IOWA DEPARTMENT OF TRANSPORTATION

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DATE: Nov. 8, 2011

PROJECT : Floyd County
Bridge Deck Overlay
Guardrail Retrofit

SUBJECT: Preliminary Design for Guardrail Retrofit – US 218 Bridge over Drainage Ditch

This project involves the deck overlay on the US 218 Bridge over a Drainage Ditch, 0.4 miles south of the Mitchell County Line. The guardrail needs to be replaced, preferably in a way that does not require extensive deck reconstruction.

MGS Bridge Rail (Existing Crash-Tested Barrier)

The Midwest Roadside Safety Facility (MwRSF) Report No. TRP-03-226-10 addresses the development of a low-cost, energy-absorbing Bridge Rail (designated the MGS Bridge Rail). As noted in Section 3.1, “a post system was needed that would transmit loads into the bridge deck without causing damage during most impacts.” The designs were therefore based on the post yielding before the attachment to the deck was damaged.

The design selected and tested in this report utilized a S3x5.7 post which was placed in a socket attached to the edge of the bridge deck. A standard weak-post connection from guardrail to post was selected. As noted in Section 4.1, the components were tested “to verify that appropriate resistive forces would be developed and sufficient energy would be absorbed by the system to safely redirect the vehicle.” For the initial impact tests, this basic design is MGSBRB-7. The full-scale crash tests performed in Sections 10 and 11 showed this system was acceptable according to the TL-3 safety performance criteria.

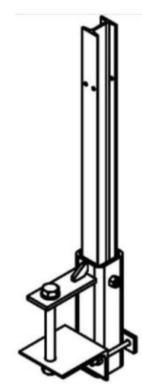

Fig. 1 - MGS Bridge Rail Post

The vertical, through-deck attachment bolt requires additional bars in the deck reinforcement to prevent the concrete from shear failure. It is noted in Section 4.4.3 that due to this special reinforcement, “an effort to develop a connection that could be retrofitted to existing decks . . . was discontinued.”

It appears this bridge deck post can be treated as two separate components. The vertical tube used to support the S3x5.7 post and the post itself was successfully crash-tested to perform as needed for a TL-3 barrier. The desirable outcome of a guardrail crash is that the tube and deck attachment will be undamaged, allowing the guardrail to be easily replaced. Therefore it is proposed that the tube and post assembly be used as developed in the MwRSF report, and the attachment to the deck be modified as necessary for this retrofit situation.

It is proposed that the tube be welded to a backing plate which is then bolted to the side of the deck. The Floyd US 218 deck is a continuous concrete slab, with an edge deck depth of 17” (see original plan set FN-369), allowing plenty of depth for multiple rows of bolts if necessary. The MwRSF Report tested a similar attachment in Section 4.4.1, designated as MGSBRB-6, which failed by tensile force in the anchor bolts and deformation of the backing plate. That design used only two ¾" rods and a ¼" plate.

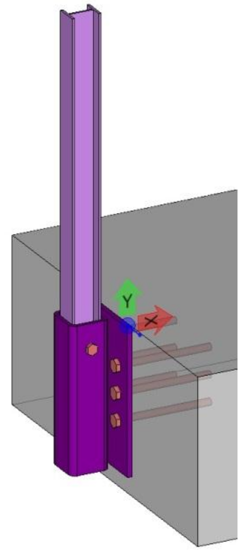

Fig. 2 – Proposed Bridge Rail

Concerning welds, the MGS Bridge Rail used $^3/_8$" welds to attach the tube to the mounting brackets. In the full-scale crash test, one mounting bracket was destroyed when the pickup wheel snagged on the mounting tube. It is recommended in Section 14 that increasing the size of the weld may help avoid weld failure. However, since our proposed attachment would place the tube level with the deck, the wheel snag should not be a problem.

Proposed Barrier Attachment

To design a new bracket assembly for the barrier attachment to the deck, the required moment resistance must first be determined.

The S3x5.7 section has an F_y = 36 ksi and Z_x = 1.94 in^3. This gives a plastic bending moment of:

$$M_p = F_y * Z_x = 69.8\ kip \cdot in = 5.82\ kip \cdot ft$$

Using the 1.7 Live Load Factor from ACI 318-11 App. C and an Impact Height of 25.375 in. above the top of the tube, the factored force to generate this moment is:

$$1.7 * P\ =\ 1.7 * M_p/d = 1.7 * \frac{69.8\ kip \cdot in}{25.375\ in} = 4.68\ kips$$

However, the tube assembly must also withstand the moment throughout the failure of the post. From the Bogie Impact Tests in the MwRSF report, the Peak Force experienced by test MGSBRB-7 was 6.7 kips, at an Impact Height of 24.875 in. above the deck.

$$M_U = P * d = 6.7 kips * 25.375\ in = 167\ kip \cdot in = 14.2\ kip \cdot ft$$

It is assumed that the bracket assembly must resist this Peak Force (6.7 kips) at the Impact Height. Since this force is larger than the factored plastic bending moment force, this assumption is conservative.

The proposed plate is 11" wide by 15" tall by 0.625" thick. Three pairs of $^3/_4$" anchor rods (A_b = 0.334 in^2) are proposed (see Figs. 2 and 3). Vertically, the pairs of rods are located at 5.00", 7.75", and 10.50" from the bottom of plate. Horizontally, the rods are offset 2" from the edge of plate. These locations satisfy the edge limits in LRFD 6.13.2.6 and AISC J3.3 for steel as well as the requirements in ACI 318-11 Sec. D.3.3 for concrete anchors.

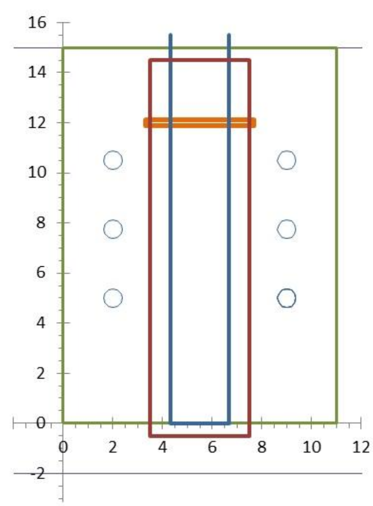

Fig. 3 - Bolt Layout

Torsional moment is neglected. It is assumed that the guardrail will limit the rotation of the post before yielding occurs.

Anchor Rod Tensile Strength

The Neutral Axis for the Bolt Group and Plate is found using the force and moment equations by trial and error.

$$\sum F_x = -6.7\ kips\ applied\ load$$

$$\sum M_z = -6.7 * (24.875" + 15" - d_{NA})\ k \cdot ft\ moment\ due\ to\ applied\ load$$

By trial and error, d = 2.121" from the bottom of the plate.

This gives a value of c = 8.379" from the neutral axis. The Moment of Inertia I_x of the bolt group and compression block about the Neutral Axis is:

$$I_x = \sum A * d^2 + \sum I_{x-i}$$

$$I_x = 0.334 in^2 * 2 * [(10.5" - 2.12")^2 + (7.75" - 2.12")^2 + (5" - 2.12")^2] + \frac{4" * 2.12"^3}{3}$$

$$I_x = 86.3\ in^4$$

Assuming the post and tube act as a unit, the moment about the neutral axis is:

$$M_{U,NA} = P * d = 6.7 kips * (24.875 in + (15 in - 2.12 in) = 253.0\ kip \cdot in = 21.1\ kip \cdot ft$$

The tensile force per bolt is:

$$r_{ut} = \frac{M_{U,NA} * c}{I_x} * A_b = \frac{253.0\ kip \cdot in * 8.38\ in}{86.3\ in^4} * 0.334\ in^2 = 8.20\ kips$$

The available tensile strength of the anchor bolts is obtained from AISC Equation J3-1.

$$\emptyset R_n = \emptyset * F_n * A_b$$

$$\emptyset = 0.75$$

From Table J3.2, the F1554 Anchor Bolt meets the requirements of AISC A3.4, and it is assumed the threads are not excluded from the shear plane. Therefore:

$$F_n = F_{nt} = 0.75 * min(F_U, 1.9 * F_Y)$$

For F1554 Anchor Bolts, Grade F_y=36 ksi, the lower limit of F_u is 58 ksi. Therefore:

$$\emptyset R_{nt} = 0.75 * min(0.75 * 58\ ksi, 1.9 * 35 ksi) * 0.334\ in^2$$

$$\emptyset R_{nt} = 10.9\ kips$$

$$(\emptyset R_{nt} = 10.9\ kips) > (r_{ut} = 8.20\ kips)$$

OK: The Tensile Strength is sufficient for the load.

Anchor Rod Shear Strength

The Shear force applied on the bolts is the weight of the post and guardrail.

Component	Weight/Unit	Size	Weight
Guardrail	6.82 lb/ft	3.25 ft	0.022 kips
Post	5.70 lb/ft	3.92 ft	0.022 kips
Tube	17.20 lb/ft	1.29 ft	0.022 kips
Thru-Bolt	1.05 lb/cf	0.50 ft	0.001 kips
Plate	490 lb/cf	0.060 cf	0.029 kips
		Total, P_u:	0.096 kips

From AISC pg. 7-8, the shear force is distributed among the bolts.

$$r_{uv} = \frac{P_u}{n} = \frac{0.096\ kips}{6\ bolts} = 0.016\ kips/bolt$$

The available shear strength of the anchor bolts is obtained from AISC Equation J3-1.

$$\emptyset R_n = \emptyset * F_n * A_b$$
$$\emptyset = 0.75$$

From Table J3.2, the F1554 Anchor Bolt meets the requirements of AISC A3.4, and it is assumed the threads are not excluded from the shear plane. Therefore:

$$F_n = F_{nv} = 0.4 * F_U$$

For F1554 Anchor Bolts, Grade F_y=36 ksi, the lower limit of F_u is 58 ksi. Therefore:

$$\emptyset R_{nv} = 0.75 * 0.40 * 58\ ksi * 0.34\ in^2$$
$$\emptyset R_{nv} = 5.81\ kips$$
$$(\emptyset R_{nv} = 5.81\ kips) > (r_{uv} = 0.016\ kips)$$

OK: The Shear Strength is sufficient for the load.

NOTE: As the Shear Load is insignificant, the concrete failure due to shear force on the anchor bolts is not investigated.

Anchor Rod Concrete Breakout Strength

From ACI 318-11 Sec. D.4.2.3, the effective embedment depth h_{ef} is limited as no less than 4*d_a and no greater than 20*d_a for this bond model. Assume an effective embedment length of 12 in.

$$(4 * d_a = 3") \leq h_{ef} = 12" \leq (20 * d_a = 15")$$

Since the applied force satisfies ACI 318-11 App. C and supplementary reinforcement is not provided (Condition B), the strength reduction factor for tension, ϕ=0.65 as shown in ACI D.4.4 (post-installed anchor, Category 2 medium reliability).

The distance from the top bolts to the top of concrete is c_{a1} = 4.5".
The distance from the top bolts to the top of concrete is c_{a2} = 7.0".
Therefore, $c_{a,min}$ = 4.5".

The nominal concrete strength for a group of anchors is given in ACI D.5.2, Equation (D-4).

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} * \Psi_{ec,N} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$$

$$A_{Nco} = 9 * h_{eff}^2 = 9 * (12")^2 = 1296\ in^2 \qquad \text{(D-5)}$$

$$A_{Nc} = \left(1.5 * h_{ef} + s_1 + 1.5 * h_{ef}\right) * (c_{a1} + s_2 + c_{a2}) \leq n * A_{Nco}$$

$$A_{Nc} = (1.5 * 12" + 7" + 1.5 * 12") * (4.5" + 5.5" + 7") \leq 6 * 1296\ in^2$$

$$A_{Nc} = 731\ in^2 \leq 7776\ in^2$$

The basic concrete strength of a single anchor in cracked concrete, N_b, is given in (D-6).

$$N_b = k_c * \lambda_a * \sqrt{f'_c} * h_{ef}^{1.5}$$

For post-installed anchors, $k_c = 17$ (Sec. D.5.2.2).
For normal weight concrete, $\lambda_a = 1.0$.
For concrete deck, $f'_c = 4000$ psi.

$$N_b = 17 * 1 * \sqrt{4000} * 12^{1.5}/1000 = 44.7\ kips$$

$$e'_n = \left|\frac{M_{U,NA}}{n * r_{ut}} + d_{NA} - y_{cg}\right| = \left|\frac{251.6\ kip \cdot in}{6 * 8.0\ kips} + 2.33" - 7.75"\right| = 0.177"$$

Eq. (D-8):

$$\Psi_{\mathrm{ec,N}} = \left(\frac{1}{1 + \frac{2 * \mathrm{e'_n}}{3 * \mathrm{h_{ef}}}} = 0.990\right) \leq 1.0$$

Eq. (D-10):

$$\Psi_{\mathrm{ed,N}} = \left(0.7 + 0.3 * \frac{\mathrm{c_{a,min}}}{1.5 * \mathrm{h_{ef}}} = 0.775\right) \leq 1.0$$

For cracked concrete, $\Psi_{c,N} = 1.0$ (sec. D.5.2.6).
For adhesive anchors, $c_{ac} = 2*h_{ef} = 24"$ (sec. D.8.6).
Eq. (D-12):

$$\Psi_{\mathrm{cp,N}} = \left(\frac{\max(1.5 * \mathrm{h_{ef}}, \mathrm{c_{a,min}})}{\mathrm{c_{ac}}} = 0.750\right) \leq 1.0$$

$$\emptyset N_{cbg} = 0.65 * \frac{1296}{731} * 0.99 * 0.775 * 1 * 0.75 * 44.7\ kips$$

$$\left(\emptyset N_{cbg} = 9.43\ kips\right) > (r_{ut} = 8.00\ kips)$$

OK: The Anchor Rod design is sufficient to resist concrete pullout.

Anchor Rod Adhesive Bond Strength

From above, ϕ=0.65
From above, $c_{a,min}$ = 4.5".

The nominal bond for a group of anchors anchor in cracked concrete is given in ACI 318-11 Sec. D.5.5, Equation (D-19).

$$N_{cbg} = \frac{A_{Na}}{A_{Nao}} * \Psi_{ec,Na} * \Psi_{ed,Na} * \Psi_{cp,Na} * N_{ba}$$

The values for bond stresses may be found in the HILTI *Product Technical Guide*. It is assumed that the HIT-RE 500-SD Epoxy Adhesive Anchoring System is used. From HILTI Sec. 3.2.4, Table 9:
Bonding stress in cracked concrete, τ_{cr} = 345 psi.
Bonding stress in uncracked concrete, and τ_{uncr} = 714 psi.

Eq. (D-21)

$$c_{Na} = 10 * d_a * \sqrt{\frac{\tau_{uncr}}{1100}} = 10 * 0.75 * \sqrt{\frac{714}{1100}} = 6.04\ in$$
$$A_{Nao} = (2 * c_{Na})^2 = (2 * 6.04")^2 = 146\ in^2$$
$$A_{Na} = (c_{Na} + s_1 + c_{Na}) * (c_{a1} + s_2 + c_{Na}) \leq n * A_{Nao}$$
$$A_{Na} = (6.04" + 7" + 6.04") * (4.5" + 5.5" + 6.04") \leq 6 * 146\ in^2$$
$$A_{Na} = 306\ in^2 \leq 876\ in^2$$

The basic adhesive bond strength of a single anchor, N_{ba}, is given in (D-22).

$$N_{ba} = \lambda_a * \tau_{cr} * \pi * d_a * h_{ef}$$

For normal weight concrete, λ_a = 1.0.

$$N_b = 1.0 * 345\ psi * \pi * 0.75" * 12"/1000 = 9.75\ kips$$

From above, e'_n = 0.177".
Eq. (D-23):

$$\Psi_{ec,Na} = \left(\frac{1}{1 + \frac{e'_n}{c_{Na}}} = 0.972\right) \leq 1.0$$

Eq. (D-25):

$$\Psi_{ed,N} = \left(0.7 + 0.3 * \frac{c_{a,min}}{c_{Na}} = 0.923\right) \leq 1.0$$

For cracked concrete, $\Psi_{cp,Na}$ = 1.0 (sec. D.5.5.4).

$$\emptyset N_{ag} = 0.65 * \frac{306}{146} * 0.972 * 0.923 * 1 * 9.75\ kips = 11.92\ kips$$
$$\left(\emptyset N_{ag} = 11.92\ kips\right) > (r_{ut} = 8.00\ kips)$$

OK: The Anchor Rod design is sufficient to resist adhesive bond pullout.

Base Plate Thickness

From AISC pg. 9-5, the base plate must be at least match the shear rupture strength of the base metal (for a fillet weld with F_{EXX} = 70 ksi). The fillet weld is only on one side of the connection.

$$t_{min} = \frac{3.09 * D}{F_u}$$

where:

D = 5, assuming a $^5/_{16}$" weld is used
F_u = 58 ksi, tensile strength of element, assuming A36 steel.

Therefore,

$$t_{min} = \frac{3.09 * 5}{58} = 0.27"$$
$$(t_{min} = 0.27") < t = 0.5"$$

From AISC pg. 9-10, the prying action of the anchor rods on the plate must be considered. The minimum thickness to eliminate prying action is given as:

$$t_{min} = \sqrt{\frac{4.44 * T * b'}{p * F_u}}$$

where:

T = r_{ut} = 8.0 kips, tension force per bolt
b' = 1.13", distance from face of stem to edge of bolt hole
p = 2.75", tributary length per pair of bolts (in y-dir.)
F_u = 58 ksi, tensile strength of element, assuming A36 steel.

Therefore,

$$t_{min} = \sqrt{\frac{4.44 * 8.00\ kips * 1.13"}{2.75" * 58\ ksi}} = 0.50"$$
$$(t_{min} = 0.50") < (t = 0.50")$$

OK: The Base Plate is thick enough to resist prying action and element shear rupture.

Weld Thickness

From AISC pg. 8-14, the required shear resistance in the weld can be found by resolving the shear and moment shears into a single vector. It is assumed that the tube is welded to the plate the entire length of contact. This results in two 15" welds, located 4" apart on either side of the vertical tube. The welds are $^5/_{16}$" thick with F_{EXX} = 70 ksi. The neutral axis of the weld is located midway between the top and the bottom of the plate. Therefore:

$$M_u = P_x * e = 6.7\ kips\ *\ (24.875" + 15"/2) = 216.9\ kip \cdot in = 18.1\ kip \cdot ft$$
$$P_u = P_y = 0.096\ kips$$

$$I_p = \frac{L_w * (3 * b^2 + L_w{}^2)}{6} = \frac{15" * (3 * 4"^2 + 15"^2)}{6} = 682.5\ in^3$$

$$c = 15"/2 = 7.5"$$

$$r_u = \sqrt{r_p^2 + r_m^2}$$

$$r_{uw} = \sqrt{\left(\frac{P_u}{2 * L_{weld}}\right)^2 + \left(\frac{M_u * c}{I_p}\right)^2} = \sqrt{\left(\frac{0.096\ kips}{2 * 15"}\right)^2 + \left(\frac{216.9\ kip \cdot in * 7.5"}{682.5\ in^3}\right)^2}$$

$$r_{uw} = 2.384\ kips/in$$

The available strength is given in Sec. J2.4, and can be converted for unit length as follows:

$$\emptyset R_n = \emptyset * \frac{F_w * A_w}{L_{weld}} = \emptyset * F_w * t_{eff}$$

$$\emptyset = 0.75$$

$$F_w = 0.60 * F_{EXX} = 0.6 * 70\ ksi = 42\ ksi$$

$$t_{eff} = 0.707 * w = 0.707 * \frac{5}{16}" = 0.221"$$

$$\emptyset R_{nw} = 0.75 * 42\ ksi * 0.221" = 6.96\ kips/in$$

$$(\emptyset R_{nw} = 6.96\ kips/in) > (r_{uw} = 2.384\ kips/in)$$

OK: The weld is easily thick enough to resist required moment. The weld size could be decreased slightly if desired. The horizontal weld across the top of the tube has not been included in these calcs, but it will create additional strength at the point of maximum stress.

Hinge Bolt Thickness

The MGS Bridge Rail design calls for a 5/8" bolt to support the post in the tube. It does not appear from the crash-test that this bolt was a critical point for any of the attachments (it did not appear to fail in any post). This part of the design is therefore carried over from the MGS Bridge Rail design. The check below is from AISC, page 7-13.

Just before the post yields, the applied rail force at the top of the post will create a lever with the other end of the post against the side of the tube and the hinge bolt as the fulcrum (the post does not touch the tube at the top). It is assumed that both the post and the hinge bolt are sacrificed in a guardrail impact; both may fail as long as the tube remains essentially intact. The hinge bolt supports the post vertically; by the time the bolt or post web fails in an impact, the horizontal resistance will be provided by the side of the tube against the flange of the post.

The guardrail is attached $7^1/_8$" below the top of the post, or 47" - $7^1/_8$" = $39^7/_8$" from the bottom of the post (y_1). The hinge bolt is 12" from the bottom of the post (y_2).

In the MwRSF bogey tests, the maximum force exerted on the post was 6.7 kips. Therefore, using the moments about the bottom of the post, the force applied at the hinge bolt is:

$$V_{bolt} = \frac{P_x * y_1}{y_2} = \frac{6.7\ kips\ *\ 39.875"}{12"} = 22.3\ kips$$

Strength of Bolt:
The available strength of the bolt is given in AISC Table 7-1:

$$\emptyset F_{nv} = 11.0\ kips$$
$$(\emptyset F_{nv} = 11.0\ kips) > (V = 22.3\ kips)$$

NG: The Bolt fails in shear, which is acceptable as the bolt is sacrificial. Therefore, the force on the post web and the tube at this location is limited by ϕF_{nv} of the bolt.

The available strength of the tube is given in AISC page 7-13. In this equation, F_y refers to the yield strength of the tube. For a typical HSS section, F_y = 46 ksi.

$$\emptyset R_n = \emptyset * 1.8 * n * F_y * d * t$$
$$\emptyset = 0.75$$
$$\emptyset R_n = 0.75 * 1.8 * 1 * 46 ksi * 0.625" * 0.375"$$
$$(\emptyset R_{nw} = 14.6\ kips) > (\emptyset F_{nv-bolt} = 11.0\ kips)$$

OK: The Hinge Bolt will fail before the Tube Wall fails.

The available strength of the post web is given in AISC Eq. (J3-6a), considering deformation at factored loads. In this equation, F_u refers to the ultimate strength of the post. For an A36 post, F_u = 58 ksi. The clear distance, L_c, from the bolt hole to the top of the web (before the flange) for a S3x5.7 post is 1.21". The thickness of the post web is 0.17"

$$\emptyset R_n = \emptyset * 1.5 * L_c * t * F_u \leq \emptyset * 3 * d * t * F_u$$
$$\emptyset = 0.75$$
$$\emptyset R_n = 0.75 * 1.5 * 1.21" * 0.17" * 58\ ksi \leq 0.75 * 3 * 0.625" * 0.17" * 58\ ksi$$
$$\emptyset R_n = 13.4\ kips \leq 13.9\ kips$$
$$(\emptyset R_{nw} = 13.4\ kips) > (\emptyset F_{nv-bolt} = 11.0\ kips)$$

OK: The Hinge Bolt will fail before the Post Web fails. As noted above, the Hinge Bolt did not appear to fail on any post in the MwRSF crash testing.

References:

1. MwRSF, *Development of a Low-Cost, Energy-Absorbing Bridge Rail*, TRP-03-226-10, August 11, 2010.
2. AISC, *Steel Construction Manual*, 13^{th} Ed., 2005
3. ACI, *Building Code Requirements for Structural Concrete (ACI 318-11)*. 2011.
4. HILTI, *North American Product Technical Guide, Vol.2: Anchor Fastening Technical Guide*, 2011 Edition.
5. AASHTO, *AASHTO LRFD Bridge Design Specifications*, 5^{th} Edition, 2010.
6. Iowa DOT, *General Specifications*, 2009 Edition.