Midwest States Pooled Fund Program Consulting Quarterly Summary

Midwest Roadside Safety Facility

04-01-2014 to 07-01-2014

Driving Posts Through MSE Wall Reinforcement

Question

State: WY
Date: 04-02-2014

Following up on a previous question to the pooled fund regarding AASHTO LRFD requirements for guardrail posts placed in MSE walls dated February 13, 2014. The response referenced report TRP-03-235-11, “Development of an Economical Guardrail System for use on Wire-Faced MSE Walls". The response also mentioned that the posts should be installed in accordance with AASHTO LRFD 11.10.10.4. In this section, it discussed penetrations through the reinforcement and recommended steel boxes be fabricated around “leave-outs" in the mesh, tying them to the mesh to provide continuity of the reinforcement. This creates many problems including trying to accurately locate these “leave-outs" when installing the posts and it could be very expensive. I am wondering if it is O.K. to drive the posts through the mesh and any material separation fabric. This would create the smallest possible penetration area through the mesh. This would likely only penetrate one to two layers of mesh located at the top of the wall where the lateral soil forces are fairly low anyway. A concern that has been raised is if this could pull the mesh out of position when the posts are driven through it. I noticed in the MwRSF report on gabions (marked “Draft"), it shows that the steel guardrail posts were driven through the wire reinforcement mesh and the material separation fabric. There was not discussion about this however. Did MwRSF experience any problems with this installation? Is this an acceptable way to install the posts by just driving them through the mesh and fabric? We have had our field personnel asking about installing sonotubes and neatly cutting the reinforcement around the tubes. I think this is a bad idea since it would introduce larger “leave-outs" in the mesh and confine soil around the posts making them overly stiff.
Attachment: http://mwrsf-qa.unl.edu/attachments/e85af4c02a8bbbb70123a412eebc6559.docx

---

Response

Date: 04-11-2014
During the installation (and repair) of the MGS system placed on top of an MSE wall, MwRSF installed the steel posts utilizing a post driver. This most like led to the post tearing through small areas of the top layer of wire-mesh of the MSE wall. Throughout the installation of the system, repairs to the system, and 2 full-scale crash tests, no movement was observed from the MSE wall. Thus, driving the posts through the wire mesh did not appear to have any negative effects. However, please note that this was for steel posts only. As noted in a follow-up report (TRP-03-256-12), further evaluation would be necessary for recommendations concerning driving wood posts through the wire mesh due to the larger cross section area, softer material, and difficulties with removal of damaged/fractured posts. Additionally, without further study, MwRSF would not recommend the use of sonotubes as this will affect a larger area of the MSE wall's wire mesh.

Head ejection application to 36

Question

State: MN
Date: 04-10-2014

We are considering increasing the height of our standard F-shape 32" concrete bridge rail.

Because of the TTI crash test (Test Report 9-1002-5), we have decided that 36" would be the minimum height. The F-shape however has the instability issue with the bottom “heal" section, so we are also considering a single slope solution similar to Wisconsin's design.

However, for both cases there are some concerns about the Head Ejection Criteria, based on the MwRSF report TRP-03-194-07.

Based on your knowledge and experience on this issue, does the Head Ejection concern apply to 36" tall barriers (f-shape or single slope)?

This is a hot and urgent issue, so a quick response is appreciated, if you can.

Thanks

Response

Date: 04-10-2014
The following is a summary of our discussion we had over the phone today, April 10th.

You had questions about how to apply the Head Ejection Envelope (as detailed in report no. TRP-03-194-07) to a 36" tall single slope barrier. We discussed 2 possible solutions to this issue. The first, and more safety conservative, approach would be to apply the Head Ejection Envelope directly to the single slope barrier even though it was initially created for vertical faced barriers. This process includes cutting out a triangular wedge from the top front corner of the barrier to keep the 36" tall barrier from violating the Head Ejection Envelope. The cut out wedge would measure 1.5" vertically (as the envelope begins at 34.5") and would extend 4" laterally into the top of the barrier. This cut out should not alter the performance of the barrier for TL-4 impacts as the 36" height will still be there to apply a vertical force the SUT vehicle's box and prevent the vehicle from rolling over the barrier.

The second option would be to leave the 36" single slope barrier as is – no cut out at the top. This could be justified by the fact that the barrier is single sloped, not vertical faced. The sloped face of the barrier would induce a small amount of vehicle climb for the impacting side front tire. This little bit of climb coupled with slight vehicle roll away from the barrier that would go with it, may be enough to keep an occupant's head from impacting the top of the barrier. However, I cannot say this with any form of certainty. Further, the vehicle may not roll or climb much at all during non-tracking impacts (vehicle is yawing just prior to impacts). As such, this approach is not as safety oriented as the first option.

We also briefly discussed the head ejection effects on 36" tall F-shape barriers. Due to the bottom toe the barrier, vehicles impacting F-shapes tend to climb up the face of the barrier and roll away from the barrier. As such, head ejection is not a problem for safety shaped barriers (NJ and F-shape). Side note: The vehicle roll and climb that eliminates concerns for head slap is the same instability issues that have led to vehicle rollovers and the main reason that we now recommend single slope and vertical faced barriers over safety shaped barriers.

---

chain link on temp concrete barrier

Question

State: OH
Date: 04-16-2014

Are there any existing crashworthy standards for chain link mounted on top of temp concrete barrier? A contractor has proposed the version attached. I'm not sure if they've made this up on their own, or if it was tested and used elsewhere.

Thanks!

~Maria

614-466-2847

Attachment: http://mwrsf-qa.unl.edu/attachments/b05ab753c82f45b2ed150f2d324f99dc.JPG

Response

Date: 04-22-2014
To the best of our knowledge, no testing has been conducted on chain link fencing attached to temporary concrete barrier. Previous research at TTI was conducted on a chain link fence mounted on a permanent concrete barrier under PL-2 impact conditions. This would be considerably different in terms of barrier deflection and vehicle extension over the barrier as compared a TCB system evaluated under current test criteria.

There are concerns with mounting fence structure on TCB's. First, the vehicle may interact with the fence structure causing snag. This may pull the fence down on the vehicle or cause deceleration or instability of the vehicle that is undesirable. it is essentially a zone of intrusion issue with the vehicle interacting with the fence. TTI has tested sign supports attached to TCB successfully, but the vehicle did interact significantly with the sign support. A second concern with a fence structure is that it may affect the performance and deflection of the TCB.

The fence structure you have shown may potentially work. However, it is difficult to determine its safety based on the concerns above without further investigation and analysis, and I would lean towards crash testing to evaluate this type of attachment.

Attachment: http://mwrsf-qa.unl.edu/attachments/96dd457a160660769ea05ec0440ec0cb.pdf

---

Downstream Bridge Attachment

Question

State: MO
Date: 04-22-2014

MoDOT usually leaves the downstream ends of bridge on divided highways unshielded. The fill slopes associated with many bridges, however, warrant guardrail on the downstream end. Several of these locations are older and consist of a W-Beam mounted directly to the concrete parapet (as shown in the attached photo). Since a crash in this situation would involve exiting a rigid structure and entering a semi-rigid environment, pocketing does not seem to be a concern. I do have a concern about rail rupture if a crash were to bend the rail around the sharp concrete corner.
Is it acceptable to leave the installation described above in place?

Response

Date: 05-06-2014
We have had similar inquiries in the past regarding guardrail on the trailing end of a bridge. As noted, the concern for pocketing associated with approach transitions is low. However, the answer to the amount of load in the rail and the anchorage of the w-beam to the bridge rail has not been fully defined. See below.

http://mwrsf-qa.unl.edu/view.php?id=338

We have not previously addressed the potential for rupture of the W-beam rail due to increased tensile loading and/or bending of the rail around the end of the bridge, but the concern may be valid. This concern could be mitigated somewhat by the location of the first post downstream of the bridge rail. By placing the first downstream post closer to the end of the bridge rail, the propensity for the rail to be bent around the end of the bridge would be lowered. Thus it may be worth considering placement of the first post 3.125 ft (quarter post spacing) or less from the end of the downstream bridge end. However, rail tensile loads may still be relatively high.

The some state DOT's have observed W-beam guardrail tears in vehicle accidents downstream from a stiff point in the barrier system (e.g., a larger post or a post installed in asphalt/concrete). These situations can be similar to the departing ends of concrete bridge rails on one-way roadways where the W-beam is directly attached to the parapet. In this scenario, the rigid concrete barrier will not deflect, thus potentially producing high tensile and/or shear forces in the rail at the edge of the rigid parapet that may result in tearing or rupture. Thus, full-scale crash testing may be necessary to fully evaluate this concern.

Illinois proposed this as a problem statement in Year 22 of the Midwest Pooled Fund, but it was not funded.

---

concrete strenght for post mounted on concrete

Question

State: WI
Date: 04-23-2014

I found this in TRP-03-114-02:

“The concrete used for the culvert's top slab and curb consisted of a Nebraska 47-BD Mix with a minimum compressive strength of 31.03 MPa (4500 PSI). The actual concrete compressive strength for the culvert's top slab and the curb on test day, as determined from concrete cylinder testing, were found to be approximately 6992 MPa and 41.64 MPa (6039 PSI), respectively."

From my understanding our structures department designs concrete using f'c=3,500 psi.

For the adhesive option when the slab is 10" what is the minimum f'c?

For the bolt through option is there a minimum slab thickness and what should the minimum f'c be for the slab?

Response

Date: 04-24-2014

With the through bolt option, the thickness of the slab shouldn't make a difference to the attachment strength. I say this assuming you do not have slab thicknesses less than 8" (the minimum found during a recent review of state DOT culvert standards) and you utilize the bottom washer plate on the under side of the culvert top slab.

Report TRP-03-278-13 recommends a minimum f'c of 4,000 psi for use with the epoxy anchorage option (8" embedment depth with a minimum epoxy bond strength of 1,300 psi). Changes to the minimum f'c could be dealt with by variations of embedment depth and/or epoxy strength. Using the design procedure for epoxy anchorages found in ACI-318-11, you would first calculate the strength of the recommended anchorage design, then lower the f'c and adjust the embedment depth and the epoxy strength until the estimated strength is equal to or greater than the original design.

Response

Date: 04-24-2014

If everything else was the same:

What depth of embedment would I need with the same bond strength and f'c=3500 concrete?

What type of bond strength would I need with the same embedment and f'c=3500 concrete?

Response

Date: 04-24-2014

using the ACI-318 procedures by the book: a concrete with f'c = 3500 requires 8.5" of embedment with an epoxy strength of 1300 psi. The failure mode is calculated as a concrete breakout failure, so 3500 psi concrete with a stronger epoxy will not produce an equivalent anchor strength to the 4000 psi concrete without a deeper embedment depth.

In report TRP-03-264-12, some dynamic loading coefficients where proposed to account for the increase in material strengths during a dynamic loading event. Note, the ACI code was written for statically loaded connections. Using these coefficients for steel, concrete, and epoxy strength under dynamic conditions, the failure mode for the anchors is epoxy bond failure for either concrete strength, f'c = 3500 psi or 4000 psi. Thus, if you wanted to use these dynamic coefficients, you would not need to change anything for a 3500 psi concrete slab.

Response

Date: 04-25-2014

For the bolt through option, what is the f'c that the concrete was designed for? Or does that not matter because the loading is going to be dynamic as well?

Response

Date: 04-25-2014

The mix was specified as a 4,500 psi concrete mix. However, utilizing a 3,500 psi mix will probably have minimal effect on the performance and damage. Make sure you have a minimum slab thickness of 7" (the thicker the better for weaker concrete), the slab has adequate reinforcement (similar to the as tested culvert slab - shown in TRP-03-114-02), and use the large bottom washer plate to distribute the load out over a wide area.

Response

Date: 04-25-2014

I've been working with our structural department on implementing the bolt through and the adhesive post systems for beam guard.

Below is some more question that they have. Could you please answer their questions below?

Looking at our culvert standard, it appears we may have less reinforcement in the top slab than the tested top slab. Does this mean we go to 8" min. and 3500 psi for the thru bolt system? Aren't they supposed to be providing a clear definition of min. requirements to meet this TL-3 level in a document form rather than a back and forth guessing game? When Don Faller (hope I got his name correct) completed his crash test analysis of a timber rail anchored to a concrete slab, he published a drawing with all pertinent details that were required (on-line), and all we had to do was select the bar steel in the slab near the anchors. I don't see that detail being made available this time.

Response

Date: 04-25-2014

From your comments below, it appears that you are talking about future culvert and guardrail installations. My mistake on earlier responses – I assumed you were asking about existing structures.

To answer your minimum top slab strength question: You can utilize the design from the report as a baseline – 7" thick slab, f'c = 4,500 psi, and reinforcement as shown in the report. If you want to use a 3,500 psi concrete mix, the slab should be thickened and/or additional reinforcement should added until the bending strength of the slab is equivalent (or greater). Now, if you want to utilize the epoxy anchorage attachment, the minimum embedment depth is 8", as detailed previously. As such, a thicker slab will be required for the epoxy anchorage detail.

---

10 Gauge Thrie Beam on Bridge Anchor

Question

State: MO
Date: 05-05-2014

MwRSF has tested a number of steel guardrail-to-concrete-barrier (bridge or median) transitions. Most of the designs call for a length of double-nested 12 ga. thrie beam panel in advance of the concrete. Is a length of single 10 ga. panel a reasonable alternative to this design?

Response

Date: 05-06-2014
Testing has been conducted on a variety of thrie beam approach guardrail transitions using both nested 12 gauge and 10 gauge rails. Review of that testing has shown no concerns with the use of 10 gauge thrie beam in lieu of nested 12 gauge. Thus, it is believed that either configuration is acceptable.

---

cable barrier line post specification

Question

State: WI
Date: 05-12-2014

I'm updating my specification for cable barrier line post footing. I would like to incorporate some of the information that MwRSF has done with bogie testing of socketed foundations.

What I want to specify is:

“Provide line post anchor design that limits line post footing movement to “X" inches, when a dynamic force of “Y" is applied to the strong axis of the post at height “H" above the ground."

Unfortunately, I don't remember X,Y or H. I believe that the strong axis is the one that we want the load to be applied to, but I could be wrong with that as well.

Response

Date: 05-22-2014
The critical impact that we at MwRSF have selected is:

An impact height, H, of 11" to represent a small car bumper

Restrict foundation movement, X, to less than 1" to prevent resetting of foundation during system repairs

Posts were impacted to create strong axis bending and continued until the bogie overrode the post. The specific load magnitude, Y, will vary depending on the post. For an S3x5.7 post, the peak load has typically been between 17 and 20 kips. For the new cable post, the MWP, peak loads were between 8 and 10 kips.

MGS Box Culvert Mounting on Tall Headwall

Question

State: WY
Date: 05-19-2014

We are rehabbing some box culverts and attempting to incorporate the recently developed MGS Low Cost Box Culvert/Bridge Rail Design as a replacement railing. The particular box culvert in question will require adding a taller headwall to provide a 1V:10H slope into the face of the side mounted MGS Railing. Our Bridge Personnel would like to know if it is better to design the sockets so that the bottom connection is into the top of the box, rather than the headwall since it is rather tall. This design is shown in the attached drawing. Or is it O.K. just to stay with the standard design? In any case we would like to standardize the length of the posts.
Attachment: http://mwrsf-qa.unl.edu/attachments/1c171b7a27998130ecdd223219410168.docx

Attachment: http://mwrsf-qa.unl.edu/attachments/101fd4e46d6523a460e7941dde84b95f.pdf

---

Response

Date: 05-23-2014

Your proposed design, shown in the PDF "Preliminary Design", should work great as a retrofit to the original weak-post guardrail to culvert attachment system. By extending the length of the socket with the height of the headwall, you will limit/reduce the lateral load transferred into the headwall by the top anchor rod (longer moment arm between tension at top of socket and compression at bottom of socket equals lower loads with same bending strength in post). Since it sounds like you are retrofitting the headwall to an existing box culvert, limiting the loads imparted to the headwall and the connection to the culvert is probably the best course of action.

Footing for road closure gate

Question

State: NE
Date: 05-22-2014

Please forward me the testing of the Road Closed Gates.
I need a footing design of the road closure gates.

Response

Date: 05-22-2014

http://mwrsf.unl.edu/researchhub/files/Report128/TRP-03-44-94.pdf

Attached is the report for the road closure gate testing we did on the South Dakota system. I don't believe that we tested the system with a specific footing design. Bernie Clocksin at SDDOT should have details on the appropriate footing. We also dealt with an alternative footing detail for Kansas.

See herehttp://mwrsf-qa.unl.edu/view.php?id=287

---

cable diameter on Bullnose

Question

State: WI
Date: 05-23-2014

I was reviewing my drawings against MwRSF's report drawings and noticed on TRP-03-095-00 that figure G-8 Cable Details and Cable plates call out for a 15.875 diam. 7x19 wire rope.

This converts to 5/8" diameter. Is 5/8" diameter what you want here or would 3/4 inch wire cable (like what we use in the BCT cable) appropriate here?

If 5/8 wire rope is what is needed what ASTM should the cable follow?

Also the cable clips, what ASTM should they be?

Response

Date: 05-27-2014
We have used a 5/8" diameter 6x19 or 7x19 XIPS IWRC wire rope in the tested and approved bullnose system.

Based on that testing, 5/8" diameter rope of that grade is sufficient. End users could specify larger 3/4" diameter 6x19 or 7x19 IWRC XIPS rope. However, the end button ferrules may increase in size and be more difficult to fit behind the cable anchor plates.

Carbon steel wire rope is manufactured in various grades, including Improved Plow Steel (IPS), Extra Improved Plow Steel (EIPS or XIPS) and Extra Extra Improved Plow Steel (EEIPS or XXIPS). The EIPS or XIPS grade is most commonly used for the 5/8" rope, it should have a breaking strength of 20.6 tons or 41.6 kips.

An Independent Wire Rope Core (IWRC) consists of a smaller wire rope core within the strands of the outer wire rope.

The u-bolts are ASTM A307 or equivalent.

Long Span Guardrail Using Thrie-Beam

Question

Date: 05-27-2014

I was asked a question by a Wisconsin contractor who is being asked by the Wisconsin resident engineer on a project to use thrie-beam in what they are referring to as a long span installation. The job has both w-beam (31") and thrie-beam installations and on one of the thrie-beam runs they encountered a concrete footer of one of the bridge piers. (I believe it is from a railroad bridge.) They are estimating the span to get by this concrete footer is between 13' to 20'. As I understand the contractor, the resident engineer wants him to install this similarly as that of the w-beam long span MGS, except using thrie-beam. The contractor is questioning this and is looking for some guidance as to whether this is the proper type installation or is there another alternative. I can see what the resident engineer is wanting to do and does seem to make some sense, but like the contractor I too cannot find anything to really support this in an installation using thrie-beam. Can you offer some guidance and advice on proper installation in such a situation.

Response

Date: 05-27-2014

The W-beam long span and MGS long span systems can accommodate 13 to 20 ft obstructions below grade. However, the dynamic deflections and working widths for these two systems are larger than observed for the baseline guardrail systems without posts removed. You note the use of a 31" W-beam guardrail as well as thrie beam guardrail. We have not yet developed long-span systems for use with thrie beam. However, there exists a potential for this type of system to meet impact safety standards. Improved performance would likely be achieved with the use of similar post embedment depth to MGS (40"), deep blockouts (12" versus 8"), and shortened thrie beam blockouts similar to those used in bullnose system as well as the newer thrie beam approach guardrail transitions. The guardrail height may need to be slightly higher than 31" and as used in some of the prior successful thrie beam crash tests under 350. With these considerations, there would be a more reasonable chance that a thrie beam long-span system would meet AASHTO MASH or NCHRP 350. Of course, this specific system has not been crash tested, and crash testing is the best method for evaluating crashworthiness.

These are my preliminary thoughts on this topic. I will scan through our Pooled Fund Consulting Website to see if we have provided similar guidance in the past.

Increased Load Height for Guardrail

Question

State: MO
Date: 05-29-2014

A safety project on a narrow two-lane road will require certain locations to be shielded with guardrail. On this particular road, there is neither a shoulder nor any useable flat area to offset the barrier from the edge of traveled way. A 12 in. offset to the face of the rail was chosen so as not to further constrict the width of the traveled way which is as narrow as 9 ft. in certain locations. As shown in the attached figure, a 2H:1V inslope begins immediately at the edge of pavement, resulting in nearly 1.25 ft. of additional exposure/decreased embedment.

Since this arrangement would yield more post exposed than embedded, it is not likely to be acceptable at the 7 ft. post length as shown.

Question: Could a guardrail system with 9 ft. posts on 3 ft. 1-1/2 in. spacing, placed 24 in. down a 2H:1V slope be reasonably expected to perform at MASH TL-3?

Attachment: http://mwrsf-qa.unl.edu/attachments/467b68588cc709967e26fd5d04d374f3.bmp

---

Response

Date: 06-05-2014

We would agree that the system shown in the attached detail would not likely perform acceptably under MASH TL-3 impact conditions based on current testing of the MGS system adjacent to slopes at both MwRSF and TTI. The main issues are the rail height, the post offset down the slope, and the overall post length/embedment.

Current TL-3 MASH testing of the MGS system has shown acceptable safety performance when tested with:
1. Standard post spacing, 12" deep blockouts, 9 ft long posts installed with the post at the slope break point of a 2:1 slope, and a 31" rail height, as tested at MwRSF.
2. Standard post spacing, 8" deep blockouts, 8 ft long posts installed with the face of the rail at the slope break point of a 2:1 slope, and a 31" rail height, as tested at TTI.

Testing of the MGS with standard post spacing, 12" deep blockouts, 9 ft long posts installed with the post at the slope break point of a 2:1 slope, and a 27.75" rail height did not meet the MASH criteria due to vehicle instability.

Based on these tests, there are concerns that the lower rail height, short embedment, and increased post and rail offset shown would not be acceptable due to concerns for vehicle capture, increased barrier deflections, and vehicle stability.

The use of 9' posts at 1/2 post spacing may improve concerns with barrier deflections, but the large rail off set and the rail height would still be a concern. No testing has been done to date with the larger proposed rail offset and concerns exist regarding vehicle capture and stability. The rail height would pose similar concerns. Thus, it is difficult to recommend that configuration with respect to MASH TL-3 safety performance.

Currently the most aggressive installation available at standard post spacing would be the MGS installed with standard post spacing, 8" deep blockouts, 8 ft long posts installed with the face of the rail at the slope break point of a 2:1 slope, and a 31" rail height. Rail heights below 31" and the larger offsets could not be recommended at this time. It should be possible to integrate this design with your existing guardrail by transitioning the guardrail from 29" to 31" top rail height over 25'-50' of guardrail as the area with the 2:1 slope is approached and employing the longer 8' posts and shorter blockouts tested in the TTI system. I have attached the TTI report on that system.

Attachment: http://mwrsf-qa.unl.edu/attachments/58e34bfb0dfe5ed53dd043eeb42eebdc.pdf

Socketed Foundations on Slope

Question
State: WI
Date: 06-11-2014

Cable barrier installations are often placed on cross-slopes, typically 6:1 and potentially 4:1 slopes. When installing socketed foundations for the posts on these slopes, the back side of the foundations often protrude a few inches above the ground line. Under the right conditions, this protrusion could result in a violation of the 4" maximum stub height established by AASHTO and the RDG. Is there a way to address this issue?

Attachment: http://mwrsf-qa.unl.edu/attachments/13e47d5cb5eac4064fa96f80e8e66ccd.jpg

---

Response
Date: 06-11-2014

To minimize the extent of the foundation protruding above ground line, it is recommended that the foundation be installed with the center of the foundation (top) be installed level with the surrounding terrain, as shown in the attached sketch. This position also ensures the post and cables are installed at the correct height relative to ground line.

Attachment: http://mwrsf-qa.unl.edu/attachments/f40bdd0c798b1c4ae2d10ae1c6f120b7.jpg

---

Nebraska Tubular Thrie Beam Bridge rail

Question

State: NE
Date: 06-18-2014

NDOR previously received guidance regarding the Nebraska Tubular Thrie beam bridge rail. Can you resubmit that inofrmation.

Response

Date: 06-18-2014

I found the original report from 1987. I have attached 2 pages for design details from the report. In addition, I have attached a sampling of prior correspondence on this matter. Ralph Hansen of Speece-Lewis has updated this railing over the last 5 years or so. I do not have a copy of their final version.

Attachment: http://mwrsf-qa.unl.edu/attachments/82195c3027d14bdfb956cfff4e4ee8bd.zip

---

Wire strand reinforcement

Question

Date: 06-26-2014

color:#002060">I request your opinion on the attached Colorado DOT standard detail.

color:#002060"> Question: Have you folks crash tested R.C. barrier using wire strand

color:#002060"> (see note 8 sheet 1of4).

Attachment: http://mwrsf-qa.unl.edu/attachments/c5c946e3551d96b6cf6026beef166b4f.pdf

---

Response

Date: 06-27-2014

We have never tested barriers using wire strand as the reinforcing steel but we believe it may be feasible. The closest thing we have seen to this is pre-stressing strands used in reinforced concrete design. in this case they are only used as reinforcement.

The strength of the specified strand appears to be equal or greater than the Grade 60 rebar, so that should not be an issue. We cannot see any direct reasons why it would not work, but we can note some potential differences between rebar and the wire strand.

1. Development length may differ for the wire strand as compared to rebar and thus may change the interaction or bond to the concrete somewhat. It is difficult to say how different that would be, but it may affect the performance of the reinforcement to some level. You would need to consider this when splicing the strands or other areas where development of the reinforcement was critical.

2. The modulus or elastic stiffness of the wire strand will be considerably less than that of the rebar due to the construction of the strand. Thus, while the strength of the rebar and strand are similar, the stiffness of the wire strand would be less. As the reinforcement severs mainly as a tension member, this may not affect ultimate capacity, but may change the flexural and longitudinal stiffness for the reinforced concrete section.

3. The difference in the modulus of the rebar and the strand may lead to increased cracking under flexural loading and temperature shifts.

Let me know if we can help you further.