held in position by two eight penny nails driven into the post and bent over the top of the plate.
All dimensions are in inches (millimeters) unless otherwise shown.
Illinois Department of Transportation
PASSED January 1, 2010
ENGINEER OF POLICY AND PROCEDURES
APPROVED January 1, 2010
ENGINEER OF DESIGN AND ENVIRONMENT
ISSUED 1-1-97
DATE | REVISIONS
1-1-10 | Corrected hole locations for steel tube, modified steel tube detail.
1-1-09 | Changed length of steel tube. Switched units to English (metric).
TRAFFIC BARRIER TERMINAL, TYPE 2
STANDARD 631011-06

Figure 9.

Problem # 8 – Mixed Guardrail Post Types

State Question:

Greetings from Ohio,

I have a question about the deflection of our generic guardrail. Our current standard allows the use of four different post the 6x8 wood, 8" round wood, W6x9, and W6x8.5 with a rail height at 27.75". For new construction the contractor must use the same post for the entire guardrail run. When guardrail is being repaired after a crash contractor have asked if they can mix post types. Nick Artimovich said this is fine if the deflection distances are close. Where can I get a copy of the crash test reports for the 8" round wood post or the W6x9 steel post? Thanks for your time.

Post type	Deflection
6x8	2.7'
8" round	
W6x9	
W6x8.5	3'

Michael Bline
OhDOT

MwRSF Response:

NCHRP 350 Testing

27¾" rail height, 7.25" round SYP post – 3.7 ft D.D. – 2000P TL-3 test by TTI for Arnold Forest Products

27"?? rail height, 6x8 rectangular wood post – 2.7 ft D.D. – 2000P TL-3 test by TTI (G4(2W))

27¾" rail height, W6x8.5 steel post w/ 6x8 wood blockout – 3.3 ft D.D. – 2000P TL-3 test by TTI (Modified G4(1s))

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

Midwest Roadside Safety Facility (MwRSF)
University of Nebraska-Lincoln
130 Whittier Research Center

Problem # 9 – PCB Transition

State Question:

Greeting Bob,

Hope all is well and welcome to the Big Ten.

I would appreciate any comments you have on the barrier transition drawing below. There is a need for Ohio to improve the way we are transition from PCB to rigid structures. I have included some pictures from construction projects in Ohio. We know that additional testing is needed but until then we would like to do the best we can. We took the results of the K barrier report and created layouts of our Jersey shaped PCB to different rigid structures. The anchor holes are 1.125" in diameter so we changed the soil anchors to 1" in diameter.

Michael Bline
OhDOT

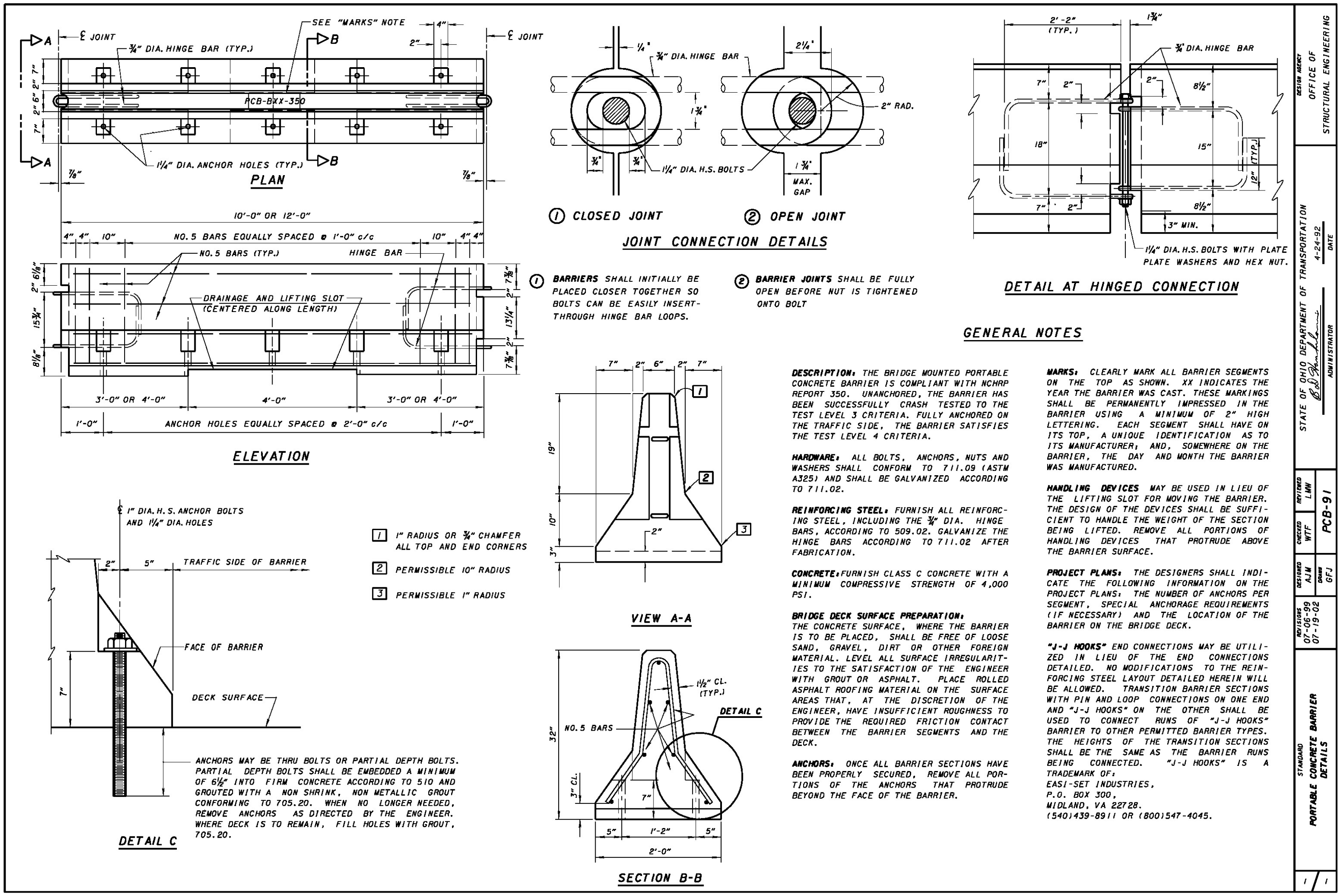


Figure 10. OhDOT PCB Transition Details

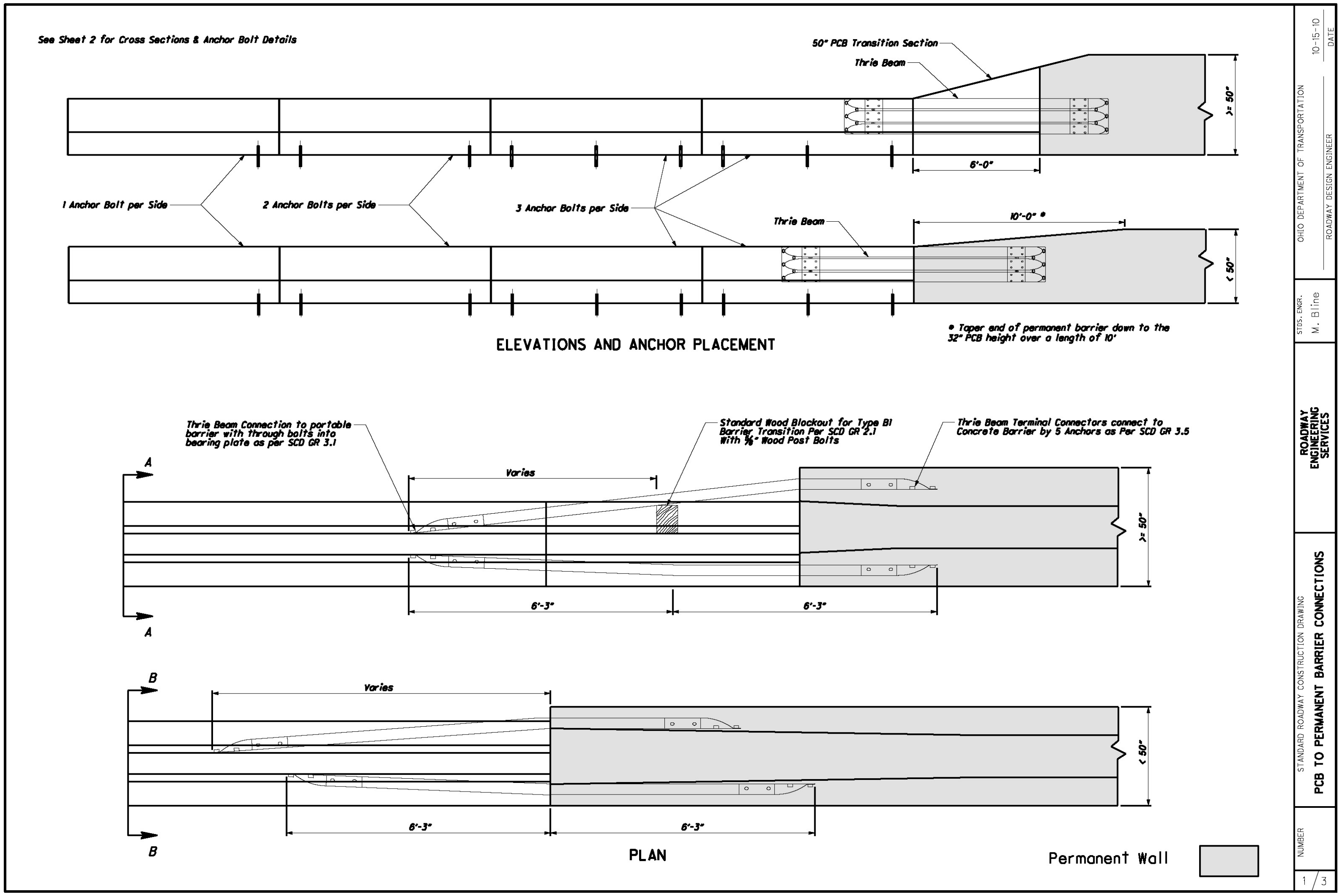


Figure 11. OhDOT PCB Transition Details

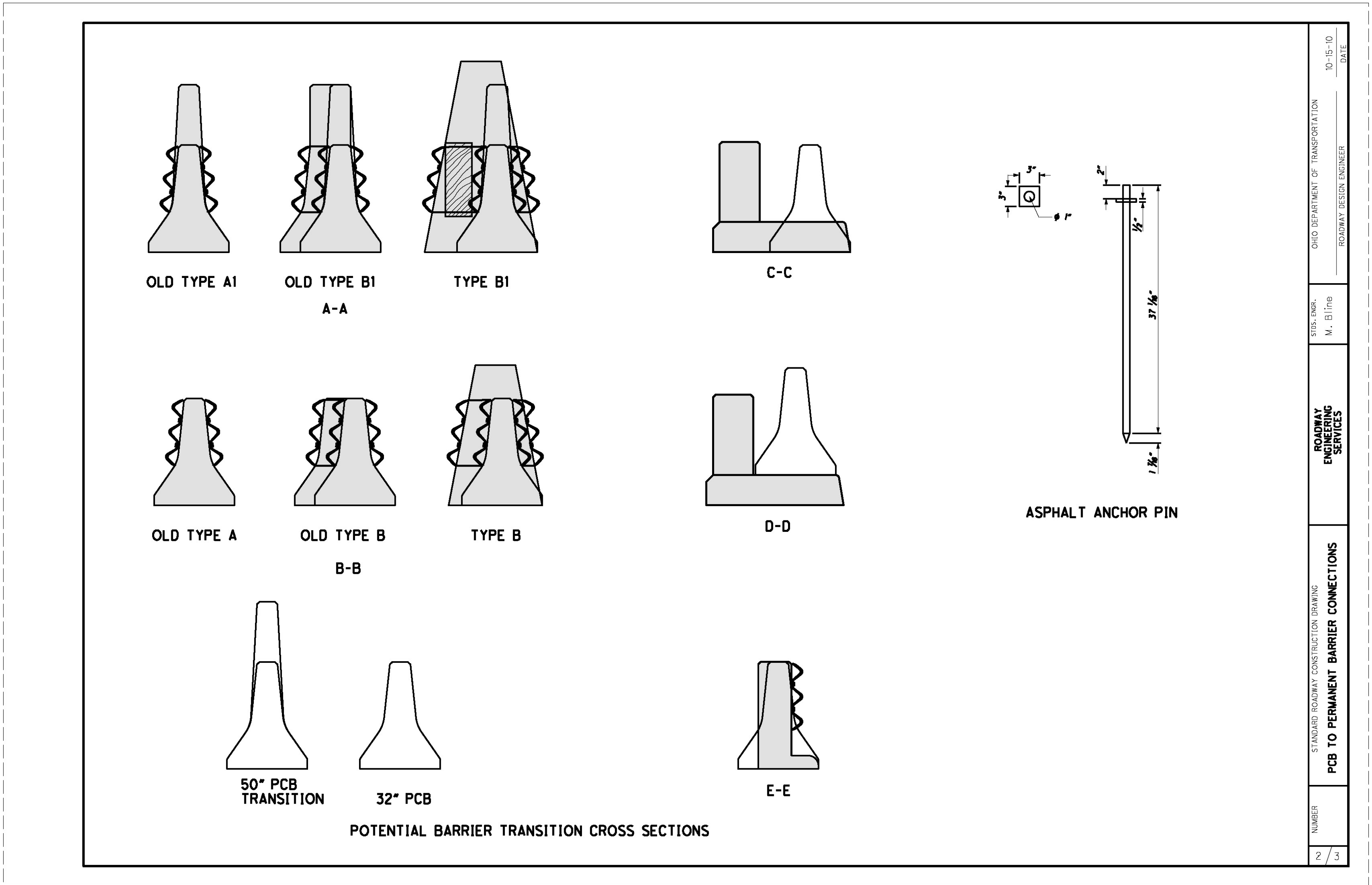


Figure 12. OhDOT PCB Transition Details

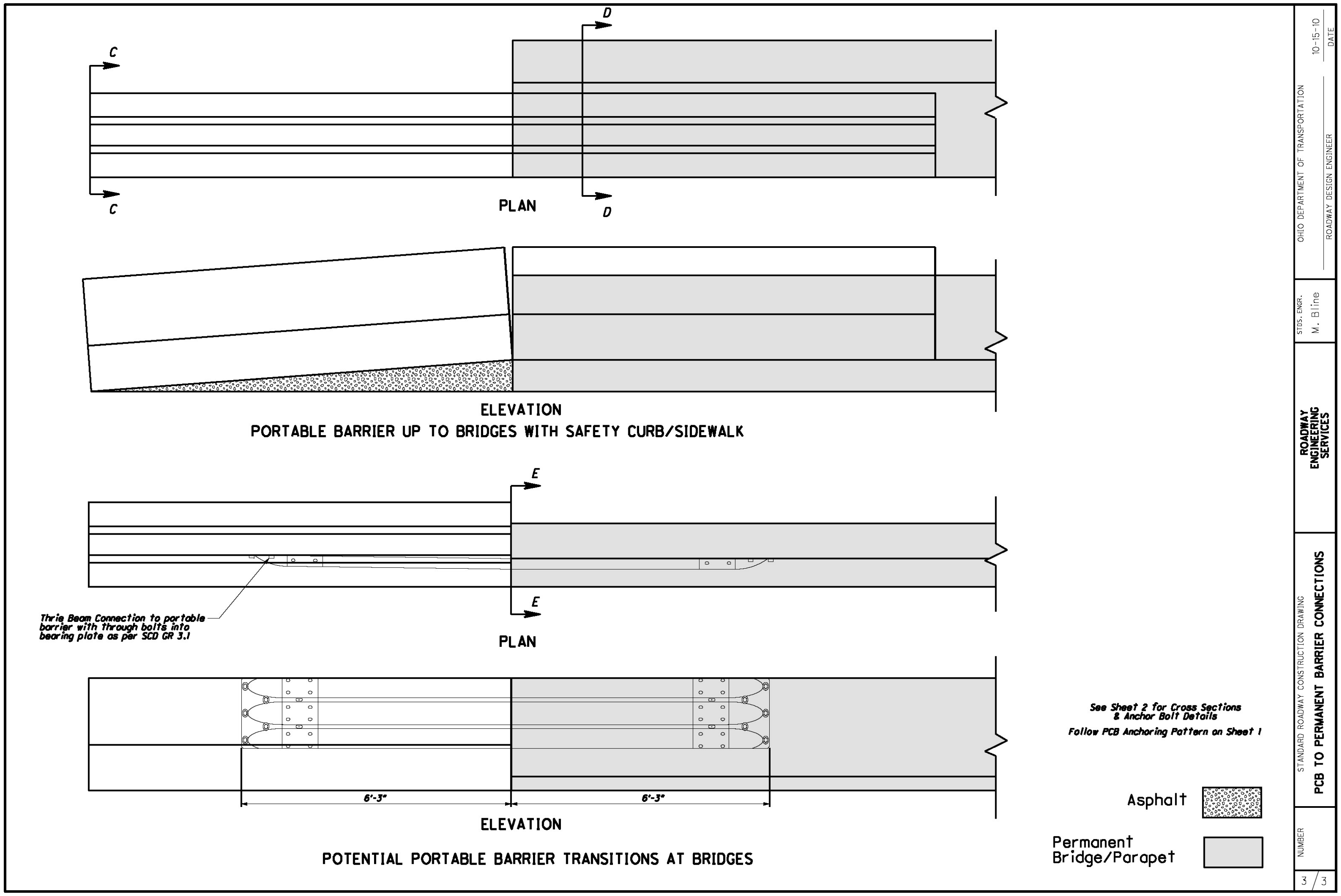


Figure 13. OhDOT PCB Transition Details

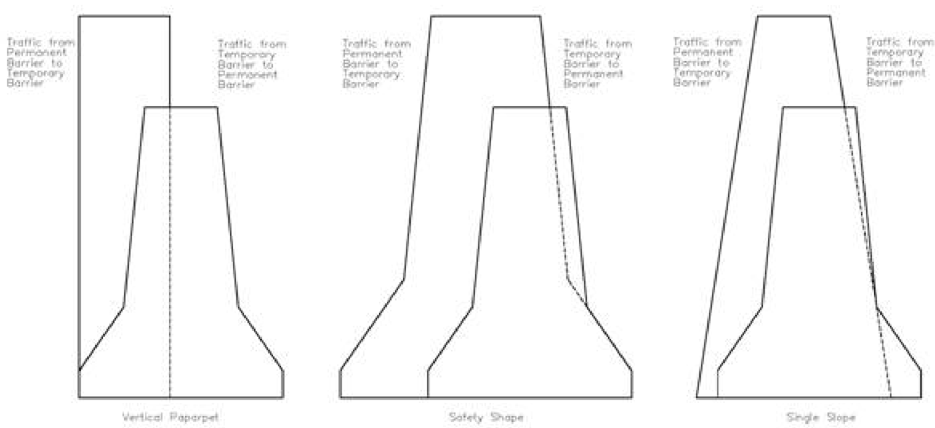
MwRSF Response:

Hi Michael,

Thanks for the welcome to the Big 10. Nice opening schedule you worked out for us.

I have gone through the transition plans that you sent and I have a few concerns.

1. The overall layout of the transition looks acceptable based on the design that we have developed here using the F-shape barrier. However, the barrier alignment will need some adjustment for some of the installation cases you have shown. The approach transition design was tested with the 42-in. tall, CA single-slope median barrier because this barrier was identified as the most critical barrier design for the transition. However, there are other permanent concrete median barriers that can be attached to the approach transition as long as the following guidelines are applied.
    a. If the permanent median barrier is 32-in. high, the sloped, steel transition cap is not required for the transition. For barriers with heights greater than 32-in. high, the steel transition cap if required. The cap design can be adjusted for different height and shape barriers as long as adjusted cap provides equivalent slope, permanent barrier coverage, barrier overlap, structural capacity, and anchorage as the original design.
    b. Alignment of the temporary barrier system with the permanent barrier may also change when the transition is applied to different permanent barrier geometries, as shown in below. When attaching to a single-slope barrier profile, the slope break point between the toe of the barrier and the main face of the barrier should be aligned flush with the oncoming traffic side of the single-slope barrier. For safety shape barriers, the toe of the temporary barrier should be aligned flush with the toe of the oncoming traffic side of the median barrier. Vertical median barriers require that the toe of the temporary barrier segments on the reverse direction traffic side be aligned with the base of the permanent barrier on the reverse direction traffic side. These alignments will prevent vehicle snag for oncoming traffic on the permanent median barrier while preventing snag on the toe of the barrier for reverse direction impacts.



Note: (1) The 32" temporary F-shape barrier is shown as the front barrier.

c. The thrie beam sections that span the gap between the end of the temporary barrier and the permanent median barrier should be used in all instances.

2. I would recommend that you check taper on the vertical transition section to higher median barriers. The system we tested used a vertical taper of 11.4 degrees. We would not recommend tapering to the taller barrier at a rate faster than that.
3. I have concerns with the size of the anchors used in the design. Asphalt pins in your system are much smaller than tested (1" diameter versus 1.5" diameter pins). This may cause significantly lower pin reaction forces and thus lower constrain of the barrier. I understand that you use a different barrier section, but the pins you are using will are not likely to perform similarly to those used in the testing.
4. Similarly, the threaded rod anchors smaller shown in your details appear to be smaller diameter than the A307 threaded rod anchors we have used in the past, but they are listed as high strength. What specifically is high strength? These may be acceptable.
5. In reviewing your barrier details, it appears that the barrier reinforcement is insufficient around the pin or bolt pockets. The F-shape barrier we tested with had specific reinforcement loops for those areas. During testing, those loops have been shown to be the primary restraint that contains the pins. Without that reinforcement in place, I do not believe that the tie-down system will function.
6. The barrier you use appears to be a NJ shape barrier. We cannot recommend this barrier for use with the tie-down system without further testing. Testing of the tie-down and transition systems has shown that the sloped face and toe of the barrier can rotate back during impact causing vehicles to ride up the barrier and increase instability. We believe that the NJ shape will make this behavior worse with its higher toe section. As such, we would not recommend NJ shapes with the tie-down systems shown.
7. We also have concerns with JJ-hooks connections with a tie-down system. We would not recommend this connection for use in an anchored barrier system. The JJ-Hooks connection is fine for free-standing systems. However, to be safely used in an anchored barrier or approach transition, the barrier joints must have comparable or greater torsional rigidity about the longitudinal barrier axis when compared to that of the as-tested configuration. JJ Hooks connection is not similar in torsion to the Kansas barrier joint, and the JJ Hooks connection is also non-symmetric in that it has different capacities depending on the direction it is loaded.

   There are certain barrier types out there that use cable loops or other types of connections such as JJ Hooks that have significantly different torsional capacity than the F-shape we were working with. Thus, we feel the need to warn that applying tie-down anchorages to a barrier with less torsionally stiff joints could promote vehicle instability through barrier rotation or snag.

Please review the information above and contact me with any additional thoughts and or questions.

Thanks

Bob Bielenberg, MSME, EIT
Research Associate Engineer

Problem # 10 – MGS Steel Post Grades

State Question:

Dear MwRSF,

I was looking at some on line information about 6PWE06-07 post (I've attached a link). On the backside, there is a note that confuse me.

I may be mistaken, but isn't Grade 50W is weathering steel? If the bolt that connects the beam guard to the block and post is galvanized, wouldn't you want a galvanized post?

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

MwRSF Response:

The AASHTO and ASTM steel specifications for determining the appropriate steel specifications for the W6x9 and W6x8.5 steel posts used within the MGS should be cleared up. Originally, we would have specified ASTM A36/A36M materials that used Fymin=36 ksi and Fumin=58 ksi. The TF13 Hardware guide designation page now also shows AASHTO 270 (ASTM A709) 50W in the 2006 version, which would denote Grade 50. Since there is a difference in the 50S and 50W designations, we need to be clear on whether or not weathering steel is specified. Also, the new W6x9 MGS posts have been being supplied with ASTM A992 (AASHTO ??) steel materials with Fymin=50 ksi. We may need to consider updating the TF13 details if the material specifications have changed.

Further clarification on steel grades:

A709 grade 250 is essentially the same steel as A36.
A709 grade 50S is essentially the same steel as A992
A709 grade 50W has nearly identical properties to 50S but is corrosion resistant (weathering steel)

So, perhaps the specifications should state: (note both 30 ksi and 50 ksi steel are already approved for use in specs)

For 36 ksi steel – A36/A36M or A709/A709M grade 250

For 50 ksi steel – A992/A992M or A709/A709M grade 50S. If corrosion resistant steel is desired, use A709/A709M grade 50W

I am not sure why we would have listed weathering steel other than it was copied from a prior specifications prepared for the steel guardrail posts already in the hardware guide. Mixing the two different components would likely be disastrous. We would like to not have anyone using corten guardrail steel or posts altogether. Thus, the specification should refer to A36, A992, A709 50S which provide options for using either Grade 36 (36 ksi) or Grade 50 (50 ksi) steel materials for prior guardrail systems.

We will need to look into why the note is currently written as it is and hopefully get it fixed. Karla will be meeting with AASHTO TF13 on Monday and Tuesday and can raise this issue with the entire guardrail community. We would not want to use up the zinc coating of the rail, bolts, nuts, etc. to first prevent A588 steel material from rusting the outer post layer.

Problem # 11 – Questions about Temporary Barrier Transition to Permanent Barrier

State Question:

Dear MwRSF,

I'm working on incorporating the transition from temporary barrier to permanent barriers. I've run across some questions (see attached) that I would like MwRSF to address.

Sincerely,

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

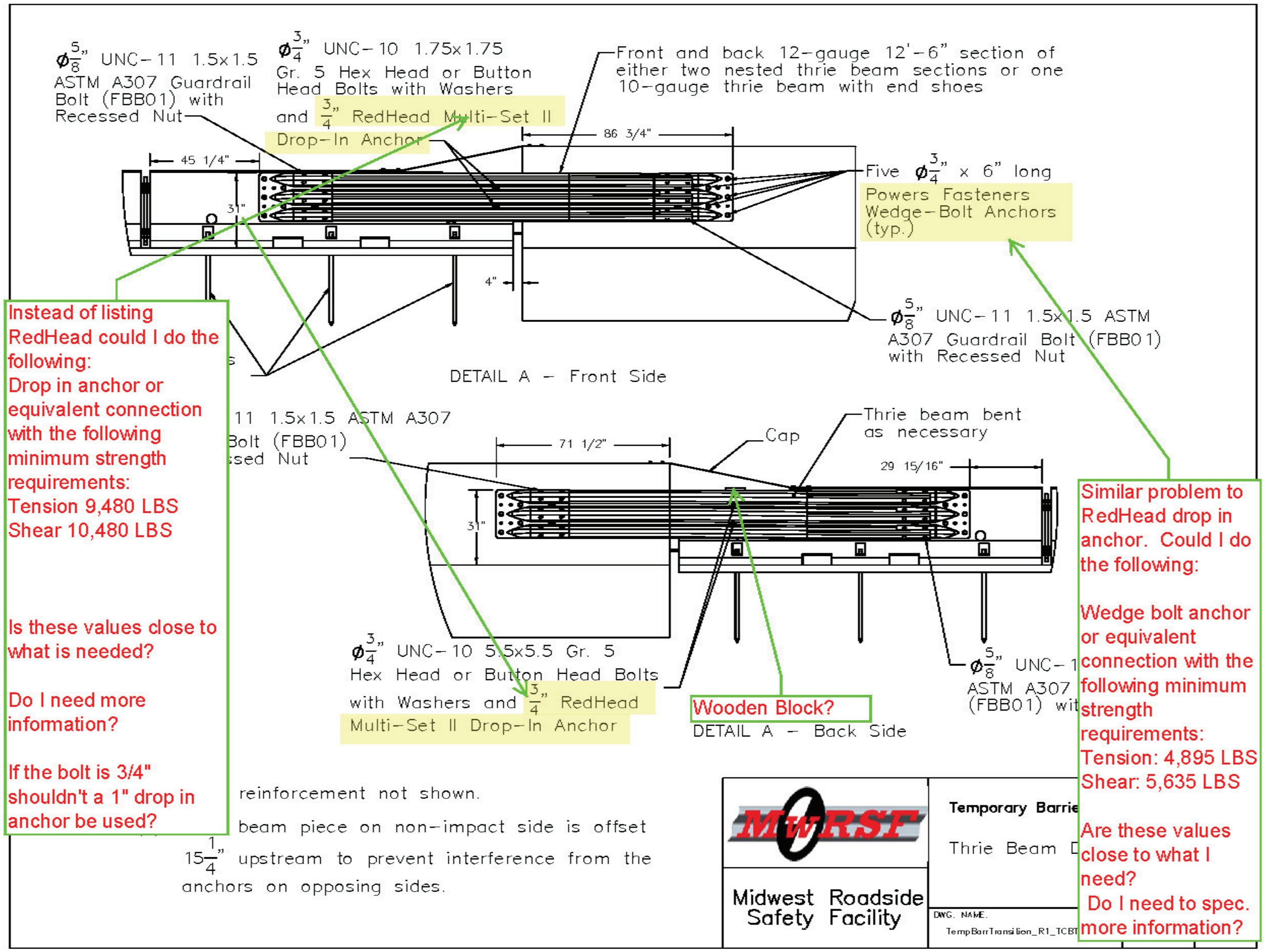


Figure 14. PCB Transition Questions

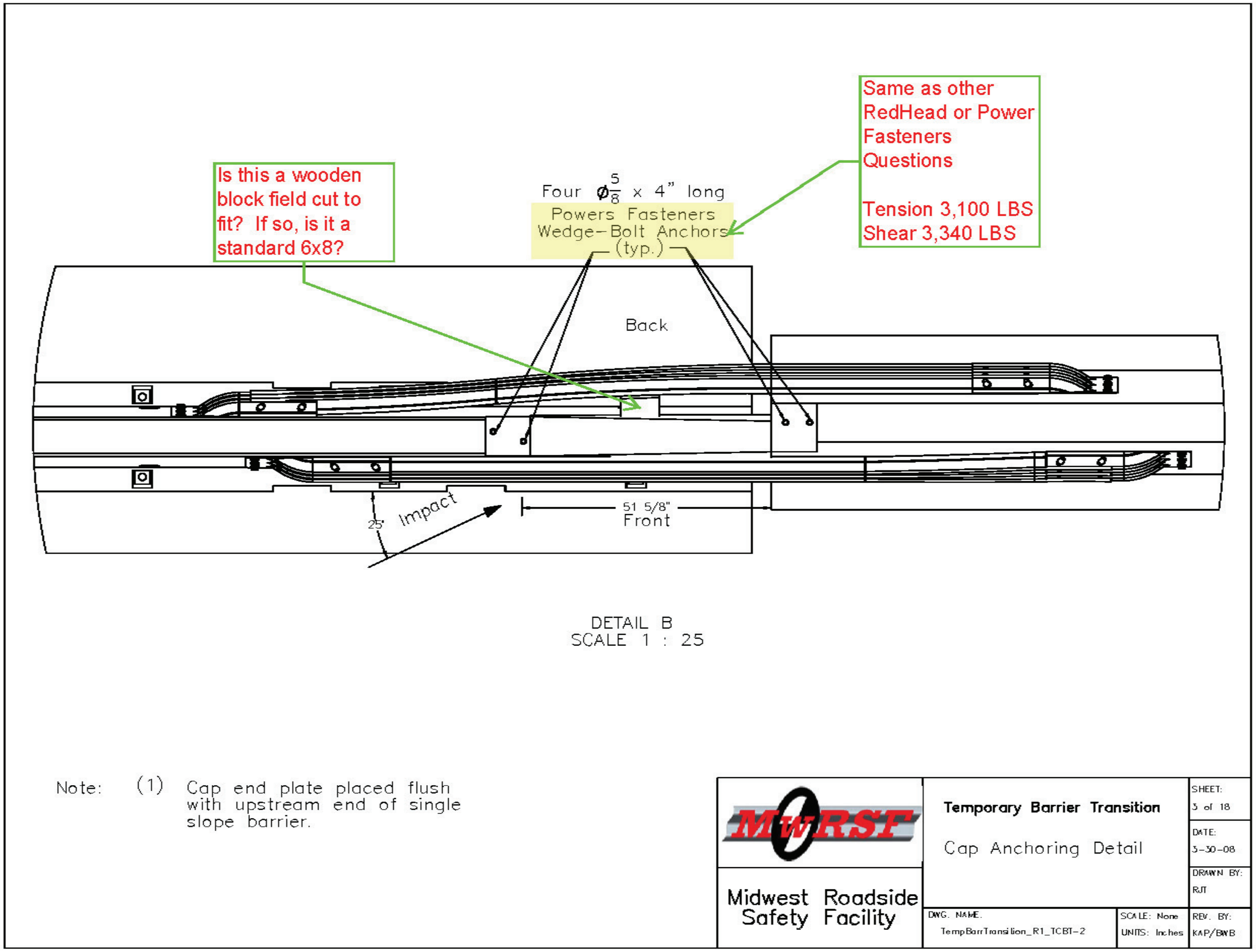


Figure 15. PCB Transition Questions

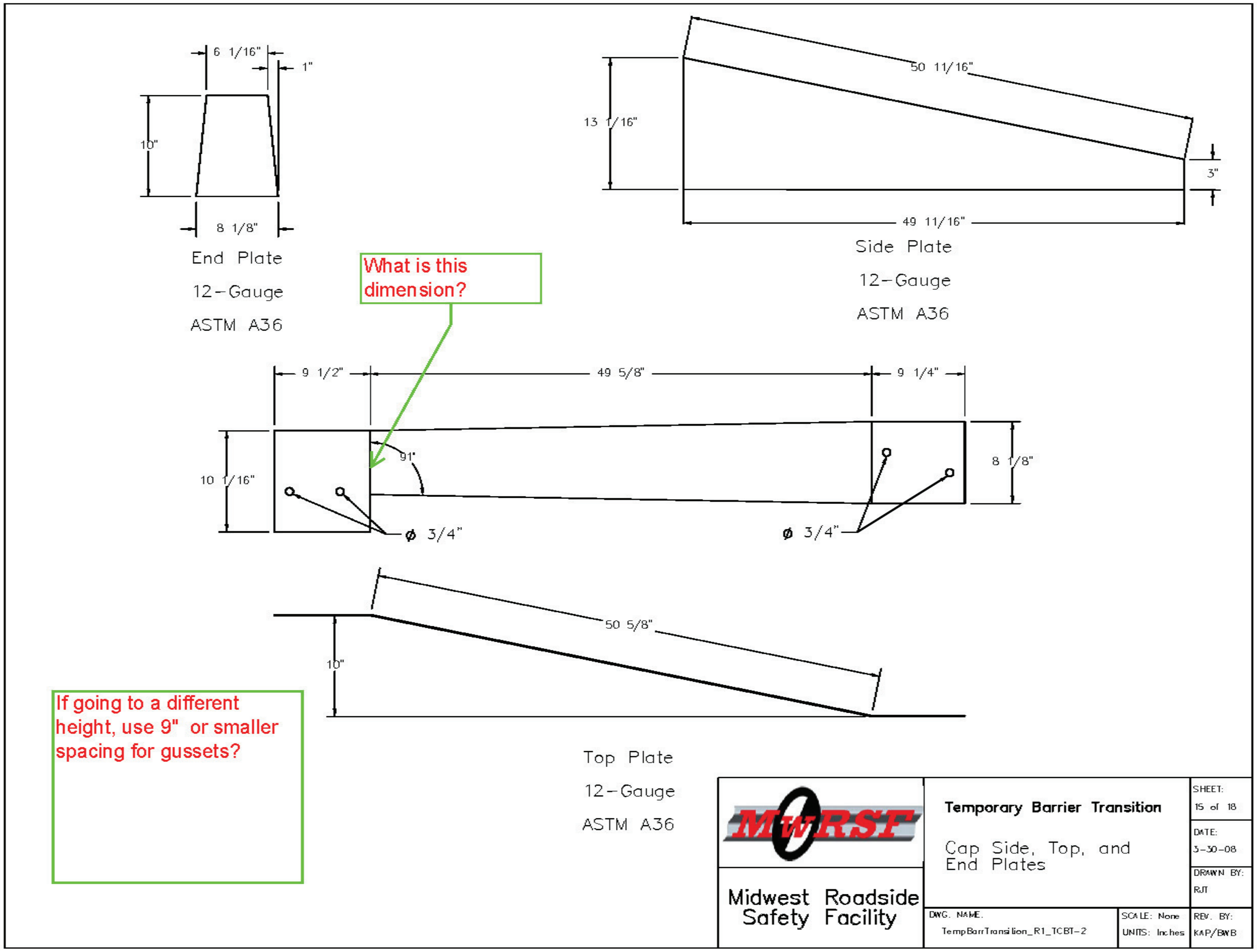


Figure 16. PCB Transition Questions

MwRSF Response:

Hi Eric,

I have reviewed your questions and have some answers below.

With respect to the Red Head drop in anchor, we would recommend that any drop-in anchor that has equal or greater ultimate shear and tensile capacity (as listed by the manufacturer) when compared to the tested anchor would be allowable. The ¾" bolts used would remain A325.

With respect to the Wedge Bolt anchor, we would recommend that any mechanical screw in type anchor that has equal or greater ultimate shear and tensile capacity (as listed by the manufacturer) when compared to the tested anchor would be allowable. We would want to use the same diameter anchors as tested in order to be consistent.

The trimmed blockout used in the design was fabricated from an existing thrie-beam timber blockout.

The missing dimension on the top of the cap should be 7.25".

If you need to go to a different height, there are several factors to consider. First, you want to keep the slope of the cap the same as the tested design. Second, you will want equal gusset spacing to the tested design. Third, increasing the height while maintaining the slope of the cap may require increasing the length significantly. Thus, we would recommend extending the sides of the cap down another 3" along the side of the barrier to allow for placement of intermediate anchors that attach the side of the cap to the side of the barrier. The anchors can be the same 5/8" mechanical anchors used elsewhere for the cap. We would recommend having these intermediate anchors approximately every 50" along the barrier. These additional anchors will prevent the cap from disengaging from the temporary barrier and allowing snag on the permanent barrier as the length of the cap increases.

Thanks

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

Problem # 12 – Oregon PCB

State Question:

Hello Ron:

Attached are 2 sketches for your consideration with the following queries:

- Temporary Precast Concrete barrier Pin and Loop/Anchors 2 (ODOT)
- Securing Concrete Barrier to Roadway (ODOT)

Are you aware if this Barrier has been crash tested for highways of 80kM/hr to NCHRP Report 350, and/or S6-06? (Bridge Barrier)

For a separated highway (I.e. 2 lanes in each direction, barriers on each side), would it be acceptable to use the Single Pin-Shoulder Installation for the Fast lane Barrier protections for construction workers operating in the area between the highway (I.e. Median)?

Or should the Median Installation (2 pins) be used, even though there are "two layers of defense"? (Note 7 indicates that 2-pins are needed for a median of less than 8', but does this include 2 barriers?)

Does this detail in conjunction with a sliding distance of 500mm away from the impact side provide a crash tested barrier that should be considered safe for use in protecting median construction work and workers?

Is it necessary to have a design engineer seal the proposed design for a temporary use?

Your valuable input is appreciated if possible.

Thanks in advance. Let me know if you have questions or need some clarification.

Regards,

Michael Roberts, P.Eng
Specialty Structural Engineering

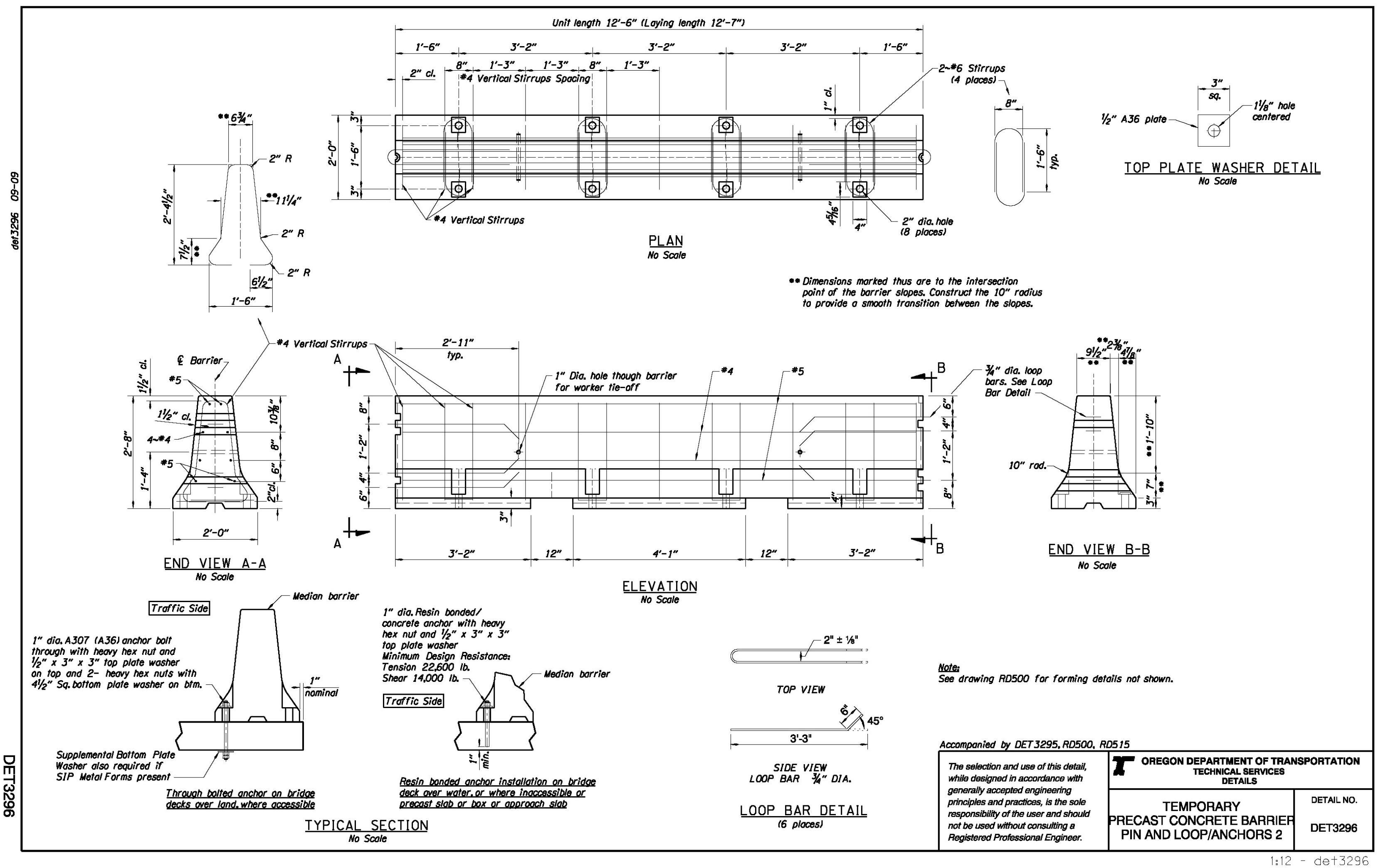


Figure 17. Oregon PCB

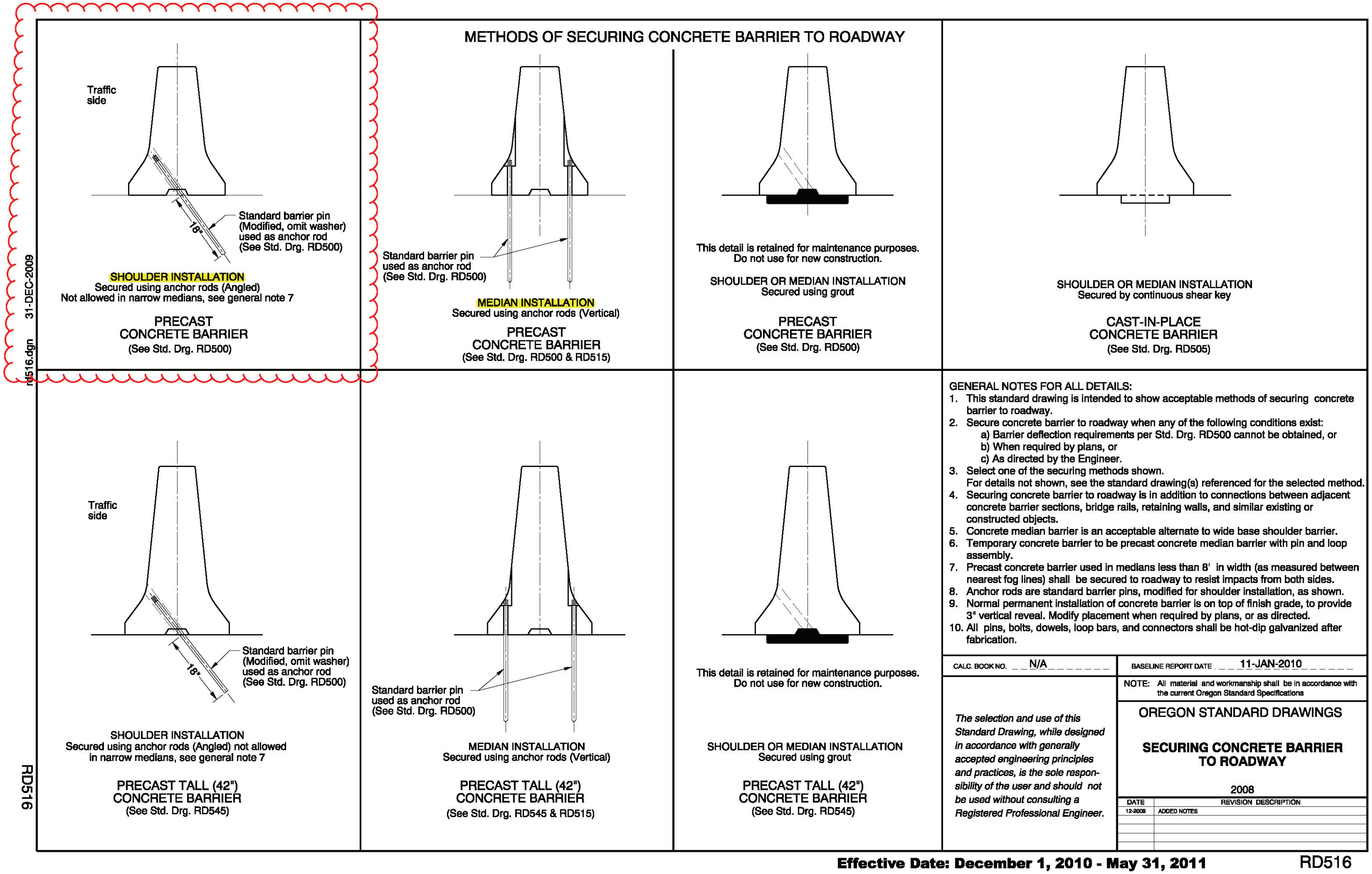


Figure 18. Oregon PCB

MwRSF Response:

Michael:

I am not aware of any crash testing programs directed toward the evaluation of the Oregon TCB in anchored/tied-down configurations. However, some of the Oregon anchorage details are similar in concept to those used by MwRSF for an Midwest Pooled Fund F-shape TCB as well as by TTI in other TCB designs.

In general, anchorage options can be adapted over to alternative TCBs without actual crash testing if several conditions are believed to be met, including demonstration that equivalent or greater structural capacity is provided. Thus, one would require that the Oregon barrier provides equal or greater flexural capacity, shear strength, torsion resistance, etc. as compared to the as-tested TCBs where anchorage systems were evaluated. The Oregon joint detail should provide equal or greater strength between adjacent segments as compared to the as-tested TCBs. Barrier regions surrounding the openings where the vertical or sloped anchors are inserted in the Oregon barrier should provide equal or greater capacity as compared to the as-tested TCBs. The anchorage hardware should also provide equivalent structural capacity. Barrier lengths should also be similar. If these general conditions are met, then it would seem reasonable to adapt prior crashworthy anchorage options to similar TCBs.

MwRSF could provide you with research/test reports for anchorage systems that have been developed and crash tested at 100 km/hr for use with the Midwest F-shape TCB. TTI researchers would have similar information for those systems that were developed and tested by their personnel. Please let me know if you need further details and information. Thanks!

Ron

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

Midwest Roadside Safety Facility (MwRSF)
University of Nebraska-Lincoln
130 Whittier Research Center
2200 Vine Street
Lincoln, Nebraska 68583-0853

Attached are 2 sketches for your consideration with the following queries:

- Temporary Precast Concrete barrier Pin and Loop/Anchors 2 (ODOT)
- Securing Concrete Barrier to Roadway (ODOT)

Are you aware if this Barrier has been crash tested for highways of 80kM/hr to NCHRP Report 350, and/or S6-06? (Bridge Barrier)

**Comments provided above. I am not aware of the Oregon TCB being crash tested under NCHRP Report No. 350 when installed with various anchorage options. However, it should be noted that some options shown in the attachments have similarity to anchor methods used with other crash tested barriers.

For a separated highway (I.e. 2 lanes in each direction, barriers on each side), would it be acceptable to use the Single Pin-Shoulder Installation for the Fast lane Barrier protections for construction workers operating in the area between the highway (I.e. Median)?
**Yes, as long as the Oregon barrier provided equivalent or greater safety performance compared to the pinned TCB configured and tested by TTI.

Or should the Median Installation (2 pins) be used, even though there are "two layers of defense"? (Note 7 indicates that 2-pins are needed for a median of less than 8', but does this include 2 barriers?)
**TCBs pinned on both sides have only been subjected to limited crash testing when used with a median TCB transition between free-standing TCBs and permanent barrier.

Does this detail in conjunction with a sliding distance of 500mm away from the impact side provide a crash tested barrier that should be considered safe for use in protecting median construction work and workers?
**I am not sure that I understand the question.

Is it necessary to have a design engineer seal the proposed design for a temporary use?
**This is a question better suited for the DOTs. However, someone needs to ensure that the barrier hardware and its placement meet current safety practices.

Problem # 13 – Shielding of Cut Slopes

State Question:

Hi Ron,

Do you know if we need to protect the "cut slopes" steeper than 1:3 within the clear zone (everything I have seen is for "fill sections")? Could you please point me to some references on this topic?

Thank you
Mohammad

Mohammad Dehdashti, P.E.
Minnesota Department of Transportation
Design Engineer

MwRSF Response:

Mohammad:

The 2006 RDG provides limited discussion on the use on backslopes or cut slopes – see Section 3.2.2 and Figures 3.6 and 3.7. In general, the RDG states that the backslope may be traversable depending on its relative smoothness and the presence (or lack) of fixed obstacles.

Some time back, I recall briefly investigating this issue for Dave Little. His email and my response is contained below, along with his original attachment. The noted references are NCHRP Report No. 158 and the 1996 RDG. From this, the guidance suggests the use of a maximum cut slope of 2:1.

Ron

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

Midwest Roadside Safety Facility (MwRSF)
University of Nebraska-Lincoln
130 Whittier Research Center

From: Ronald K. Faller [mailto:rfaller1@unl.edu]
Sent: Wednesday, September 17, 2008 10:31 AM
To: 'Little, David [DOT]'
Cc: 'Dean Sicking-Work'; rbielenberg2@unl.edu
Subject: RE: Cut Slopes

Dave:

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.

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

Ron

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

Midwest Roadside Safety Facility (MwRSF)
University of Nebraska-Lincoln
527 Nebraska Hall
Lincoln, Nebraska 68588-0529

(402) 472-6864 (phone)
(402) 472-2022 (fax)
rfaller1@unl.edu

From: Little, David [DOT] [mailto:David.Little@dot.iowa.gov]
Sent: Wednesday, September 17, 2008 10:01 AM
To: Ronald K. Faller
Subject: Cut Slopes

Ron:

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.

Dave

Problem # 14 – MASH Temporary Barrier Deflection Vs NCHRP 350 Deflection

State Question:

Dear MwRSF,

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

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

MwRSF Response:

Hi Erik,

We have seen an increase in the deflected of the F-shape PCB when impacted with MASH vehicles.

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

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

This increase in deflection is due to a couple of factors

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

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

Bob Bielenberg, MSME, EIT

Research Associate Engineer
Midwest Roadside Safety Facility

Problem # 15 – Temporary Sand Barrel Arrays

State Question:

Dear MwRSF,

I'm looking into providing additional guidance for our staff on the use of temporary crash cushions and sand barrel arrays.

During my reviews, I found NCHRP Report 358 Recommended Practices for Use of Traffic Barriers and Control Treatments for Restricted Work Zone (see attached). I have the following questions:

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

Sincerely,

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

MwRSF Response:

Hi Eric,

I have responded to your questions below.

Bob Bielenberg, MSME, EIT
Research Associate Engineer

---

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

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

2. The guidance on what treatment to use to protect the blunt end of the temporary barrier was based on the barrier being installed for 1 year or less. If a project will have temporary barrier installed for more than a year what steps should be taken by a designer?
    - The end treatment guidance in NCHRP 358 was based on benefit/cost analysis. Thus, the longer the sand barrel array was installed, the more likely that a more robust, long term attenuator would be worth installing. That said, we do not believe that leaving the sand barrels in place for a period over one year hugely problematic. If the barrel array is installed for a much longer time than one year, then you may want to rethink which type of system you use.
3. Would the charts (figures 4.17-4.24) have significant changes to the break points between different end treatments because of the new MASH vehicles? Or do these charts represent the most current state of the art for temporary barrier end treatment protection?
    - The charts mentioned in NCHRP 358 are currently the best guidance for barrier flare rates n the work zone. No further analysis has been done to update those tables with more recent accident data or to make considerations for MASH.

Problem # 16 – Design Considerations for Prevention of Cargo Tank Rollovers

State Question:

Dear MwRSF,

I received the attached memo from FHWA today (INFORMATION: Design Considerations for Prevention of Cargo Tank Rollovers - September 3, 2010). I have only skimmed through the document, but I am concerned that FHWA is recommending the use of taller vertical barrier without considering the effect of head slap on smaller vehicles.

If MwRSF could review view this document and provide comments it would be appreciated. I wish to send a letter back to FHWA indicating my concerns about the use of taller barrier wall without considering head slap.

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

MwRSF Response:

Erik:

I have briefly reviewed the attached information. From my review, FHWA has noted the concern for head slap against taller TL-6 barriers such as a 90-in. configuration tested at TTI years ago.

However, this same concern does not appear within the guidance to correspond to TL-5 barriers which have commonly been configured with 42-in. high RC parapets. You have correctly pointed out that 42-in. parapets can also pose risks of head ejection and contact against taller barriers. I believe that Dean has uncovered the risks associated with ejected passengers resulting in serious injuries and/or fatalities after analyzing accident data for the State of Kansas as part of the median barrier study.

I believe that the roadside safety community needs to be careful about blindly placing a large number of tall, rigid barriers in more locations in an effort to contain the rare occurrence of a tractor-tank trailer into piers and other structures, especially if it results in much greater risk of injury/fatality for occupant of passenger vehicles to have partial body/head ejection against tall, rigid barriers. If deemed necessary, it would seem reasonable to utilize TL-5 barrier designs which can both prevent catastrophic crashes as well as reduce/prevent head slap against tall parapets for the occupants of passenger vehicles.

Ron

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

Problem # 17 – Breakaway Devices Behind the MGS

State Question:

Ron,

Is there guidance for clearance behind the MGS for hardware that has a breakaway base/devices such as light poles or ground mounted signs?

Have any of these devices been tested with the MGS?

On many locations along the IL Tollway the typical section consists of a v-shaped gutter at the edge of outside shoulder, a noise abatement wall that is 5' from the back of gutter. The guardrail post is set back 6" from the back of gutter, so the distance from the back of post to face of noise wall is 4'. This 4' space is where light poles and single post ground mounted signs are placed. If the light pole is centered in the 4' space, the distance from the back of guardrail post to near edge of light pole is approx. 1'-7". This is obviously less than the 28" minimum clearance distance recommended in the MGS documentation.

Thanks,
Tracy Borchardt
AEC OM
IL Tollway GEC

MwRSF Response:

Tracy:

There exists guidance for use in placing the MGS in front of various obstacles. In the absence of crash testing with poles, trees, supports, etc., it would be recommended that the Working Widths be used. The working width is measured from the original front face of the barrier system. I have enclosed a table with available working widths from the MwRSF crash testing programs.

If pole placement is desired within the published WW values, then full-scale crash testing would be necessary to verify acceptable safety performance with the alternative pole placement.

Ron

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

System Description	Post Spacing	Post Type[1,2]	Post Length	Test Designation	Test Criteria	Dynamic Deflection	Working Width	Maximum Vehicle Roll
Standard Post Spacing	6 ft - 3 in.	Steel W6x9	6 ft	Test 3-11	350	43.1 in.	49.6 in.	-4.8 deg
	6 ft - 3 in.	Steel W6x9	6 ft	Test 3-11	MASH-08	43.9 in.	48.6 in.	-6.3 deg
	6 ft - 3 in.	Steel W6x9	6 ft	Test 3-11	MASH-08[4]	57.0 in.	57.3 in.	-4.8 deg
	6 ft - 3 in.	Round DF	5 ft 9 in.	Test 3-11	350	60.2 in.	60.3 in.	14.2 deg
	6 ft - 3 in.	Round PP	5 ft 9 in.	Test 3-11	350	37.6 in.	48.6 in.	10.7 deg
Half Post Spacing	3 ft - 1 1/2 in.	Steel W6x9	6 ft	BARRIER VII Simulation	350	27.8 in.	43.1 in.	NA
Quarter Post Spacing	1 ft - 8 3/4 in.	Steel W6x9	6 ft	Test 3-11	350	17.6 in.	36.7 in.	4.7 deg
2:1 Fill Slope	6 ft - 3 in.	Steel W6x9	9 ft	Test 3-11	MASH-08	57.5 in.	62.6 in.	8.3 deg
8:1 Approach Slope	6 ft - 3 in.	Steel W6x9	6 ft	Test 3-11	350	57.6 in.	82.8 in.	38.1 deg
Long-Span Over Culvert	25-ft Unsupported Length with 6 ft - 3 in.	CRTs[3]	6 ft	Test 3-11	MASH-08	92.2 in.	93.4 in.	9.9 deg
		CRTs[3]	6 ft	Test 3-11	MASH-08	77.5 in.	84.0 in.	-10.7 deg
5:1 Flare Rate	6 ft - 3 in.	Steel W6x9	6 ft	Test 3-11	350	75.6 in.	97.4 in.	-10.1 deg
7:1 Flare Rate	6 ft - 3 in.	Steel W6x9	6 ft	Test 3-11	350	75.8 in.	87.9 in.	-19.9 deg
13:1 Flare Rate	6 ft - 3 in.	Steel W6x9	6 ft	Test 3-11	350	66.3 in.	70.6 in.	-16.2 deg
MGS Over 6-in. Curb	6 ft - 3 in.	Steel W6x9	6 ft	Test 3-11	350	40.3 in.	57.2 in.	-18.8 deg

[1] - Steel W6x8.5 sections can be substituted for W6x9 sections.
[2] - Round Southern Yellow Pine (SYP) timber posts were also crash tested and evaluated with the standard MGS.
[3] - Three timber CRT posts are installed on each side of the unsupported length measuring 25 ft or less.
[4] - 2270P GMC 2500 3/4-ton, 2-Door, Pickup Truck.

Figure 19. MGS Working Widths

Problem # 18 –MGS Minimum Length

State Question:

Bob et al,

I get asked this question pretty often. Does the work that you are doing for Wisconsin address the minimum length issue?

I am considering placing a 100' minimum length requirement in the new manual, exclusive of terminals, when installation is not attached to a structure (parapet, barrier wall). This essentially provides 137.5' of w-beam guardrail toward the length of need. Are you comfortable with this or should we use a longer minimum installation?

Thanks,
Tracy Borchardt
IL Tollway GEC
AECOM

MwRSF Response:

Tracy:

MwRSF has recently obtained funding to investigate the minimum guardrail length issue. For the minimum guardrail length study, system lengths less than 175 ft will be evaluated in terms of effectiveness, working width, and dynamic barrier deflection.

Based on crash tested system lengths, we have suggested that a minimum system length of 175 ft be used for TL-3 applications. This system length includes the use of guardrail end terminals. Guardrail end terminals can be used to configure the overall system length. For example, the interior guardrail length would be 100 ft if each end included the use of a 37.5 ft long FLEAT guardrail end terminal. We recognize that there exists is strong likelihood that system lengths less than 175 ft will be capable of containing and redirecting heavy passenger vehicles occurring at high speeds and angles. However, we are unable to definitively provide guidance on the minimum barrier length for W-beam guardrail systems or on placement in front of hazards. For now, I would recommend that you maintain the use of the 175 ft system length.

Ron

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

Problem # 19 –Bridge Rail Retrofit

State Question:

Dear MwRSF,

WisDOT has a project where an existing thrie beam bridge rail was installed too low. Regional staff has asked if:
1. The existing longitudinal channel on top of the bridge rail could be removed.
2. A small box beam or steel tube could be bolted to the existing post (i.e. to get the correct rail height)
3. Existing longitudinal channel is reinstalled.

From what I understand, the existing deck bolts and nuts are very rusty and difficult to remove. This will make it difficult to remove the existing post and replace with new taller posts. In addition the taller post are more expensive to fabricate than the smaller box beam/steel tubes.

An example of the retro fit is attached (W Rail Retrofit.pdf)

An example of our current thrie beam retro fit is also attached (3002.pdf).

The rail used on the existing bridge is also attached.

Sincerely,

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

MwRSF Response:

The Wisconsin design is similar to the Missouri thrie beam and channel bridge rail tested by TTI. However, the bridge rail plans show the system as 4 inches shorter than it was tested at previously. Thus, the addition of the 4 in. tall spacer block to the top of the shorter post allows the rail to be installed at the correct height. The use of four 5/8 in. diameter bolts to connect the block to the post should provide more than enough strength to prevent shear failure during an impact.

The anchorage for the Wisconsin bridge rail seems to be a modification of the tested system as well. Tested used three 1 in. diameter A307 bolts, while the current drawings show 4 7/8 in. diameter A449 bolts. Noting that A449 provides a 20-50% increase in strength (depending on grade), the Wisconsin bridge rail design should provide equal or greater anchorage strength.

Therefore, the proposed bridge rail design appears to be of comparable strength and geometry to that of the tested Missouri thrie beam and channel system.

Scott Rosenbaugh
Midwest Roadside Safety Facility (MwRSF)
University of Nebraska – Lincoln

Problem # 20 –Updated Barrier Design for Higher Weight Limits

State Question:

Ron,

We have been asked by our management to consider how barrier designs may change as a result of recently-proposed higher weight limits on our interstate highways. Of particular interest is whether additional reinforcement will be required in our TL-5 barriers in order to withstand an impact from a tractor-trailer weighing up to 97,000 pounds.

If I have understood the literature correctly, staff at Lincoln use a defined-magnitude force applied over a certain length, at a specified height, when determining the minimum amount of reinforcement required in a barrier. Would you be able to provide me with updated values for these forces if we were to assume a 97,000 pound tractor-trailer impacting our 44" F-shape concrete barrier under TL-5 conditions? Here is a link to our standard drawing: http://www.iowadot.gov/design/SRP/IndividualStandards/eba100.pdf

Let me know if you have any questions.

Thanks,

-Chris

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Chris Poole, P.E.
Roadside Safety Engineer
Office of Design
Iowa Department of Transportation

MwRSF Response:

Chris:

In a prior analysis and design effort, MwRSF researchers determined a linear relationship for the lateral design impact load for tractor trailer vehicles striking rigid parapets. Of course, these results were determined using historical crash testing data for both the total IS values as well as the IS value for the tractor's rear tandems. From TRP-03-149-04, we reported design loading for PL-3 (50,000-lb vehicle) and TL-5 (79,366-lb vehicle) impact conditions was 153 to 153 kips and 243 to 248 kips, respectively. Based on this evidence, we prepared some preliminary TL-5 barrier designs using 217-kip design load based on the structural capacities of prior TL-5 barriers.

Later, MwRSF designed and crash tested a TL-5 open concrete bridge railing as well as a concrete median barrier using a reduced load condition similar to that provided above. Thus, it seemed reasonable to use existing yield-line analysis procedures (YLAP) in combination with the reduced lateral impact load.

You noted below that a vehicle weight of 97,000 lbs is being considered by the IaDOT. As such, you inquired as to the predicted design load with such a weight increase. Based on an increased IS of (97000/79366)=1.22, the new design load would be approximately 265 kips if using existing YLAP.

Please let me know if you have any additional questions regarding the information contained above. Thanks!

P.S. – Please note that the design of expansion joints are treated differently than interior sections since fewer yield lines can be developed due to the discontinuous rail.

Ron

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