

REPORT

Ministry of Forests, Lands and Natural Resource Operations

Development and Testing of CL-2 and CL-3 Barriers, Rev.1



November 2013

ASSOCIATED ENGINEERING QUALITY MANAGEMENT SIGN-OFF	
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1 Introduction

The **Ministry of Forests, Lands and Natural Resource Operations** (the Ministry) retained **Associated Engineering** (AE) to develop standard bridge barrier and connection details for low volume industrial bridges. Previous phases of this project highlighted current practices for the design of low volume bridge barriers in North America, proposed a Barrier Classification System and made recommendations regarding the selection and design of bridge barriers for use on low volume industrial bridges. We documented this work in the following three reports:

- Phase I: Development of Standard Curb Design Parameters - Literature Review Summary, Associated Engineering, 2010.
- Phase II: Development of MFR Standard Curb Design Parameters - Barrier Selection and Design Philosophy, 2010.
- Phase III: Guideline for Barrier Selection and Design, 2011.

Concurrently, the University of British Columbia (UBC) completed laboratory testing focused on evaluating the strength of the Ministry's existing field barrier configurations (side mounted timber barriers), and modified barriers (timber risers, and steel HSS post and rails on side mounted steel brackets). UBC summarized the results of this work in their 2011 report, "*Experimental Evaluation of Concrete Decks with Guard Rail Systems*". Following the completion of the testing, AE reviewed the test results and recommended in our 2012 technical memorandum, "*Classification of HSS Guide Retrofit Bridge Rail*" that the Ministry classify the HSS Guide Retrofit Bridge Rail as a CL-2 barrier. In addition, we also recommended that the Ministry consider minor modifications to improve the performance (increase the resistance) of the barrier.

During the testing program, UBC at its own accord modified the HSS Guide Retrofit Rail by adding a 600 mm long knee-brace, which projected under the concrete deck and engaged the girder flange. This modification resulted in the barrier capacity increasing by approximately 230% and the barrier achieving the minimum required lateral resistance to be classified as a CL-3 barrier.

At the request of the Ministry, AE reviewed these modifications and concluded in our 2012 technical memorandum "*Review of Modified HSS Guide Retrofit Rail*" that a maximum practical knee-brace projection length of 300 – 400 mm was more appropriate, to avoid girder flange overlap. As a result, the modified HSS Guide Retrofit Rail with the reduced knee brace length would likely not provide the required resistance to be classified as a CL-3 barrier. As an alternative to the tested knee-brace configuration, we recommended that the Ministry develop a side mounted barrier connection that met the requirements for classification as a CL-3 barrier.

Following the completion of the review of the UBC testing, the Ministry retained AE and UBC to complete:

- Additional analytical and experimental work to evaluate the recommended modifications to the Ministry’s CL-2 barrier to improve its performance.
- The development and testing of a CL-3 barrier connection.

This report present the results and recommendations related to modifications to the CL-2 barrier and a proposed CL-3 barrier connection.

2 Barrier Classification

Table 2-1 presents the proposed bridge barrier design criteria included in AE’s 2011 Phase III Report, “*Guideline for Barrier Selection and Design*”.

**Table 2-1
Barrier Design Criteria**

Factored Design Forces	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
Note:			
1. When completing an analytical evaluation of a barrier, these forces represent factored forces; resistances should be calculated assuming nominal material strengths.			
2. Height measured from travel surface.			

3 Modification to Experimental Set-Up (2013)

While evaluating the results of UBC’s initial testing program, we postulated that the height of the applied load relative to the deck varied by up to 50 mm during the test (Figure 3.1a). Given the sensitivity of the barrier capacity to the height of the applied load, we replaced the HSS beam with a steel bars (Figure 3-1b). The new configuration reduces the variation in the testing lever arm to a maximum of 20 mm (typically 10 mm variation at yield load). With the exception of the loading point detail, the experimental set-up remained constant throughout all phases of testing. UBC’s 2010 report, “*Experimental Evaluation of Concrete Decks with Guard Rail Systems*” details the experimental configuration.

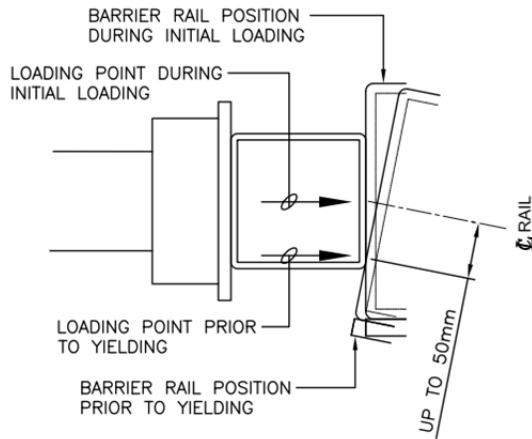


Figure 3-1a
Original Load Application

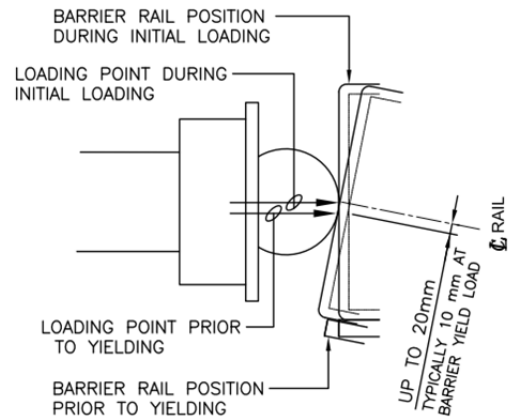


Figure 3-1b
Modified Load Application

4 CL-2 Testing (2013)

This Section presents a summary of the previously completed testing on the HSS Guide Retrofit Rail and the results of additional testing of recommended modifications to the CL-2 barrier connection included in AE's 2012 technical memorandum "*Classification of HSS Guide Retrofit Bridge Rail*".

4.1 PREVIOUS CL-2 EXPERIMENTAL AND NUMERICAL RESULTS

Table 4-1 presents the experimental and numerical findings relating to the CL-2 barrier (HSS Guide Retrofit Rail) documented in our 2012 technical memorandum, "*Classification of HSS Guide Retrofit Bridge Rail*", and included in Appendix B.

**Table 4-1
CL-2 Theoretical and Experimental Barrier Resistances (2010)**

Specimen	Capacity			Comments
	Theoretical ^{1,2,3}	Experimental		
		Actual ⁴	Adjusted ⁵	
2.1	47	64.2	60.6	A307 bolts connecting barrier and concrete panel. <i>Failure Mode:</i> Bolts ruptured in tension.
2.11	51	68.1	64.3	A325 bolts connecting barrier and concrete panel. <i>Failure Mode:</i> Spalling of concrete in vicinity of inserts.
2.2	47	65.7	62.1	A307 bolts connecting barrier and concrete panel. <i>Failure Mode:</i> Bolts ruptured in tension.
2.21	51	57.3	54.1	A325 bolts connecting barrier and concrete panel. <i>Failure Mode:</i> Spalling of concrete in vicinity of inserts.
2.3	47	56.4 ²	53.3	A307 bolts connecting barrier and concrete panel, Vertical/Horizontal loading ratio = 1.08:6. <i>Failure Mode:</i> Bolts ruptured in tension.
2.4	47	54.9 ²	51.9	A307 bolts connecting barrier and concrete panel, Vertical/Horizontal loading ratio = 1.08:6. <i>Failure Mode:</i> Bolts ruptured in tension.
<p>Note:</p> <ol style="list-style-type: none"> 1. Based on tested concrete compressive strength $f_c = 56$ MPa. 2. Capacities calculated assuming nominal material strengths. 3. Load application height = 450 mm 4. Load application height = 425 mm 5. Since the tested load application height did not match the prescribed load application height (Table 2-1), we adjusted the measured experimental capacity to reflect the prescribed load application height (450 mm) by assuming the measured failure moment remains constant. 				

4.2 PROPOSED MODIFICATIONS TO CL-2 BARRIER

As discussed in our 2012 technical memorandum, “*Classification of HSS Guide Retrofit Bridge Rail*”, we recommended that the Ministry consider the following modifications to increase the resistance of the CL-2 barrier:

- Relocate the drip groove to 300 mm from the deck edge to prevent premature spalling of the concrete located in the compression region of the deck panel;
- Improve the anchorage of the reinforcing bars which are connected to the couplers by tack welding a nut to the free ends; and
- Reducing the width of the bracket, from 680 mm to 550 mm, to reduce the cost of the bracket.

Based on these recommendations, UBC tested the following:

- Panel Type A – 680 mm wide bracket.
- Panel Type B – 550 mm wide bracket.
- Panel Type C – 550 mm wide bracket, with nuts welded to the end of the reinforcing bars which project from the couplers.

For all the panels, we relocated the drip groove 300 mm for the deck edge and incorporated 2 U – shaped reinforcing bars on either side of each Burrard coupler to replicate the Ministry’s standard reinforcing details. UBC’s 2013 report, “*Testing of FLNR Standard Curb Systems*”, included in Appendix A, provides complete details of the specimens tested.

4.3 CL-2 EXPERIMENTAL AND THEORETICAL RESULTS

Table 4-2 presents the theoretical and experimental results for the each of the tested configurations. Compression tests of the field cured cylinders indicated a concrete compressive strength of 40 MPa which was lower than previously tested, i.e. 40 MPa compared with 56 MPa. This allowed us to compare the effects of concrete compressive strength on the resistance of the barrier.

**Table 4-2
CL-2 Theoretical and Experimental Barrier Resistances (2013)**

Panel Type	Description	Capacity (kN)		
		Theoretical ^{1,2,3}	Experimental ⁴	Adjusted ⁵
A	680 mm Bracket	43.9	47.5	44.1
			48.5	45.0
B	550 mm Bracket	42.2	51.6	47.9
			44.3	41.1
C	550 mm Bracket and Bar Anchors	42.2	49.1	45.6
			47.3	43.9
			40.7	37.8
			43.9	40.7

Note:

1. Based on tested concrete strength $f_c = 40$ MPa.
2. Capacities calculated assuming nominal material strengths.
3. Load application height = 450 mm
4. Load application height = 485 mm
5. Since the tested load application height did not match the prescribed load application height (Table 2-1), we adjusted the measured experimental capacity to reflect the prescribed load application height (450 mm) by assuming the measured failure moment remains constant.

Comparing the theoretical and experimental values in Tables 4-1 and 4-2, it is evident that:

- The numerical model provides a good estimate of the resistance of the bracket for the 40 MPa concrete (Table 4-1), but underestimates the capacity of all specimens using a higher strength concrete (56 MPa) (Table 4-2).
- The capacity of the side mounted steel barrier appears sensitive to the compressive strength of the concrete.
- Reducing the bracket width from 680 mm to 550 mm reduces the connection resistance (mean values) by approximately 4% (experimentally and numerically).
- There is a 26% and 19% variation in experimental values for the 550 mm and 680 mm wide brackets respectively suggesting that the strength of the bracket is sensitive to fabrication tolerances, experimental set-up and testing and/or material variations.
- Improving the anchorage of the embedded reinforcing anchors does not increase the resistance of the barrier.

Given the difference in concrete strength between the two test series, we could not determine the effect of relocating the drip groove. However, we did observe that the concrete compressive failure (crushing) terminated at the location of the drip groove, regardless of its location. This suggests that relocating the drip groove further from the panel edge resulted in engaging a larger area of concrete to act in compression.

4.3.1 Experimental Observations

Figure 4-1 presents the typical failure pattern and load-displacement response for the CL-2 barrier connection. The lateral resistance of the system increased until the concrete failed in compression below the bracket. This was followed by concrete spalling on the underside of the panel and extensive rotation of the barrier.

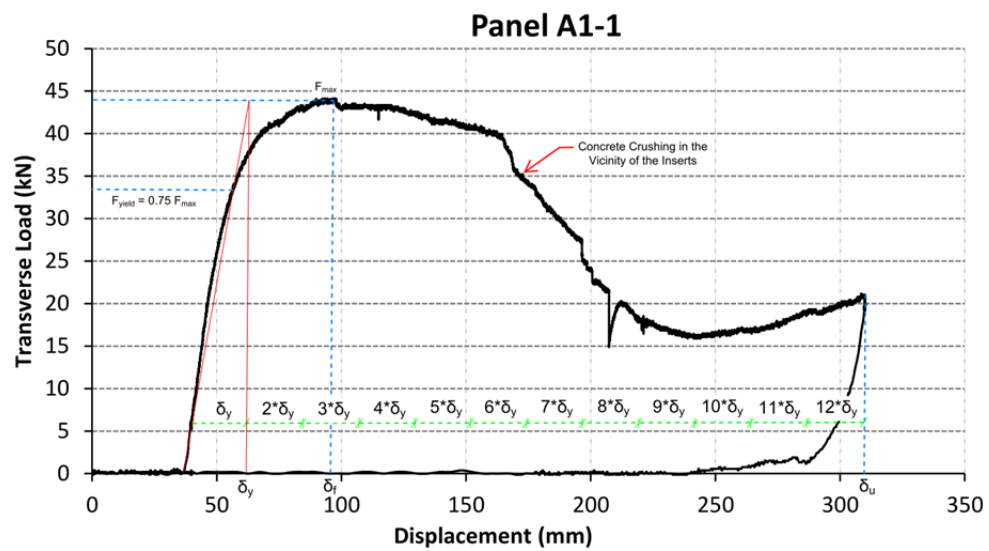


Figure 4-1
Typical CL-2 Specimen Crack Pattern & Load-Displacement Plot

All specimens displayed good displacement ductility characteristics (typically $\mu > 5$), where μ represents the ratio of yield displacement (δ_y) to ultimate displacement (δ_u) suggesting that the tested configuration provides good energy dissipation while retaining load carrying capacity.

UBC's 2013 report, "Testing of FLNR Standard Curb Systems", included in Appendix A, provides a more detailed description of the failure.

4.4 RECOMMENDATIONS BASED ON THE CL-2 EXPERIMENTAL INVESTIGATION

While the observed resistances from the initial testing (Table 4-1) suggest that the system is capable of achieving the required lateral resistance to be classified as a CL-2 barrier; a reduction in concrete compressive strength (35-40 MPa) suggests that the capacity of the barrier is sensitive to concrete strength and that at these concrete strengths, the barrier does not provide the required lateral resistance. Considering the experimental and analytical findings we recommend that the Ministry:

- Reduce the required minimum lateral resistance for the CL-2 barrier for 60 kN to 45 kN.
- Relocate the drip groove to 300 mm from the deck edge.
- Reduce the width of the bracket from 680 mm to 550 mm.

5 Development and Testing of a CL-3 Barrier (2013)

During the previous experimental phase, UBC strengthened the HSS Guide Retrofit Rail by adding a 600 mm long knee-brace (Figure 5-1). Full details of the modifications are presented in UBC's 2011 report, "*Experimental Evaluation of Concrete Decks with Guard Rail Systems*".

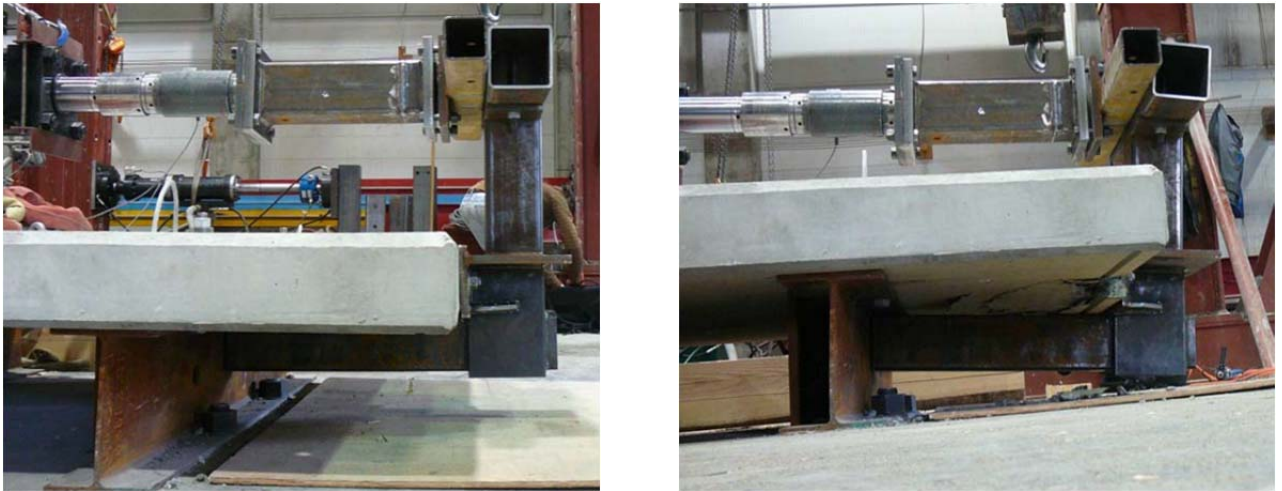


Figure 5-1
Side Mounted Steel Bracket with Knee Brace

The modifications resulted in a 230% increase in the lateral resistance of the barrier with the anchor bolts failing in shear rather than the previously observed concrete compressive failure or tensile failure of the anchor bolts. Further, the modified barrier achieved the resistance required to be classified as a CL-3 barrier.

Given these encouraging results, AE investigated the feasibility of incorporating a knee brace into the HSS Guide Retrofit Rail. We concluded that in addition to field installation challenges, the maximum practical knee brace length would be limited to 300-400 mm to avoid bearing on the girder flange. Our analysis suggested that the reduction in the knee-brace length may result in a 20% reduction in lateral resistance of the barrier and it would no longer meet the minimum CL-3 barrier design criteria (120 kN). Therefore, we recommended the Ministry consider a side mounted CL-3 barrier incorporating an embedded steel angle and deformed bar anchors to improve the resistance of the barrier connection to the deck panel. Our 2012 technical memorandum, “*Review of Modified HSS Guide Retrofit Rail*” included in Appendix C includes details of our evaluation of the knee-brace and recommendations for a side mounted CL-3 barrier connection. The following sections describe the proposed connection and associated testing.

5.1 PROTOTYPE CL-3 BARRIER

Figure 5-2 illustrates the proposed CL-3 side mounted connection. The prototype CL-3 barrier bracket incorporates an embedded angle with vertical headed studs and horizontal Nelson Deformed Bars (NDB's). The embedded angle and vertical headed studs confine the concrete along the edge of the panel resulting in an increased compressive resistance. The horizontal NDB's provide the required tensile resistance.

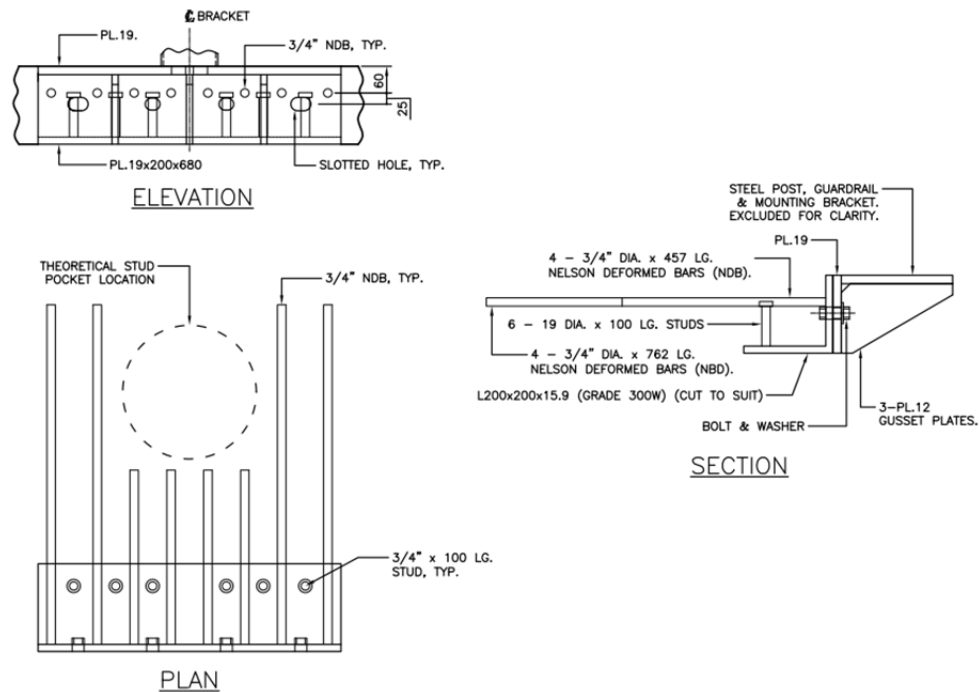


Figure 5-2
Proposed CL-3 Bracket (Post and Rail Details Excluded for Clarity)

Since the numerical analysis suggested that the 175 mm thick panel may not achieve the required 120 kN lateral resistance we also recommended testing a 200 mm thick panel. Based on these recommendations, UBC tested the following panels:

- Panel Type D – 175 mm thick panel with 680 mm wide bracket.
- Panel Type E – 200 mm thick panel with 680 mm wide bracket.

For all the panels we located the drip groove 300 mm from the deck edge.

5.2 CL-3 THEORETICAL AND EXPERIMENTAL RESULTS

Table 5-1 presents the theoretical and experimental results for the each of the tested configurations.

Table 5-1
CL-3 Theoretical and Experimental Barrier Resistances

Panel Type	Description	Capacity (kN)		
		Theoretical ^{1,2,3}	Experimental ⁴	Adjusted ⁵
D	175 mm Panel	133	141.5	134.6
			117.9	112.1
			137.0	130.3
			122.6	116.6
E	200 mm Panel	166	143.4	136.4
			150.7	143.3
			148.0	140.7
			160.5	152.6

Note:

1. Based on tested concrete strength $f_c = 40$ MPa.
2. Capacities calculated assuming nominal material strengths.
3. Load application height = 510 mm
4. Load application height = 485 mm
5. Since the tested load application height did not match the prescribed load application height (Table 2-1), we adjusted the measured experimental capacity to reflect the prescribed load application height (450 mm) by assuming the measured failure moment remains constant.

A review of the Table 5-1 indicates:

- The connection does not consistently provide the required resistance for the 175 mm thick panel.
- The connection provides sufficient resistance on the 200 mm thick panel.
- There is reasonable correlation between the predicted and measured capacity of the connection although for both panel thicknesses, the theoretical analysis tends to overestimate the capacity of the connection.

5.3 EXPERIMENTAL OBSERVATIONS

Figure 5-3 presents a typical failure and load-displacement response for the proposed connection.

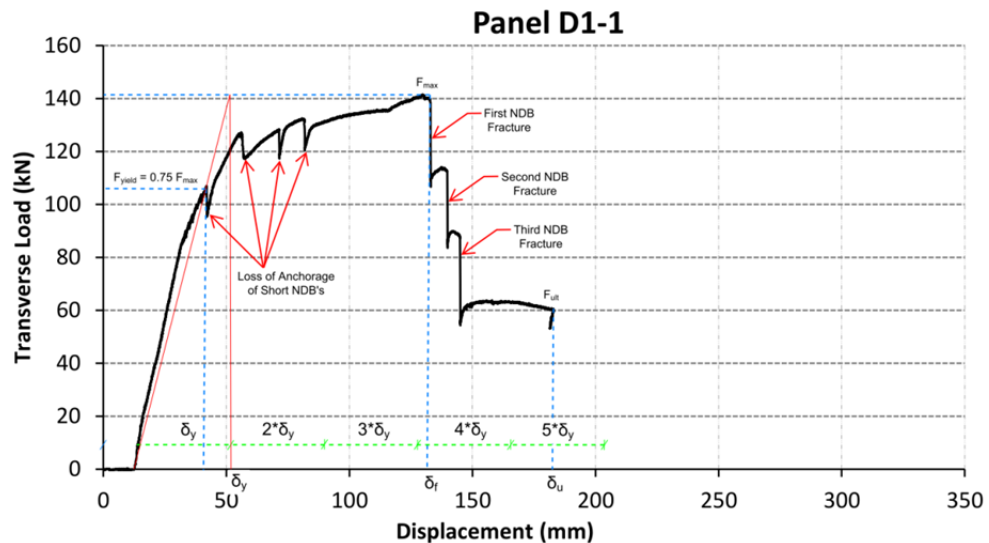


Figure 5-3
Typical CL-3 Connection Failure and Load-Displacement Plot

During all the tests we heard 3-4 “popping” sounds that coincided with the minor dips in resistance on the ascending branch of the load-displacement plot (Figure 5-3). Based on the load displacement plot and post-failure observations, we believe the “popping” sounds were associated with the bond failure of the four short NDB’s (Figure 5-4).

The connection achieved peak load after the bond failure of the short NDB’s and prior to the fracture of one or more of the long NDB’s at the weld location. The specimens failed by fracturing the long NDB’s and compressive failure of the concrete around the embedded angle.



Figure 5-4
Short NDB's Post Failure

The load-displacement plot indicates that the connection has residual resistance (i.e. approximately 40% of the peak resistance); this is a good indication of the systems post-failure ductility and energy dissipation characteristics. Some specimens achieved post-failure displacement ductility, $\mu > 4$ (where μ represents the ratio of yield displacement (δ_y) to ultimate displacement (δ_u)); it is likely all specimens would exceed this displacement ductility if the actuator stroke had not been exceeded.

UBC's 2013 report, "*Testing of FLNR Standard Curb Systems*", included in Appendix A, provides a more detailed description of the failure.

5.4 TESTING OF MODIFIED CL-3 BARRIER

As noted in the previous Section, the experimental findings suggest that if the short NDB's had adequate anchorage (bond length), the connection may achieve a higher peak resistance. To increase the resistance of the four short NDB's we recommended replacing them with four pairs of stacked headed studs (Figure 5-5). To verify this assumption, UBC tested the following panels which incorporated the recommended modifications:

- Panel Type G – 175 mm thick panel with 680 mm wide bracket.
- Panel Type H – 200 mm thick panel with 680 mm wide bracket.

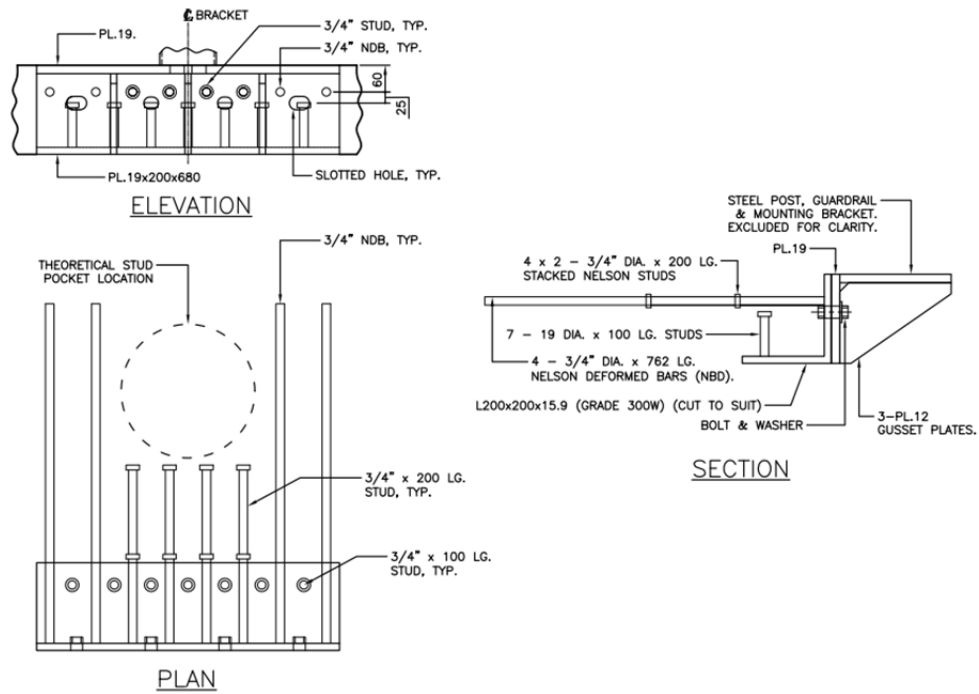


Figure 5-5
Re-Designed CL-3 Bracket

Table 5-2 presents the theoretical and experimental results for all the CL-3 barrier connections.

Table 5-2
Maximum CL-3 Theoretical and Experimental Barrier Resistances

Panel Type	Description	Capacity (kN)		
		Theoretical ^{1,2,3}	Experimental ⁴	Scaled ⁵
D	175 mm Panel with only NDB's	133	141.5	134.6
			117.9	112.1
			137.0	130.3
			122.6	116.6
G	175 mm Panel with NDB's & Stacked Studs	168	165.0	156.9
			161.4	153.5
E	200 mm Panel with only NDB's	166	143.4	136.4
			150.7	143.3
			148.0	140.7
			160.5	152.6
H	200 mm Panel with NDB's & Stacked Studs	218	193.0	183.6
			172.8	164.3

Note:

1. Based on tested concrete strength $f'_c = 40$ MPa.
2. Capacities calculated assuming nominal material strengths.
3. Load application height = 510 mm
4. Load application height = 485 mm
5. Since the tested load application height did not match the prescribed load application height (Table 2-1), we adjusted the measured experimental capacity to reflect the prescribed load application height (450 mm) by assuming the measured failure moment remains constant.

Figure 5-6 presents a typical failure and load-displacement response for the revised CL-3 barrier connection.

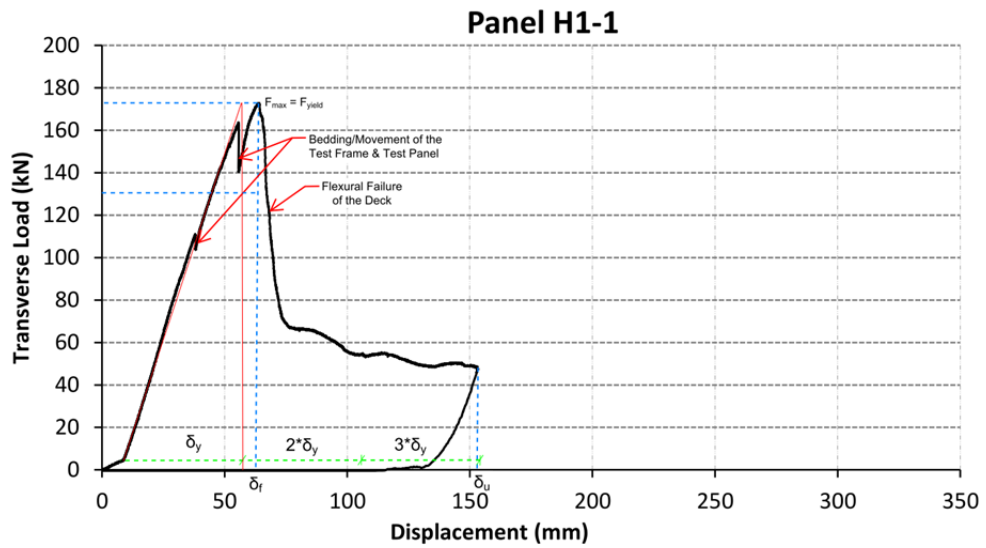


Figure 5-6
Typical Revised CL-3 Failure and Load-Displacement Plot

The modified CL-3 barrier connection failed due to yielding/pull-out of the stacked headed studs and NDB's, and the loss of the top cover concrete. This was followed by the compressive failure of the concrete on the underside of the panel and extensive rotation of the bracket. The NDB's did not fracture during any of the tests.

The load-displacement plot indicates that the modified connection has residual resistance (i.e. approximately 30% of the peak resistance); this is a good indication of the systems post-failure ductility and energy dissipation characteristics. The modified CL-3 configuration tests were terminated prior to the full actuator stroke length being achieved. It is likely the specimens would have achieved a post failure displacement ductility, $\mu > 4$ (where μ represents the ratio of yield displacement (δ_y) to ultimate displacement (δ_u)), similar to the original CL-3 configuration. Although the original and modified CL-3 barrier connections display similar post-failure performance characteristics; the modified connection displayed a brittle failure with limited yielding near the peak load.

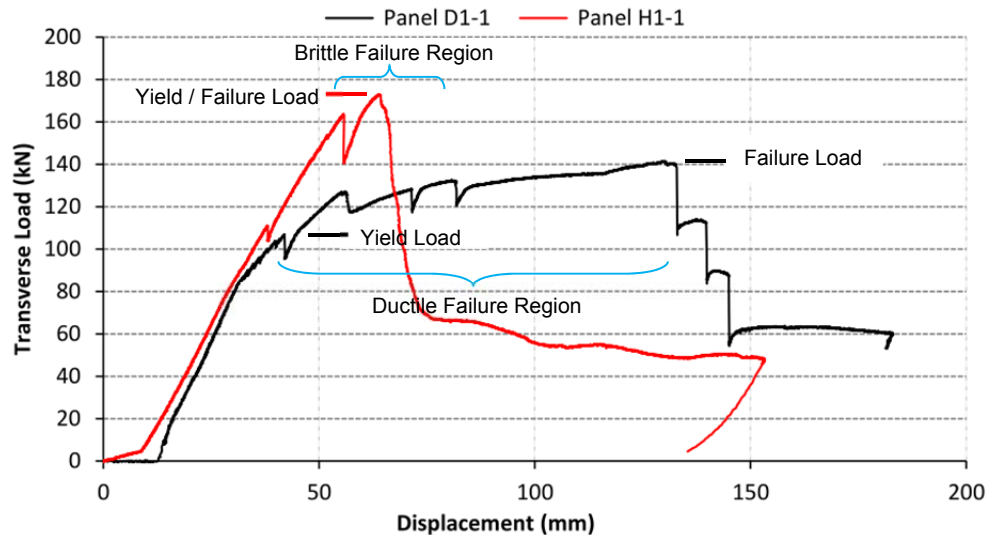


Figure 5-7
Original and Revised CL-3 Configuration Load-Displacement Plots

Comparing the load-displacement plots for the original and revised CL-3 configurations (Figure 5-7), it can be seen that the revised configuration (Panel H1-1) fails in a brittle manner compared to the ductile failure for the original configuration (Panel D1-1). This is typical for the failure of an embedded stud. The failure leads to a sudden drop in resistance, at yield/failure, compared to a prolonged yielding plateau for the original configuration incorporating only NDB reinforcement. The brittle failure is of no concern if the post and rail assembly is designed to yield prior to failure of the bracket.



Figure 5-8
Comparison of Original and Modified CL-3 Barrier Connections Failures

Figure 5-8 provides a photographic comparison of the failures of the original and modified CL-3 barrier connections. The change in failure mechanism is highlighted by the extensive cracking on the top surface of the concrete panel for the modified configuration (with stacked studs).

In addition to the visual observations, a review of the numerical and experimental capacities of the modified CL-3 barrier connection suggests that the numerical model consistently over-estimates the connection capacity. The numerical model predicted a concrete compressive failure on the underside of the panel while we observed tensile cracking and spalling of the top cover concrete when the connection achieved its peak load (Figure 5-6). This suggests that the connection was not able to develop the predicted tensile capacity of the stacked headed studs assumed within the numerical model. As a result, the connection failed at a reduced load.

5.5 RECOMMENDATIONS BASED ON THE CL-3 EXPERIMENTAL INVESTIGATION

Based on the experimental findings and numerical analysis for the CL-3 barrier connection, we recommend that the Ministry adopt a modified connection that incorporates a combination NDB's and stacked headed studs similar to that illustrated in Figure 5-5. The proposed connection achieves the minimum required lateral resistance for a CL-3 barrier (i.e. 120 kN) for the 175 and 200 mm thick panels.

The failed connection provides good residual strength (approximately 30% of the maximum lateral resistance of the bracket) and good post-failure ductility. The connection also provides sufficient over-strength, to facilitate the design of a post and rail assembly that will yield prior to the failure of the bracket of concrete panel.

6 Conclusions and Recommendations

6.1 CL-2 BARRIERS

Based on the experimental testing, we were not able to confirm that the recommended modifications to the Ministry's standard CL-2 barrier including improving the anchorage of the embedded anchors and relocating the drip groove away from the deck edge resulted in improved performance. However, the testing did highlight the sensitivity of the connection capacity to the compressive strength of the concrete. A reduction in the compressive concrete strength from 56 MPa to 40 MPa resulted in the tested capacity decreasing from approximately 60 kN to 45 kN, suggesting that the barrier no longer meets the minimum criteria to be classified as a CL-2 barrier.

Given the observed reduction in the capacity of the connection and recognition that the majority of precast concrete deck panels have a compressive concrete strength of 35-45 MPa, we recommend that the Ministry:

- Reduce the required minimum lateral resistance for the CL-2 barrier for 60 kN to 45 kN.
- Relocate the drip groove to 300 mm from the deck edge.
- Reduce the width of the bracket from 680 mm to 550 mm.

Given the recommended reduction in the factored lateral design load for the CL-2 barrier, the Ministry should revise the Barrier Design Criteria (Table 2-1). We propose reducing the CL-2 Factored Transverse Load and eliminating the design parameters for the CL-1 barriers. As a result, only Ministry mandated CL-1 barriers would be considered acceptable i.e. timber and W-beam barriers. Table 6-1 summarizes the proposed Barrier Design Criteria.

6.2 CL-3 BARRIERS

Based on the testing, we developed a suitable barrier connection detail that will allow the development of a barrier that meets the minimum design requirements for the CL-3 barrier. The connection incorporates an embedded angle anchored into the deck edge using Nelson Deformed Bars and stacked headed studs. The connection also provides sufficient over strength to allow the design of a post and rail assembly that will yield prior to failure of the bracket or concrete deck. We therefore recommend that the Ministry develop a CL-3 barrier that incorporates the proposed connection.

Since the proposed CL-3 barrier connection can only be used for new bridge construction the Ministry may wish to review the knee-brace configuration to allow it use as a CL-3 retrofit barrier.

While this assignment focussed on the development and testing of a side mounted CL-3 barrier connection, there are alternative bridge barriers that may also meet the CL-3 design criteria including top mounted barriers on an integral concrete curb and continuous concrete curbs. We recommend that the Ministry complete a minimum of four tests to confirm the performance of each proposed CL-3 barrier.

If the Ministry wishes to classify the CL-3 barrier as a PL-1 barrier in accordance with the requirements of the Canadian Highway Bridge Design Code, the Ministry should modify the testing to incorporate the simultaneous application of the transverse, vertical and longitudinal loads.

Based on discussion with the Ministry, we recommend reducing the minimum barrier height from 685 mm to 585 mm to reflect the proposed rail configuration. Table 6-1 summarises the minimum design requirements for a CL-3 barrier configuration.

6.3 PROPOSED BARRIER DESIGN CRITERIA

Table 6-1 summarizes the proposed Barrier Design Criteria.

**Table 6-1
Proposed Barrier Design Criteria**

Factored Design Forces	Containment Level		
	CL-1 ³	CL-2	CL-3
Transverse Load, F_T , kN	-	45	120
Longitudinal Load, F_L , kN	-	20	40
Vertical Load, F_V , kN	-	20	20
Load Application Height, mm ²	-	450	510
Minimum Barrier Height ²	-	525	585
Note: 1. When completing an analytical evaluation of a barrier, these forces represent factored forces; resistances should be calculated assuming nominal material strengths. 2. Height measured from travel surface. 3. Only MFLNRO standard drawings shall be used for CL-1 containment level design.			

7 Closure

The services provided by Associated Engineering in the preparation of this report were conducted in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practicing under similar conditions. No other warranty expressed or implied is made.

Respectfully submitted,
Associated Engineering (B.C.) Ltd.

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REPORT

Appendix A - UBC Testing Report "Testing of FLNR Standard Curb Systems"

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Testing of FLNR Standard Curb Systems



By

B. Robazza & S.F. Stiemer, Dept. of Civil Engineering, UBC

August 2013

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Purpose of testing

Laboratory testing shall be conducted in order to evaluate the strength of existing field barrier configurations (side mounted timber barriers), and modified barriers (timber risers, and steel HSS post and rails on side mounted steel brackets). The following was to be conducted:

- theoretical analysis and lab testing of FLNR Standard Curb Systems to determine and confirm strengths of existing and proposed systems consistent with proposal by Associated Engineering (BC) Ltd. dated September 5, 2012;
- work in collaboration with both the ministry and Associated Engineering (BC) Ltd., in developing testing protocols for the bridge barrier systems to be tested. The Civil Engineering department of UBC shall be responsible for developing the test protocol and conducting the actual testing. The Associated Engineering (BC) Ltd. will design the test specimens and support development of the test protocol and collaborate with UBC in the analysis of test results;
- test protocol(s) to be implemented shall be agreed upon by the Contractor, Associated Engineering (BC) Ltd. and the ministry representative prior to proceeding with any testing.

Test setup

Concrete test panels representative of concrete bridge decking will be subjected to static loading on bridge barrier assemblies. The loading will be steadily increased until failure of barrier, bracket, attachment, or panel section. The failures will be recorded with photos, video, and load/displacement records of the load ram.

Details: see the following and Appendix.



Figure 1: Overview of Test-Setup

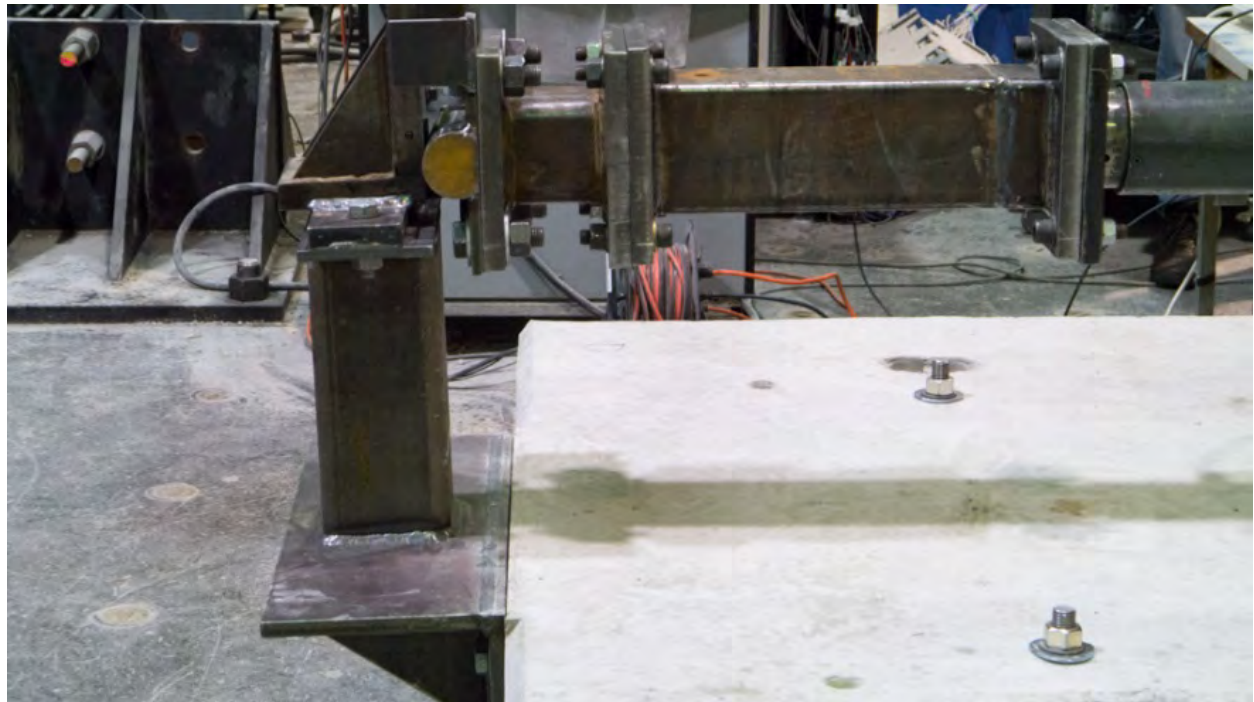


Figure 2: Detail of Load Application



Figure 3: Panels Stored Outside



Figure 4: Intensive Testing Efforts

Barrier Testing Matrix

Overview

Table 1: Panel Types, Parameters, Numbers

Panel Types			
Type A Panel: 175 thick c/w three inserts for 680 wide bracket			
Type B Panel: 175 thick c/w three inserts for 550 wide bracket			
Type C Panel: 175 thick c/w three inserts and bar terminators 550 wide bracket (to be confirmed - could do one on each side of panel)			
Type D Panel: 175 thick c/w CL3 insert			
Type E Panel: 200 thick c/w CL3 insert			
Description	No. Tests	Panel Type	No. Panels
Required Test			
175mm Panel w/ 680 Wide Bracket c/w relocated drip groove	2	A	1
175mm Panel w/ 550 Wide Bracket c/w relocated drip groove	2	B	1
175mm Panel w/ 550 Wide Bracket & Terminators	4	C	2
175mm Panel w/ 680 Wide CL-3 Bracket	4	D	2
200mm Panel w/ 680 Wide CL-3 Bracket	4	E	2
175mm Panel	2	G	1
200mm Panel	2	H	1
total	20		10
Optional Test			
Nuts at end of anchors	2	C	1

Test Records

Table 2: Overview of Tests, Dates, Max. Loads

Panel Type / Test Number	Date	max. Load [kN]	Remarks
D-1-1	20.02.2013	140	
D-1-2	21.02.2013	118	
D-2-1	25.02.2013	137	
D-2-2	25.02.2013	122	
E-1-1	26.02.2013	143.4	
E-1-2	27.02.2013	99.8	
E-2-1	28.02.2013	147	
E-2-2	06.03.2013	160.5	
A-1-1	08.03.2013	44.1	loading height 3 mm less than D panels E
A-1-2	11.03.2013	45.5	loading height 3 mm less than D panels E
B-1-2	12.03.2013	47.9	
B-1-1	13.03.2013	41.1	
C-1-1	13.03.2013	45.6	
C-1-2	15.03.2013	43.9	
C-2-1	22.03.2013	37.9	
C-2-1	22.03.2013	40.7	left bolt broke at 180 mm displacement
G-1-1	08.04.2013	164.6	two bolts ripped out at 110 [kN], use of slightly longer bolts
G-1-2	08.04.2013	161.4	
H-1-1	12.04.2013	179.2	weld at bottom of post failed, after reinforcement, o.k., max. value from first test, second test only 172.8 [kN]
H-1-2	12.4.2013	189.6	two center bolts ripped out at 189 [kN], after re-cut insert, test successful

Individual Panel Tests

Panel / Bracket Test A-1-1

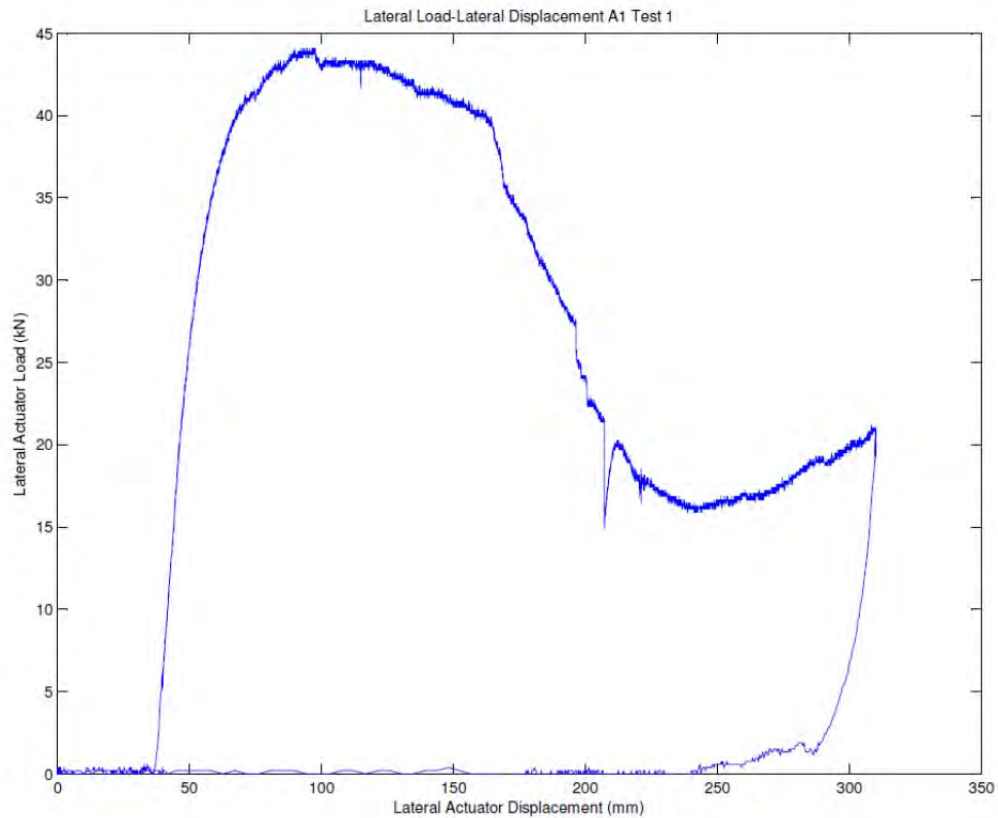


Figure 5: Load/Deflection Curve of Top of Bracket, A-1-1 (Note that initial drop in load corresponded to when the compression concrete on the slab face crushed to the drip groove)



Figure 6: Images from A-1-1

Panel / Bracket Test A-1-2

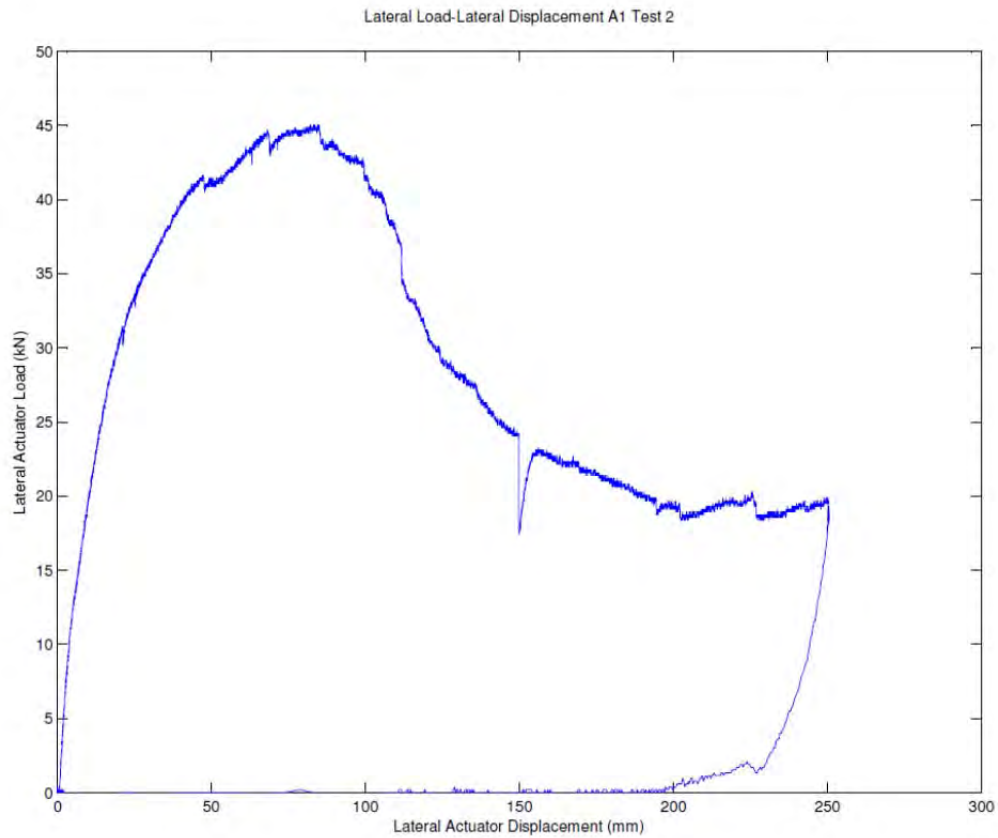


Figure 7: Load/Deflection Curve of Top of Bracket, A-1-2 (Note that initial drop in load corresponded to when the compression concrete on the slab face crushed to the drip groove)

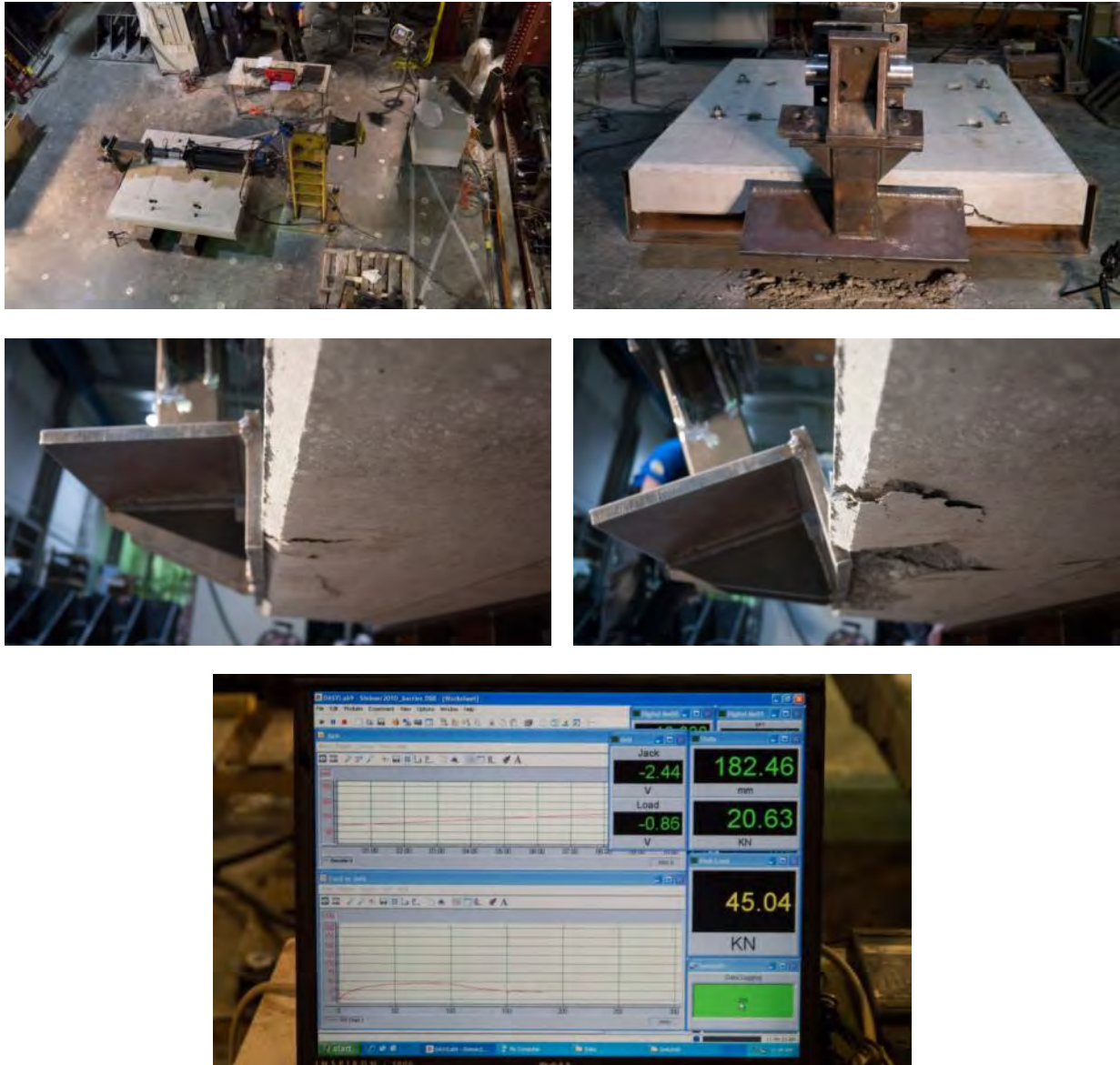


Figure 8: Images from A-1-2

Panel / Bracket Test B-1-1

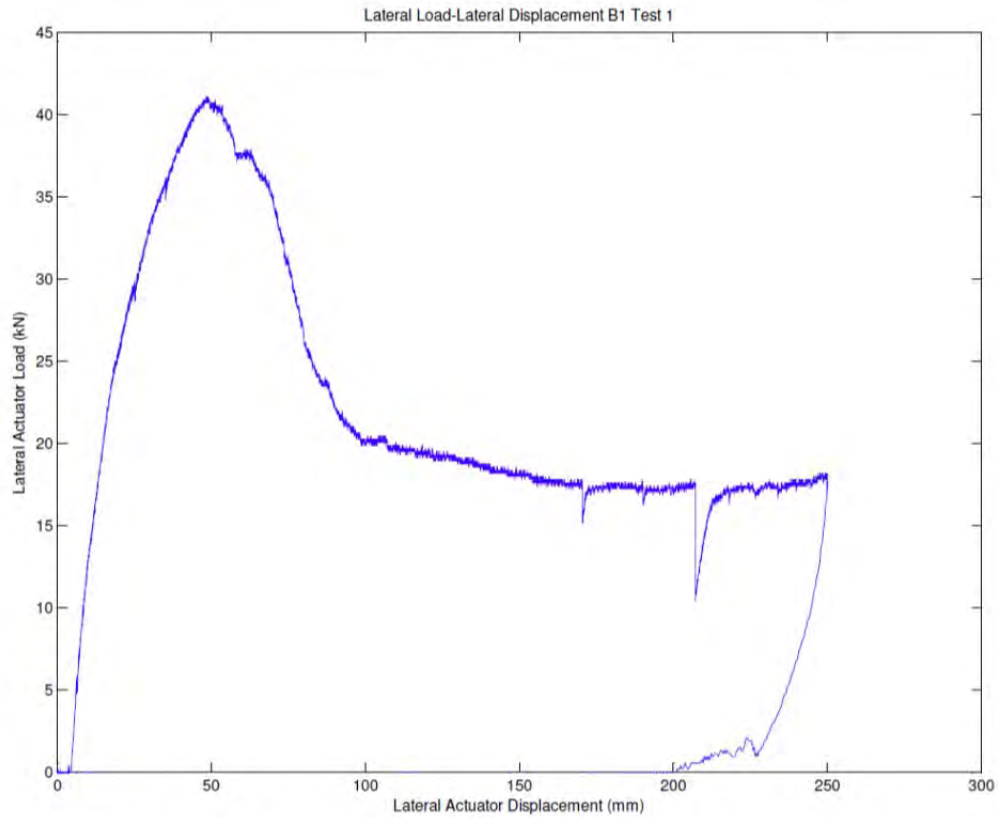


Figure 9: Load/Deflection Curve of Top of Bracket, B-1-2 (Note that initial drop in load corresponded to when the compression concrete on the slab face crushed to the drip groove)

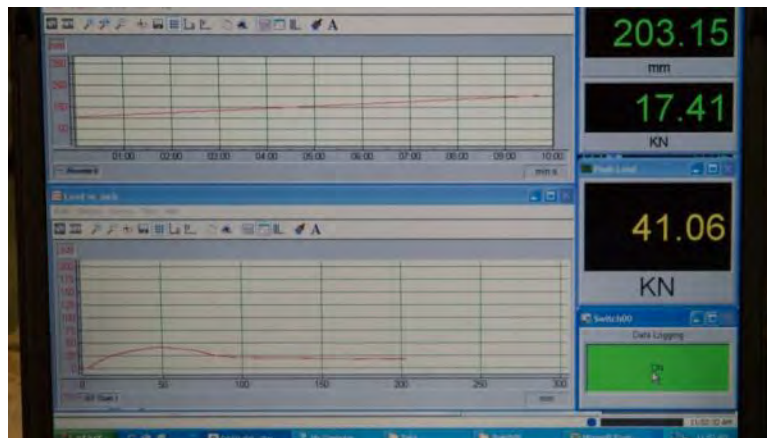
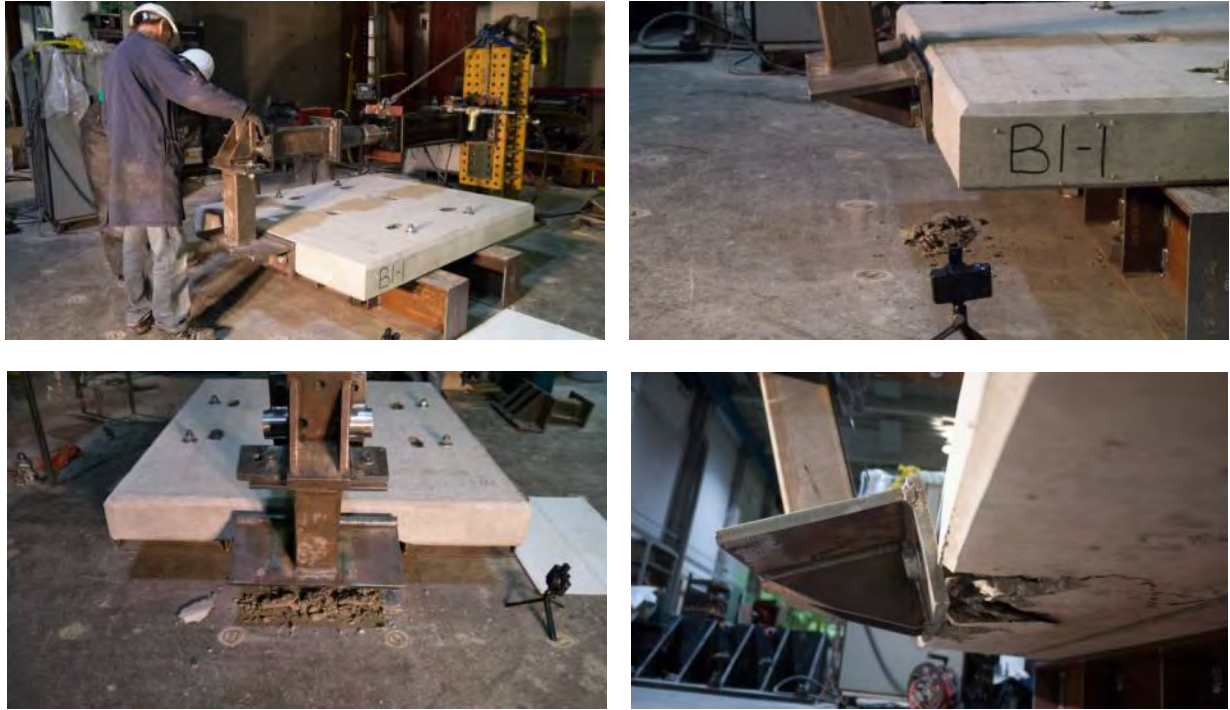


Figure 10: Images from B-1-1

Panel / Bracket Test B-1-2

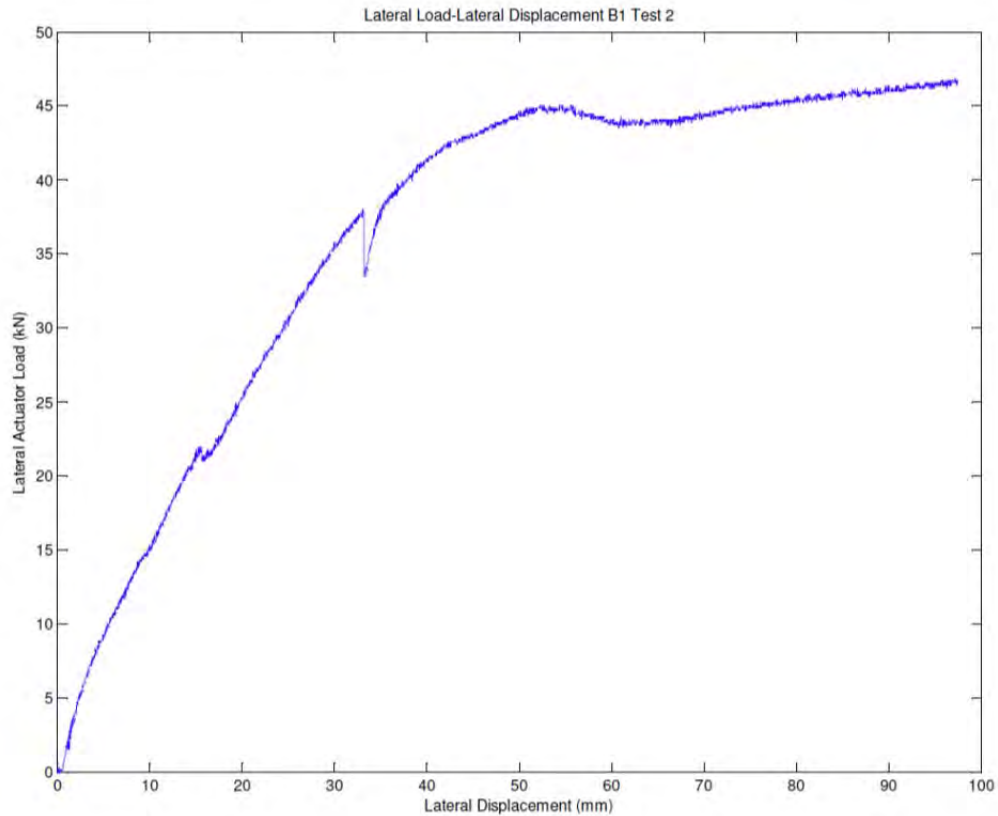


Figure 11: Load/Deflection Curve of Top of Bracket, B-1-2 (Note that drop in stiffness corresponded to when the compression concrete on the slab face crushed to the drip groove, after this point the load was entirely carried by the bending of the bolts. The test was halted as safety became a concern after this point)



Figure 12: Images from B-1-2

Panel / Bracket Test C-1-1

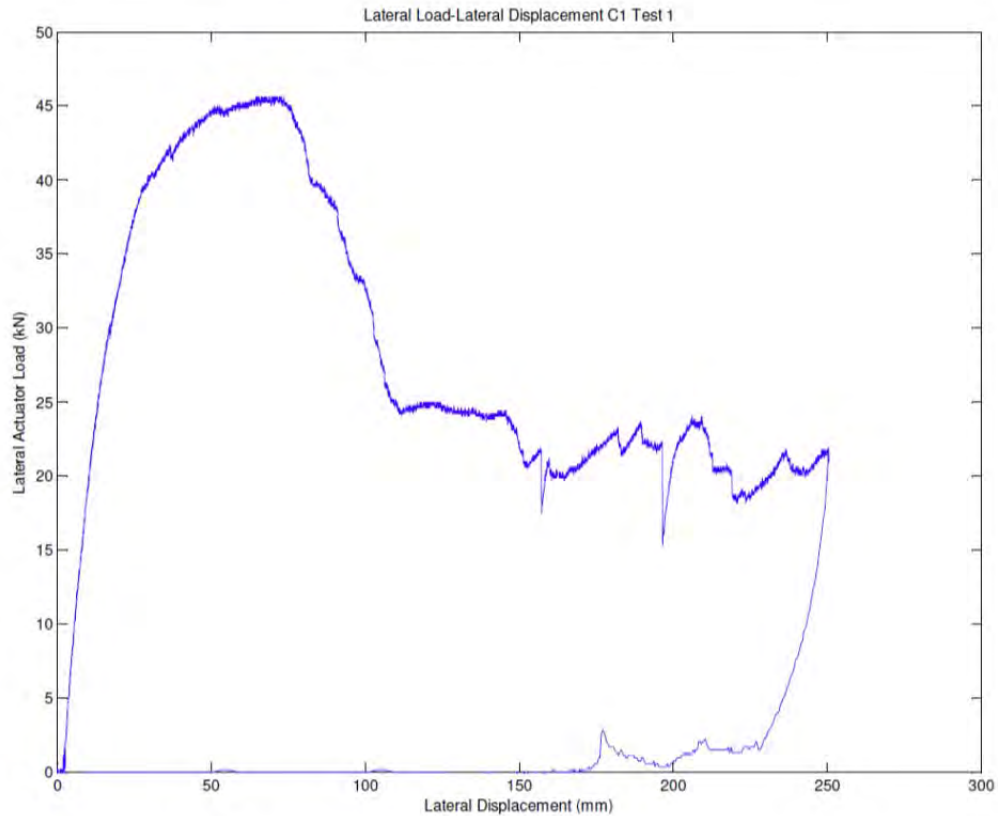


Figure 13: Load/Deflection Curve of Top of Bracket, C-1-1 (Note that initial drop in load corresponded to when the compression concrete on the slab face crushed to the drip groove)



Figure 14: Images from C-1-1

Panel / Bracket Test C-1-2

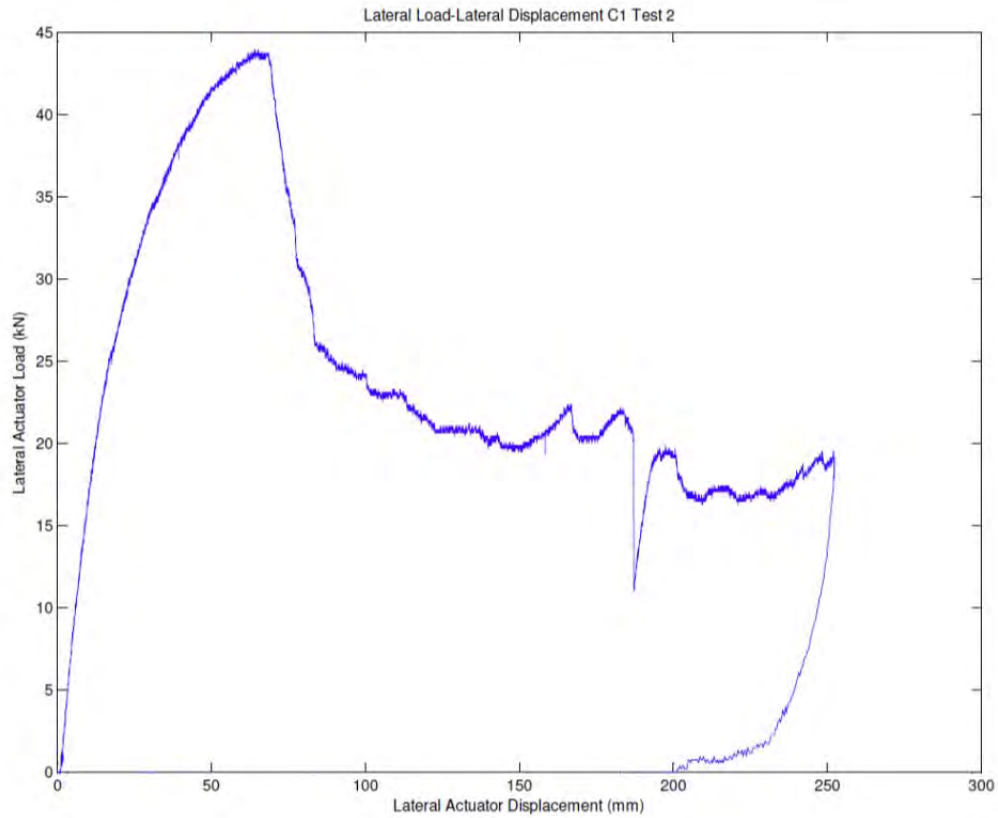


Figure 15: Load/Deflection Curve of Top of Bracket, C-2-1 (Note that initial drop in load corresponded to when the compression concrete on the slab face crushed to the drip groove)



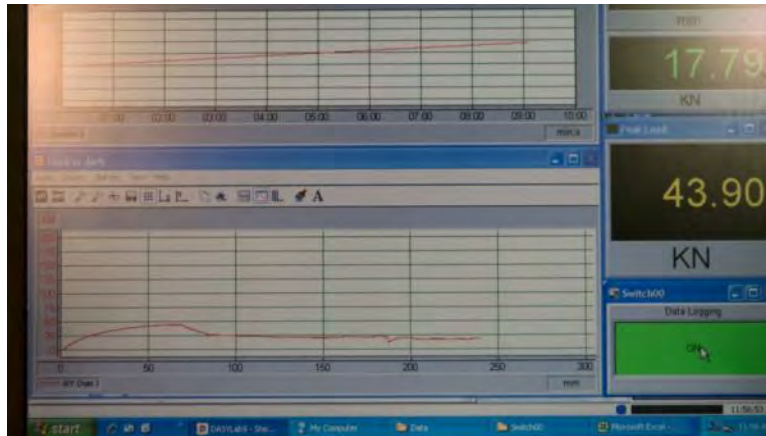


Figure 16: Images from C-2-1

Panel / Bracket Test C-2-1

Figure 17: Load/Deflection Curve of Top of Bracket, C-2-1

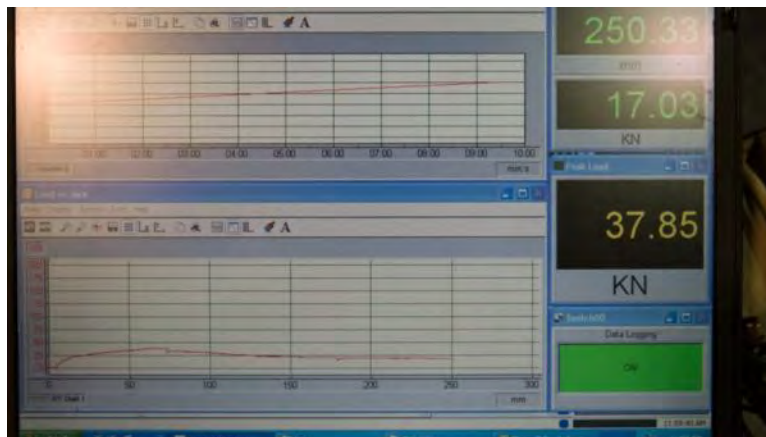
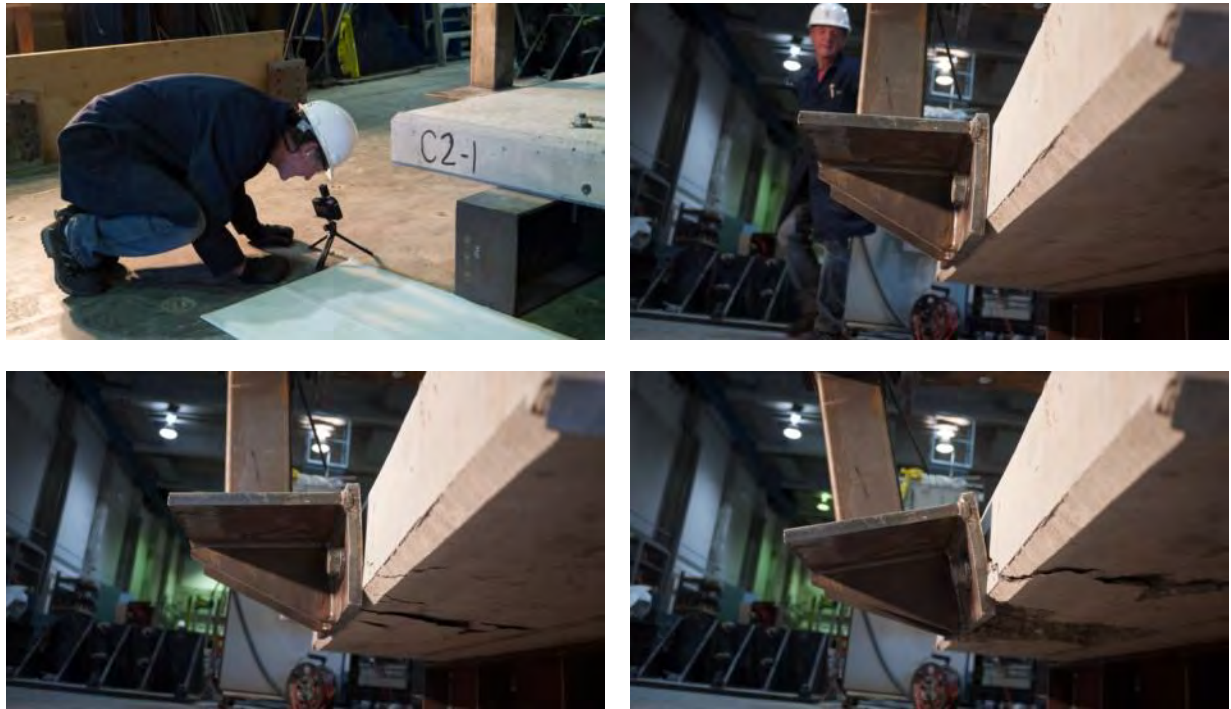


Figure 18: Images from C-2-1

Panel / Bracket Test C-2-2

Figure 19: Load/Deflection Curve of Top of Bracket, C-2-2

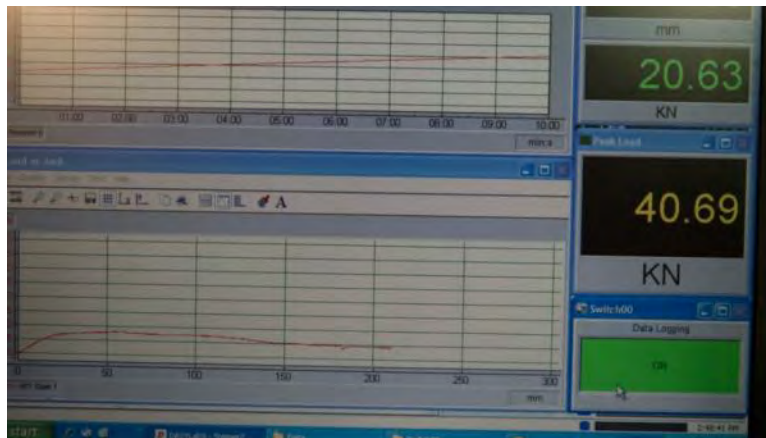
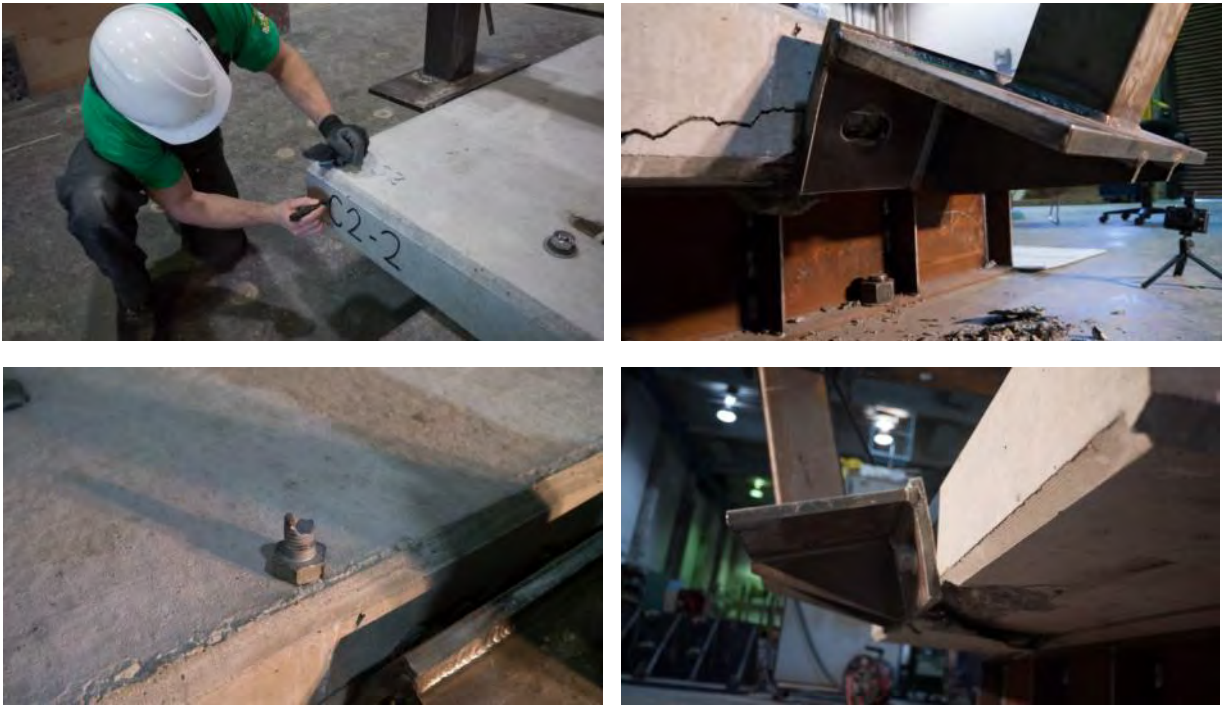


Figure 20: Images from C-2-2

Panel / Bracket Test D-1-1

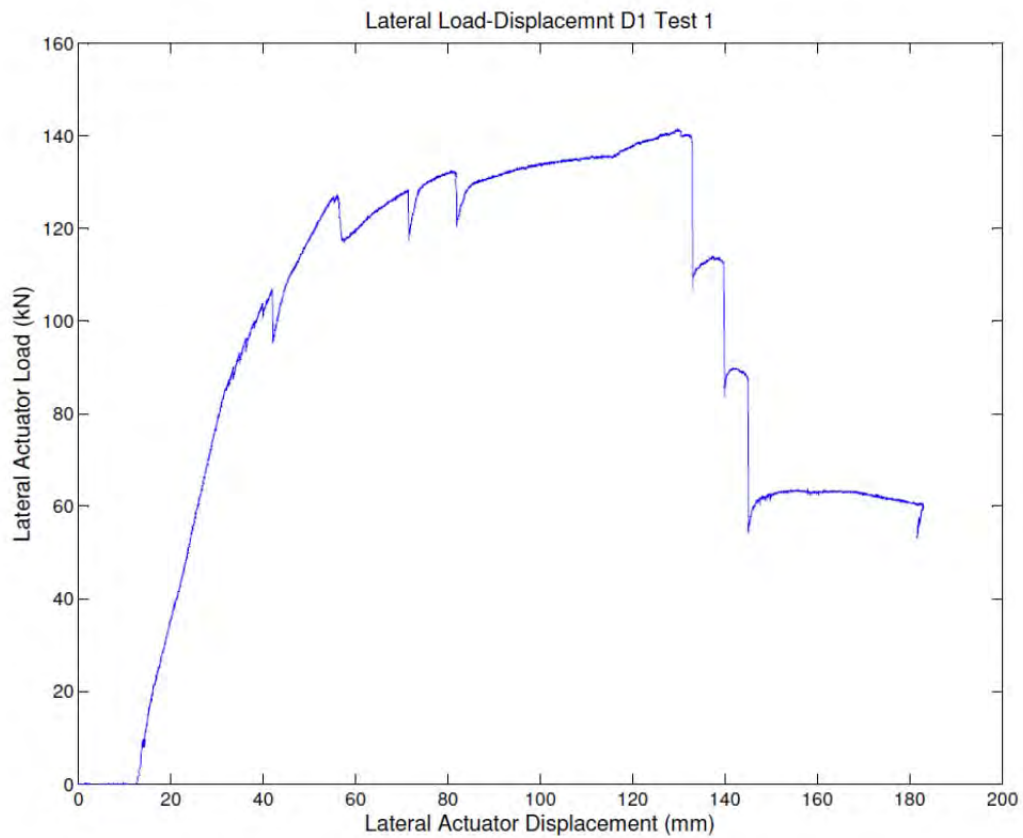


Figure 21: Load/Deflection Curve of Top of Bracket, D-1-1 (Note drops in load correspond to roughly the capacity of one of the four main anchor rods as they fractured in sequence)



Figure 22: Concrete panel attachment detail failing

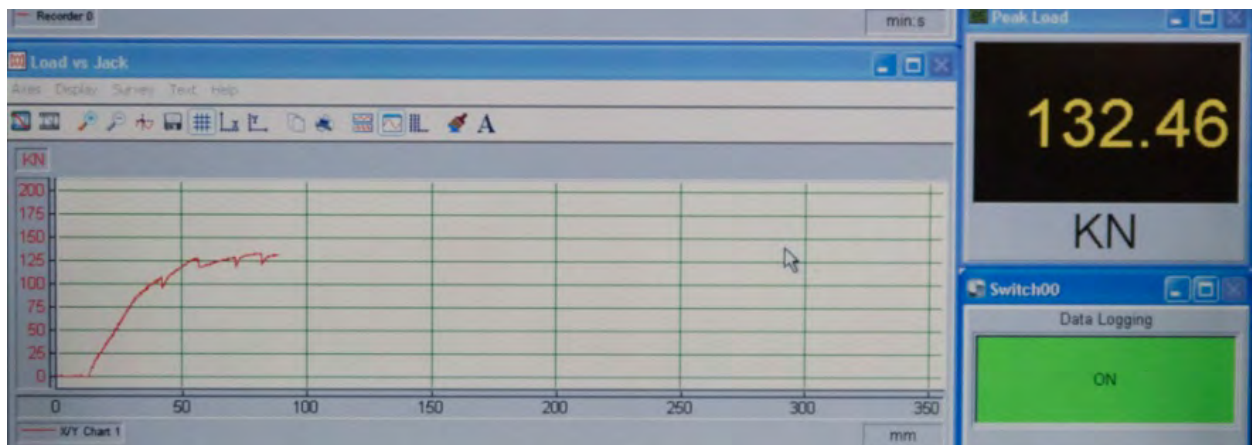


Figure 23: Example of computer screen readout during end phase of test

Panel / Bracket Test D-1-2

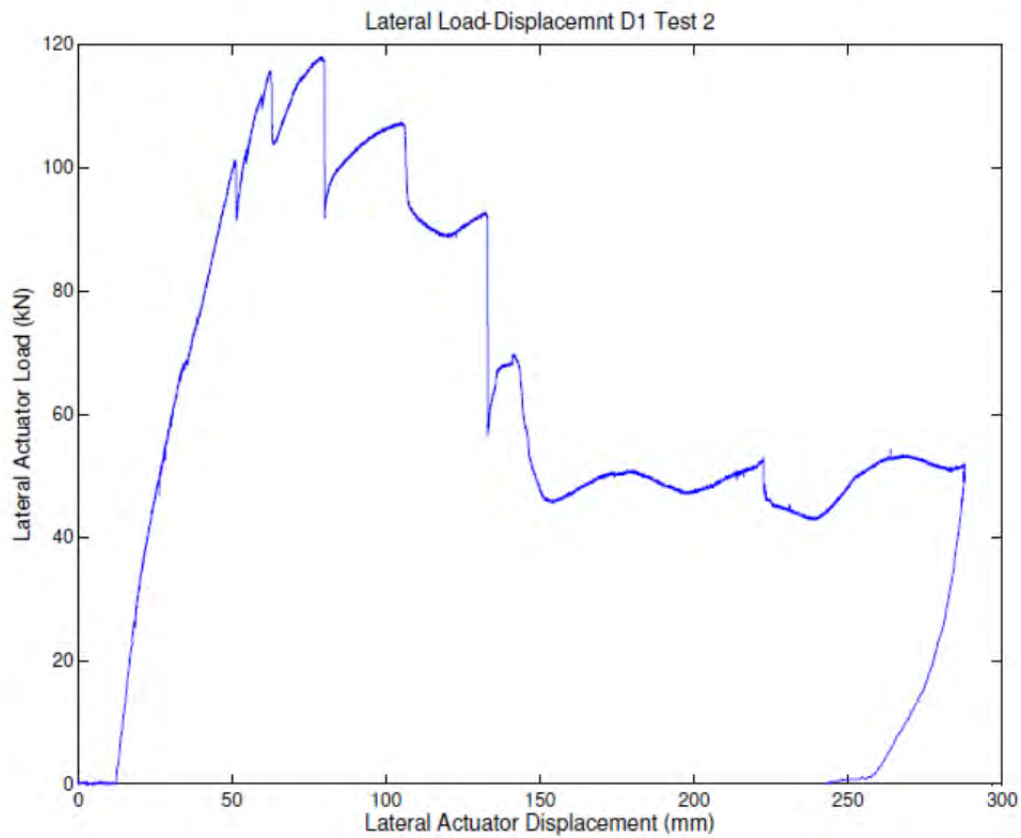


Figure 24: Load/Deflection Curve of Top of Bracket, D-1-2 (Note that the major drops in load correspond to anchor fracture and/or slippage)



Figure 25: Images from D-1-2

Panel / Bracket Test D-2-1

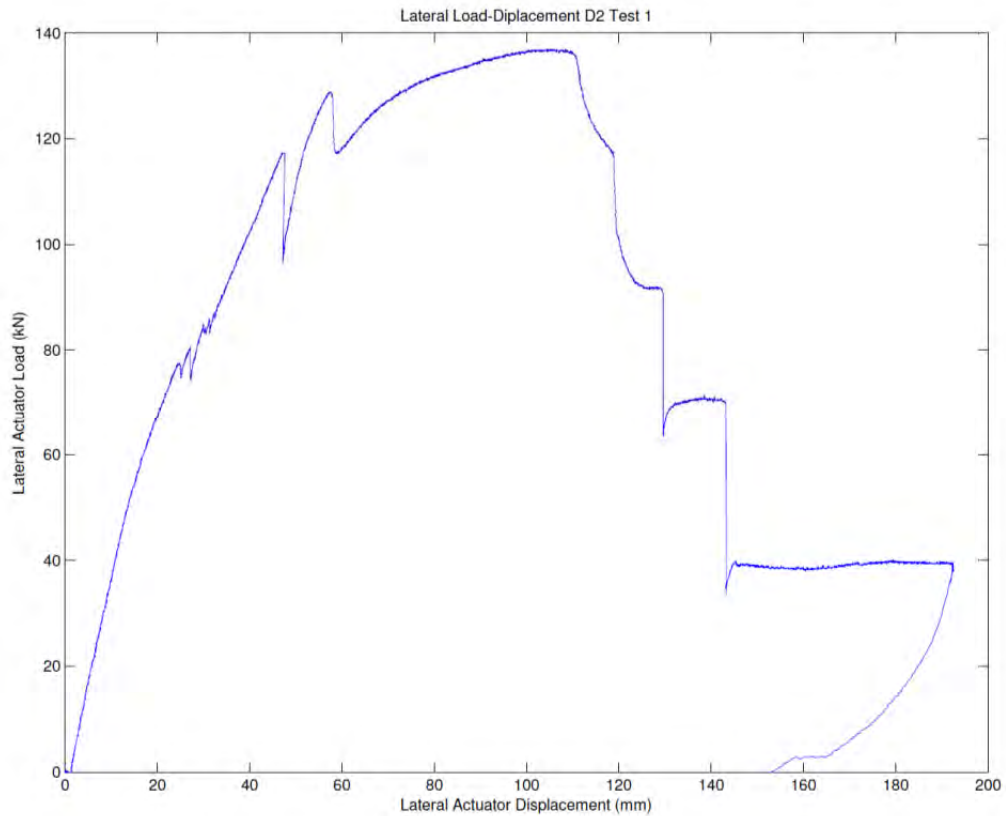


Figure 26: Load/Deflection Curve of Top of Bracket, D-2-1 (Note drops in load correspond to roughly the capacity of one of the four main anchor rods as they fractured in sequence)



Figure 27: Images from D-2-1

Panel / Bracket Test D-2-2

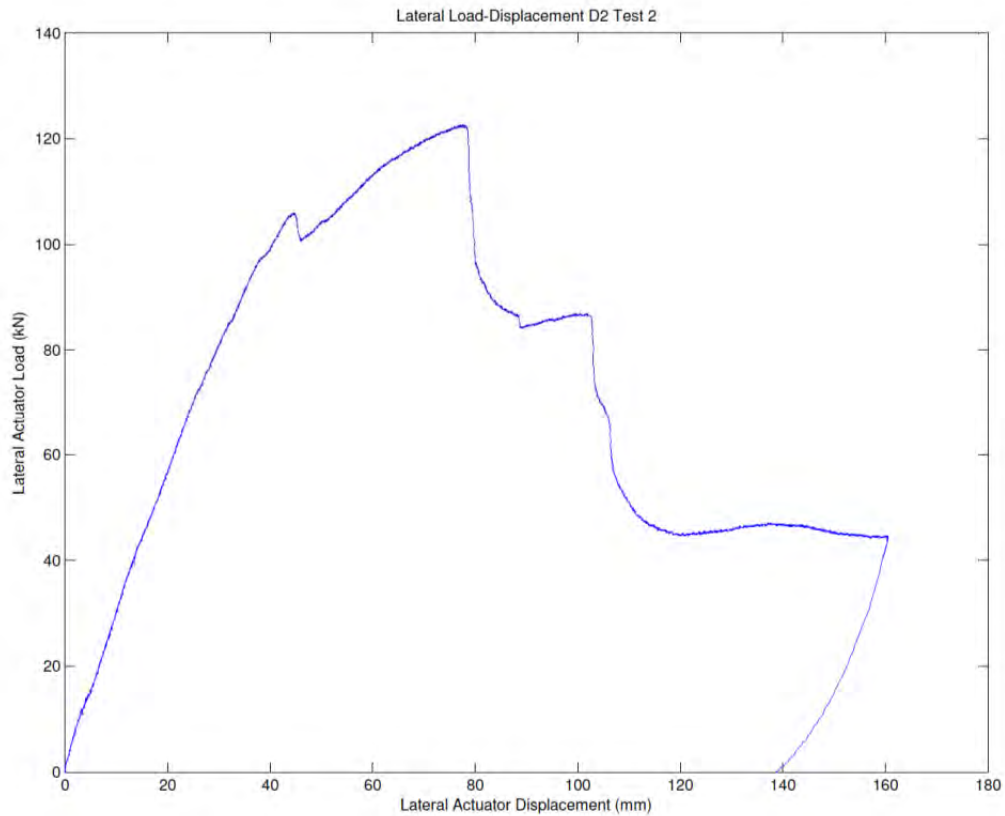


Figure 28: Load/Deflection Curve of Top of Bracket, D-2-2 (Note drops in load correspond to roughly the capacity of one of the four main anchor rods as they fractured in sequence)

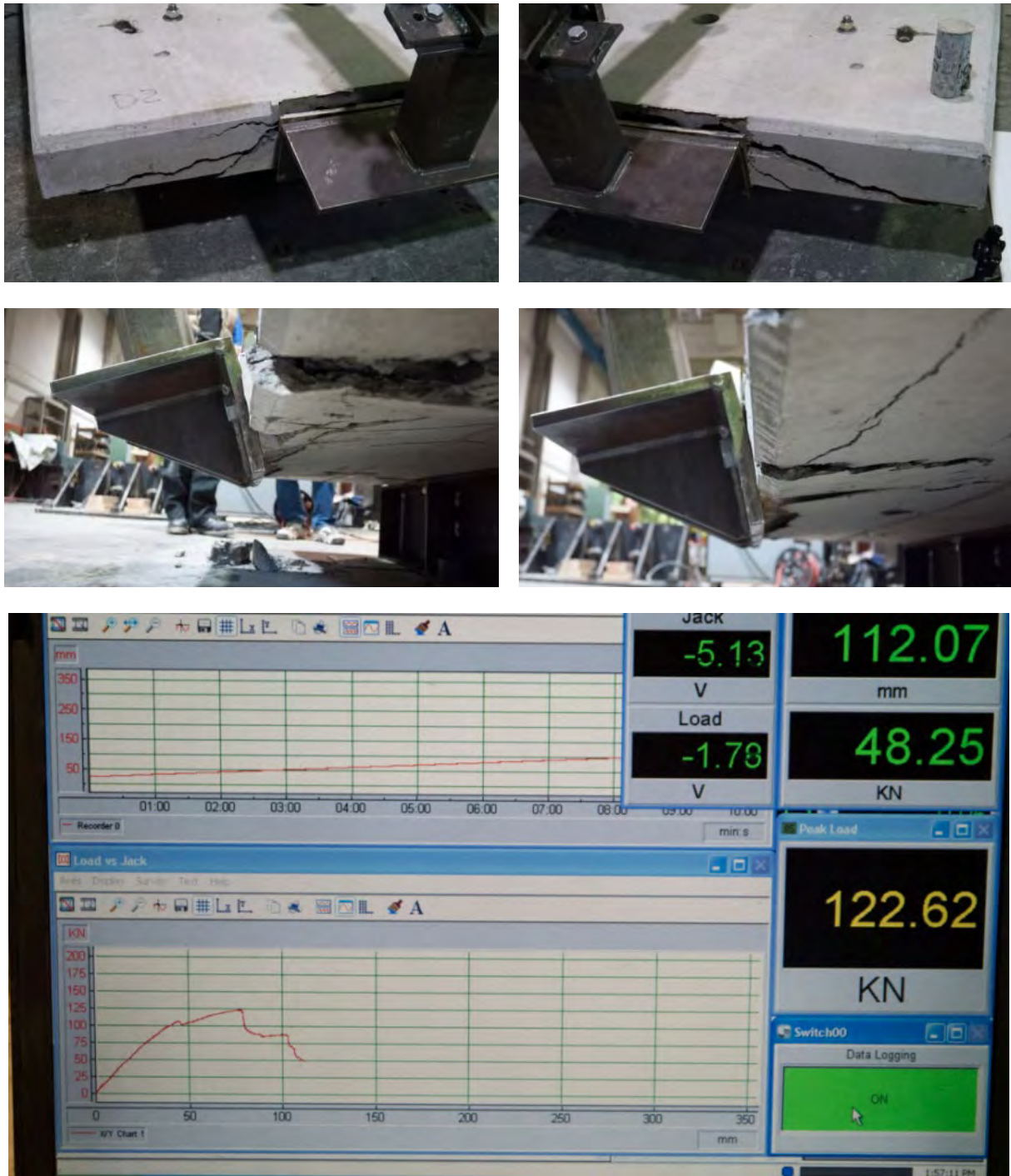


Figure 29: Images from D-2-2

Panel / Bracket Test E-1-1

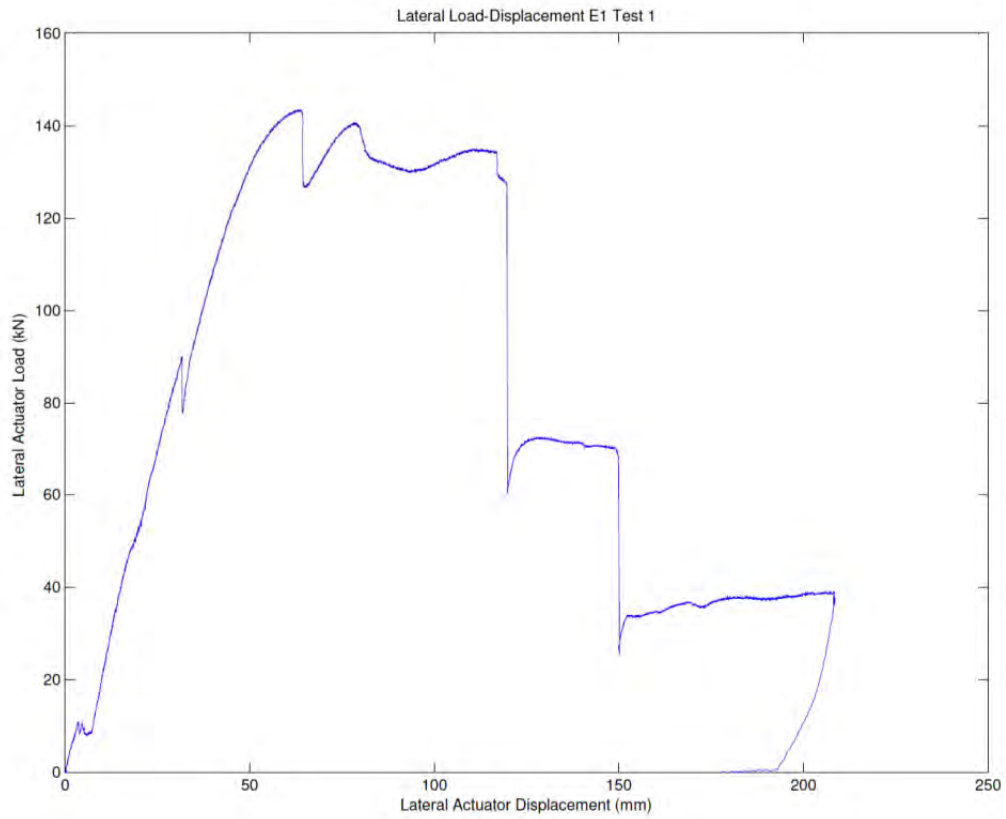


Figure 30: Load/Deflection Curve of Top of Bracket, E-1-1 (Note drops in load correspond to roughly the capacity of one of the four main anchor rods as they fractured in sequence)



Figure 31: Images from E-1-1

Panel / Bracket Test E-1-2

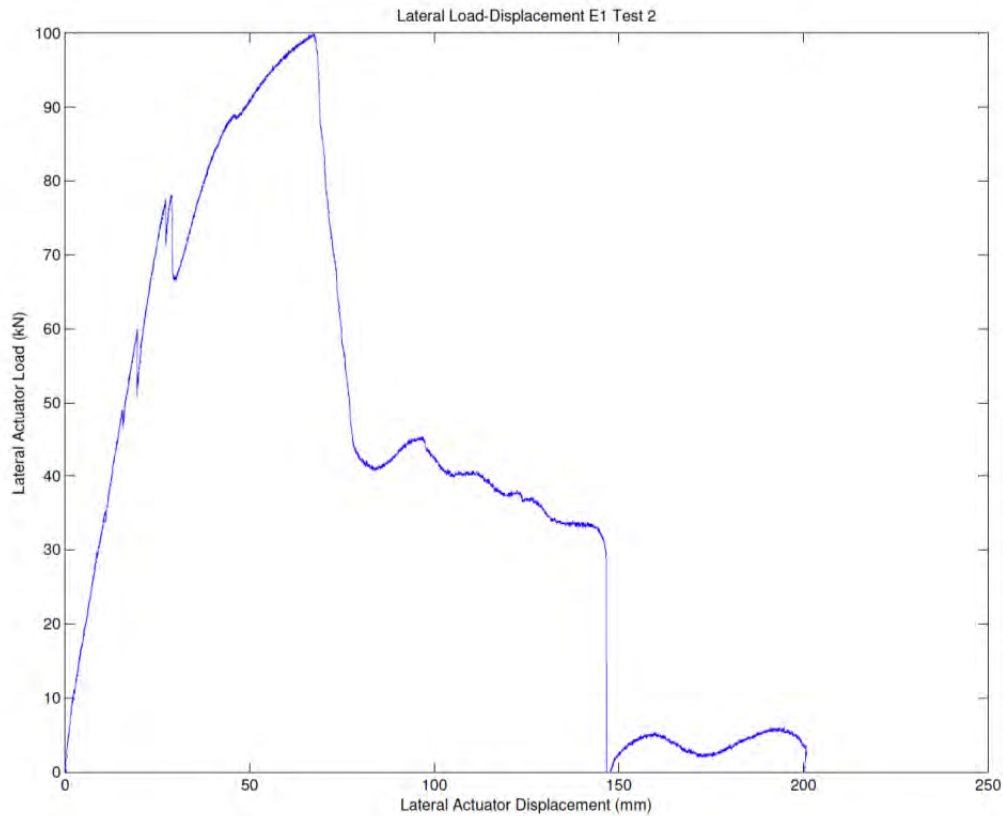


Figure 32: Load/Deflection Curve of Top of Bracket, E-1-2 (Note initial slipping of support apparatus caused initial drops in load during the initial portion of the loading curve. There may have been an initial preload of roughly 50 kN which resulted in the low lateral load capacity of this specimen)



Figure 33: Images from E-1-2

Panel / Bracket Test E-2-1

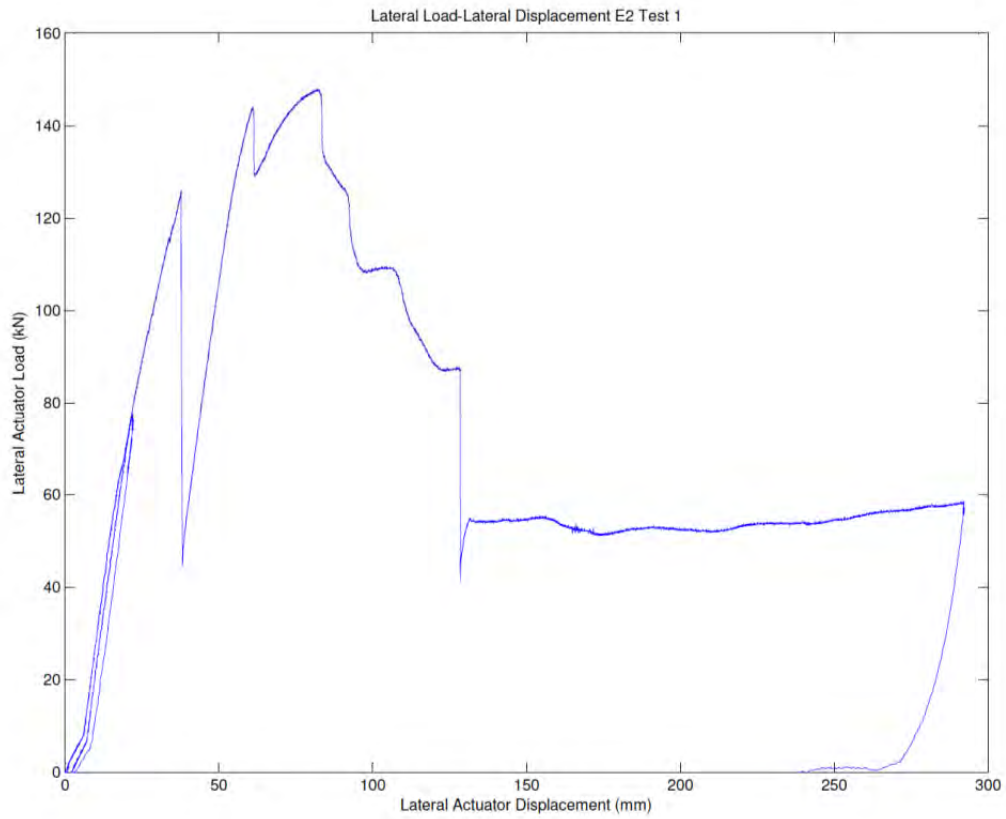


Figure 34: Load/Deflection Curve of Top of Bracket, E-2-1 (Note drop in load of 83 kN at lateral displacement of 39 mm due to panel support apparatus shifting under load)



Figure 35: Images from E-2-1

Panel / Bracket Test E-2-2

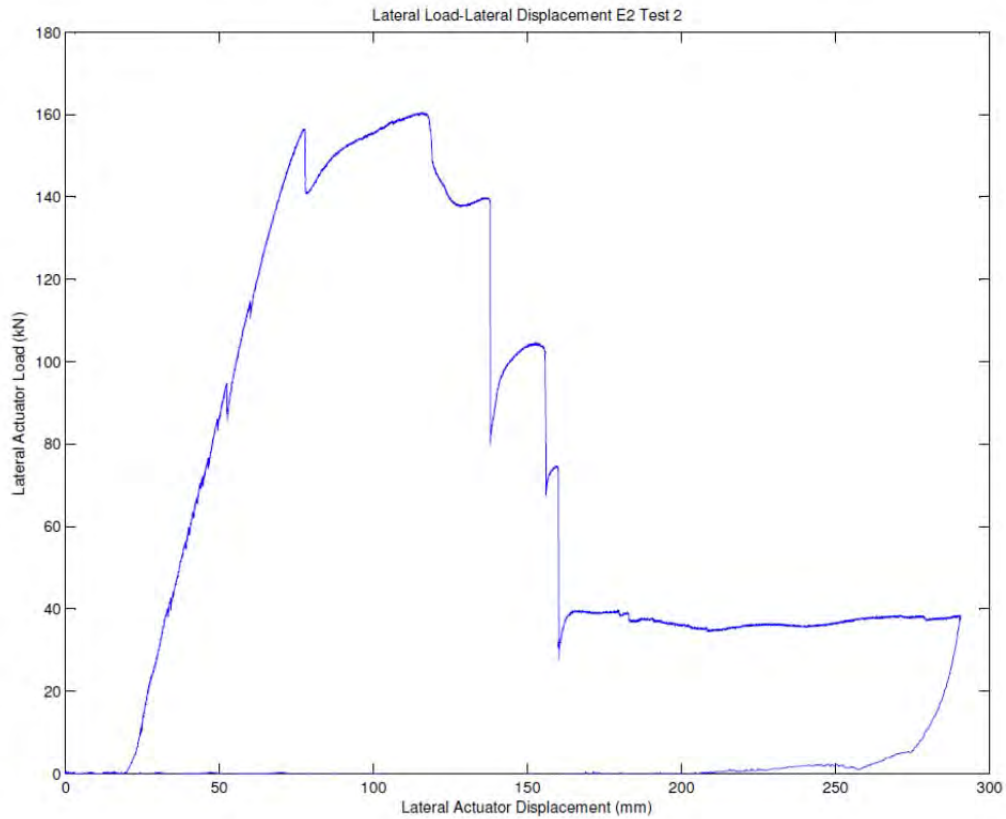


Figure 36: Load/Deflection Curve of Top of Bracket, E-2-2 (Note that the major drops in load correspond to the capacity of an anchor bar as it fractured)

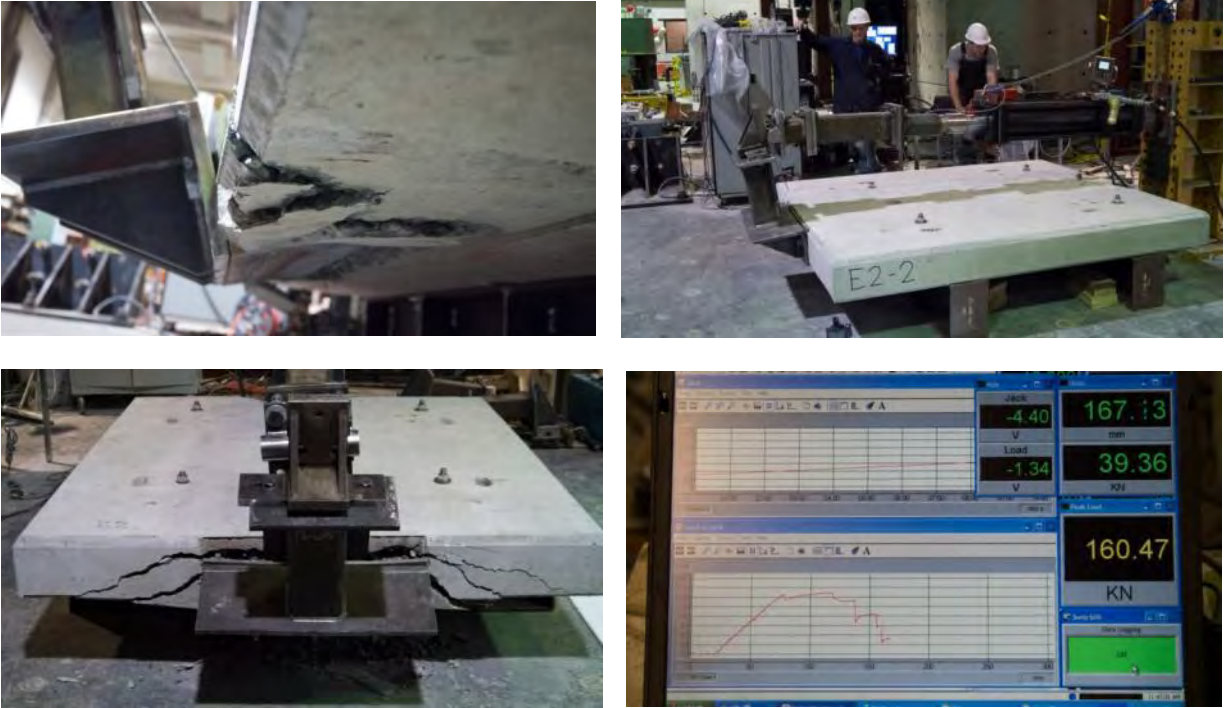


Figure 37: Images from E-2-2

Panel / Bracket Test G-1-1

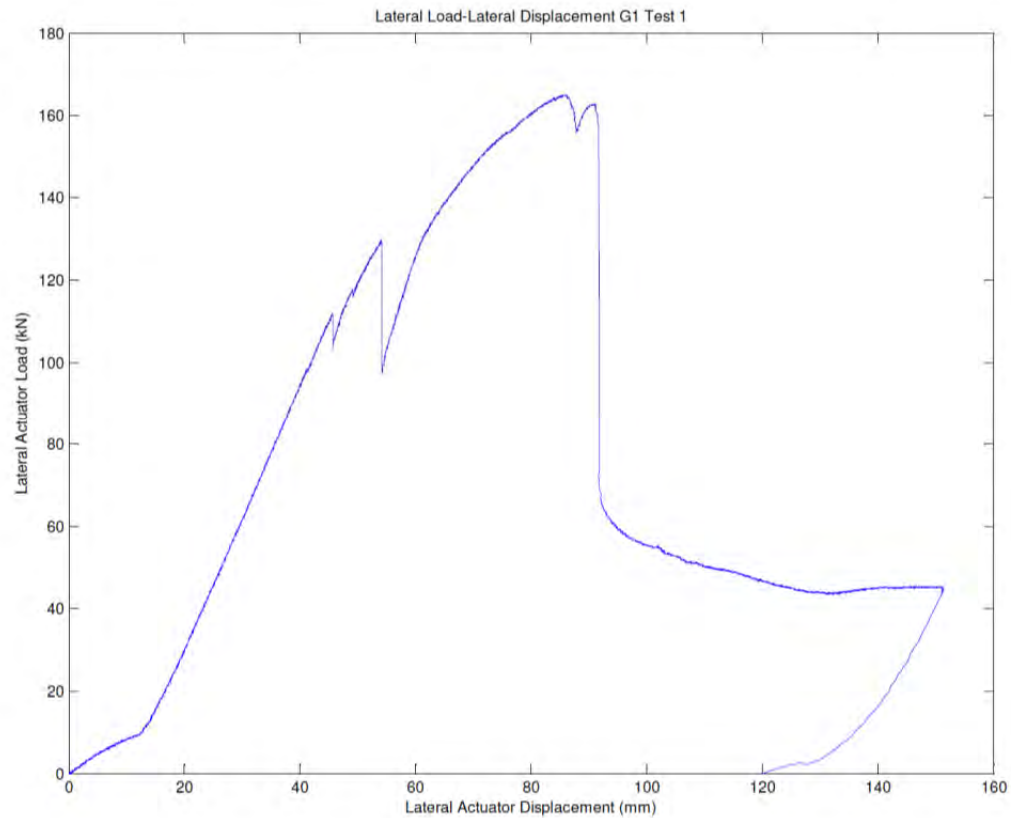


Figure 38: Load/Deflection Curve of Top of Bracket, G-1-1 (Note that the primary failure mechanism was characterized by the concrete cover above the Nelson studs breaking away as the rigid studs reached a critical curvature demand)



Figure 39: Images from G-1-1, loading to bolts stripping out of sockets

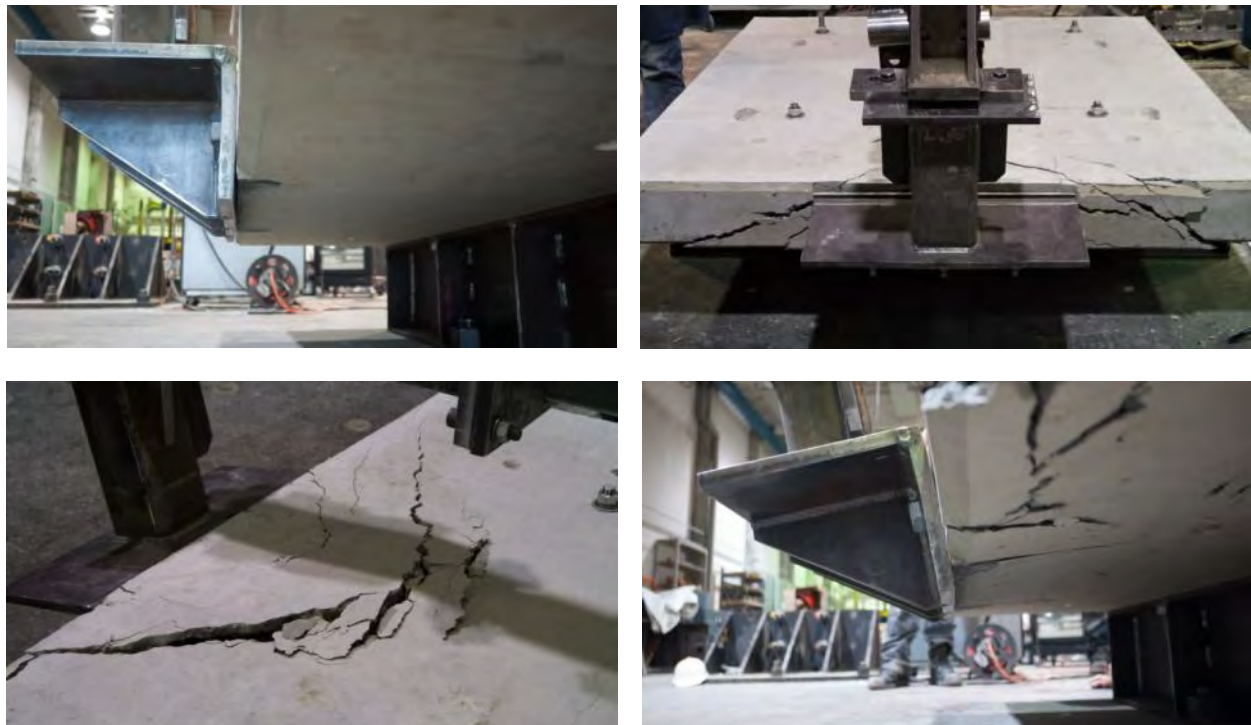


Figure 40: Images from G-1-1, new bolts, slightly longer (1/4")



Figure 41: Images from G-1-1, failure inspection after top concrete removal

Panel / Bracket Test G-1-2

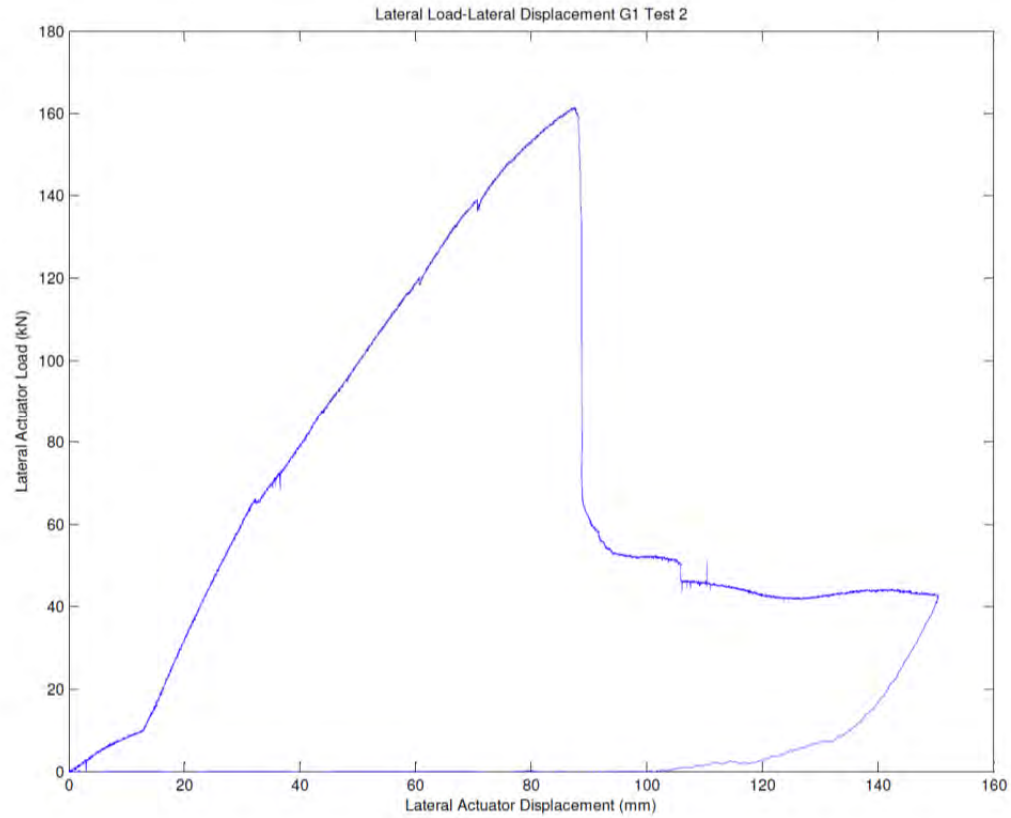


Figure 42: Load/Deflection Curve of Top of Bracket, G-1-2 (Note that the primary failure mechanism was characterized by the concrete cover above the Nelson studs breaking away as the rigid studs reached a critical curvature demand)

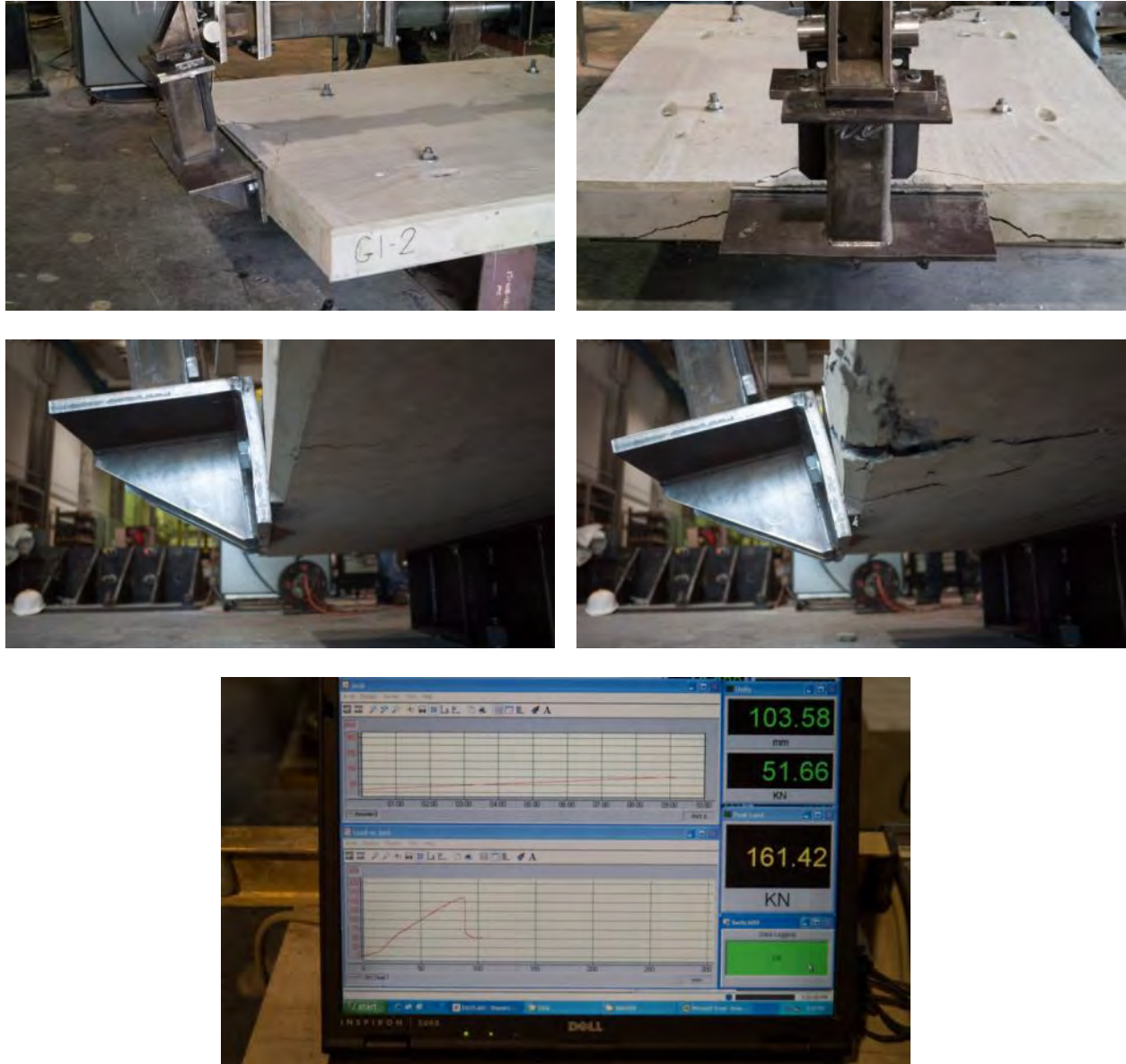


Figure 43: Images from G-1-2

Panel / Bracket Test H-1-1

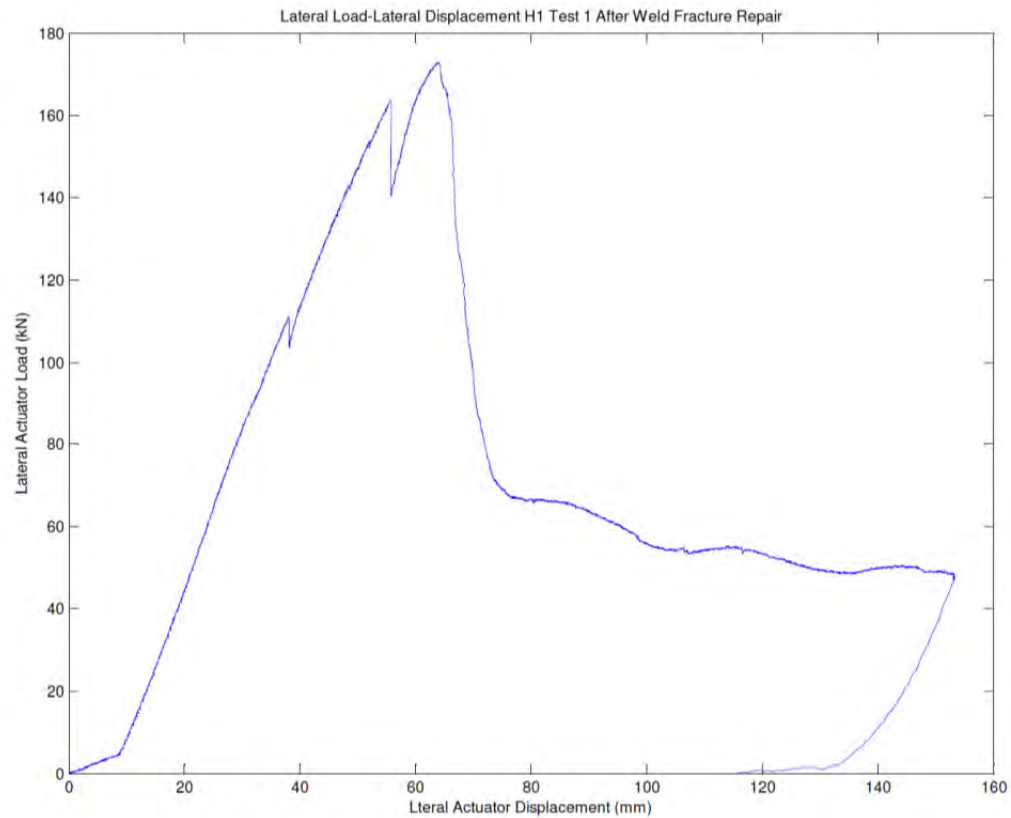


Figure 44: Load/Deflection Curve of Top of Bracket, H-1-1 (Note that the primary failure mechanism was characterized by the concrete cover above the Nelson studs breaking away as the rigid studs reached a critical curvature demand)

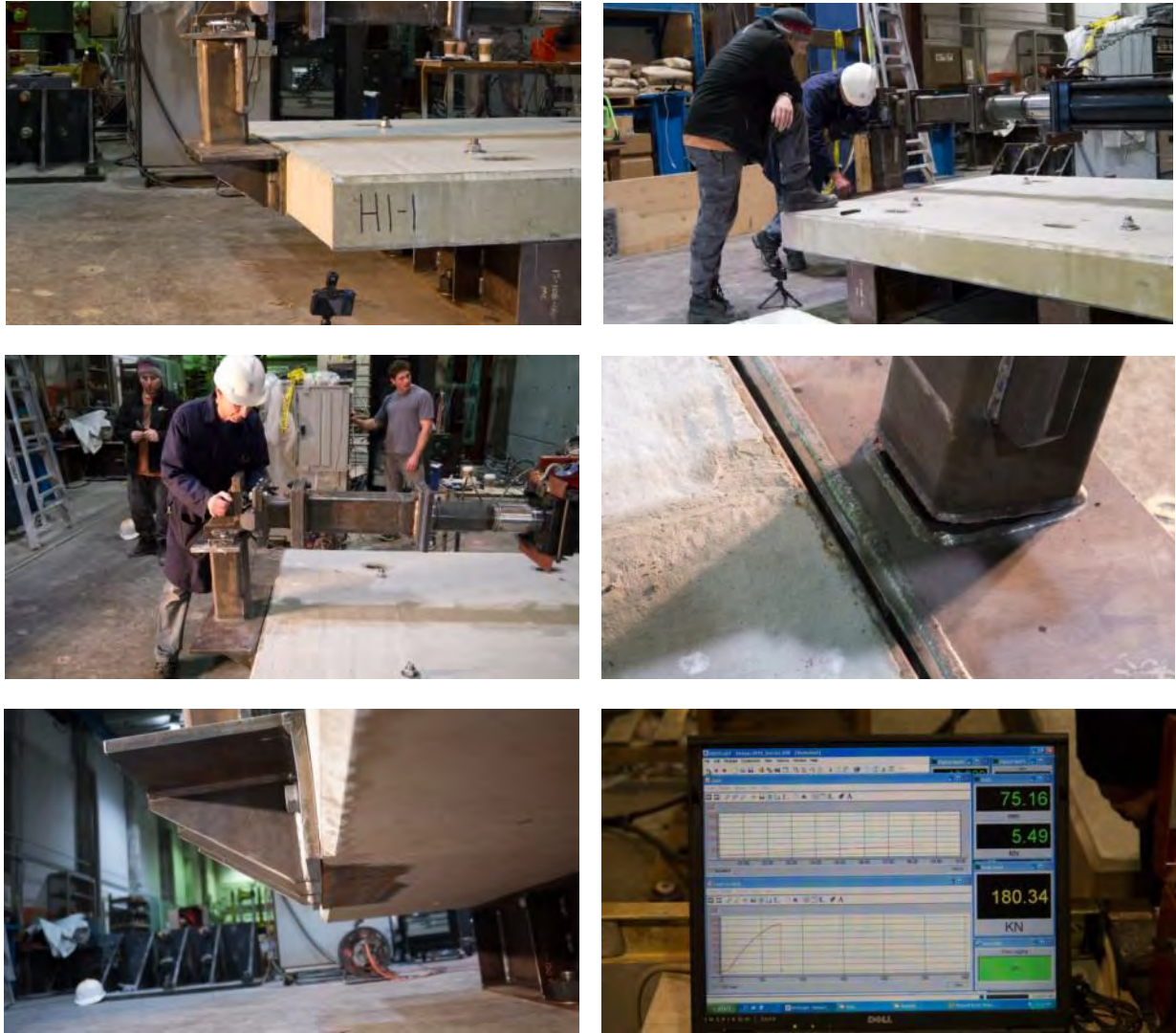


Figure 45: Images from H-1-1 to weld failure at post foot

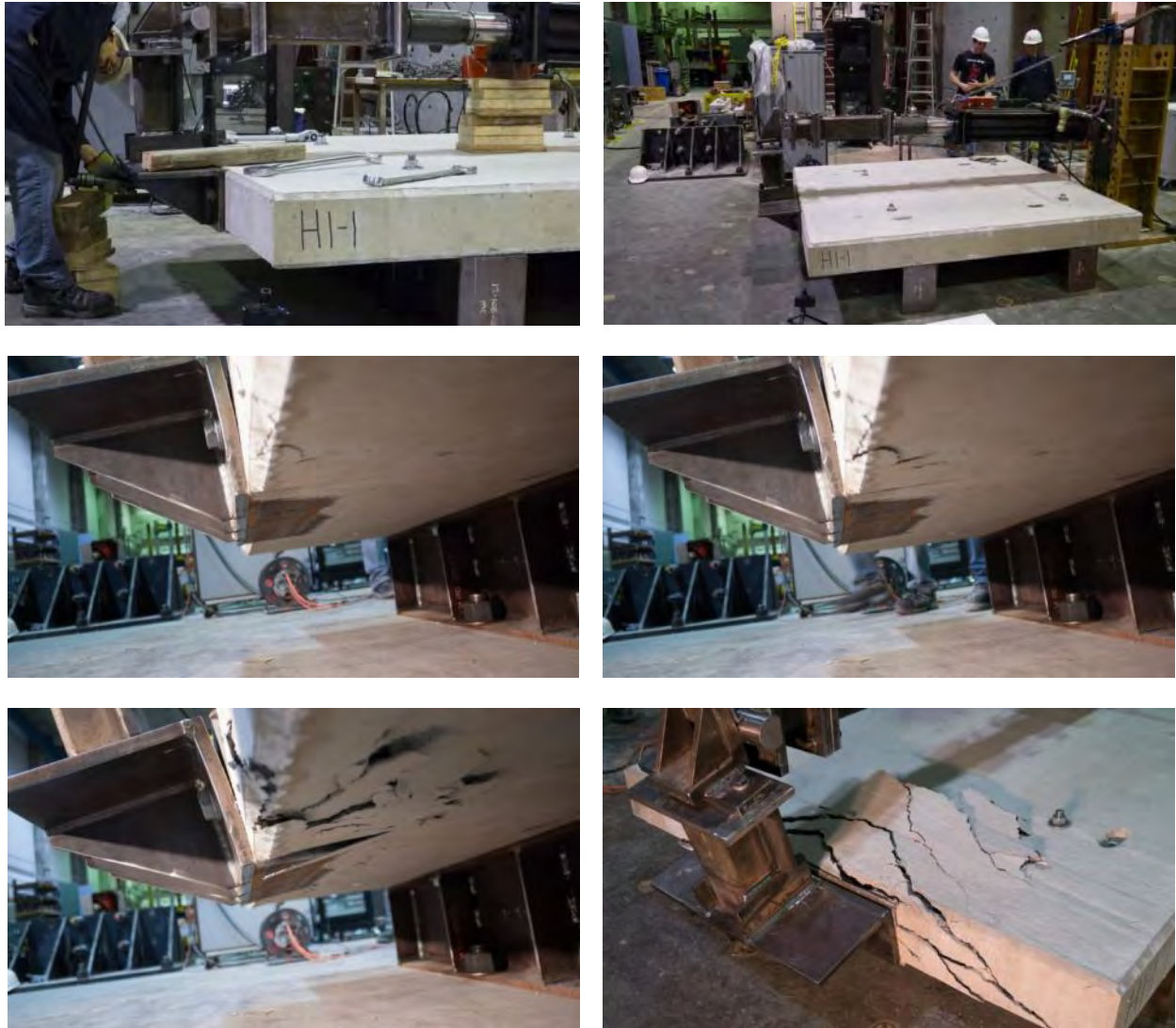


Figure 46: Images from H-1-1 with reinforced post

Panel / Bracket Test H-1-2

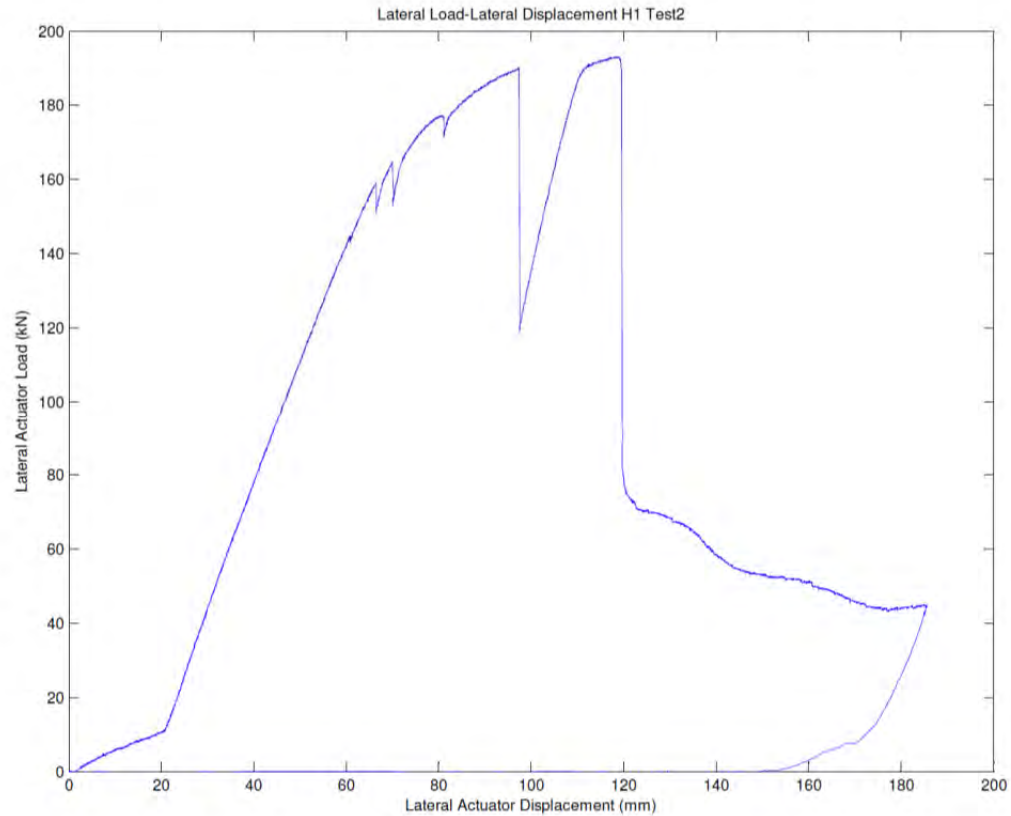


Figure 47: Load/Deflection Curve of Top of Bracket, H-1-2 (Note the major drop in load at 97 mm lateral displacement was caused by the slippage of the support apparatus. Also note that the primary failure mechanism was characterized by the concrete cover above the Nelson studs breaking away as the rigid studs reached a critical curvature demand)





Figure 48: Images from H-1-2

Interpretation of Mechanism of Failure

The failure mechanisms and bracket behaviour varied for each design. In general however, the brackets used on the panels fell into three primary categories of similar behaviour and failure mechanisms.

Panels A, B and C all failed primarily due to crushing of the compression concrete beneath the anchors and up until the inset drip groove. There was very little surface cracking on the top of the panels as the relatively small compression zone of the bottom portion of the panel failed at load levels low enough that there was little strain on the anchors. The cracking was located only on the bottom side of the panel in the form of diagonal shear cracks extending to the drip groove from which they terminated. After the compression zone crushed and spalled away, the loads were transferred almost entirely through bending to the bolts connecting the brackets to the panels. In one instance, bolt failure eventually occurred after significant bending stress and strains had been induced within the bolts.

Panels D and E failed primarily due fractures of the exterior anchors with the most development length. After the majority of these four anchors had fractured, the interior four short anchors with insufficient development length would begin to pull out along their length. Interestingly, the most exterior, well developed anchor bars would fracture prior to the adjacent bars. This behaviour is not fully understood and may be due to varying workmanship during the welding. The cracking for these panels was extensive. On the top surface, the cracking consisted of flexural cracks extending over the breadth of the panels perpendicular to the loading. These cracks developed from roughly 300 mm from the bracket to the support holes on the opposite side of the panel. On the top surface near the bracket there were diagonal shear cracks forming a semicircular shape extending 250 mm along the length of the panel and 200 mm on either side of the bracket along the breadth of the panel. On the side face of the panel there were diagonal cracks formed at approximately 30° angles on either side of the bracket. On the bottom face of the panel, diagonal shear cracks extended from the bracket to the drip groove.

Panels G and H failed due to cover spalling on the top surface as the highly rigid Nelson studs experienced high levels of curvature. There were first flexural cracks extending over the length of the panel similar to what was observed for Panels D and E above. This was followed by shear cracks forming on the top surface at a distance corresponding to the length of the Nelson studs. At the peak load, the Nelson studs would spall off the cover concrete. At this point the load would remain nearly constant as the remaining anchors pulled out of the concrete. Similar to the D and E Panels, the cracking on the sides and bottom consisted of diagonal shear cracking to the bottom of the panels and the drip groove respectively.

Conclusions and Observations

As the testing team was not involved in the design or the analysis of the tested concrete decks and the barriers, conclusions from the experimentalists should be restricted to the experimental testing. The chosen test set-up proved to be appropriate. Predictions of load levels and deflection were correct and helped to choose the proper test equipment.

Tests could be kept economical in timing and budget. Therefore a larger number of test specimens were tested than contracted. This can be attributed to proper planning and engaged contributions by students and technicians.

A close cooperation with the Ministry of Forests, Lands and Natural Resource Operations as well with the engineers from Associated Engineering enabled a flexible adjustment of test methods and targets.

The tests showed impressively the importance of tight quality control of concrete production.

Other observations during the testing lead to the following conclusions:

- The barrier resistance against loads at the end of the bracket is largely influenced by the load transfer mechanism between bracket and concrete deck. Obviously, the larger the contact area to the concrete is, the greater is the resisting moment.
- When premature spalling can be avoided, and thus avoiding a reduction in the level arm of the contact area, the bracket achieve a higher capacity.
- Similarly obvious is the direct relationship of concrete strength to connection resistance.
- Embedment of anchoring bolts is of importance, although choices in embedment length or location relative to the deck thickness are limited.
- Thickness of the deck can increase the performance of the bracket. This is theoretically directly related to the moment of inertia about the horizontal deck axis.
- It can be envisioned that other methods of connecting bridge barriers to bridge decks are more economical or provide a higher degree of safety. In particular, the bridge barriers should be investigated how they act as a system along an entire bridge, not only as one individual post. This would be a great area of novel research and development. An interdisciplinary research group consisting of members of the practicing profession and academic research might show new routes to success.

Core Compression Tests

Table 3: Concrete Strength Tests

Test Sample #	Date	Age at time of testing (days)	Specimen end faces	Failure Type	Peak Load (kN)	Peak Stress (MPa)	Sample Size	
							Dia. (mm)	Area (mm ²)
A1	8-Mar-13	44	machined	cone & split	237	43.80	83	5411
B1	13-Mar-13	49	machined	cone	262.8	48.57	83	5411
C1	13-Mar-13	35	machined	cone & shear	258.2	47.72	83	5411
D1	27-Feb-13	35	raw	shear	142.4	26.31	83	5411
D2	27-Feb-13	35	raw	cone & shear	187.6	34.70	83	5411
E1	27-Feb-13	35	raw	cone & shear	175.7	32.47	83	5411
E2	28-Feb-13	36	machined	cone & split	174.1	32.18	83	5411
G/H 1	3-Apr-13	70	machined & padded	local failure at corners	135.6	15.96	102	8171
G/H 2	3-Apr-13	70	machined & padded	columnar	90.9	11.12	102	8171
G/H 3	15-Apr-13	82	machined	cone	261.4	32.12	102	8171
G/H 4	15-Apr-13	82	machined	cone	310.7	36.58	102	8171
675D	27-Feb-13	35	raw	columnar	209.32	25.62	102	8171
676D	27-Feb-13	35	raw	columnar	217.10	26.57	102	8171
Fabricator's Compressive Test Cylinder Results								
A1*	30-Jan-13	7	--	--	294	36.3	101.	8118

							6	
B1*	20-Feb-13	28	--	--	317	39.1	101.6	8118
C1*	20-Feb-13	28	--	--	308	38.5	101.6	8118
E1*	30-Jan-13	7	--	--	258	31.8	101.6	8118
A2*	21-Mar-13	7	--	--	257	31.7	101.6	8118
B2*	11-Apr-13	28	--	--	335	41.3	101.6	8118
C2*	11-Apr-13	28	--	--	338	41.5	101.6	8118
D2*	03-Apr-13	20	--	--	320	39.5	101.6	8118
E2*	21-Mar-13	7	--	--	233	28.7	101.6	8118

Note: Samples 675D, 676D, D1, D2, and E1 had raw ends, which was causing earlier failure due to uneven loading.

*Fabricators naming, not related to panel name. A1, B1, C1 and E1 for Panels A,B,C,D and E. A2, B2, C2, D2 and E2 for Panels G and H.

Testing of the specimens should be done according to CSA A23.1/A23.2 *Concrete materials and methods of concrete construction/Test methods and standard practice for concrete*. Grinding (machining) of specimen end face to produce uniform bearing as consistent with the CSA standard is acceptable (according to e-mail from Brian Chow, March 14).

Note: The padded concrete cylinder specimens in the above table (G/H 1 and G/H 2) utilized neoprene pads on their end contact surfaces during the cylinder testing. It was determined that these pads negatively affected the cylinder testing results by causing preemptive columnar and local corner failures.

Table 4: Remarks and Observations for Strength Tests

Test Sample #	Observations / Remarks
A1	Machined smooth cylinder faces exhibiting cone and split type fracture
B1	Machined smooth cylinder faces exhibiting cone type fracture
C1	Machined smooth cylinder faces exhibiting cone and shear type fracture
D1	Originally cast cylinder faces exhibiting shear type fracture
D2	Originally cast cylinder faces exhibiting cone and shear type fracture
E1	Originally cast cylinder faces exhibiting cone and shear type fracture
E2	Machined smooth cylinder faces exhibiting cone and shear type fracture
G/H 1	Machined smooth cylinder faces, rubber pads used during testing, specimen failed in localized zone at top and bottom corners
G/H 2	Machined smooth cylinder faces, rubber pads used during testing, and multiple columnar type fractures
G/H 3	Machined smooth cylinder faces exhibiting cone type fracture
G/H 4	Machined smooth cylinder faces exhibiting cone type fracture
675D	Originally cast cylinder faces exhibiting columnar type fracture
676D	Originally cast cylinder faces exhibiting columnar type fracture



Figure 49: Compression Test Specimen

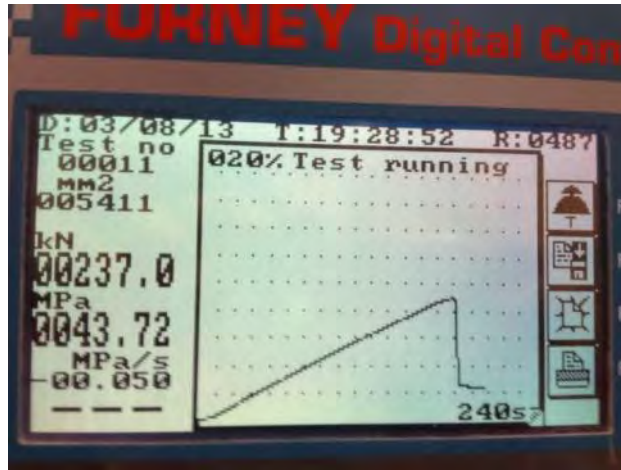


Figure 50: Core Compression Specimen A-1, precision machined compression surfaces

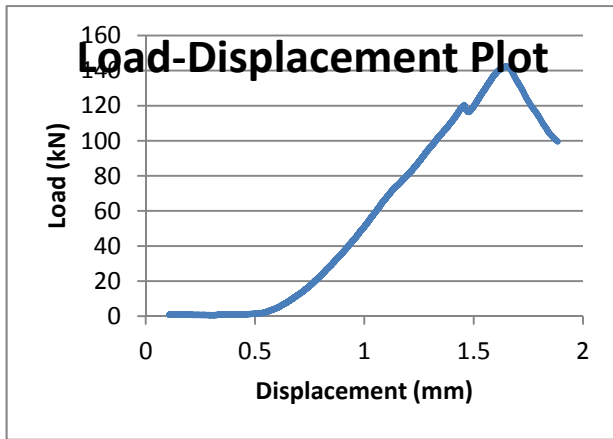


Figure 51: Core Compression Test D-1

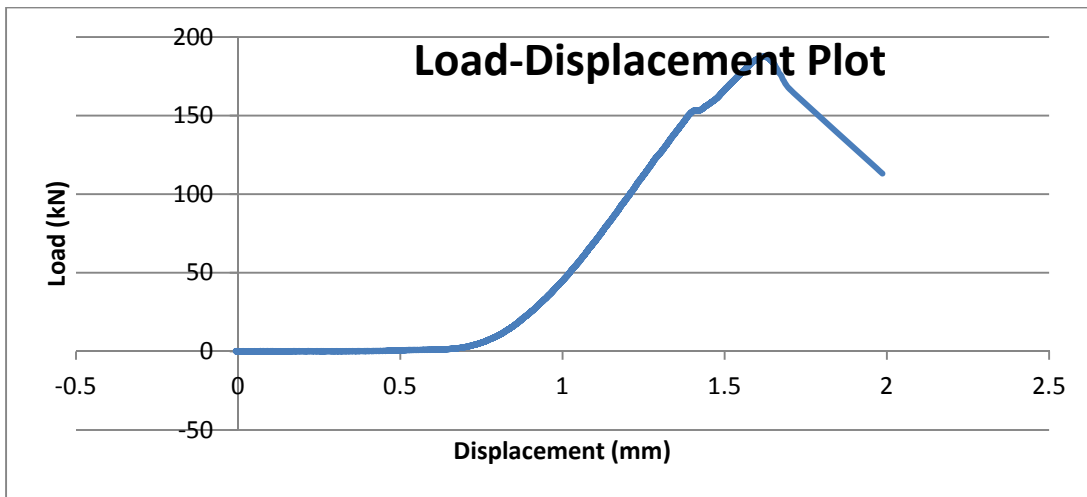


Figure 52: Core Compression Test D-2

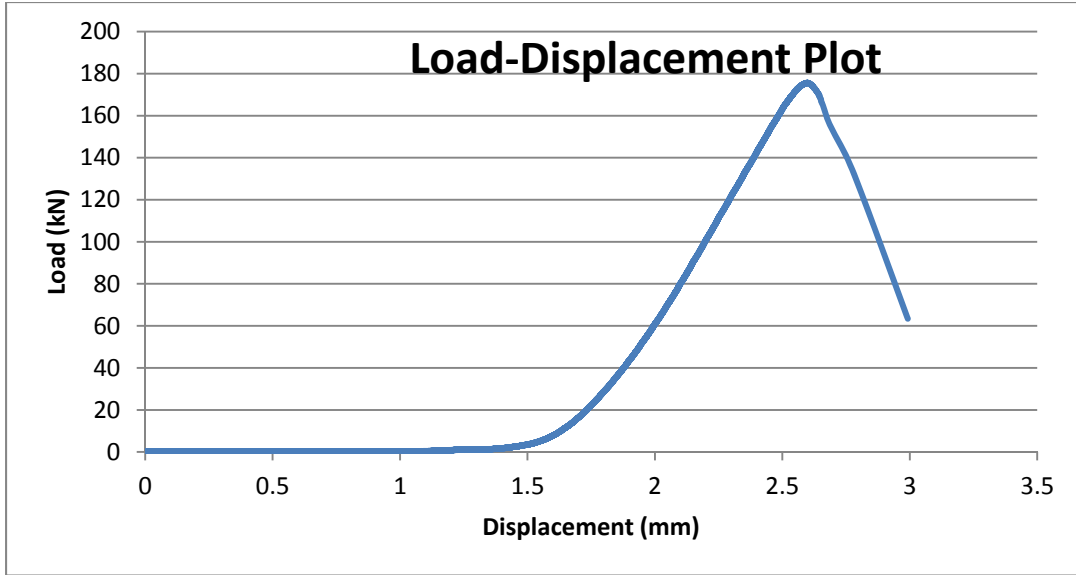


Figure 53: Core Compression Test E-1

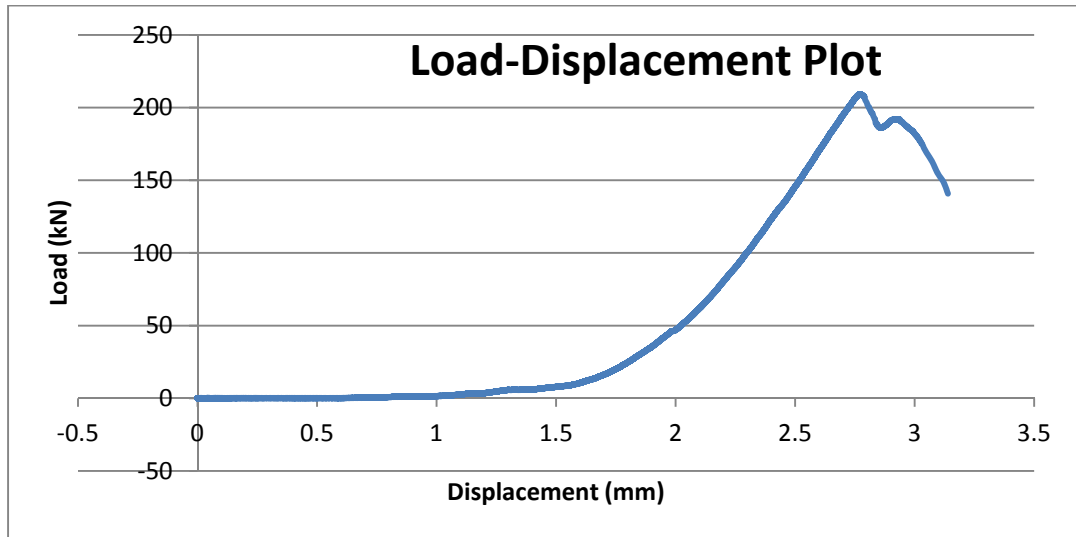


Figure 54: Core Compression Test 675D

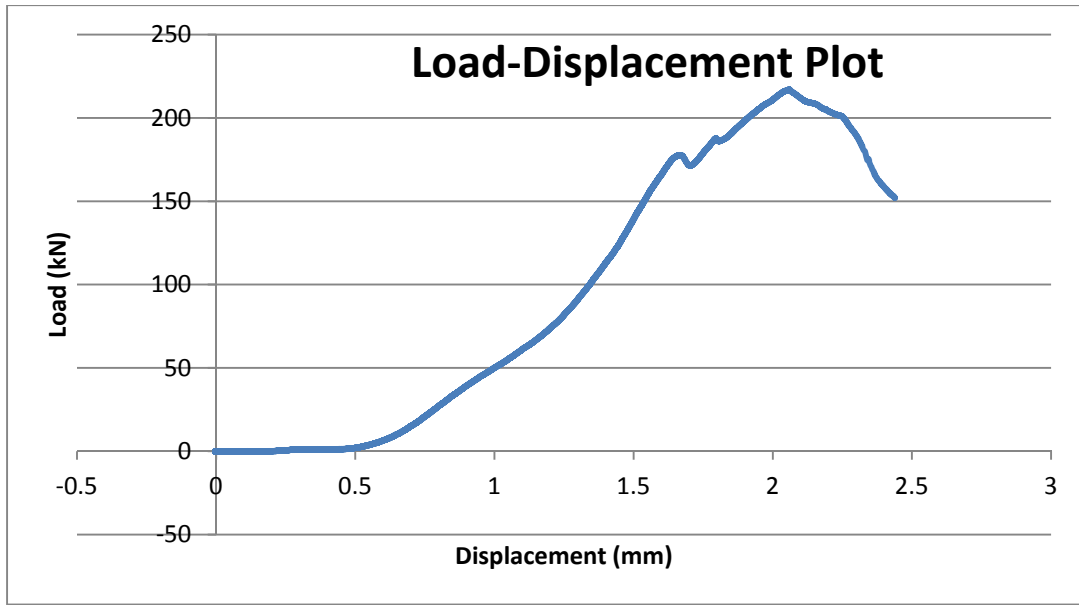


Figure 55: Core Compression Test 676D

Rebar System Photos









Consulting and General Services Contract

<p>CONTRACT./FILE NO: 1070-20/OT13FHQ340</p>	<p>THIS AGREEMENT DATED FOR REFERENCE THE 26th DAY OF SEPTEMBER, 2012.</p>
<p>PROJECT DESCRIPTION: RESEARCH, THEORETICAL ANALYSIS AND LAB TESTING OF FLNR STANDARD CURB SYSTEMS TO DETERMINE AND CONFIRM STRENGTHS OF EXISTING AND PROPOSED SYSTEMS.</p>	

BETWEEN:

HER MAJESTY THE QUEEN IN RIGHT OF THE PROVINCE OF BRITISH COLUMBIA, as represented by the MINISTER OF FORESTS, LANDS AND NATURAL RESOURCE OPERATIONS

Engineering Branch

3rd Floor, 1520 Blanshard Street, Victoria, BC V8W 3K2

PO Box 9525 Stn Prov Govt, Victoria, BC V8W 9C3

Phone Number: (250) 953-4370.....FAX Number: (250) 953-3687

Ministry Representative: Brian Chow

E-mail Address: Brian.Chow@gov.bc.ca

(the "Province", "we", "us", or "our" as applicable)

AND:

University of British Columbia

6250 Applied Science Lane, Vancouver, BC V6T 1Z4

Phone Number: (604) 822-6301.....FAX Number: (604) 822-6901

E-mail Address: sigi@civil.ubc.ca

Contractor Representative: Siegfried F. Stiemer, Dr.Ing. (Ph.D), Professor of Civil Engineering

Corporate Business Number:

WorkSafe BC and/or Personal Optional Protection Number:

(the "Contractor", "you", or "your" as applicable)

The Province wishes to retain the Contractor to provide the Services specified in Schedule A and, in consideration for the remuneration set out in Schedule B, the Contractor has agreed to provide those Services, on the terms and conditions set out in this Agreement.

(complete contract definition in Document *Testing-Standard Curb Systems-UBC.PDF*.)

Schedule A - Services

File: 1070-20/OT13FHQ340

Attachment to the Agreement with [University of British Columbia](#) for Research, Theoretical Analysis and Lab Testing of FLNR Standard Curb Systems to determine and confirm strengths of existing and proposed systems.

1. THE SERVICES

- 1.01 The Contractor shall conduct theoretical analysis and lab testing of FLNR Standard Curb Systems to determine and confirm strengths of existing and proposed systems consistent with proposal by Associated Engineering (BC) Ltd. dated September 5, 2012 (pages 2 of 4 attached).
- 1.02 The Contractor will work in collaboration with both the ministry and Associated Engineering (BC) Ltd., in developing testing protocols for the bridge barrier systems to be tested. The Civil Engineering department of UBC shall be responsible for developing the test protocol and conducting the actual testing. The Associated Engineering (BC) Ltd. will design the test specimens and support development of the test protocol and collaborate with UBC in the analysis of test results.
- 1.03 Test protocol(s) to be implemented shall be agreed upon by the Contractor, Associated Engineering (BC) Ltd. and the ministry representative prior to proceeding with any testing.

2. KEY PERSONNEL

The Services shall be performed by the following "Key Personnel":

- [Siegfried F. Stierner, Dr.-Ing. \(Ph.D\), Professor of Civil Engineering, University of British Columbia](#) and there shall be no substitution for the person(s) listed above without the prior consent of the Province.

3. CONSULTING AND PROFESSIONAL INDEMNITY

The Contractor and the Province agree that Section 11.01 of the Agreement is deleted and replaced with the following:

The Contractor hereby agrees to indemnify and save harmless the Province, its successor(s), assign(s) and authorized representative(s) and each of them from and against all losses, claims, damages, actions and causes of action (collectively referred to as "claims") that the Province may sustain, incur, suffer or be put to at any time either before or after the expiration or termination of this Agreement, that arise out of errors, omissions or negligent acts of the Contractor or its subcontractor(s), servant(s), agent(s) or employee(s) under this Agreement, excepting always that this indemnity does not apply to the extent, if any, to which the Claims are caused by errors, omissions or the negligent acts of the Province, its other contractor(s), assign(s) and authorized representative(s) or any other person.

Deck Panels, Requirements & Specifications

Ministry of Forests, Lands and Natural Resource Operations

Precast Concrete Bridge Test Deck Panels

Requirements & Specifications

Ministry Structure Number(s): Eng Br Test Panels 2012/13

Scope of Work

Fabricate and supply 7 precast concrete bridge deck test panels for Engineering Branch, Ministry of Forests, Lands and Natural Resource Operations (MFLNRO) and deliver to Dept. of Civil Engineering, UBC, 6250 Applied Science Lane, Vancouver, BC. Fabrication of test bridge deck panels to be consistent with practices for fabrication of Ministry of Forests, Lands and Natural Resource Operations concrete deck panels.

Terms and Conditions

Contractor General Qualifications

As these test panels must be fabricated in a manner consistent with typical practices to emulate “real” standard concrete bridge deck panels, bidders, as identified in their quote, must have successfully fabricated, supplied and delivered, to the Ministry of Forests, Lands and Natural Resource Operations, on time, at least 10 bridges utilizing Ministry of Forests, Lands and Natural Resource Operations standard concrete deck panels, within the past 2 years.

- * Proof for the purposes of the foregoing is required to be submitted within 4 business days of a request from the ministry, and must include, but is not necessarily limited to:
 - a. evidence that the bidder has successfully fabricated, supplied and delivered at least 10 precast concrete deck on steel girder bridges to the Ministry of Forests, Lands and Natural Resource Operations;
 - b. evidence that the bidder has successfully carried out and completed works of a similar nature or is otherwise fully capable of fulfilling a contract having the necessary qualifications;
 - c. a list of relevant fabricating equipment (and its condition) that the bidder intends to use to fulfil the contract;

- d. evidence that the personnel being utilized by the bidder to perform the works for this contract have the necessary professional standing, technical and trade qualifications, or licenses necessary to fulfil a contract; and,
- e. the name and contact information of the Professional Engineer who took responsibility for the design of the relevant products specified above.

The ministry shall be the sole and final judge of the sufficiency of the proof provided.

- The ministry may, at any time and from time to time, after closing time of this Invitation to Quote, require any bidder, or successful bidder, to satisfy the ministry, in its sole discretion, that they have the necessary qualifications, finances, equipment, fabrication site, material, personnel, and resources available to carry out the fulfillment of any contract resulting from this Invitation to Quote in a safe, competent manner, within the time limits, and any other requirements specified in the Invitation to Quote, including by delivering information to the ministry in writing. Any bidder, or successful bidder, asked to provide this information must comply with the request within 4 business days from the date on which the request was made. The ministry reserves the right to reject the quote of any bidder, or to terminate the contract with any successful bidder, that does not provide information to the satisfaction of the ministry, in its sole and absolute discretion, in response to any such request.
- The ministry, at its sole discretion, may elect to have the bidder's fabrication facility and equipment reviewed to satisfy itself of a bidder's likely ability to carry out the terms and conditions of this tender.

Subcontractor Qualifications

- Use of a sub-contractor will not be acceptable for the purposes of this project without express written approval from the ministry.

Welding Qualifications

- Bidders responsible for shop welded construction must be certified, at the time of tender and for the duration of fabrication, for Division 1 or Division 2 of CSA Standard W47.1, *Certification of Companies for Fusion Welding of Steel Structures*, with the following exceptions: fabrication of bridge railings, shear connectors for concrete slab bridges, and miscellaneous steelwork for all-timber portable superstructures may be undertaken by companies certified for Division 3 of CSA W47.1.
- Bidders must provide proof of appropriate Canadian Welding Bureau (CWB) certification within 2 business days of a ministry request.

Precast Concrete Qualifications

- Fabricators responsible for precast concrete fabrication (except for concrete roadside barriers and unreinforced interlocking blocks) must be certified, at the time of tender and for the duration of fabrication, in accordance with CSA A23.4 Precast Concrete- Materials and Construction. Companies must be certified by the Canadian Standards Association (CSA), or the Canadian Precast/Prestressed

Concrete Institute (CPCI). Bidders must provide proof of certification within 2 business days of a ministry request.

General

- The successful bidder shall not deliver the fabricated materials beyond the dates shown in the schedule without the prior written consent of the ministry.
- The successful bidder shall warrant all material fabricated and supplied against defects in materials and workmanship for a period of one year from the completion of manufacture. All defective products must be repaired or replaced to the satisfaction of the ministry as soon as is practicable, at the successful bidder's own expense.

Schedule for works

- Upon request from the ministry, a bidder, or successful bidder, must supply, within 4 business days of the request, a schedule for works which conforms to the required delivery dates of the tender. The schedule must include, but is not necessarily limited to: material receipt dates, fabrication commencement date, a minimum of 3 critical intermediate fabrication milestone dates, and a fabrication completion date. The schedule shall also include timelines for submissions of designs, for ministry approval. The schedule shall provide a minimum of 5 business days for ministry review of designs. Failure to provide a satisfactory schedule may result in rejection of the bid, or termination of the contract, at the ministry's sole discretion. The ministry shall be the sole and final judge of the sufficiency of the schedule provided.

Project Reference Documents

- Associated Engineering Drawings, Curb Connection Test Panels 2012/13, drawing numbers: 20102698-01-3-101 through 20102698-01-3-107
- Standard ministry references:
 - Ministry standard drawings
 - *Ministry Interim Bridge Design Guidelines (IBDG)*
 - *Forest Service Bridge Design and Construction Manual (FSBDCM)*
 - The standard drawings, IBDG, and FSBDCM are available for downloading at:
http://www.for.gov.bc.ca/hth/engineering/Bridges_And_Major_Culverts.htm

In-Plant Quality Assurance Inspection

- All materials must conform to the current ministry standards and shall not be acceptable without in-plant inspection by the ministry's in-plant inspection agency (below):

McElhanney Consulting Services Ltd

#500 – 3960 Quadra Street, Victoria BC V8X 4A2

Telephone (250) 370-9221

- The successful bidder must contact the ministry's in-plant inspection agency, to arrange for inspection prior to commencement of fabrication.
- The ministry's in-plant quality assurance inspections during fabrication are not substitutes for, but are supplemental to, the successful bidder's own required quality control measures as specified by and conforming to the various standards and specifications applicable to this contract.
- Where the ministry's in-plant inspector identifies deficiencies with the successful bidder's work, the deficiencies shall be corrected at the successful bidder's expense, including the cost of any additional inspection works undertaken by the ministry's in-plant inspector. The cost of the additional inspection work, required in order to assure the ministry that deficiencies are acceptably rectified shall be deducted by the ministry from the supplier's invoice(s) for the works.
- Bridge materials shall not be shipped to the ministry until the products have been reviewed and accepted by the ministry's in-plant quality assurance inspector as having been fabricated in conformance with the required fabrication standards, designs and specifications for the works. Prior to shipping of bridge materials, the supplier shall be responsible to confirm that all non-conformances, if any, have been rectified or accepted to the satisfaction of the ministry's in-plant quality assurance inspector.

Material Specifications

- All materials utilized in fabrication shall be new, not previously used in any application.

Steel

- All steel products to meet CSA G40.21M *Structural Quality Steel* unless equivalent specification has been pre-approved in writing by the Ministry Bridge Engineer.
- All steel plates and sections shall be atmospheric corrosion resistant steel (350 A or 350 AT as appropriate) unless specifically noted otherwise in this specification, or on the specified drawings.

Steel Components for Guardrail Systems

- Steel plates and sections for guardrail mounting plates, brackets, posts and HSS rail shall have the following steel grades and types, and coating options for corrosion resistance:

Steel	Coating
Guardrail Component	Uncoated (bare)

Brackets	350A
Posts	350A
HSS Rails	Not Applicable

- For posts:
 - ASTM A500 Grade C shall be considered equivalent to CSA G40.21M 350W
 - ASTM A847 shall be considered equivalent to CSA G40.21M 350A

Welding

- All welding must conform to CSA W59 *Welded Steel Construction (Metal Arc Welding)*
- Fillet weld leg size shall be a minimum of 6 mm unless noted otherwise.
- Inspection of welding shall meet the requirements of CSA W59.
- All tension butt welds shall be radiographically or ultrasonically tested.
- The welding procedure data sheets, as per CSA W47.1, shall be available for ministry review prior to fabrication.
- The desired objective for flange to web welds, for both I-girders and all-steel portable girders, is that they be made as continuous, uninterrupted and uniform welds free of abnormalities that could result in stress concentrations.

Generally, web to flange welds shall be made continuously by machine or automatic welding using submerged arc welding, flux-cored arc welding or metal-cored arc welding.

There may be instances where the ministry may accept girder web to flange welds with stops and starts in the deposition of weld material (e.g., at plate diaphragm locations on box girders, at certain end of girder locations with limited access, or upon occasions of unexpected power outages). However, continuous welds made by automatic or machine methods are required wherever it is reasonably physically possible (e.g., welds made on the outside of all steel portable box girders, and interior welds on all steel portable box girders except as previously noted in this paragraph).

- Where welds require repair, they may be repaired using a semi-automatic or manual process, but the repaired weld shall blend smoothly with the adjacent welds. Weld repairs shall be undertaken in accordance with CSA W59.
- I-girder flange to web welds shall be made using submerged arc welding

Concrete

- Concrete components must be fabricated and supplied in accordance with the ministry *Bridge Component Concrete Standard* located at: http://www.for.gov.bc.ca/hth/engineering/documents/Std_Br_Material_Templates/BrCompConcStd.pdf

Documentation Requirements

- All documentation shall be supplied in electronic Adobe (pdf) format.
- All documents shall be clearly labelled with the appropriate structure number pertaining to each applicable structure.
- The following documents shall be supplied to the ministry's in-plant inspection agency within specified time frames, and for each fabricated bridge:
 - Mill Certificates of structural steel plates and sections (within 2 weeks of fabrication)
 - Radiographic or Ultrasonic testing reports (within 2 weeks of fabrication)
 - Concrete Test Results including:
 - Formwork release test results (prior to shipping of fabricated concrete components)
 - 7 day concrete compressive strength test results (within 5 business days of testing)
 - 28 day concrete compressive strength test results (within 5 business days of testing)
- For concrete components, 7 day concrete compressive strength test results shall also be sent to the ministry Bridge Engineer within 5 business days of testing.

Concrete Test Panels

Ministry Assigned Structure #: Eng Br Test Panels 2012/13

Table 5: Critical Dates and Time Frames

	ITEM	DATE REQUIRED
1.1	Complete Materials Fabrication <i>(Means: Completed materials fabrication, ministry In-plant Inspection, and ministry acceptance of all materials at the fabrication facility)</i>	January 25, 2013
1.2	Billing Submission <i>(Latest date billing to be received by the ministry)</i>	February 15, 2013
1.3	Estimated Delivery Date <i>(Actual date to be specified by the ministry, with a minimum one week notice prior to required product delivery date/time.)</i>	Between January 25 and February 15, 2013
1.4	Maximum Storage Period <i>(Possible storage by fabricator prior to delivery.)</i>	Until February 28, 2013

Table 6: General Information

	ITEM	DESCRIPTION
2.1	Bridge Engineer responsible for design and fabrication review	John Deenihan, PHD, EIT Structural Engineer, Associated Engineering (BC) Ltd Ph: (604)293-1411 e-mail: deenihanj@ae.ca
2.2	Structure Number	Eng Br Test Panels 2012/13

Other Test Reports



Certificate of Compliance Interim In-Plant Quality Assurance Product Acceptance

Ministry Structure Number: _____

Bridge / Project Name: UBC Curb Connection Panels

Ministry of Forests, Lands and Natural Resource Operations Bridge Engineer: Brian Chow, P.Eng

- As Quality Assurance Technician for the above-noted structure, I have performed In-Plant Quality Assurance Services on behalf of McElhanney Consulting Services Ltd. for the BC Ministry of Forest, Lands and Natural Resource Operations.
- In-Plant Quality Assurance services involved field reviews consisting of observations and/or sampling of a representative portion of the work performed by Pioneer Precast Products Ltd. during the fabrication of the following components. *(Name of Fabricator)*

Component Description
Test Panels x 2 (1 x Type G, 1 x Type H)

- During my observations, the fabrication of the foregoing components including any remedial work was performed in accordance with the requirements of the Contract Documents.

Noted exceptions are stated as follows:

- Confirmation of 28 day concrete strength test results
- One plastic bolt sleeve was out of position by 5mm. Ministry Engineer has been notified.
-

Dan Robek, P.Eng
Print Name

Signature of Quality Assurance Technician

March 22, 2013

Date

Note:

In-Plant Quality Assurance Services performed by a representative of the BC Ministry of Forests, Lands and Natural Resource Operations is not a substitute for the Contractor's or his Subcontractor's Quality Control, including their obligation to perform the work in accordance with the requirements of the Contract.

This Interim Product Acceptance is subject to final review of documentation by the Quality Assurance Review Engineer.



McElhanney Consulting Services Ltd.

3960 Quadra Street, Unit 500
Victoria BC
Canada V8X 4A3
Tel 250 370 9221
Fax 250 370 9223



VALLEY TESTING SERVICES LTD.
 #18 - 3275 McCallum Road ph: 1-888-855-9733
 Abbotsford, B.C. V2S 7W8 fax: (604) 855-7378



CONCRETE TEST REPORT

PROJECT NO. V2536

CLIENT PIONEER PRECAST PRODUCTS LTD. a
 C.C. REMPEL BROS CONCRETE

TO
 PIONEER PRECAST PRODUCTS LTD. a
 division of IOTA Const. Ltd.
 8190 AITKEN ROAD
 CHILLIWACK, BC
 V2R 4H5

ATTN: MR. NICK EUSTACE

PROJECT CONCRETE TESTING 2012 Q.C
 CONCRETE TESTING & INSPECTION

8190 AITKEN ROAD
 CHILLIWACK

SET NO. 675 NO. OF SPECIMENS 5 DATE RECEIVED 2013.Jan.25 DATE CAST 2013.Jan.23

SPCM NO.	SPECIMEN TYPE	CURE COND	DATE TESTED	AGE AT TEST (DAYS)	AVERAGE DIAMETER (mm) OR SIDE (mm x mm)	AVERAGE LENGTH OR SPAN (mm)	MAXIMUM LOAD (kN)	COMPRESSIVE OR FLEXURAL STRENGTH (MPa) Average	FAILURE TYPE
A	Cylinder	Lab	Jan.30	7	101.6	203.2	294	36.3	
B	Cylinder	Lab	Feb.20	28	101.6	203.2			
C	Cylinder	Lab	Feb.20	28	101.6	203.2			
D	Cylinder	Lab	Mar.20	56	101.6	203.2			
E	Cylinder	Field	Jan.30	7	101.6	203.2	258	31.8	

SPECIFIED STRENGTH 35 MPa @ 28 DAYS CEMENT CONTENT kg/m ³ CEMENT TYPE 10 POZZOLAN CONTENT kg/m ³ POZZOLAN TYPE MAXIMUM SIZE AGGREGATE 20 mm BATCH TIME 07:20 ADMIXTURES ADVA 1407 0.80		CONCRETE TEMPERATURE 16.0 °C AIR TEMPERATURE 2.0 °C SLUMP 125 mm SPEC. 130 ± 20 SLUMP FLOW mm SPEC. ± FLOW TIME sec SPEC. AIR 6.5 % SPEC. 6.0 ± 1.0 PLASTIC DENSITY kg/m ³ HARDENED DENSITY kg/m ³ CAST TIME 08:00 CAST BY VTS AS CURING CONDITIONS WOODEN		TREND GRAPH
SUPPLIER REMPEL BROS CONCRETE MIX NO. PP35C1 TRUCK NO. 689 TICKET NO. 375885 LOAD VOL. 10 m ³ CUM. VOL. 10 m ³ WATER ADDED AUTH. BY Page 1 of 1 2013.Jan.30		INITIAL CURING TEMP: MAXIMUM 22.0 °C MINIMUM 15.0 °C LOCATION 1) 20102541-15-3, 6 PANELS (A, B, C, D, E, F) 2) 20102698-01-3, 8 PANELS <i>N4-001</i> COMMENTS (A, B, C, D, E, F, G, H) TEST TAKEN @ PANEL A		

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of test results is provided only on written request.



VALLEY TESTING SERVICES LTD.

#18 - 3275 McCallum Road ph: 1-888-855-9733
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CONCRETE TEST REPORT

PROJECT NO. V2536

CLIENT PIONEER PRECAST PRODUCTS LTD. a
C.C. REMPEL BROS CONCRETE
IOTA CONSTRUCTION LTD.

TO
PIONEER PRECAST PRODUCTS LTD. a
division of IOTA Const. Ltd.
8190 AITKEN ROAD
CHILLIWACK, BC
V2R 4H5

ATTN: MR. NICK EUSTACE

PROJECT CONCRETE TESTING 2012 Q.C
CONCRETE TESTING & INSPECTION

8190 AITKEN ROAD
CHILLIWACK

SET NO. 695 NO. OF SPECIMENS 9 DATE RECEIVED 2013.Mar.15 DATE CAST 2013.Mar.14

SPCM NO.	SPECIMEN TYPE	CURE COND	DATE TESTED	AGE AT TEST (DAYS)	AVERAGE DIAMETER (mm) OR SIDE (mm x mm)	AVERAGE LENGTH OR SPAN (mm)	MAXIMUM LOAD (KN)	COMPRESSIVE OR FLEXURAL STRENGTH (MPa) Average	FAILURE TYPE
A	Cylinder	Lab	Mar.21	7	101.6	203.2	257	31.7	
B	Cylinder	Lab	Apr.11	28	101.6	203.2			
C	Cylinder	Lab	Apr.11	28	101.6	203.2			
D	Cylinder	Lab	May.09	56	101.6	203.2			
E	Cylinder	Field	Mar.21	7	101.6	203.2	233	28.7	
F	Cylinder	Field		Hold	101.6	203.2			
G	Cylinder	Field		Hold	101.6	203.2			
H	Cylinder	Field		Hold	101.6	203.2			
I	Cylinder	Field		Hold	101.6	203.2			

SPECIFIED STRENGTH 35 MPa @ 28 DAYS CEMENT CONTENT kg/m ³ CEMENT TYPE 10 POZZOLAN CONTENT kg/m ³ POZZOLAN TYPE MAXIMUM SIZE AGGREGATE 20 mm BATCH TIME 09:27 ADMIXTURES ADVA 140T	CONCRETE TEMPERATURE 18.0 °C AIR TEMPERATURE 10.0 °C SLUMP 110 mm SPEC. 130 ± 20 SLUMP FLOW mm SPEC. ± FLOW TIME sec SPEC. AIR 5.5 % SPEC. 6.0 ± 1.0 PLASTIC DENSITY kg/m ³ HARDENED DENSITY kg/m ³ CAST TIME 10:15 CAST BY VTS GM CURING CONDITIONS WOODEN	TREND GRAPH <p>STRENGTH (MPa) vs SRT NUMBER</p>
INITIAL CURING TEMP: MAXIMUM 22.0 °C MINIMUM 15.0 °C		MOULD TYPE PLASTIC
SUPPLIER REMPEL BROS CONCRETE MIX NO. PP35C1 TRUCK NO. 1680 TICKET NO. 382181 LOAD VOL. 1.7 m ³ CUM. VOL. 1.7 m ³ WATER ADDED 1 AUTH. BY Page 1 of 1 2013.Mar.21		LOCATION CURB CONNECTION TEST PANEL: 1-TYPE G:G; 1-TYPE H:H DRWG #20102698-01 COMMENTS TEST TAKEN @ TYPE G SAMPLE F, G, H, I SENT BACK TO PIONEER AS REQUESTED. VALLEY TESTING SERVICES LTD. PER.

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of test results is provided only on written request.
Report System Software Registered to: Valley Testing Services Ltd.



Associated Engineering (B.C.) Ltd.
Suite 300 - 4940 Canada Way
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TEL: 604.293.1411
FAX: 604.291.6163
www.ae.ca

September 5, 2012
File: BUR_P_2012.759

Mr. Brian Chow, M.Eng., P.Eng.
Chief Engineer
Ministry of Forests, Lands and Natural Resource Operations
Engineering Branch, Provincial Operations
3rd Floor, 1810 Blanshard Street
Victoria, BC
V8W 3K2

Re: ADDITIONAL TESTING OF CL-2 AND CL-3 BARRIERS

Dear Mr. Chow:

As discussed previously in e-mail and telephone correspondence, we agree that additional testing of both the CL-2 and CL-3 barrier configurations is required. We have recommended that the drip groove be moved 300 mm from the deck edge, thus minimizing any potential influence it may have on the resistance of the deck. Furthermore, based on our analytical evaluation we believe we can reduce the size of the bracket from a 680 mm wide plate to 550 mm without compromising the resistance of the barrier.

As discussed in our technical memorandum, titled "Review of Modified HSS Guide Retrofit Rail", it is clear that the knee-brace configuration developed by UBC is capable of achieving the required resistance of a CL-3 barrier. However, as it's unfeasible to replicate this configuration in the field due to fabrication and installation issues, the tested resistances are of limited use. Thus we proposed an alternate connection detail using the existing connection bracket, with an embedded edge plate and deformed nelson bars, details of which are presented in the technical memorandum. The additional confinement provided by the embedded plate enhances the edge compressive capacity of the concrete, and, the tensile resistance of the reinforcement is increased with the addition of nelson deformed bars. We believe these modifications will improve the resistance of the barrier but physical testing is required to verify the capacity of the configuration and if it can achieve the resistance requirements of a CL-3 barrier.

This letter shall discuss the proposed additional testing in two categories, additional CL-2 level testing and modified CL-3 level testing.

CL-2 – HSS Guide Retrofit Rail Additional Testing

We proposed the following additional tests be conducted for the CL-2 barriers:

- 550 wide bracket with reinforcing and coupler details matching previous tests.
- 550 wide bracket with a nut at the end of the insert rebar to improve bond.

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September 5, 2012
 Mr. Brian Chow, M.Eng., P.Eng.
 Ministry of Forests, Lands and Natural Resource Operations
 - 2 -

- 680 wide bracket with a nut at the end of the insert rebar to improve bond.

We recommend three tests per option, resulting in a reduction of deviation in resistances and providing additional confidence to previous experimental conclusions.

CL-3 – Modified HSS Guide Retrofit Rail

We proposed the following tests be conducted for the CL-3 barriers:

- Side mounted bracket with alternative connection detail as per our technical memorandum, titled "*Review of Modified HSS Guide Retrofit Rail*".

We recommend initially testing three modified HSS Guide Retrofit Rail Barriers to determine if the proposed configuration can achieve the resistance requirements of a CL-3 barrier. Failing this, we shall need to review the proposed connection detail and make suitable modifications based on experimental findings. Thus, it is undesirable to fabricate several modified CL-3 deck panels until initial testing can verify an approximate resistance of the proposed configuration.

Additional Testing Considerations

To improve the cost effectiveness of testing we propose two potential modifications to the deck slab.

- .1 We believe that it is possible to incorporate four barrier connections per panel. This would result in almost halving the production costs for a fixed number of tests; furthermore, it would decrease the turnaround time between tests and significantly reduce the wastage per panel.

A review of the existing deck panel would be required to determine if it's feasible to introduce additional reinforcement into the panel with the intention of making it doubly symmetric, without altering the original resistance of the panel.

- .2 Alternatively it may be possible to produce a stub panel with dimensions marginally greater than the predicted damage area. These panels would be single-test panels only, but would be significantly easier to handle, minimise wastage and increase the turnaround speed between tests. The stub panels have the advantage that any individual failure will not affect subsequent tests, whereas a panel with four barrier connections may experience deterioration during one test which may result in a compromised resistance of subsequent tests.

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Date: March 15, 2012 **File:** 00.E.05.00
To: Brian Chow, M.A.Sc., P.Eng.
From: Julien Henley, M.A.Sc., P.Eng.
Project: 20102698
Subject: Review of Modified HSS Guide Retrofit Rail

MEMO

As part of the Ministry of Forests, Lands and Natural Resource Operations development of appropriate bridge barrier design guidelines, they retained the University of British Columbia (UBC) to complete an experimental program to verify the capacity of standard bridge barriers currently in use in the forest industry in British Columbia. During the experimental program, UBC modified the HSS Guide Retrofit Rail by adding a knee-brace in an effort to increase the capacity of the rail. This modification resulted in a capacity approximately 2.3 times greater than that of a typical side mounted connection. This memorandum, provides a brief summary of Associated Engineering's review of the modifications and classification of the barrier based on the recommendations included the 2011 AE report, "Phase III – Guidelines for Barrier Selection and Design and summarised 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	500	500	510
Minimum Barrier Height ²	500	500	685

Note:

- When completing an analytical evaluation of a barrier, these forces represent factored forces and resistances should be calculated assuming nominal material strengths.
- Height measured from travel surface.

Figure 1 illustrates the general modified HSS Guide Retrofit Rail along with the theoretical free body force diagram. The post and rail component are identical to that of a standard Side Mounted HSS Guide Retrofit Rail, with the exception of the addition of the knee-brace which extended approximately 600mm under the precast concrete deck panel. Full details are presented in the 2011 UBC report titled "Experimental Evaluation of Concrete Decks with Guard Rail Systems". The rail was mounted on a 175mm thick concrete panel with reinforcing and couplers matching the MFLNRO Drawings STD-



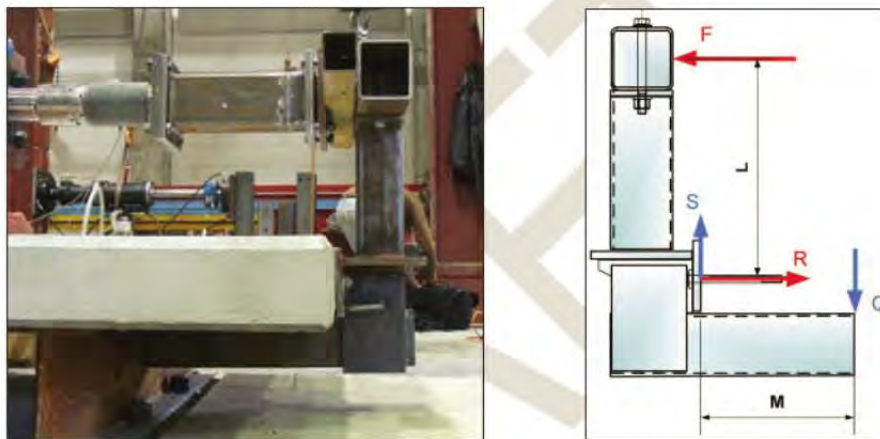
P:\20102698\00_Eval_Br_Barriers\Engineering\05.00_Design\Development Of CL-3 Barrier\DRAFT_Mem_Mflnro_Dev_Ci3_Barrier_20120315_Jh.Doc



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EC-030 Series.

Figure 1
Modified HSS Guide Retrofit Rail



Under transverse loading the standard side mounted HSS Guide Retrofit Rail reacts in compression against the concrete deck below the location of the bolt insert; this reaction force magnifies the horizontal tension load applied to the bolts as discussed in our Memorandum titled "Classification of HSS Guide Retrofit Bridge Rail", March 2012. The modified HSS Guide Retrofit Rail with knee-brace transfers the applied transverse force to the bearing location of the knee-brace via rotation about the bolt inserts. The resultant force is resisted by shear in the anchor bolts (as opposed to tension) and the reaction of the knee-brace against the underside of deck (or girder in the case of the experimental test). As a result, the failure mechanism is different to that observed to for the side mounted HSS Guide Retrofit Rail with the anchor bolt inserts punching through the underside of the deck rather than concrete crushing or bolts fracturing as previously observed. Figure 2 illustrates the observed failure mode of the modified HSS Guide Retrofit Rail.

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Memo To: Brian Chow, M.A.Sc., P.Eng.
 March 15, 2012
 - 3 -

Figure 2
Typical Observed Failure of the Modified HSS Guide Retrofit Rail



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

Table 2
Observed Peak Failure Loads and Associated Failure Mechanisms

Specimen ID ¹	Observed Peak Horizontal Load (kN)	Comments
--------------------------	------------------------------------	----------

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Memo To: Brian Chow, M.A.Sc., P.Eng.
 March 15, 2012

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Specimen ID ¹	Observed Peak Horizontal Load (kN)	Comments
4.3	154.8	<i>Description:</i> A307 bolts connecting barrier and concrete panel <i>Failure Mode:</i> Spalling of concrete in vicinity of inserts
4.4	124.1	<i>Description:</i> A307 bolts connecting barrier and concrete panel <i>Failure Mode:</i> Spalling of concrete in vicinity of inserts
4.5	164.4	<i>Description:</i> A307 bolts connecting barrier and concrete panel <i>Failure Mode:</i> Spalling of concrete in vicinity of inserts
Note:		
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.		

A review of the observed peak failure loads and comparison with the recommended resistances shown in Table 1 for the CL-3 barrier, suggests that this simple modification to the HSS Guide Retrofit Rail is sufficient to increase the strength of the barrier, resulting in its classification as a CL-3 barrier. However, after reviewing the UBC Report, associated videos documenting the testing and discussions with UBC researchers we established that the knee-brace extended approximately 600 mm under the concrete deck and was supported on the girder flange although it did not react against the supporting girder web.

Although a knee-brace of this length results in a significant reduction in the demand on the anchor bolts, it is not practical for field installations since the knee-brace would rest on the girder flange making installation and accommodation of field tolerances difficult. A review of typical steel girder and concrete deck forestry bridges suggests that the maximum practical lever arm is 300-400 mm which results in an increased demand on the anchor bolts. We completed a preliminary analysis of based on a reduced knee-brace length (400 mm) as shown in Figure 3 and determined an approximate horizontal capacity of 98 kN which suggests that it does not meet the proposed requirements for a CL-3 barrier which requires a minimum resistance of 120kN.

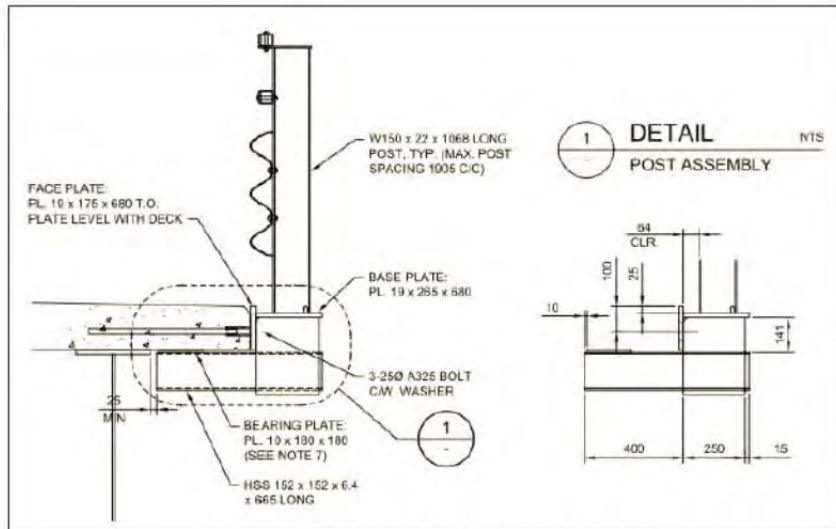
Figure 3
Modified HSS Guide Retrofit Rail with Reduced Length Knee-Brace



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With the required modifications resulting in a significant reduction in the strength, we have proposed modifying the barrier as shown in Figure 4 to increase the strength of the connection to the deck. The modifications include the addition of an embedded steel plate and nelson deformed bars to improve the shear resistance of the connection. We believe that further experimental testing will verify that these modifications will result in the barrier being classified as CL-3 barrier.

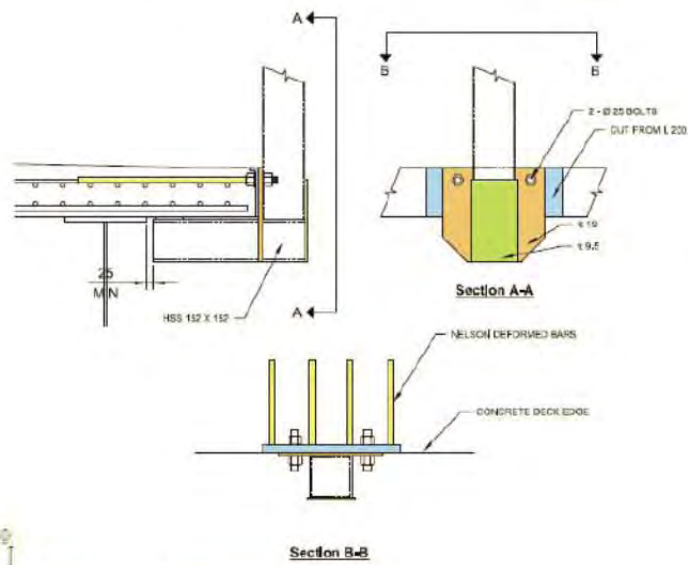




Memo To: Brian Chow, M.A.Sc., P.Eng.
 March 15, 2012
 - 6 -

Figure 4

Modified Knee-Brace Connection Detail



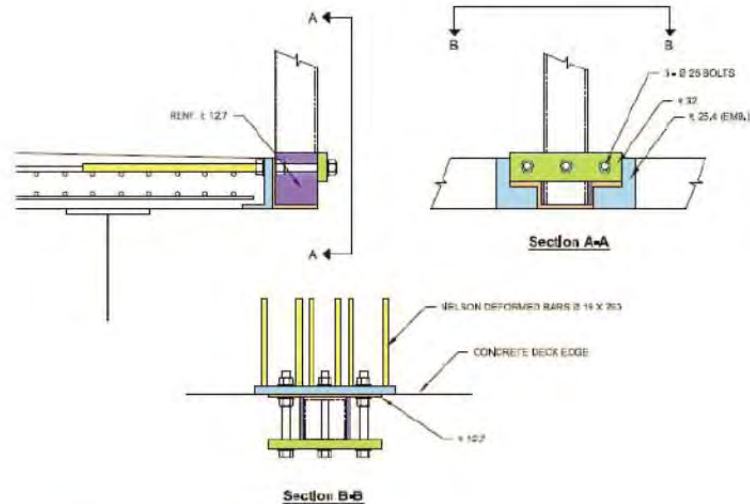
In addition, Figure 5 illustrates an alternative deck connection that eliminates the need for the knee-brace. The proposed connection includes an embedded plate and nelson deformed bars to increase the compressive strength of the deck edge. The capacity of this connection needs to be verified through experimental testing.





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 March 15, 2012
 - 7 -

Figure 5
Proposed Alternative Connection Detail



Prior to completing further experimental testing, we recommend that the Ministry discuss the two proposed details with fabricators to determine whether either is feasible and economical. In completing the review, the Ministry may also wish to compare the proposed modified rails with existing CL-3 crash tested barrier arrangements (AASHTO TL2 crash tested barriers) and possibly adopt a previously tested barrier rather than develop and test a new barrier.



DRAWING TRANSMITTAL

To: Ministry of Forests, Lands and Natural Resource Operations Transmittal No: 1
PO Box 9510, Stn Prov Govt, 3rd Floor - 1520 Blanshard Street Page: 1 of 1
Victoria, BC V8W 3K2 Date: December 7, 2012
 Attention: Mr. Brian Chow, M. Eng., P.Eng. File: 20102698.01.E.05.00
 Subject: Curb Connection Test Panels 2012/2013 Project No: 20102698

CODES: <table style="display: inline-table; vertical-align: top; margin-right: 20px;"> <tr><td>A Reviewed</td></tr> <tr><td>B Reviewed as Modified</td></tr> <tr><td>C For Correction</td></tr> </table> <table style="display: inline-table; vertical-align: top; margin-right: 20px;"> <tr><td>D Not Reviewed</td></tr> <tr><td>E For Approval</td></tr> <tr><td>F As Requested</td></tr> </table> <table style="display: inline-table; vertical-align: top;"> <tr><td>G For Your Comments</td></tr> <tr><td>H For Your Information</td></tr> <tr><td>I Issued for Construction</td></tr> </table>					A Reviewed	B Reviewed as Modified	C For Correction	D Not Reviewed	E For Approval	F As Requested	G For Your Comments	H For Your Information	I Issued for Construction
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C For Correction													
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H For Your Information													
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OTHER													
Drawing Number	Rev.	No. of Copies	Description or Drawing Title	Code									
20102698-01-3-101	0	2	Type A Panel – 175 mm Thick with 680 mm Wide Bracket	F									
20102698-01-3-102	0	2	Type B Panel – 175 mm Thick with 550 mm Wide Bracket	F									
20102698-01-3-103	0	2	Type C Panel – 175 mm Thick with 550 mm Wide Bracket & Terminations	F									
20102698-01-3-104	0	2	Type D Panel – 175 mm Thick with CL3 Inserts – Sheet 1	F									
20102698-01-3-105	0	2	Type D Panel – 175 mm Thick with CL3 Inserts – Sheet 2	F									
20102698-01-3-106	0	2	Type E Panel – 200 mm Thick with CL3 Inserts – Sheet 1	F									
20102698-01-3-107	0	2	Type E Panel – 200 mm Thick with CL3 Inserts – Sheet 2	F									
Remarks													
Forwarded by:			Copies to:										
Associated Engineering (B.C.) Ltd. 300 – 4940 Canada Way Burnaby, BC V5 G 4M5													
Associated Engineering _____			Received by _____										

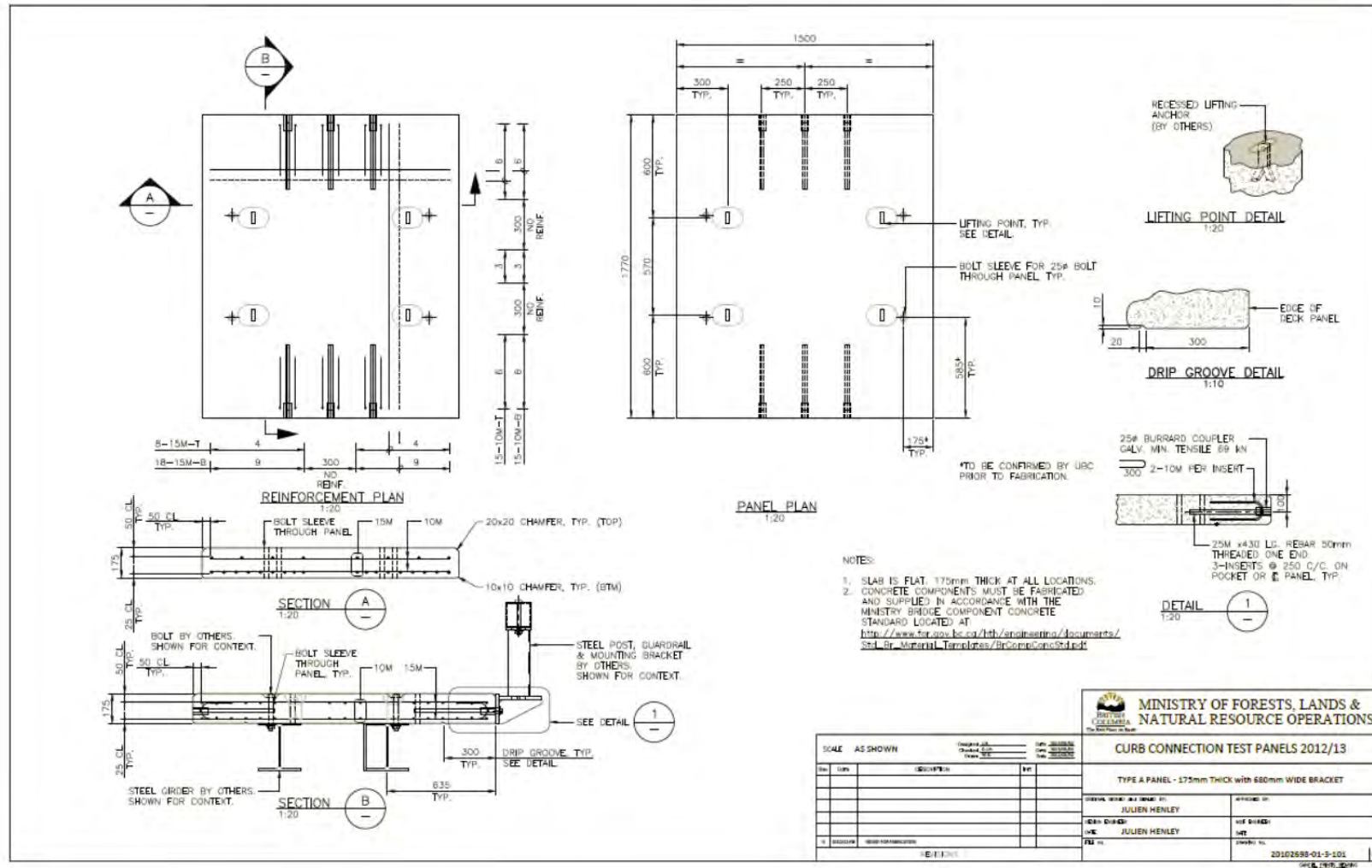
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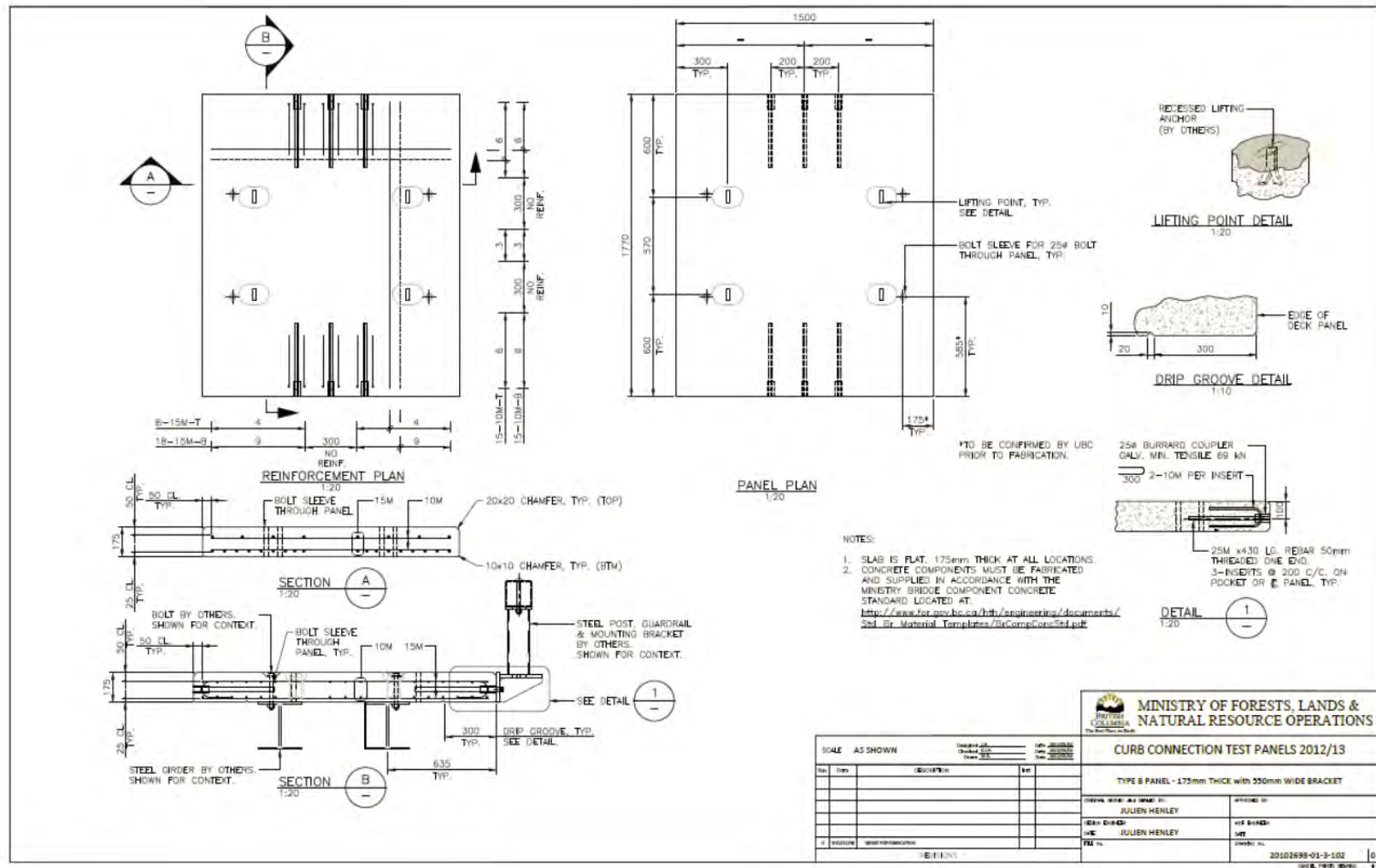
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 Please sign and return to Associated Engineering

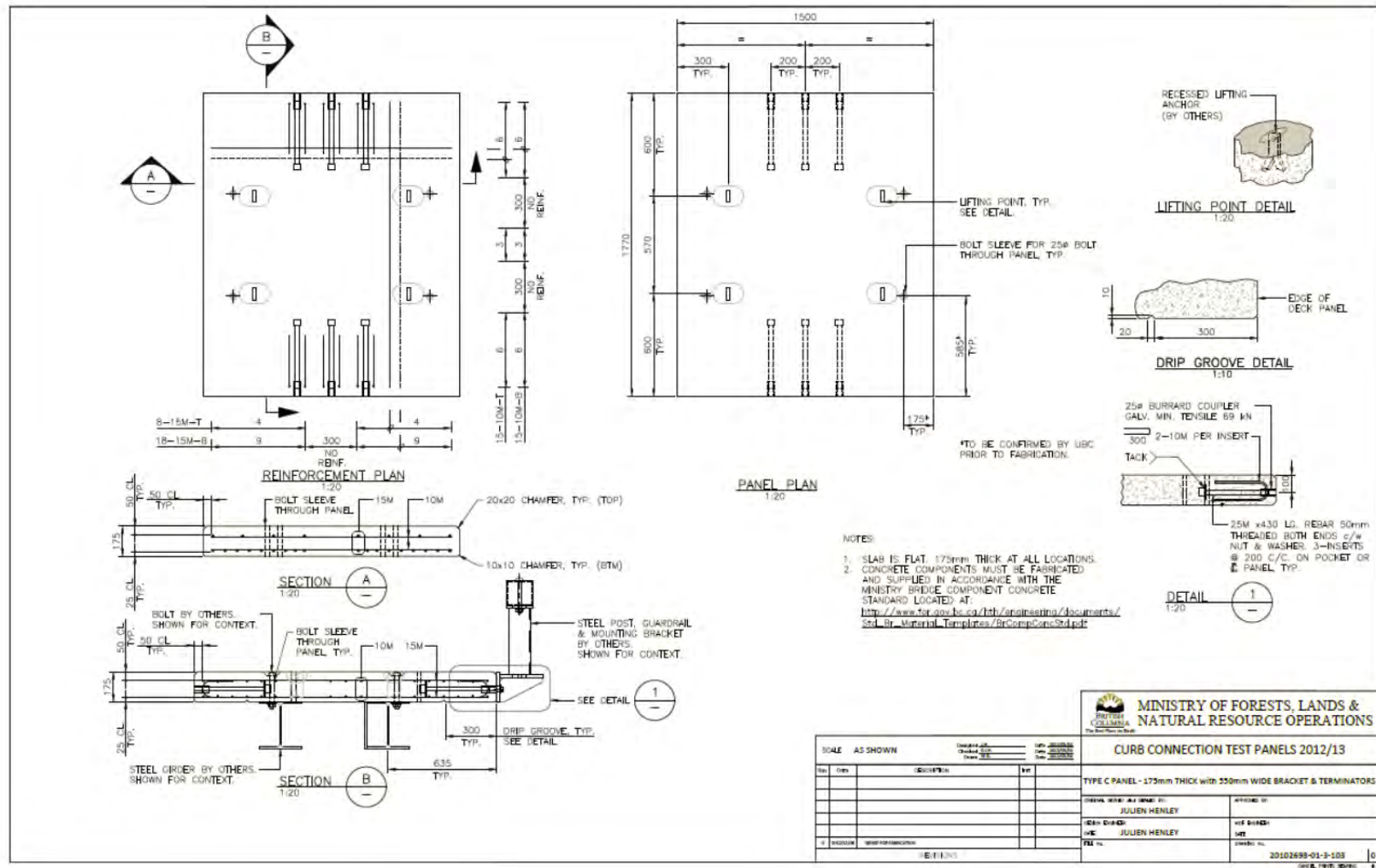
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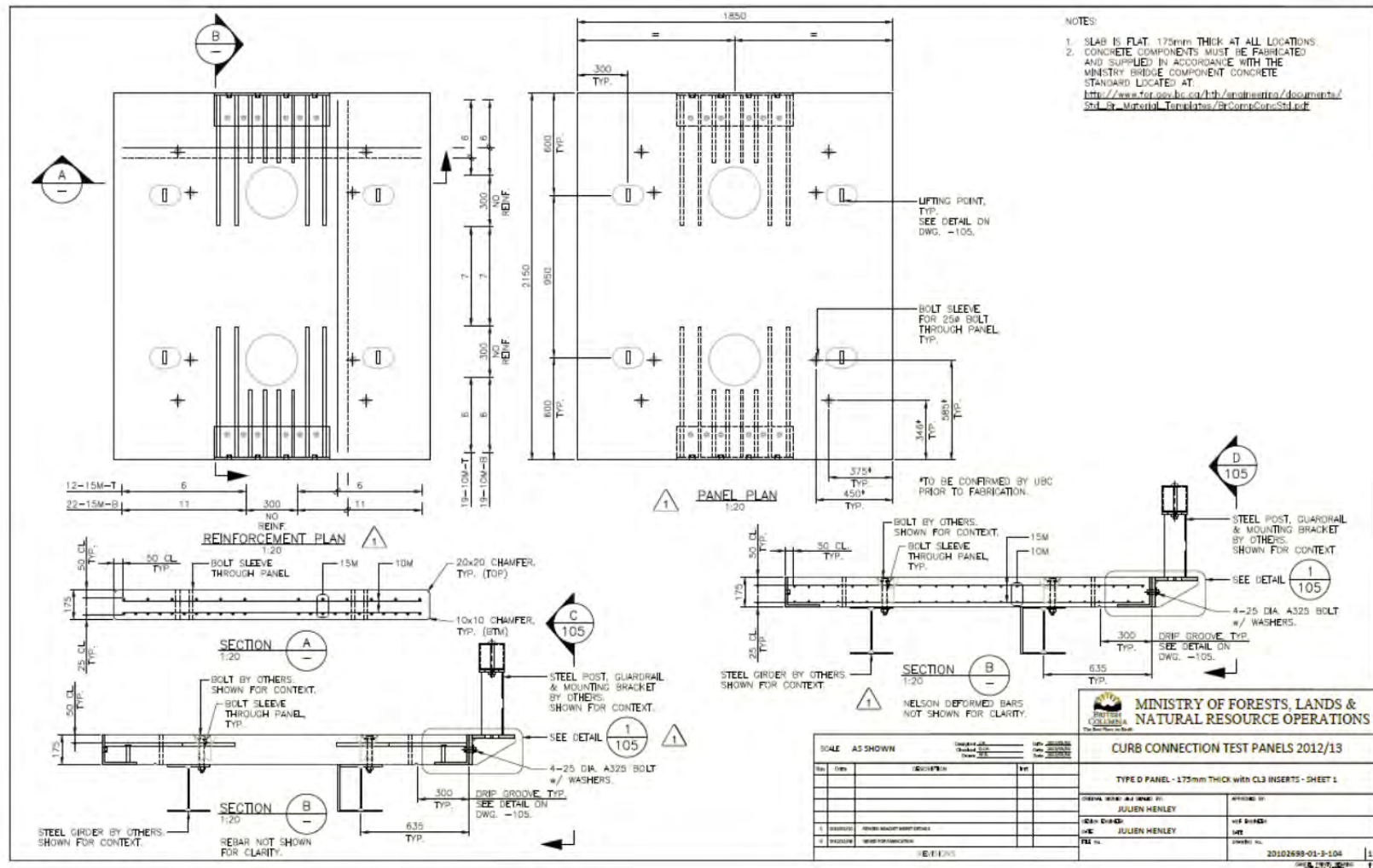
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Revised Drawings_Full Set - Jan 10th 2013

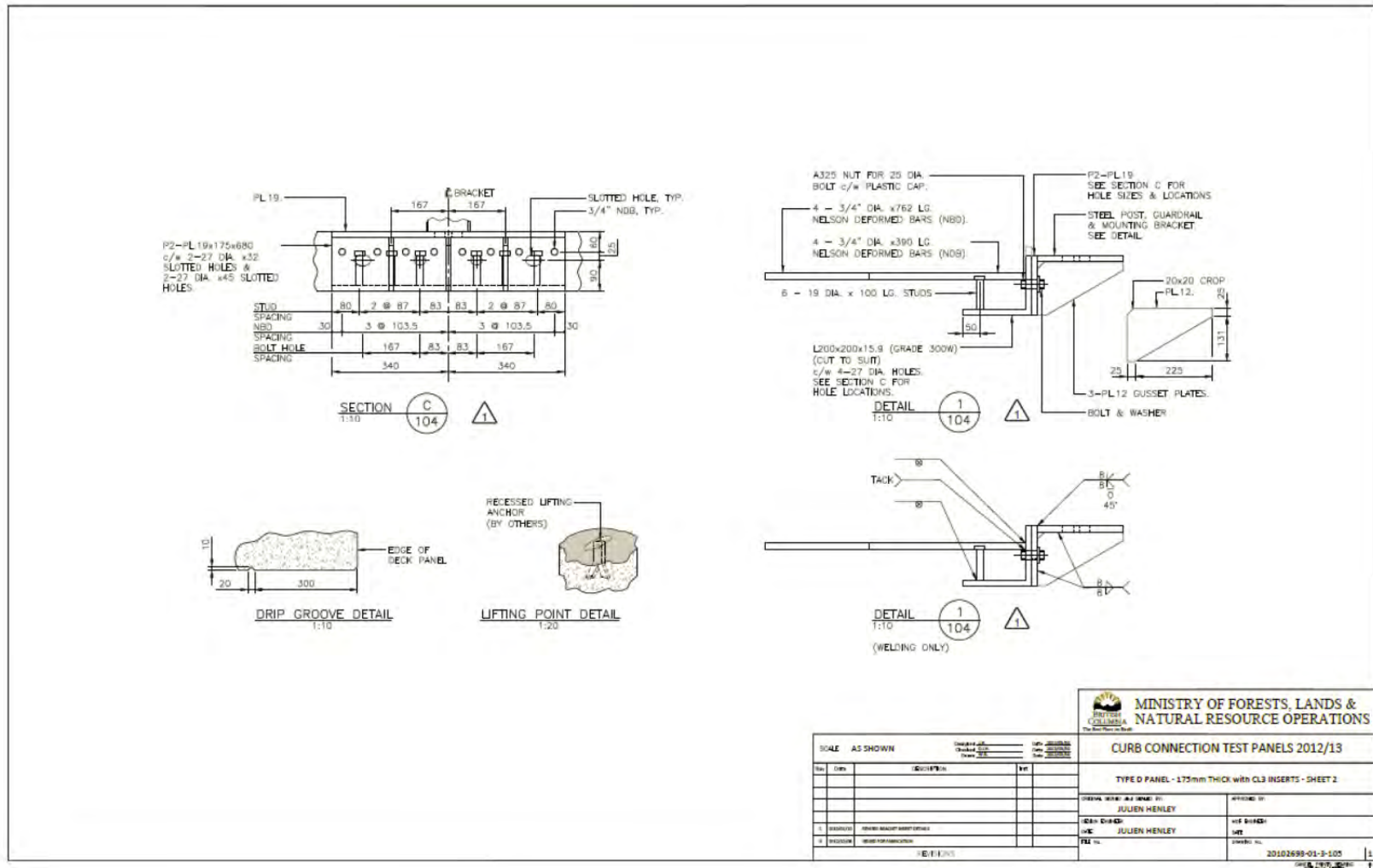


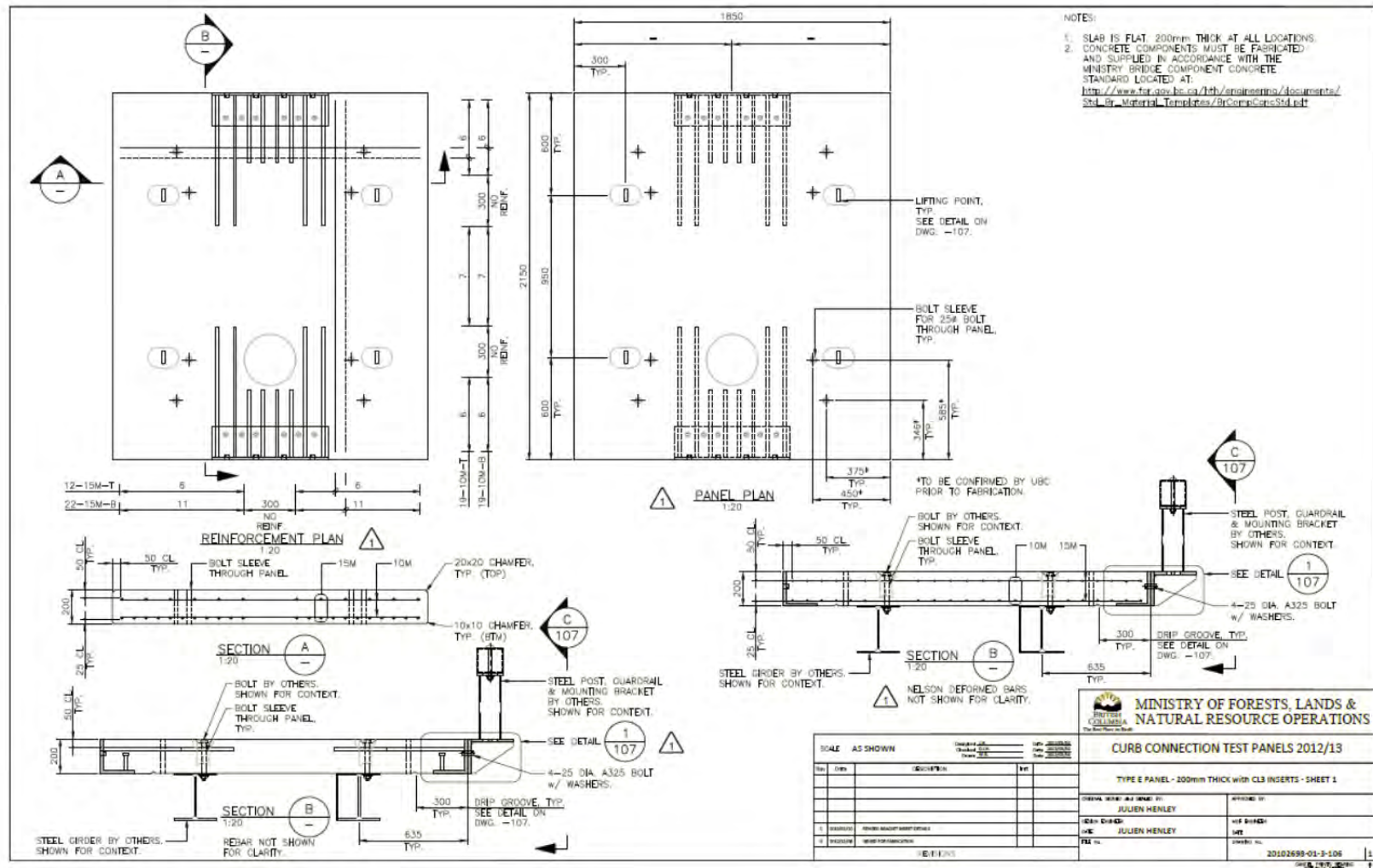


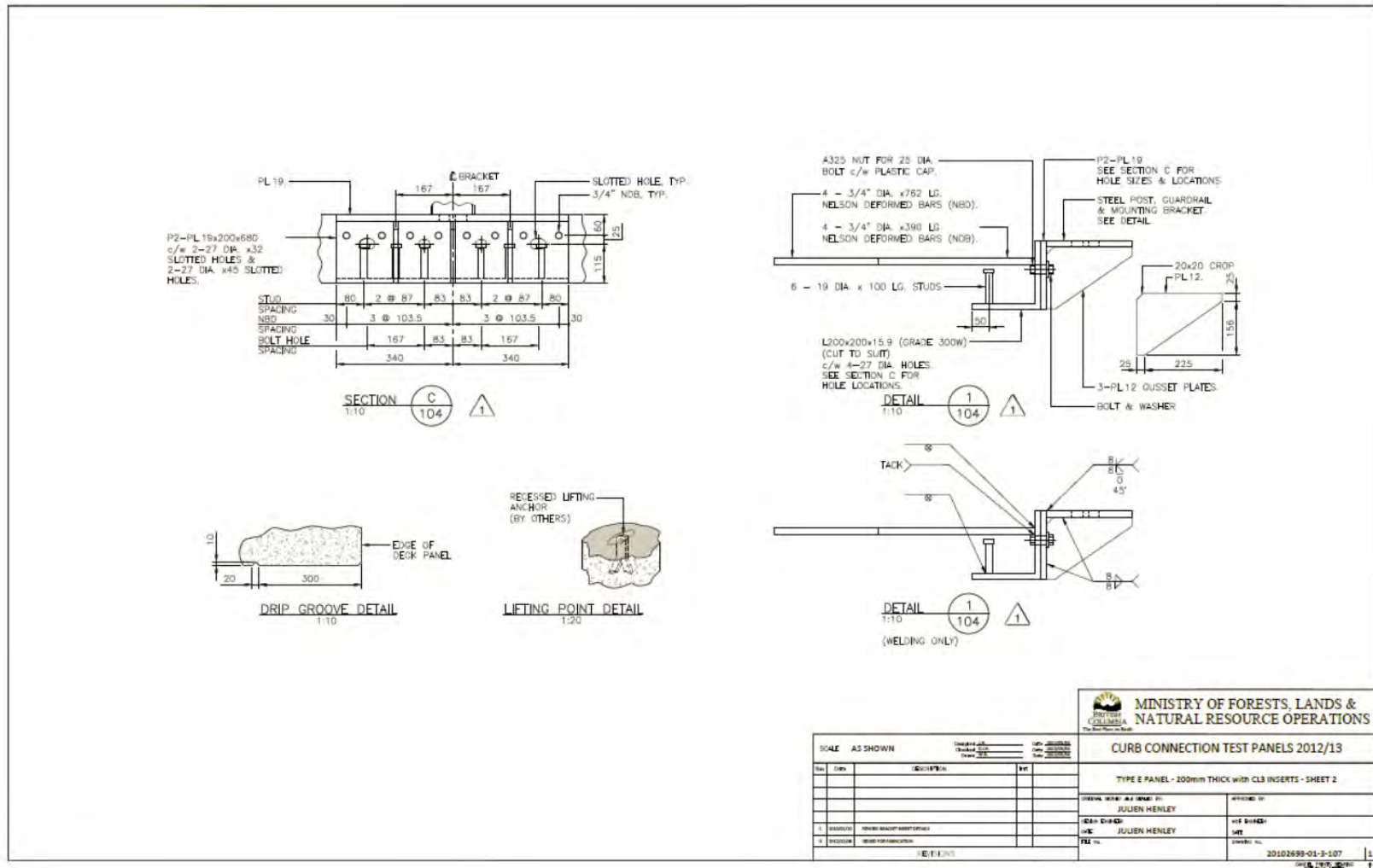


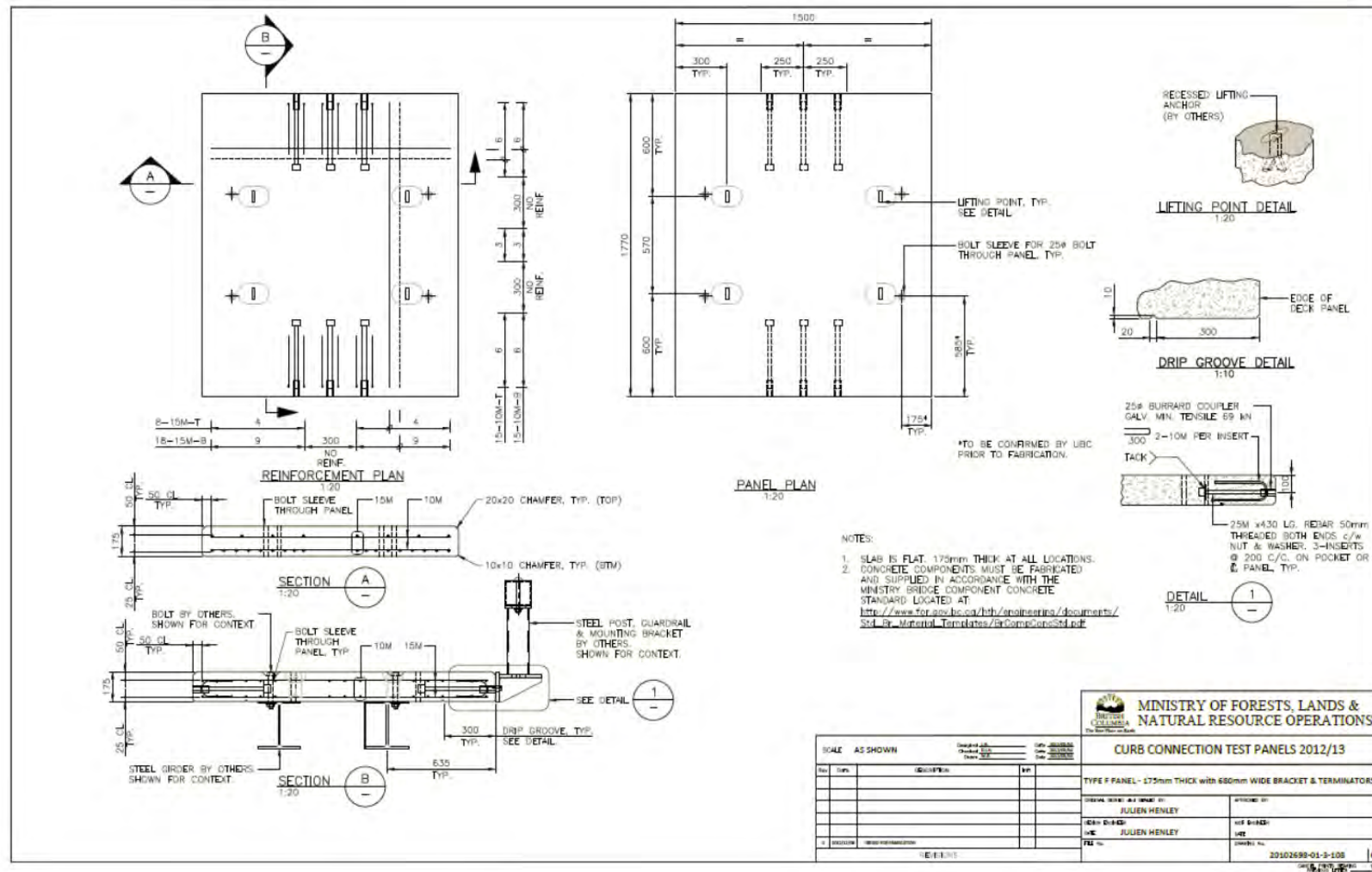


MINISTRY OF FORESTS, LANDS & NATURAL RESOURCE OPERATIONS			
CURB CONNECTION TEST PANELS 2012/13			
TYPE D PANEL - 175mm THICK WITH CL3 INSERTS - SHEET 1			
SCALE AS SHOWN	Checked By: _____ Date: 2013-04-16	Drawn By: _____ Date: 2013-04-16	APPVED BY: JULIEN HENLEY
1. REVISION: DESIGN MODIFICATION	2. REVISION: REBAR MODIFICATION	3. REVISION: _____	4. REVISION: _____
DATE: 2013-04-16		DRAWN BY: JULIEN HENLEY	
DESIGNED BY: _____		CHECKED BY: _____	
PROJECT NO. 20102698-01-3-104		SHEET NO. 11	

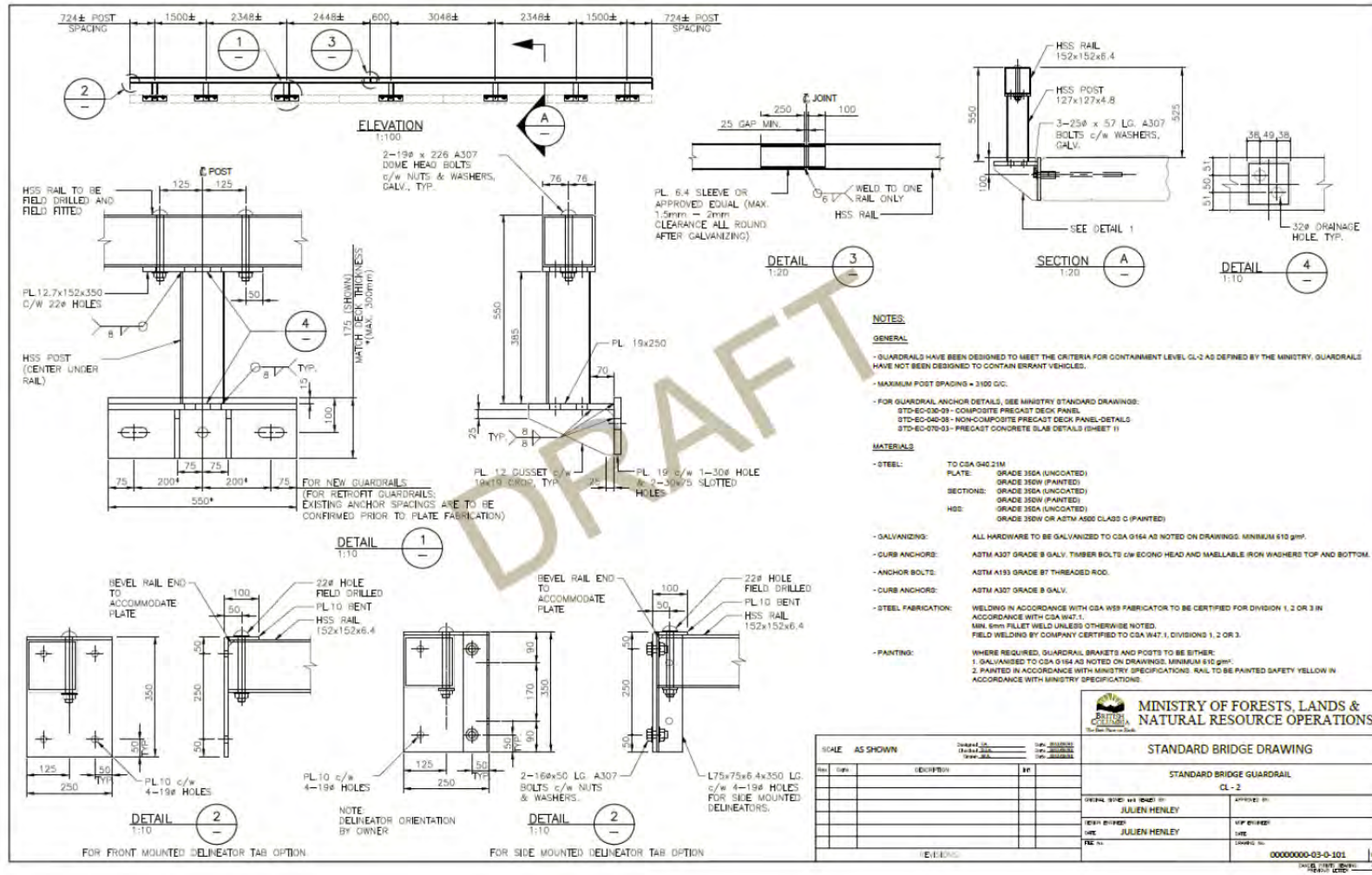




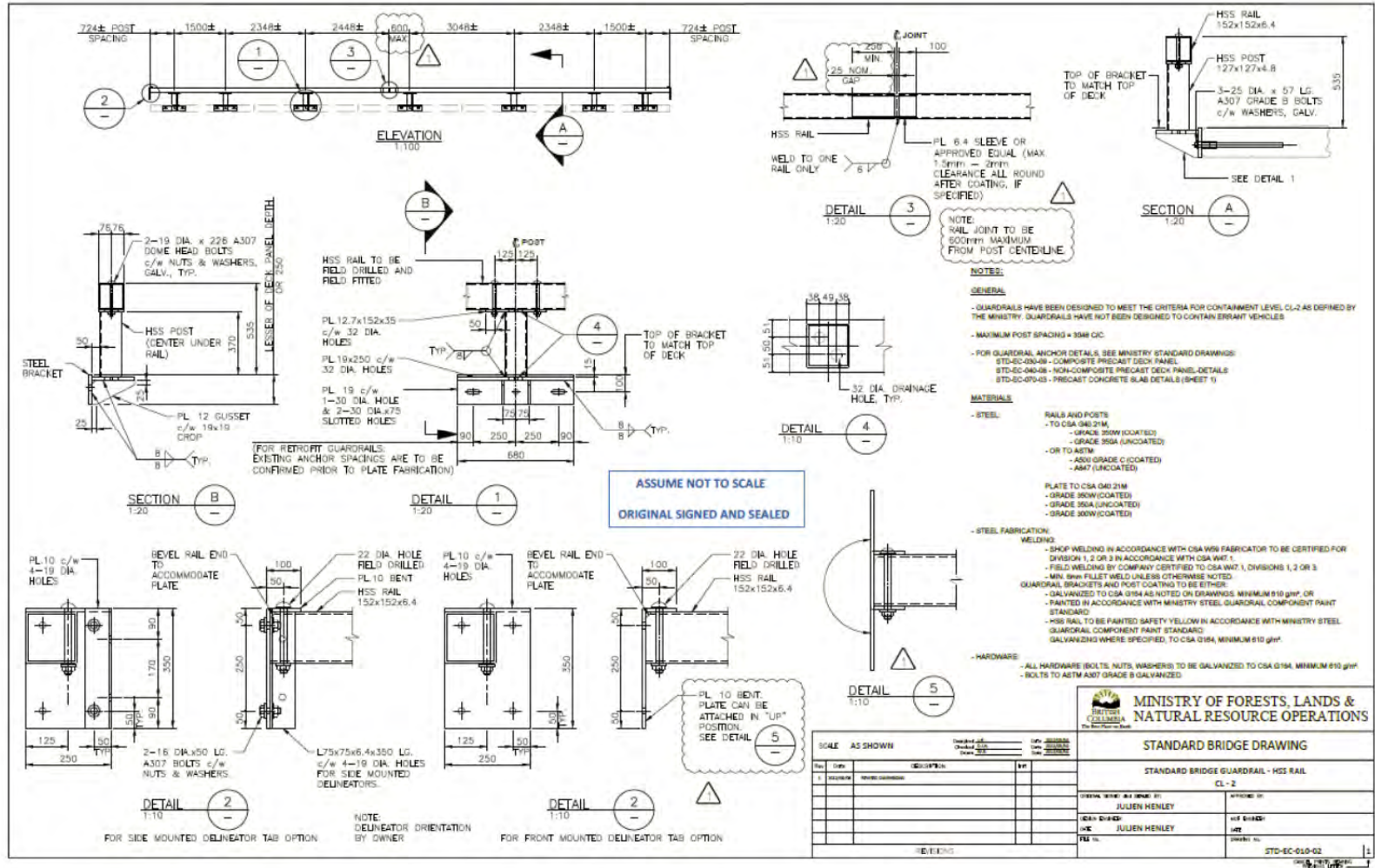




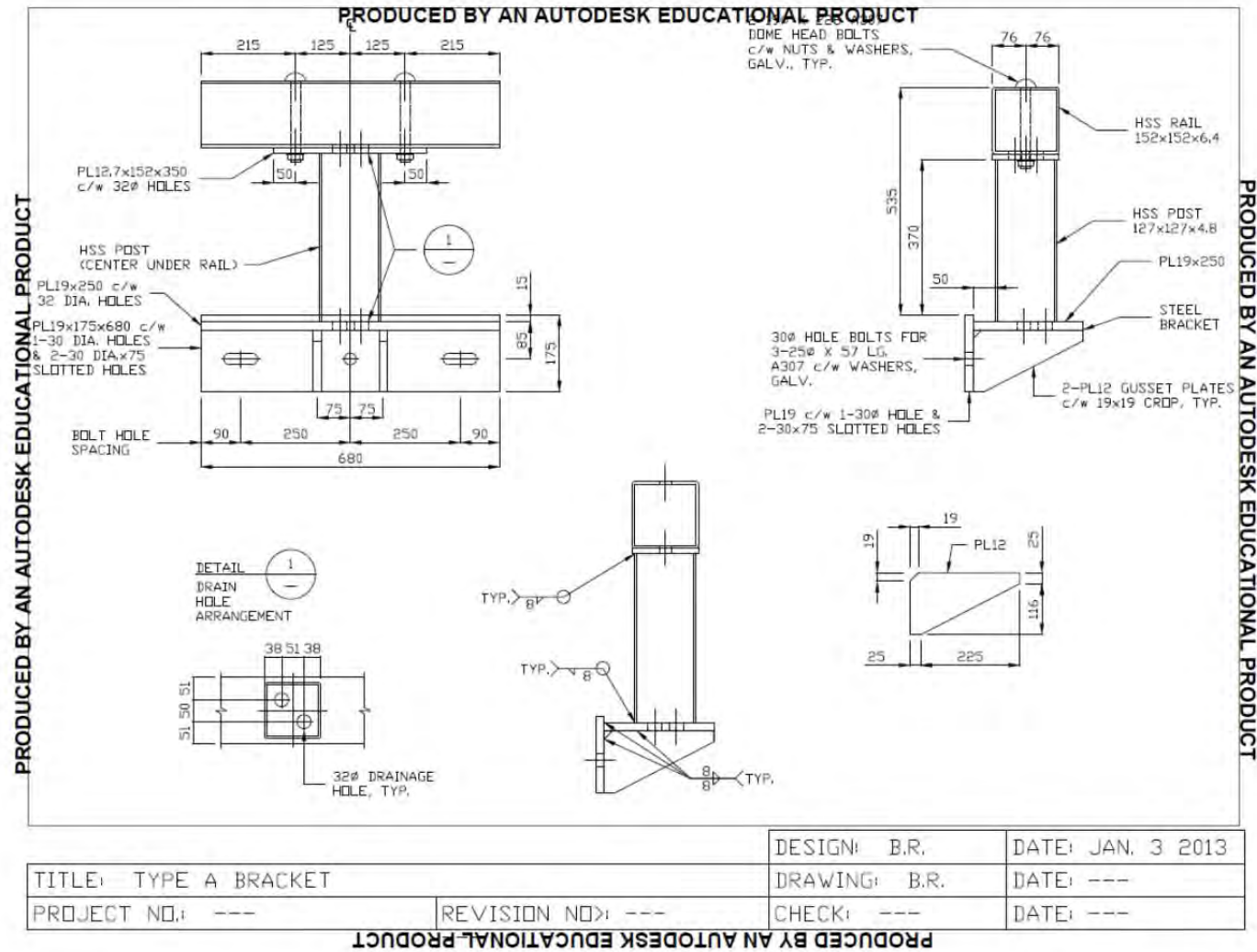
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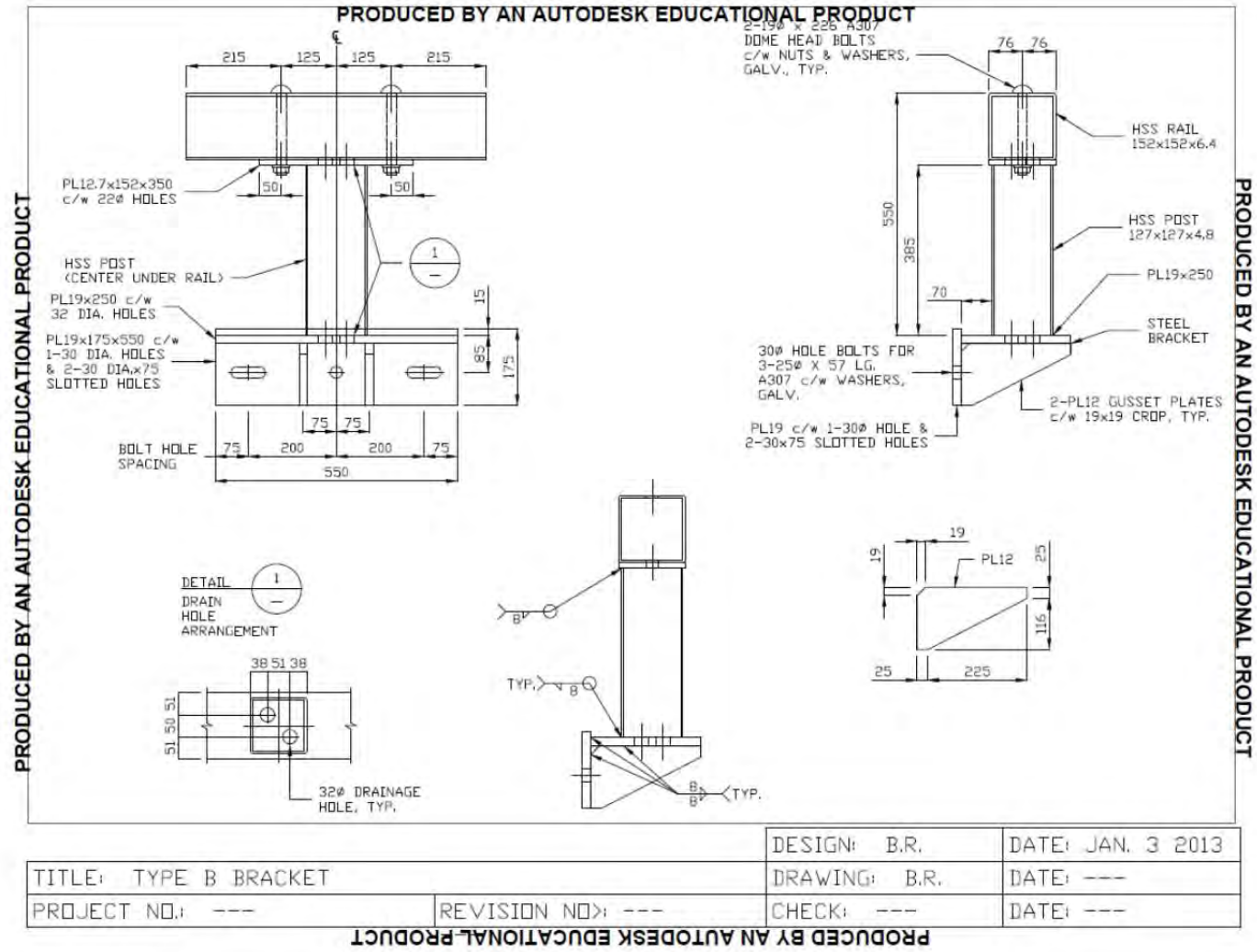
680 mm bracket



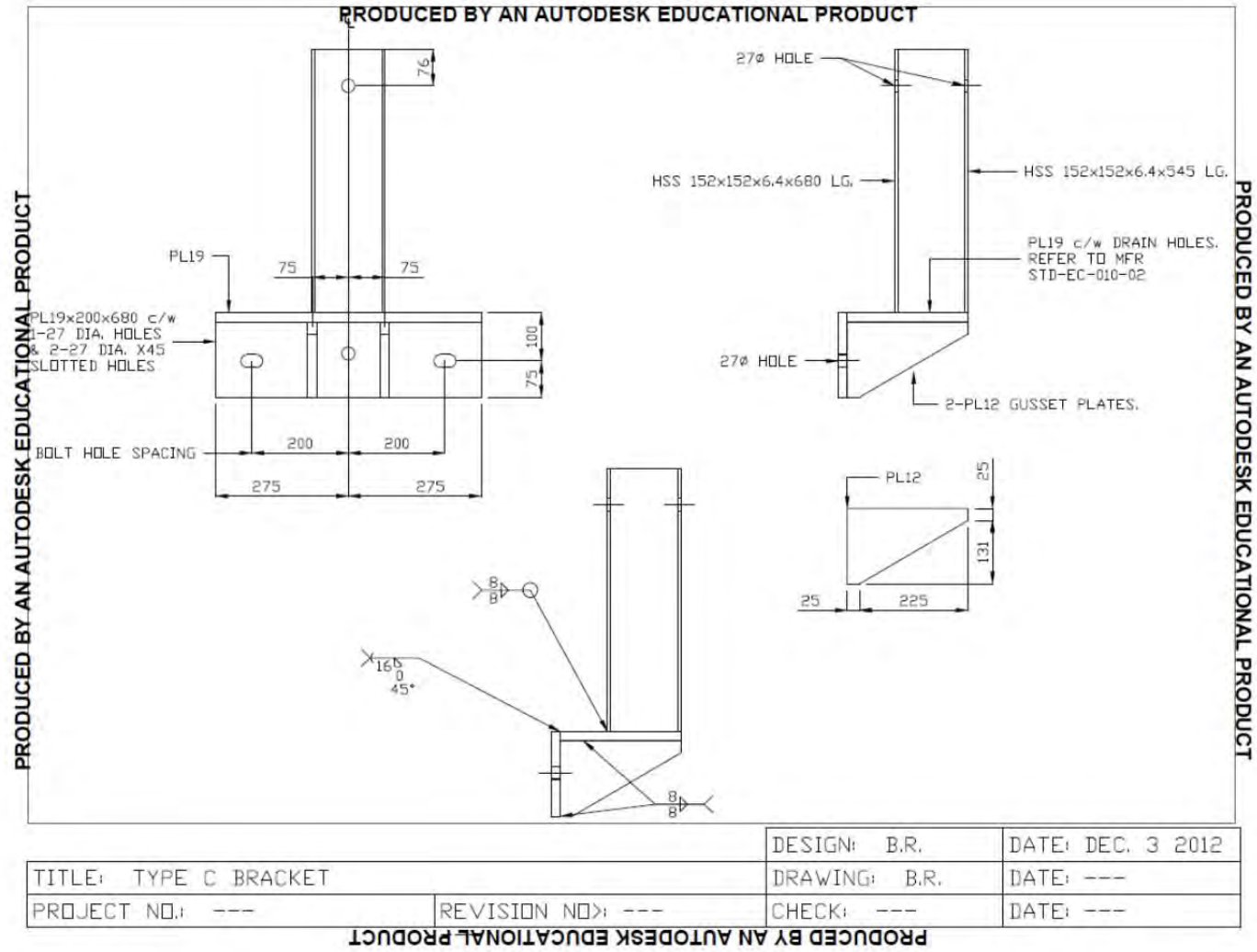
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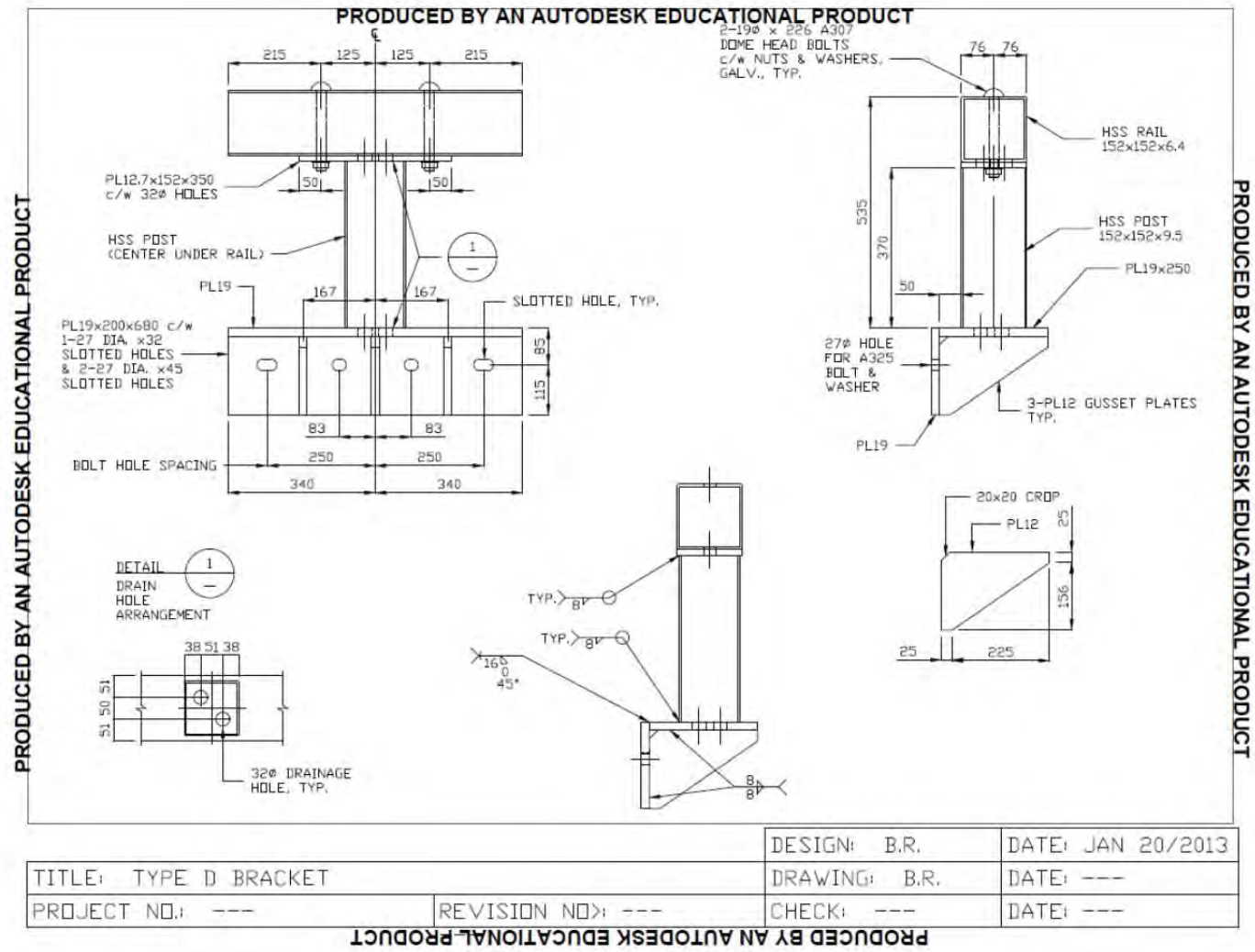
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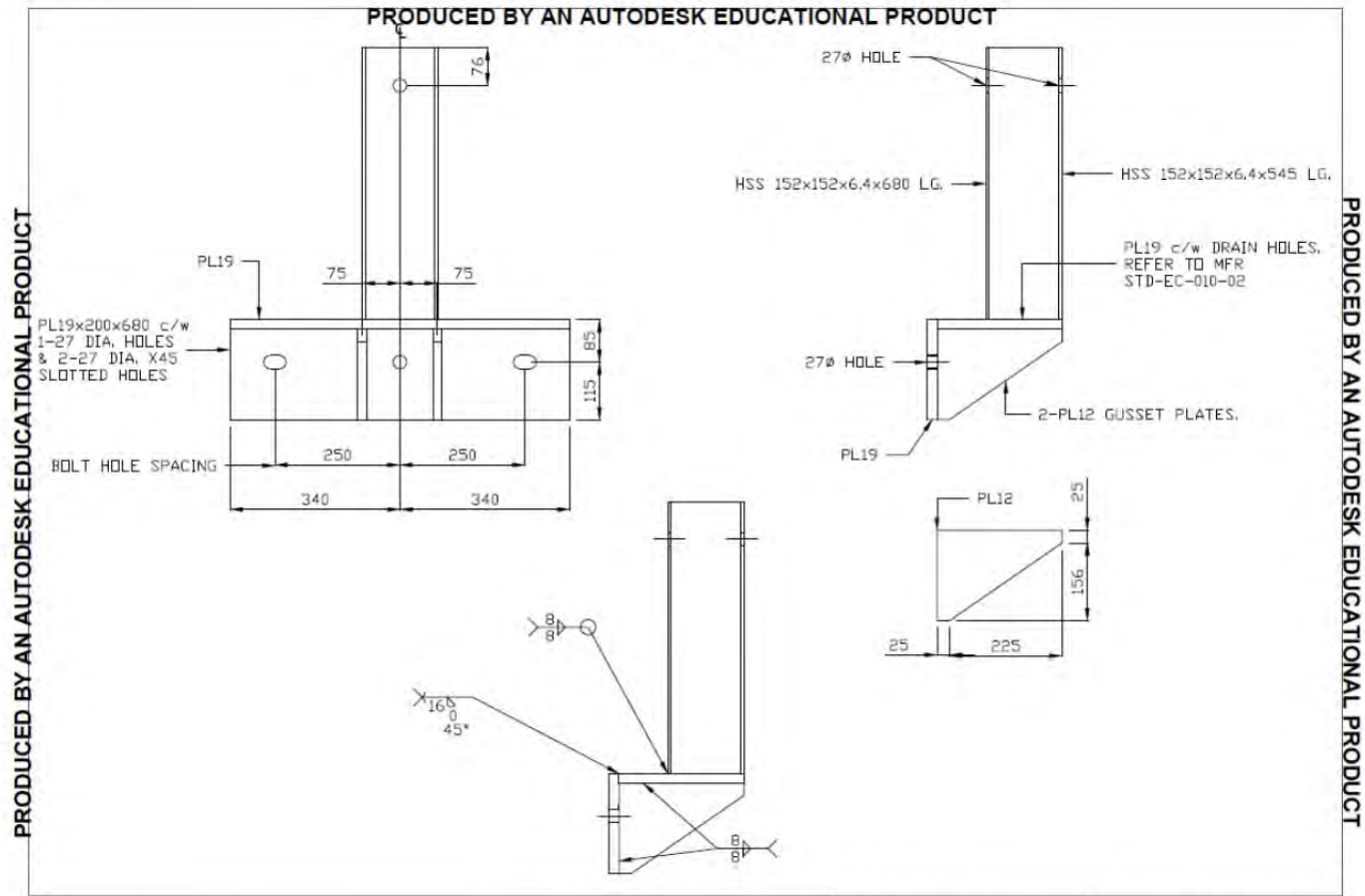
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GUARDRAIL DRAWING-Type D Bracket



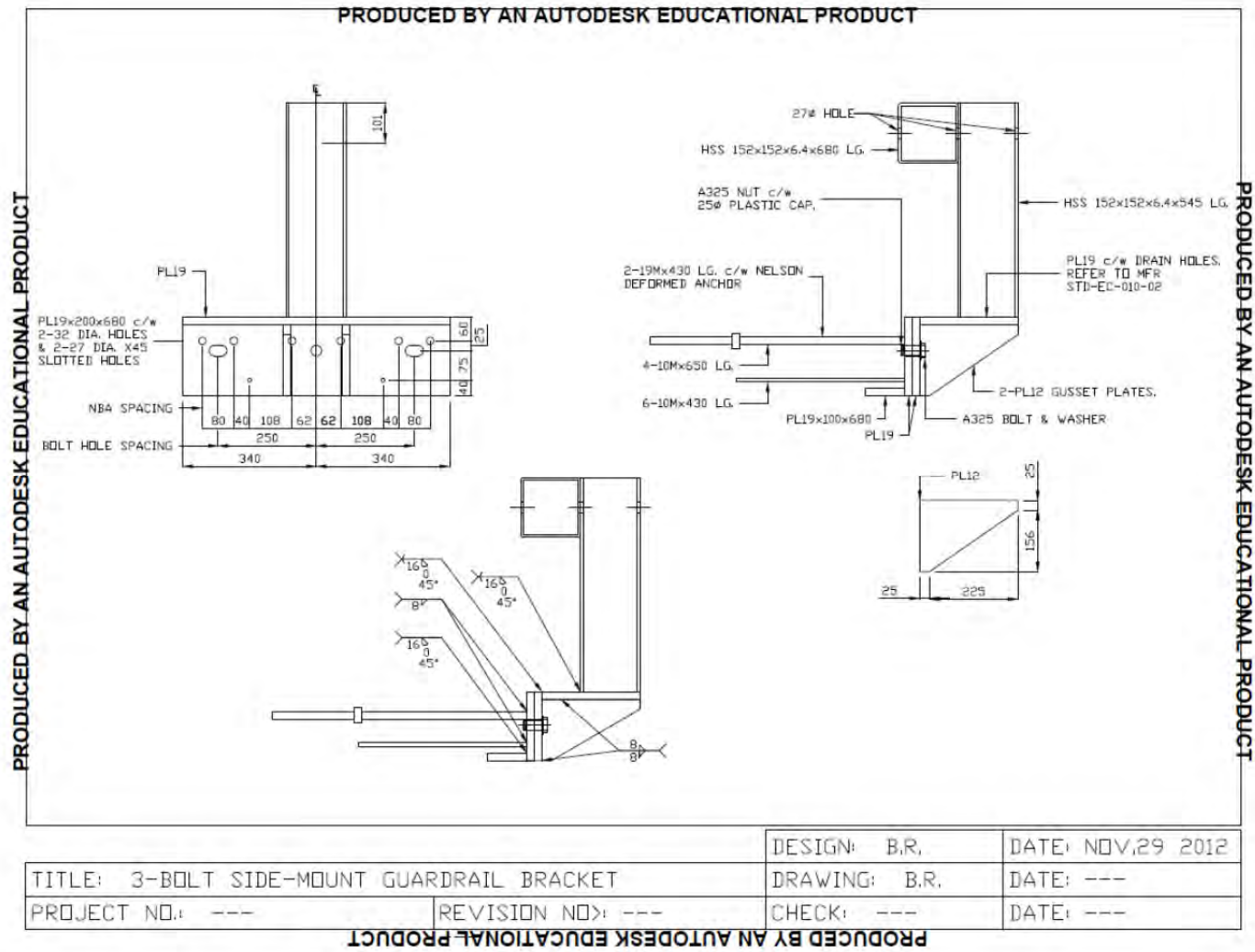
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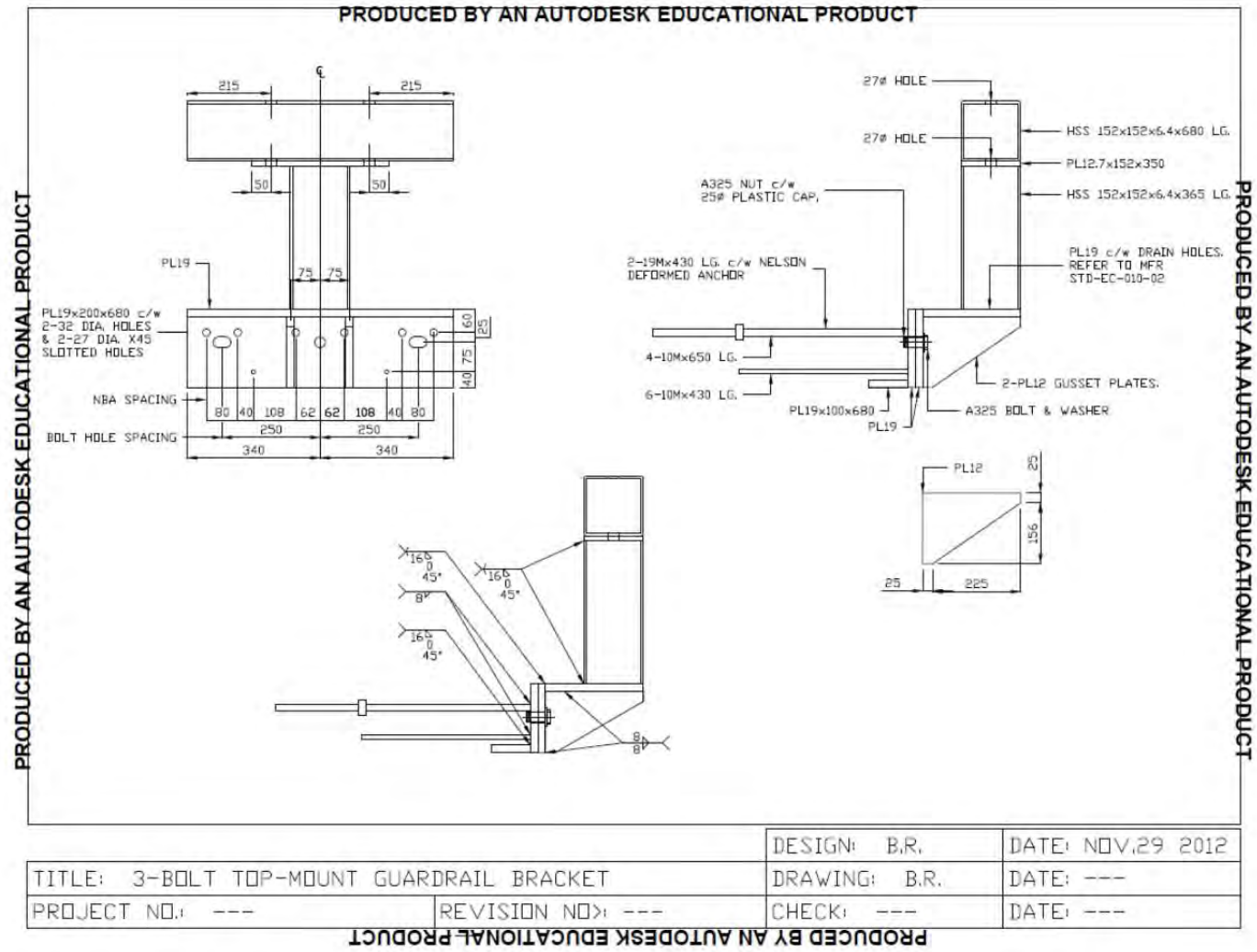
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PROJECT NO.: ---		DRAWING: B.R.	DATE: ---
REVISION NO.: ---		CHECK: ---	DATE: ---

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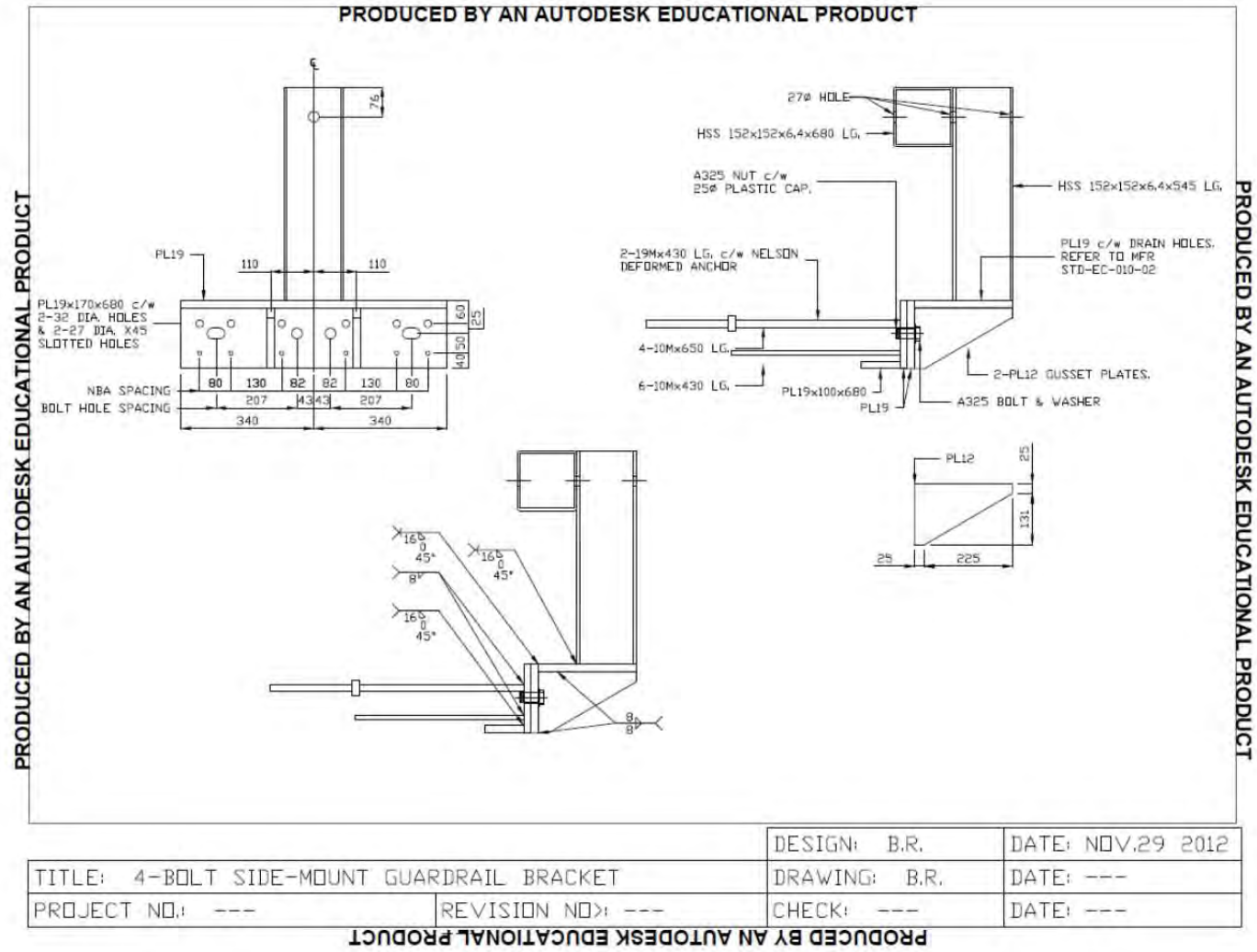
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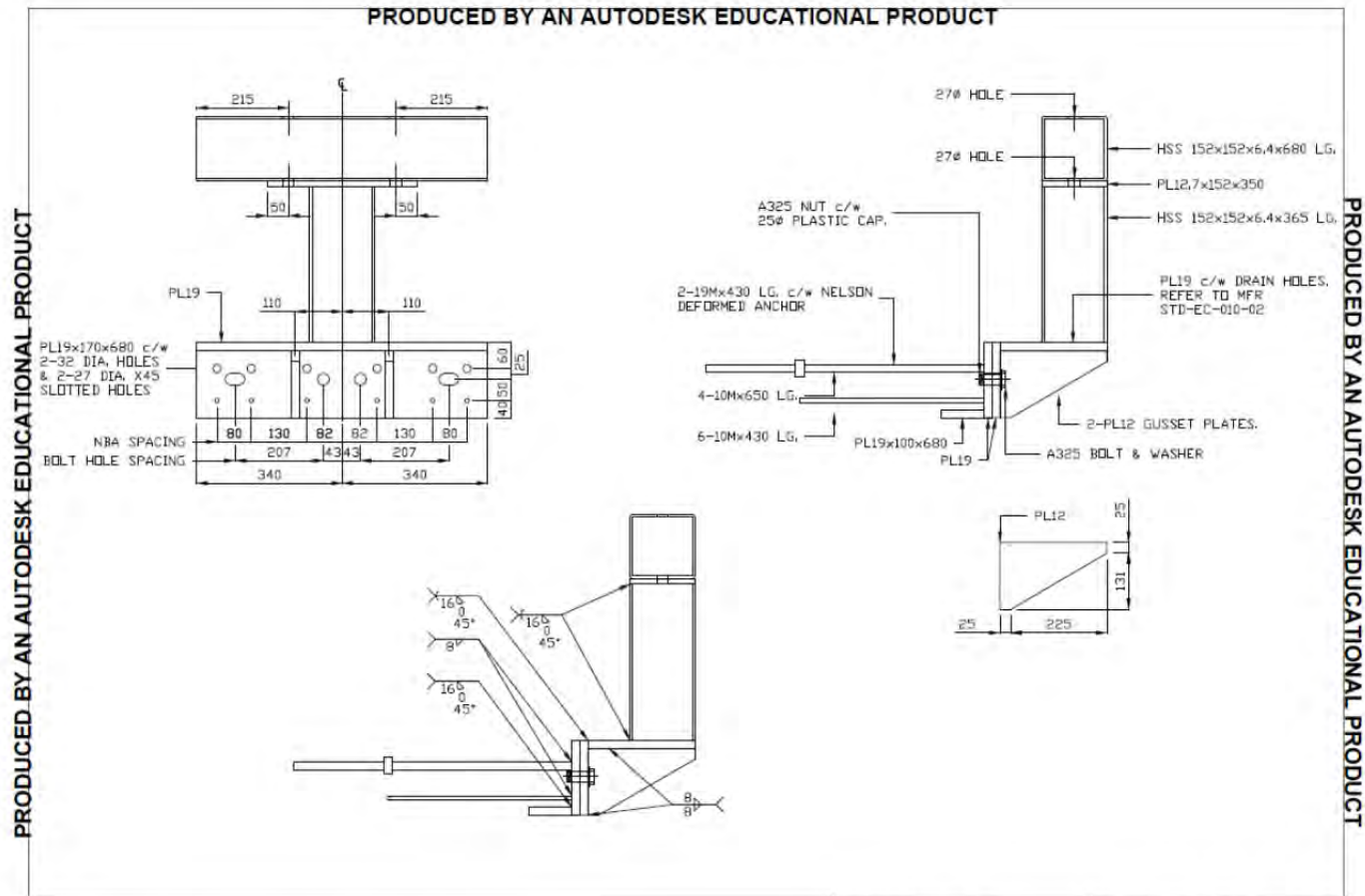
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GUARDRAIL DRAWING-4-Bolt Side-Mount Guardrail Bracket



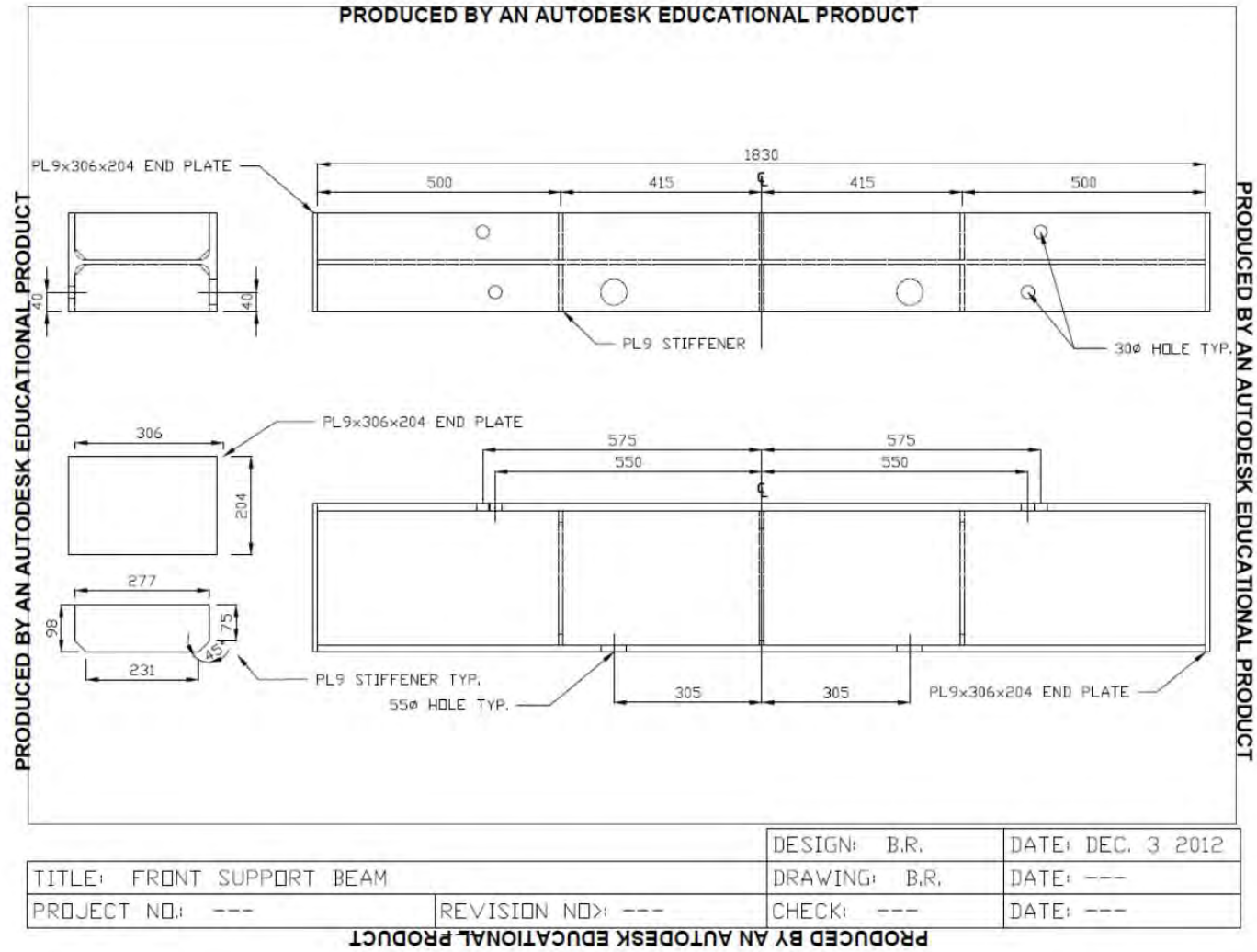
GUARDRAIL DRAWING-4-Bolt Top-Mount Guardrail Bracket



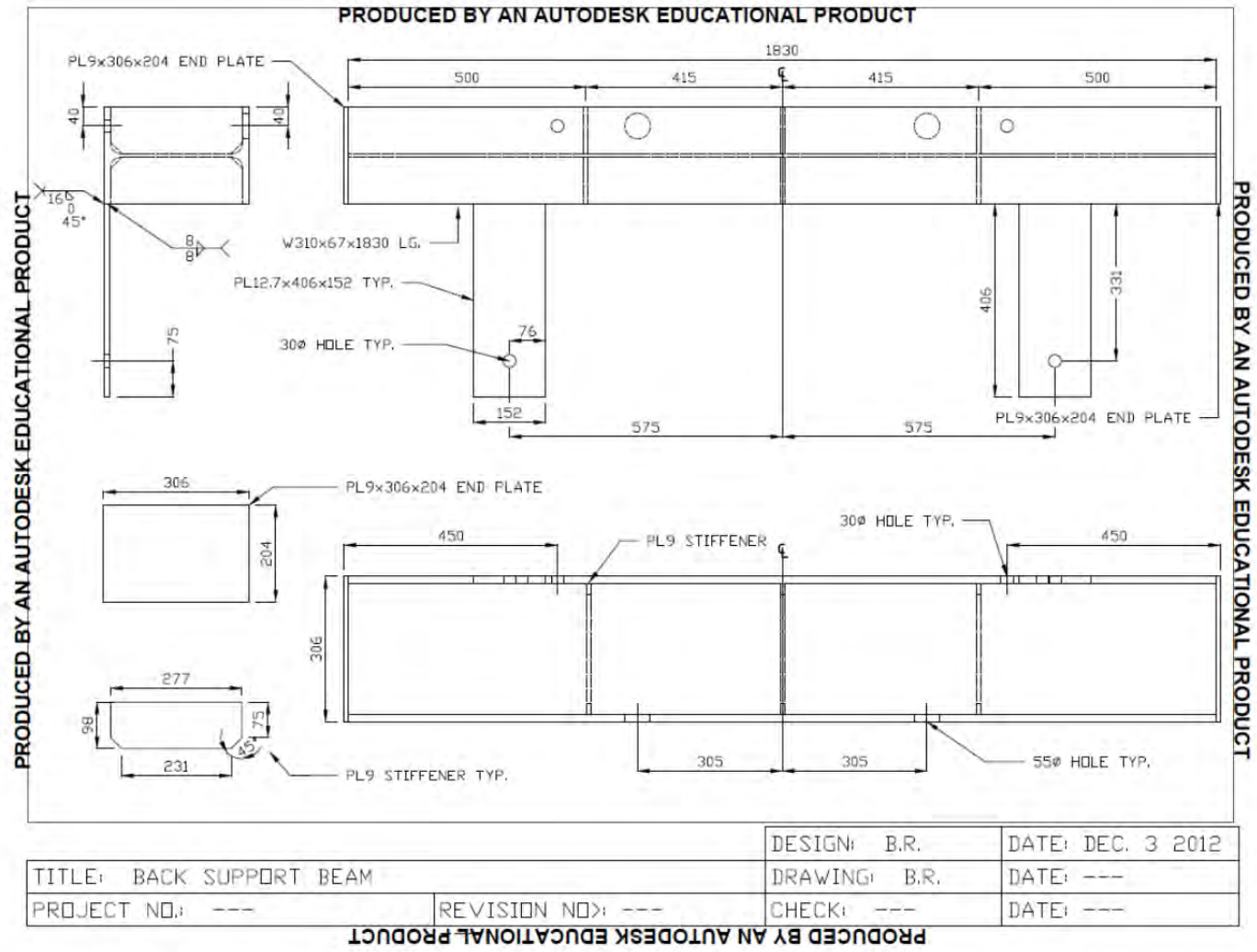
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DRAWING: B.R.		DATE: ---
PROJECT NO.: ---	REVISION NO: <>: ---	CHECK: ---
		DATE: ---

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GUARDRAIL DRAWING-Front Support Beam



GUARDRAIL DRAWING-Back Support Beam



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REPORT

Appendix B - Technical Memorandum "Classification of HSS Guide Retrofit Bridge Rail"

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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:

- When completing an analytical evaluation of a barrier, these forces represent factored forces and resistances should be calculated assuming nominal material strengths.
- 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.





Memo To: Brian Chow, M.Eng., P.Eng.

July 18, 2012

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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

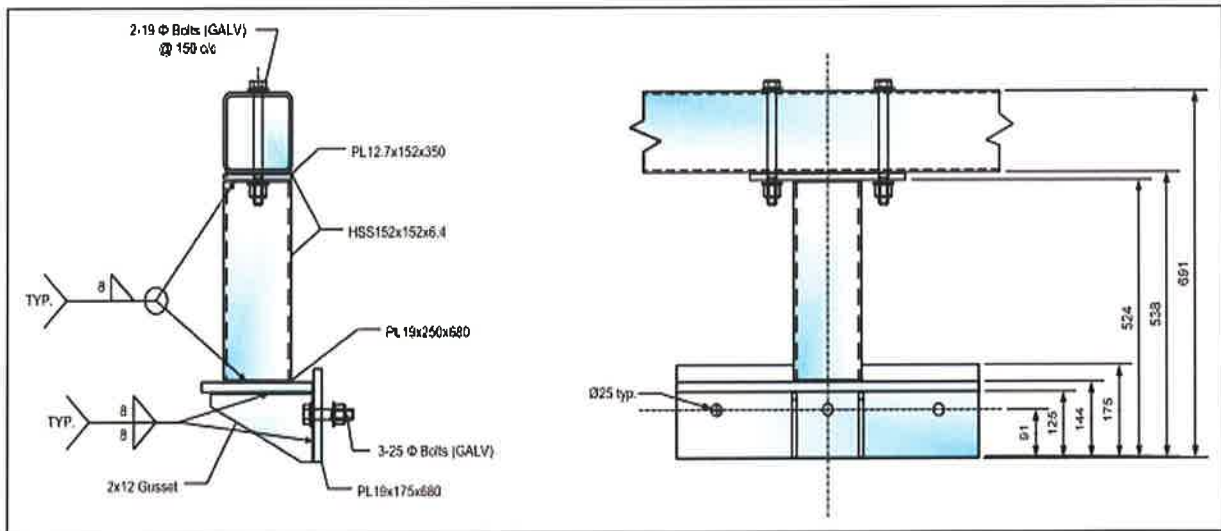


Table 2 presents the observed peak static loads recorded for each specimen during the experimental testing of the HSS Guide Retrofit Rail.

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.





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

Notes:

- The specimen ID references correspond with those assigned by UBC in the report "Experimental Evaluation of Concrete Decks with Guard Rail Systems", April 2011.
- Load applied 425 mm above travelled surface.
- 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





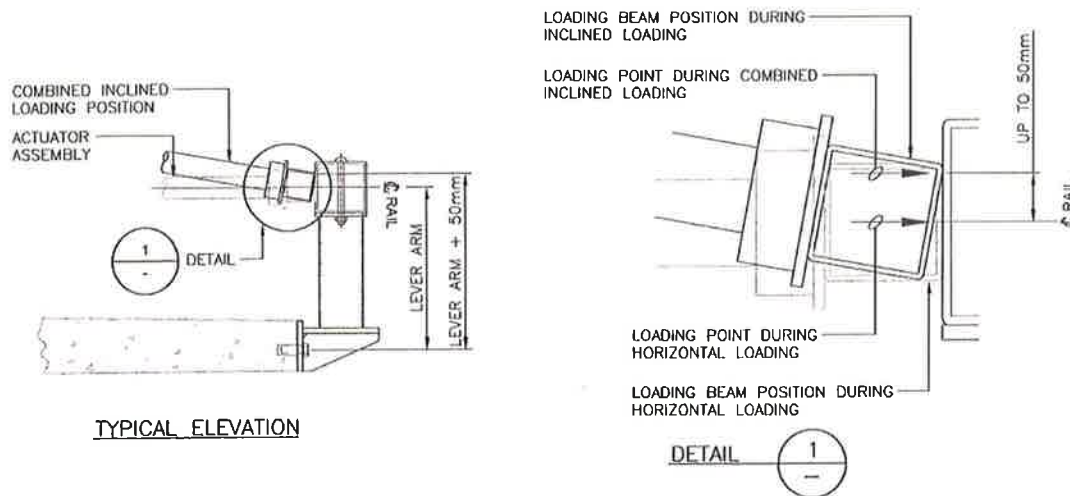
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arm would result in a 10% corresponding reduction in the applied load since the demand on the anchor bolts is inversely proportional to the height of the applied load.

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





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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 (F_y) and ultimate strengths (F_u) 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., Φ_s , Φ_c , and $\Phi_b = 1.0$.
- 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.





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

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:

- Grade A307 Bolt: $F_y = 248 \text{ MPa}$, $F_u = 414 \text{ MPa}$.
- Grade A325 Bolt: $F_y = 635 \text{ MPa}$, $F_u = 830 \text{ MPa}$.
- Assumed capacity of 25M reinforcing inserts: $F_y = 296 \text{ Mpa}$.
- Reduced effective depth to account for 20 mm chamfer and drip groove.
- No reduction in effective depth to account for chamfer and drip groove.
- Capacities calculated assuming nominal material strengths.
- 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 ($F_y = 248 \text{ MPa}$) 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





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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





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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.





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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





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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:

John Deenihan 18-Jul-2012

John Deenihan Ph.D., EIT
Structural Engineer

JD/JH/fd

Reviewed by:



2012.07.18

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



REPORT

Appendix C - Technical Memorandum "Review of Modified HSS Guide Retrofit Rail"

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Date: July 10, 2012 File: 20102698.00.E.05.00

To: Mr. Brian Chow, M. Eng., P.Eng.

From: Julien Henley M.A.Sc., P.Eng.

Project: Development of MFR Standard Curb Design Parameters

Subject: Review of Modified HSS Guide Retrofit Rail

MEMO

As part of the Ministry of Forests, Lands and Natural Resource Operations development of appropriate bridge barrier design guidelines, they retained the University of British Columbia (UBC) to complete an experimental program to verify the capacity of standard bridge barriers currently in use in the forest industry in British Columbia. During the experimental program, UBC modified the HSS Guide Retrofit Rail by adding a knee-brace in an effort to increase the capacity of the rail. This modification resulted in a capacity approximately 2.3 times greater than that of a typical side mounted connection. This memorandum, provides a brief summary of Associated Engineering’s review of the modifications and classification of the barrier based on the recommendations included the 2011 AE report, “Phase III – Guidelines for Barrier Selection and Design” and summarized 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:

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

Figure 1 illustrates the general modified HSS Guide Retrofit Rail, along with the theoretical free body force diagram. The post and rail component are identical to that of a standard Side Mounted HSS Guide Retrofit Rail, with the exception of the addition of the knee-brace, which extended approximately 600 mm under the precast concrete deck panel. Full details are presented in the 2011 UBC report titled “*Experimental Evaluation of Concrete Decks with Guard Rail Systems*”. The rail was mounted on a 175 mm thick concrete panel with reinforcing and couplers matching the MFLNRO Drawings STD-EC-030 Series.

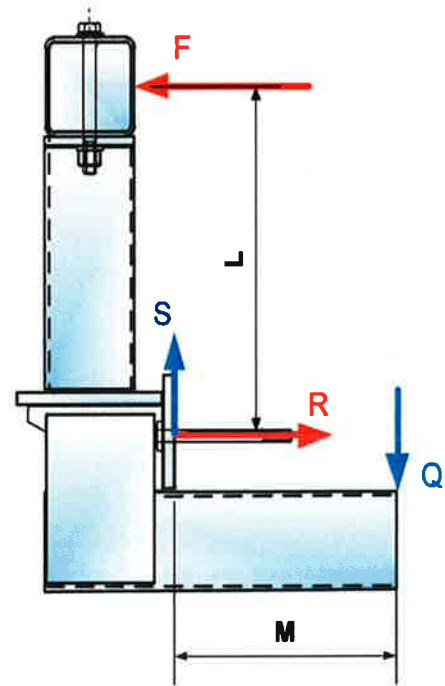
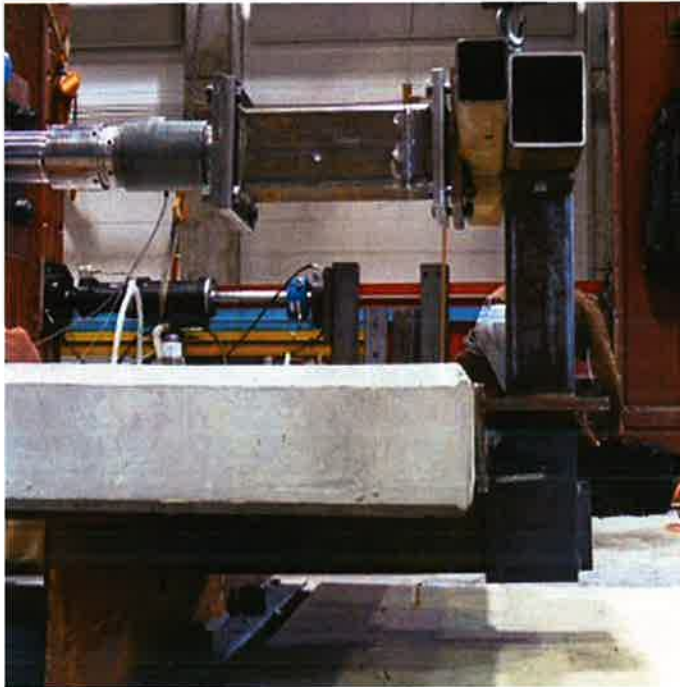


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Figure 1
Modified HSS Guide Retrofit Rail



Under transverse loading, the standard side mounted HSS Guide Retrofit Rail reacts in compression against the concrete deck below the location of the bolt insert; this reaction force magnifies the horizontal tension load applied to the bolts as discussed in our Memorandum titled “*Classification of HSS Guide Retrofit Bridge Rail*”, May 2012. The modified HSS Guide Retrofit Rail with knee-brace, transfers the applied transverse force to the bearing location of the knee-brace via rotation about the bolt inserts. The resultant force is resisted by shear in the anchor bolts (as opposed to tension) and the reaction of the knee-brace against the underside of deck (or girder in the case of the experimental test). As a result, the failure mechanism is different to that observed for the side mounted HSS Guide Retrofit Rail, with the anchor bolt inserts punching through the underside of the deck rather than concrete crushing or bolts fracturing as previously observed.

Figure 2 illustrates the observed failure mode of the modified HSS Guide Retrofit Rail.



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Figure 2
Typical Observed Failure of the Modified HSS Guide Retrofit Rail





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

Table 2 Observed Peak Failure Loads and Associated Failure Mechanisms

Specimen ID ¹	Observed Peak Horizontal Load (kN)	Comments
4.3	154.8	Description: A307 bolts connecting barrier and concrete panel. Failure Mode: Spalling of concrete in vicinity of inserts.
4.4	124.1	Description: A307 bolts connecting barrier and concrete panel. Failure Mode: Spalling of concrete in vicinity of inserts.
4.5	164.4	Description: A307 bolts connecting barrier and concrete panel. Failure Mode: Spalling of concrete in vicinity of inserts.

Note:

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.

A review of the observed peak failure loads, and comparison with the recommended resistances shown in Table 1 for the CL-3 barrier, suggests that this simple modification to the HSS Guide Retrofit Rail is sufficient to increase the strength of the barrier, resulting in its classification as a CL-3 barrier. However, after reviewing the UBC Report, associated videos documenting the testing and discussions with UBC researchers we established that the knee-brace extended approximately 600 mm under the concrete deck and was supported on the girder flange, although it did not react against the supporting girder web.

Although a knee-brace of this length results in a significant reduction in the demand on the anchor bolts, it is not practical for field installations, since the knee-brace would rest on the girder flange making installation and accommodation of field tolerances difficult. A review of typical steel girder and concrete deck forestry bridges suggests that the maximum practical lever arm is 300 to 400 mm, which results in an increased demand on the anchor bolts. We completed a preliminary analysis based on a reduced knee-brace length (400 mm), as shown in Figure 3, and determined an approximate transverse load capacity of 98 kN, which suggests that it does not meet the proposed requirements for a CL-3 barrier, which requires a minimum resistance of 120 kN (Table 1).

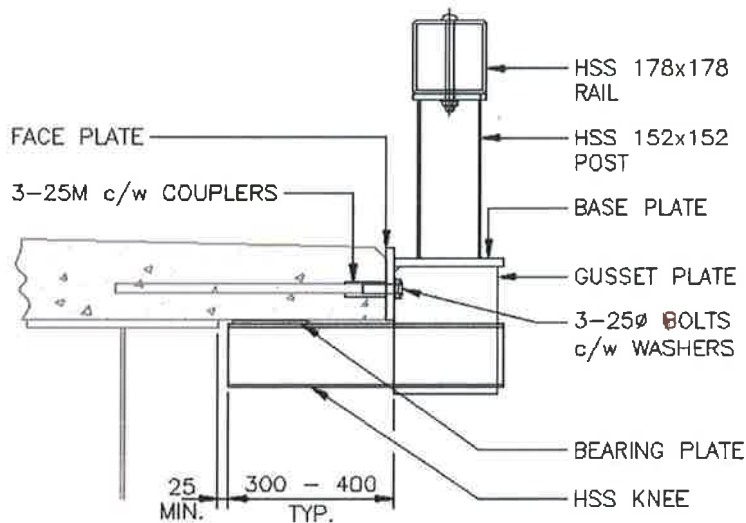


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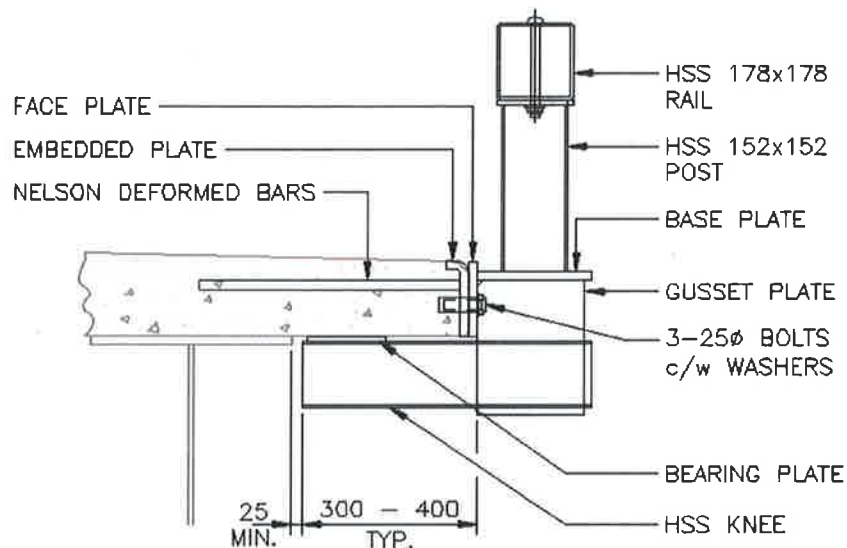
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Figure 3
Modified HSS Guide Retrofit Rail with Reduced Length Knee-Brace Length



With the required modifications resulting in a significant reduction in the strength, we have proposed modifying the barrier, as shown in **Figure 4**, to increase the strength of the connection to the deck. The modifications include the addition of an embedded steel plate and nelson deformed bars to improve the shear resistance of the connection. We believe that further experimental testing will verify that these modifications will result in the barrier being classified as a CL-3 barrier.

Figure 4
Modified Knee-Brace Connection Detail





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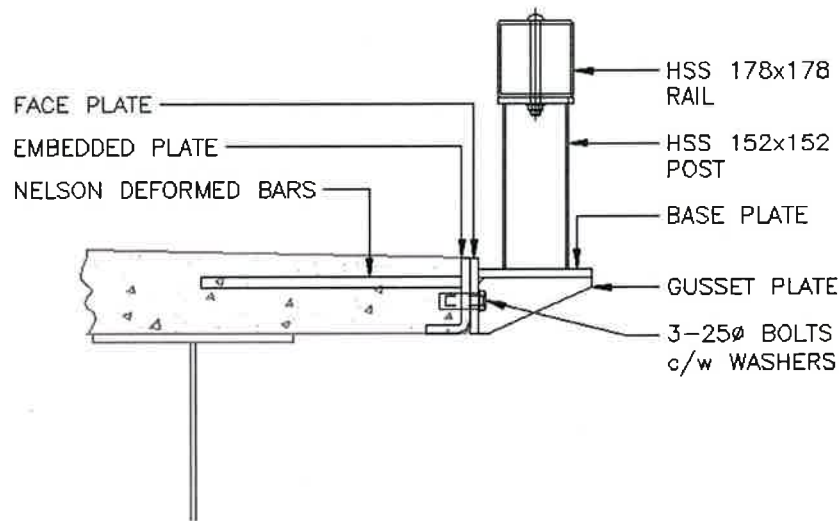
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Although the modified knee-brace configuration may provide the required containment, it poses a number of installation challenges:

- To provide the vertical adjustment to ensure that the bracket bears against the underside of the concrete deck, the design needs to incorporate vertical slotted holes, which may reduce the shear capacity of the connection, the primary mode for resisting the applied load. We did consider a slip critical connection, but the shear demand on the bolt is significantly higher than what we could practically provide with three to four anchor bolts.
- Access to the underside of the deck is required to ensure that the knee-brace is bearing against the concrete.
- Since girder flange widths vary, it is not possible to develop a "standard" knee-brace length.

Given these challenges, we briefly investigated alternative anchorages to determine whether it is possible to develop a side-mounted CL-3 barrier anchorage system for precast concrete deck panels. While our initial analysis, and review of the experimental data, suggests that it may be challenging to develop a CL-3 barrier anchorage given the significant tension and compression demands on the edge of a 175 mm thick concrete deck, **Figure 5** presents a possible solution.

Figure 5
Proposed Alternative Connection Detail



The proposed connection includes an embedded plate that increases the edge compressive capacity of the deck by confining the concrete. Further, the level arm between the compressive and tensile forces is increased by locating the nelson deformed anchors within the top mat of reinforcing, rather than at the mid-depth of the panel. Since the theoretical analysis suggests that this connection may not provide the required resistance, the capacity would need to be verified through experimental testing.





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Prior to completing further experimental testing, we recommend that the Ministry:

- Discuss the proposed details with fabricators to determine whether they are feasible and economical.
- Consider increasing the minimum deck panel thickness from 175 mm to 225 mm.
- Consider using a top mounted rail for precast concrete deck panels.
- Review existing AASHTO TL2 crash tested barriers (equivalent to the CL-3 criteria) and adopt a previously tested barrier.

Respectfully submitted,

Prepared by:

 2012.07.10

John Deenihan, Ph.D., EIT
Structural Engineer

Reviewed by:

 2012.07.10

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

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Appendix D - Summary of Barrier Test Results to Date

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Panel Type	Test ID	Test Date (mm/yy)	Bracket Width (mm)	Bracket & Barrier Description	Panel Description (Standard MoF Panel Reinforcement)	Concrete Strength (f _c)	Bracket Bolt Grade	Load Application Height (mm)	Theoretical Capacity (kN)	Theoretical Failure Mechanism	Target Capacity (kN)	Experimental Capacity (kN)	Experimental Failure Mechanism
A	1.1	02 / 11	800	Side Mounted Timber Barrier	<ul style="list-style-type: none"> 175 mm edge thickness Excluding 2 U-shaped reinforcing bars at each insert location Drip groove 50mm from panel edge 3 bolts connecting bracket to panel 	56	A307	425	1.8 ²	Bearing failure under washer	N/A	19.7	Vertical crushing of the timber blocks at the bolt washers, bottom of the timber blocks split vertically along the three bolt holes
	1.2	02 / 11							1.8 ²	Bearing failure under washer	N/A	23.5	
	1.3 ¹	03 / 11							1.8 ²	Bearing failure under washer	N/A	26.3	
	1.4 ¹	03 / 11							1.8 ²	Bearing failure under washer	N/A	23.5	
A	2.1	02 / 11	680	Side Mounted Steel Bracket w/ Steel Post/Rail	<ul style="list-style-type: none"> 175 mm edge thickness Excluding 2 U-shaped reinforcing bars at each insert location Drip groove 50mm from panel edge 3 bolts connecting bracket to panel 		A307		47	Bolts yield/fracture	60	64.2	Bracket bolts ruptured in tension
	2.11	02 / 11					A325		51	Bond failure (inserts fail/bar pullout)	60	68.1	Spalling of the concrete in the vicinity of the inserts
	2.2	02 / 11					A307		47	Bolts yield/fracture	60	65.7	Bracket bolts ruptured in tension
	2.21	02 / 11					A325		51	Bond failure (inserts fail/bar pullout)	60	57.3	Spalling of the concrete in the vicinity of the inserts
	2.3 ¹	03 / 11					A307		47	Bolts yield/fracture	60	56.4	Bracket bolts ruptured in tension
	2.4 ¹	03 / 11					A307		47	Bolts yield/fracture	60	54.9	Bracket bolts ruptured in tension
A	3.1	03 / 11	680	Top & Side Mounted Steel Bracket w/ Steel Post/Rail	<ul style="list-style-type: none"> 175 mm edge thickness Excluding 2 U-shaped reinforcing bars at each insert location Drip groove 50mm from panel edge 3 bolts connecting bracket to panel 2 additional top mounted bolts through the deck 		A307		N/A	N/A	60	50.1	Deck fails in block shear
	3.2	03 / 11							N/A	N/A	60	68.7	
	3.3	03 / 11							N/A	N/A	60	67.2	
	3.4	03 / 11							N/A	N/A	60	58.3	
A	4.1 ¹	03 / 11	680	Timber Barrier on Side Mounted Steel Bracket	<ul style="list-style-type: none"> 175 mm edge thickness Excluding 2 U-shaped reinforcing bars at each insert location Drip groove 50mm from panel edge 3 bolts connecting bracket to panel 		A307		1.8 ²	Bearing failure under washer	N/A	36.1	Vertical crushing of the timber blocks at the bolt washers, bottom of the timber blocks split vertically along the three bolt holes
	4.2 ¹	03 / 11		1.8 ²					Bearing failure under washer	N/A	42.2		
	4.3	03 / 11		Side Mounted Steel Bracket & Knee Brace w/ Steel Post/Rail		N/A		N/A	120	154.8	Spalling of the concrete in the vicinity of the inserts		
	4.4	03 / 11		N/A		N/A		120	124.1				
	4.5	03 / 11		N/A		N/A		120	164.4				
A	A1-1	03 / 13	680	Side Mounted Steel Bracket w/ Steel Post/Rail	<ul style="list-style-type: none"> 175 mm edge thickness Including 2 U-shaped reinforcing bars at each insert location 	40	A325	485	43.9	Concrete fails in the vicinity of the inserts	60	44.1	Spalling of the concrete in the vicinity of the inserts
	A1-2	03 / 13								Concrete fails in the vicinity of the inserts	60	45.0	
B	B1-1	03 / 13	550	Side Mounted Steel Bracket w/ Steel Post/Rail	<ul style="list-style-type: none"> Drip groove 300mm from panel edge 3 bolts connecting bracket to panel 		42.2		Concrete fails in the vicinity of the inserts	60	41.1		
	B1-2	03 / 13							Concrete fails in the vicinity of the inserts	60	47.9		
C	C1-1	03 / 13	550	Side Mounted Steel Bracket w/ Steel Post/Rail	<ul style="list-style-type: none"> 175 mm edge thickness Including 2 U-shaped reinforcing bars at each insert location Drip groove 300mm from panel edge 3 bolts connecting bracket to panel Anchor bolts welded to the ends of the anchor reinforcing bars 		42.4		Concrete fails in the vicinity of the inserts	60	45.6		
	C1-2	03 / 13							Concrete fails in the vicinity of the inserts	60	43.9		
	C2-1	03 / 13							Concrete fails in the vicinity of the inserts	60	37.8		
	C2-2	03 / 13							Concrete fails in the vicinity of the inserts	60	40.7		
D	D1-1	02 / 13	680	Side Mounted Steel Bracket w/ Steel Post/Rail	<ul style="list-style-type: none"> 175 mm edge thickness (Panel Type D) 200 mm edge thickness (Panel Type E) Excluding 2 U-shaped reinforcing bars at each insert location Drip groove 300mm from panel edge 4 bolts connecting bracket to panel 4 long & 4 short NDB's anchored to embedded steel angle at deck edge 		133		Embedded steel yield/fracture	120	141.5	Bond failure of short NDB's & fracture of long NDB's	
	D1-2	02 / 13							Embedded steel yield/fracture	120	117.9		
	D2-1	02 / 13							Embedded steel yield/fracture	120	137.0		
	D2-2	02 / 13							Embedded steel yield/fracture	120	122.6		
E	E1-1	02 / 13	680	Side Mounted Steel Bracket w/ Steel Post/Rail	<ul style="list-style-type: none"> Drip groove 300mm from panel edge 4 bolts connecting bracket to panel 4 long & 4 short NDB's anchored to embedded steel angle at deck edge 		166		Embedded steel yield/fracture	120	143.4		
	E1-2	02 / 13							Embedded steel yield/fracture	120	150.7		
	E2-1	02 / 13							Embedded steel yield/fracture	120	148.0		
	E2-2	03 / 13							Embedded steel yield/fracture	120	160.5		
G	G1-1	04 / 13	680	Side Mounted Steel Bracket w/ Steel Post/Rail	<ul style="list-style-type: none"> 175 mm edge thickness (Panel Type G) 200 mm edge thickness (Panel Type H) Excluding 2 U-shaped reinforcing bars at each insert location 	168	Embedded steel yield/fracture	120	165.0	Flexural failure – top concrete cover failed resulting in loss of anchorage to the stacked studs followed by concrete crushing in the vicinity of the inserts			
	G1-2	04 / 13					Embedded steel yield/fracture	120	161.4				
H	H1-1	04 / 13	680	Side Mounted Steel Bracket w/ Steel Post/Rail	<ul style="list-style-type: none"> Drip groove 300mm from panel edge 4 bolts connecting bracket to panel 4 long NDB's & 4 pairs stacked studs anchored to embedded steel angle at deck edge 	218	Embedded steel yield/fracture	120	193.0				
	H1-2	04 / 13					Embedded steel yield/fracture	120	172.8				

¹ Specimens subject to a combined vertical and horizontal loading ratio of 1.08:6

² Values not determined by Associated Engineering (AE)