Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

Date:	July 18, 2012	**File:**	20102698.00.E.05.00
To:	Brian Chow, M.Eng., P.Eng.		
From:	Julien Henley M.A.Sc., P.Eng.		
Project:	Development of MFR Standard Curb Design Parameters		
Subject:	Classification of HSS Guide Retrofit Bridge Rail–Rev. 1		

MEMO

1 INTRODUCTION

The purpose of this technical memorandum is to classify the Side Mounted HSS Guide Retrofit Rail as either a CL-2 or CL-3 Barrier, based on the criteria included in Associated Engineering's (AE) Phase 3 Report, titled "*Guideline for Barrier Selection and Design*" and listed in **Table 1**.

Table 1
Minimum Required Barrier Resistance or Factored Barrier Design Force

Applied Force[1]	Containment Level		
	CL-1	CL-2	CL-3
Transverse Load, F_T, kN	40	60	120
Longitudinal Load, F_L, kN	20	20	40
Vertical Load, F_V, kN	20	20	20
Load Application Height, mm[2]	375 (Timber Curb) 450 (Steel Rail)	450	510
Minimum Barrier Height[2]	500	525	685

Notes:
1. When completing an analytical evaluation of a barrier, these forces represent factored forces and resistances should be calculated assuming nominal material strengths.
2. Height measured from travelled surface.

This memorandum briefly summarizes the findings of the experimental research conducted by the University of British Columbia (UBC), additional numerical analysis completed by AE, and makes a recommendation regarding barrier classification based on the tested and calculated resistance of the barrier.

2 UNIVERSITY OF BRITISH COLUMBIA EXPERIMENTAL RESEARCH PROGRAM

UBC undertook an experimental program to verify the static resistance of a standard Side Mounted HSS Guide Retrofit Rail configuration using a pseudo-static rate of load application. The following presents a brief summary of the test program. A complete description of the experimental program and results can be found in the report titled "*Experimental Evaluation of Concrete Decks with Guard Rail Systems*", April 2011, produced by UBC.

Figure 1 illustrates the tested Side Mounted HSS Guide Retrofit Rail. The rail was mounted on a 175 mm thick concrete panel, with the reinforcing and couplers matching the MFLNRO Drawings STD-EC-030 Series.

Figure 1
Steel Post Barrier Layout

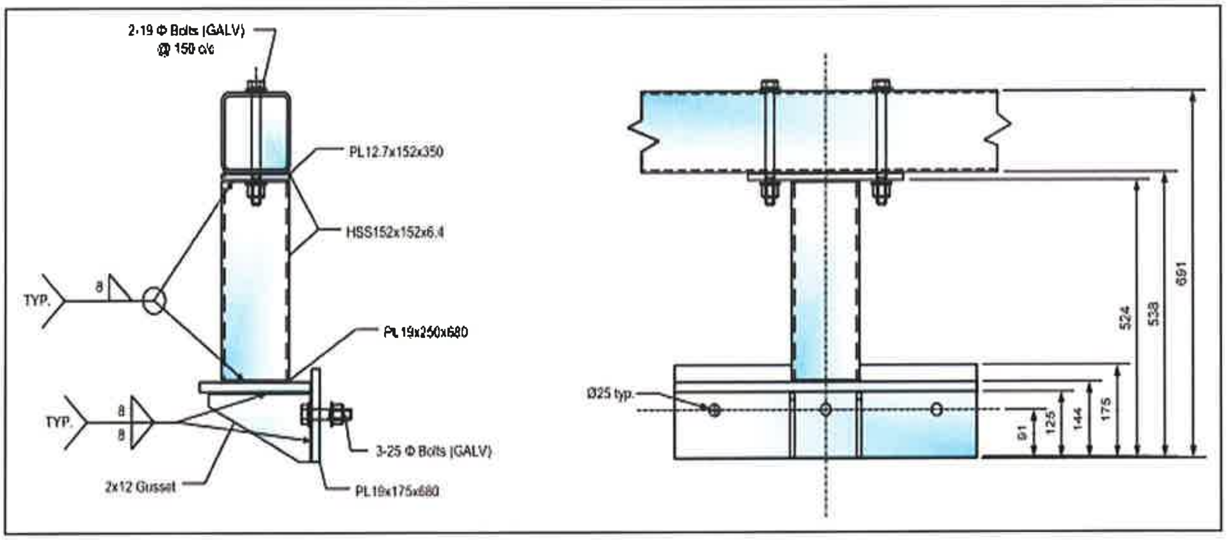


Table 2 presents the observed peak static loads recorded for each specimen during the experimental testing of the HSS Guide Retrofit Rail.

Table 2
Observed Peak Failure Loads and Associated Failure Mechanisms

Specimen ID[1]	Observed Peak Horizontal Load (kN)[2]	Comments
2.1	64.2	*Description:* A307 bolts connecting barrier and concrete panel. *Failure Mode:* Bolts ruptured in tension.
2.11	68.1	*Description:* A325 bolts connecting barrier and concrete panel. *Failure Mode:* Spalling of concrete in vicinity of inserts.
2.2	65.7	*Description:* A307 bolts connecting barrier and concrete panel. *Failure Mode:* Bolts ruptured in tension.
2.21	57.3	*Description:* A325 bolts connecting barrier and concrete panel. *Failure Mode:* Spalling of concrete in vicinity of inserts.

Specimen ID[1]	Observed Peak Horizontal Load (kN)[2]	Comments
2.3	56.4[3]	*Description:* A307 bolts connecting barrier and concrete panel, Vertical/Horizontal loading ratio = 1.08:6. *Failure Mode:* Bolts ruptured in tension.
2.4	54.9[3]	*Description:* A307 bolts connecting barrier and concrete panel, Vertical/Horizontal loading ratio = 1.08:6. *Failure Mode:* Bolts ruptured in tension.

Notes:
1. The specimen ID references correspond with those assigned by UBC in the report "Experimental Evaluation of Concrete Decks with Guard Rail Systems", April 2011.
2. Load applied 425 mm above travelled surface.
3. Values provided reflect applied horizontal load.

In summary, the test results include the following:

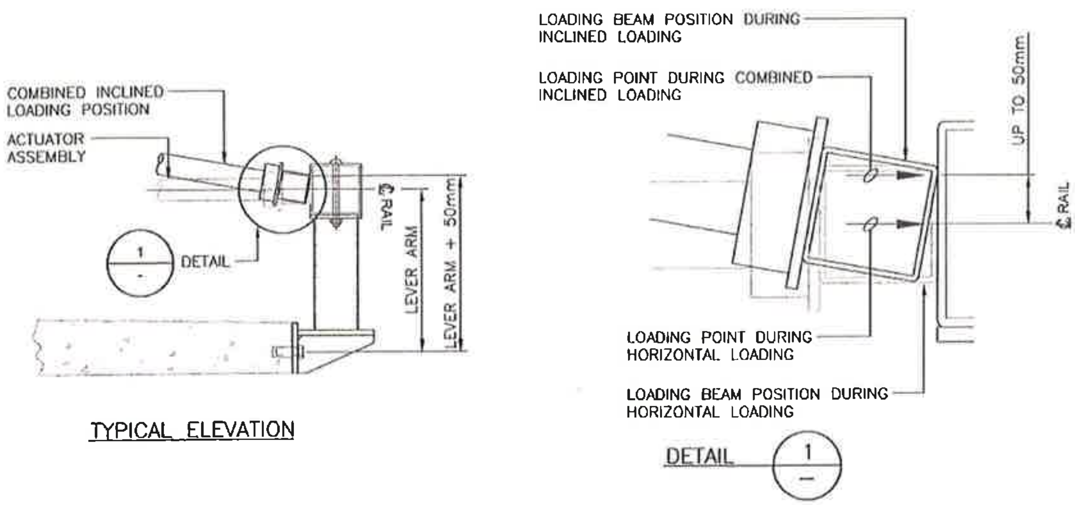
- We observed peak horizontal loads of 57.3 kN and 68.1 kN and failure of the concrete deck in compression in the two specimens that incorporated Grade A325 anchor bolts. We can likely attribute the difference in the observed peak horizontal loads (approximately 19%) to variation in the edge compressive strength of the deck panel.

- We observed a peak horizontal load of 65.7 kN and 64.2 kN, and rupture of the anchor bolts with limited damage to the concrete deck in the two specimens that incorporated Grade A307 anchor bolts and were subject to a horizontal load only.

- We observed a peak horizontal load of 56.4 kN and 54.9 kN, and rupture of the anchor bolts with limited damage to the concrete deck in the two specimens that incorporated Grade A307 anchor bolts and were subject to the simultaneous application of a horizontal and vertical load (vertical load was approximately 18% of the horizontal load). This is approximately 15% lower than the same tested configuration, when only a horizontal load was applied. The reduced capacity of this configuration could be attributed to:

    - An increase in the height of the lever arm as a result of the inclination of the hydraulic ram as illustrated by Figure 2. Since the HSS loading beam was rigidly connected to the actuator an inclination of the hydraulic ram resulted in the load being applied through the top edge of the loading beam rather than as a uniform load across the loaded edge of the loading beam. Assuming that the loading beam was a 102 x 102 HSS section, the inclination of the hydraulic ram could have resulted in an increase of approximately 50 mm to the lever arm of the applied load. This increase in lever arm would result in a 10% corresponding reduction in the applied load since the demand on the anchor bolts is inversely proportional to the height of the applied load.

Figure 2
HSS Loading Beam and Loading Contact Point



- Increasing the bolt grade from A307 to A325 only provides a marginal increase in capacity (approximately 6% increase), but results in the concrete deck failing rather than the yielding or fracture of the anchor bolts. Based on the report, it is difficult to ascertain whether the concrete deck failed in compression or the inserts failed.

- The observed peak horizontal loads for Specimen 2.1, 2.11 and 2.2 exceed the minimum horizontal resistance (60 kN) listed in Table 1, while Specimens 2.21, 2.3 and 2.4 tested on average 56.2 kN or 7% below the specified load.

3 ANALYTICAL RESULTS

As part of our review, we performed a numerical analysis of the bracket to deck connection to determine the theoretical failure loads of the Side Mounted HSS Guide Retrofit Rail. We based the analysis on the assumption that the connection behaves in a similar manner to a column base plate or a concrete beam in flexure (with the anchor bolts and inserts acting as tension reinforcement). We determined the capacity of the connection by generating a moment curvature response based on the geometry and associated material properties of the assumed section. To generate the moment curvature response:

- We assumed a concrete strength of 56 MPa as reported by UBC.
- We assumed the minimum yield (Fy) and ultimate strengths (Fu) for the bolts based on the specified bolt grades.
- We limited the maximum stress in the 25M reinforcing insert to 296 MPa, the theoretical capacity based on the provided bond length.
- We used nominal material strengths, i.e. we did not account for material resistance factors i.e., Φs, Φc, and Φb = 1.0.
- We assumed that the bolts were centred 100 mm below the top of the deck panel as detailed on the MFLNRO Standard Drawing STD-EC-030-09. This results in an effective depth to the bolt of 75 mm when measured from the underside of deck. Notwithstanding, the experimental results suggested that the drip groove resulted in premature compressive failure of the concrete and we therefore considered two scenarios:
  - An effective depth of 75 mm based on the assumption that the chamfer and drip groove do not affect the capacity of the connection.
  - A reduced effective depth of 55 mm to account for the presence of the 20 mm chamfer and drip groove.

Table 3 summarizes that calculated capacity of the connection based on these assumptions.

Table 3
Calculated Capacity of the Tested HSS Guide Retrofit Rail Deck Connection (f'c = 56 MPa)

Bolt Grade	Effective Depth		Predicted Failure Mode
	55 mm[4]	75 mm[5]	
A307	33 kN	47 kN	Bolts yield/fracture
A325	35 kN	51 kN	Bond failure (inserts fail/ bar pullout)

Notes:
1. Grade A307 Bolt: F_y = 248 MPa, F_u = 414 MPa.
2. Grade A325 Bolt: F_y = 635 MPa, F_u = 830 MPa.
3. Assumed capacity of 25M reinforcing inserts: F_y = 296 Mpa.
4. Reduced effective depth to account for 20 mm chamfer and drip groove.
5. No reduction in effective depth to account for chamfer and drip groove.
6. Capacities calculated assuming nominal material strengths.
7. Load applied 425 mm above travelled surface to allow comparison with UBC test results.

A review of Table 3 indicates the following:

- The moment curvature analysis correctly predicts the failure mode i.e., failure of the 25M reinforcing insert or yielding/fracture of the bolt although the predicted capacities are lower than the observed peak horizontal loads.

- The analytical results for the A307 anchor bolts are significantly less than the peak horizontal loads observed during testing (Table 2, Specimen 2.1 and 2.2). This is likely due to the material strength variability since A307 bolts are classified as mild steel bolts (Fy = 248 MPa) hence, the variation in strength can be significant depending on the actual material used.

We also considered the effect of the simultaneous application of the horizontal and vertical load (approximately 18% of the horizontal load) and determined that it did not result in a significant reduction in the tensile capacity of the bolts and hence the capacity of the connection.

In addition to reviewing the tested configuration, we completed an analytical review of the Side Mounted HSS Guide Retrofit Rail mounted on standard L75/BCL-625, L100 and L150/L165 precast concrete panels with deck edge thicknesses of 175, 200 mm and 225 mm, respectively. Further, based on discussions with the Ministry, we considered a 680 mm (as tested) and 550 mm (proposed) wide bracket. Table 4 summarizes the results of this analysis based on a nominal 35 MPa concrete, assuming that the drip groove is relocated away from the deck edge to ensure that its presence does not result in a reduction in the capacity of the guardrail connection to the deck. Included in the summary are the calculated capacities for three failure modes:

- Bolts yielding or fracturing i.e. the capacity of the guard rail connection is governed by the strength of the anchor bolt.
- Bond failure i.e. the capacity of the connection is governed by the pull-out strength of the 25M x 450 long Grade 400 reinforcing bar insert.
- Failure of the 25M reinforcing bar inserts by yielding i.e. the design is modified to ensure that the 25M reinforcing bar insert can be fully developed.

Table 4
Theoretical Capacity of HSS Guide Retrofit Rail Deck Connection (kN) - (f'c = 35 MPa)

Bracket Width	Bolt Grade	Deck Edge Thickness (mm)			Predicted Failure Mode
		175	200	225	
680	A307	44	62	80	Bolts yield/fracture
	A325	47	63	79	Bond failure (inserts fail/ bar pullout)
	A325[9]	60	84	105	Inserts fail – bar yield
550	A307	41	58	76	Bolts yield/fracture
	A325	45	62	77	Bond failure (inserts fail/ bar pullout)
	A325[9]	57	80	102	Inserts fail – bar yield

Bracket Width	Bolt Grade	Deck Edge Thickness (mm)			Predicted Failure Mode
		175	200	225	

Notes:

1. Assumed concrete strength f'_c = 35 Mpa.
2. Anchor bolts are located 100 mm below the travelled surface of the deck and guardrail bracket depth matches the deck edge thickness.
3. Grade A307 Bolt: F_y = 248 MPa, F_u = 414 MPa.
4. Grade A325 Bolt: F_y = 635 MPa, F_u = 830 MPa.
5. Capacity of the 25M reinforcing insert (based on bond failure), F_y = 296 MPa.
6. Capacity calculated assuming that the drip groove is relocated away from the deck edge to ensure its presence does not result in a reduction in capacity.
7. Capacities calculated assuming nominal material strengths.
8. Load applied 450 mm above travelled surface.
9. To achieve this failure mode, the 25M reinforcing insert would need to be increased in length (or modified) to provide sufficient bond length to allow development of the yield strength of the bar.

A review of Table 4 indicates:

- The bracket width can be reduced without resulting in a significant reduction in capacity.

- The capacity of the 25M reinforcing insert (pull-out resistance) is similar to the capacity of the A307 bolt (tensile resistance) resulting in similar barrier resistances even when incorporating the higher strength A325 bolt. This conclusion is similar to that drawn from the UBC Experimental Research Program.

- It may be possible to increase the capacity of the barrier by approximately 25-30% by increasing the length of the 25M reinforcing insert to provide sufficient bond length to allow the development of the yield strength of the insert.

- The Side Mounted HSS Guide Retrofit Rail incorporating A307 bolts and 450 mm long 25M reinforcing inserts theoretically has sufficient capacity to resist the mandated 60 kN Transverse Design Load (Table 1) for the Ministry standard L100 and L150/L165 precast concrete deck panels. Further, since, the UBC experimental results suggest that this configuration has a capacity ranging from 54 - 64 kN when tested on a typical L75/BCL-625 deck panel, it is likely that the tested capacity would exceed 60 kN if the drip groove was eliminated or relocated away from the edge of the panel.

4 BARRIER CONTAINMENT CLASSIFICATION

Based on our review of the UBC experimental research program test results and the numerical analysis of the Side Mounted HSS Guide Retrofit Rail with A307 anchor bolts, we recommend that the Ministry move the drip groove to 300 mm from the edge of deck to minimize the effect it has on reducing the strength of the connection. With this change, we recommend that the Side Mounted HSS Guide Retrofit Rail can be classified as a CL-2 barrier. Notwithstanding the recommendation to move the drip groove, vehicular impact will likely result in some form of concrete damage that may require the replacement of the concrete deck panel.

Should the Ministry be concerned that the test results and theoretical results do not conclusively indicate that the Side Mounted HSS Guide Retrofit Rail is capable of resisting the mandated 60 kN Transverse Design Load, consideration can be given to completing additional tests that include the suggested modifications to the guardrail and concrete deck.

As discussed, it is possible to increase the capacity of the current Side Mounted HSS Guide Retrofit Rail, by making the following changes:

- Substitute the A307 bolts with A325 bolts.

- Increase the capacity of the 25M reinforcing inserts by providing additional bond length or substituting the 25M reinforcing bar with a 450 x 25 diameter A193 Type B7 threaded rod with a nut on the embedded end. This will likely be equivalent to a fully developed 25M reinforcing bar.

Notwithstanding, these modifications will not result in the barrier being classified as CL-3 barrier.

5 MODIFICATIONS TO THE SIDE MOUNTED HSS GUIDE RETROFIT RAIL

While reviewing the classification of the existing Side Mounted HSS Guide Retrofit Rail, we also considered modifications to the existing design to reduce fabrication costs without compromising the performance of the barrier. Based on discussions with the Ministry we included the following modifications on the proposed standard drawing:

- Reduced post size from HSS152x152x6.4 to HSS127x127x4.8.

- Reduced the length of the joint sleeve from 600 mm to 400 mm.

The analytical results indicate that reducing the width of bracket from 680 mm to 550 mm (for new installations) does not significantly reduce the capacity of the barrier connection. Notwithstanding the analytical results, we recommend the Ministry test three Side Mounted HSS Guide Retrofit Rail Barriers with a 550 mm bracket to verify their resistances meet the requirements of a CL-2 barrier before modifying the standard drawings.

In addition, the Ministry should consider modifying the standard concrete deck panel drawings by shifting the drip groove away from the edge of the deck.

Respectfully submitted,

Prepared by:

18-Jly-2012

John Deenihan Ph.D., EIT
Structural Engineer

Reviewed by:

PROFESSIONAL PROVINCE OF J. HENLEY BRITISH COLUMBIA ENGINEER

2012.07.18

Julien Henley, M.A.Sc., P.Eng.
Manager – Resource Infrastructure

JD/JH/fd