Testing of FLNR Standard Curb Systems

By

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Purpose of testing

Laboratory testing shall be conduction 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

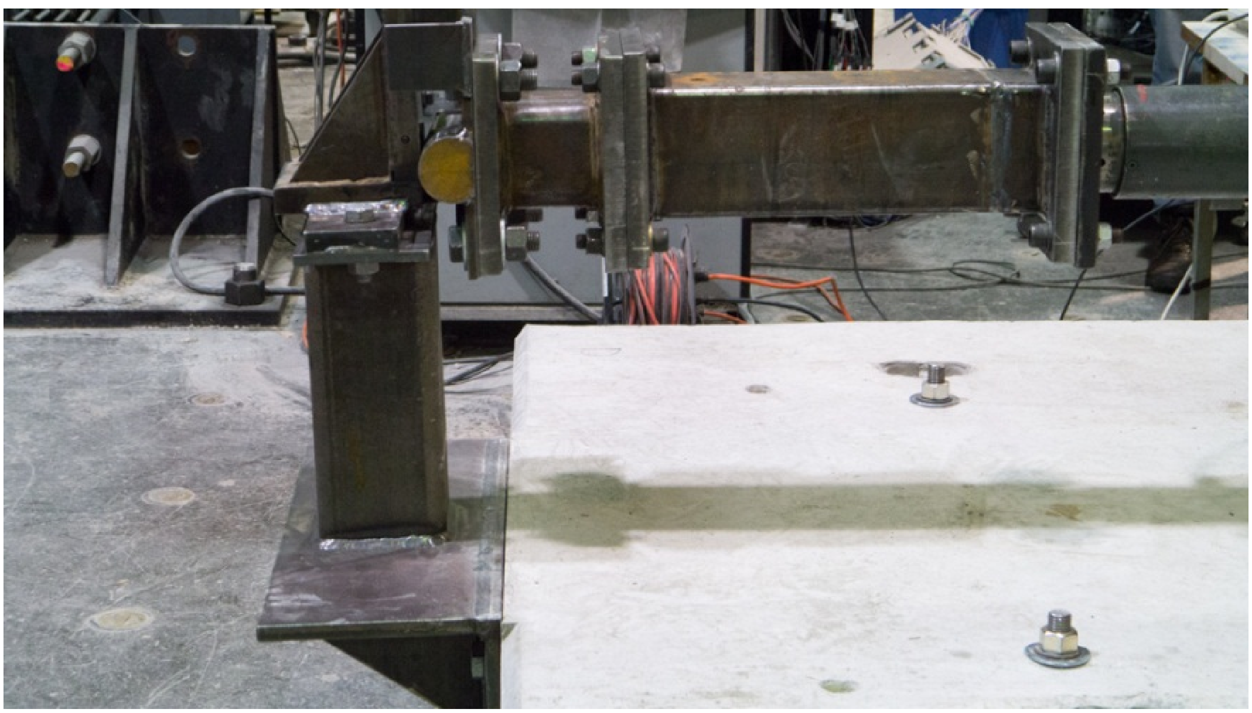


Figure 2: Detail of Load Application

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

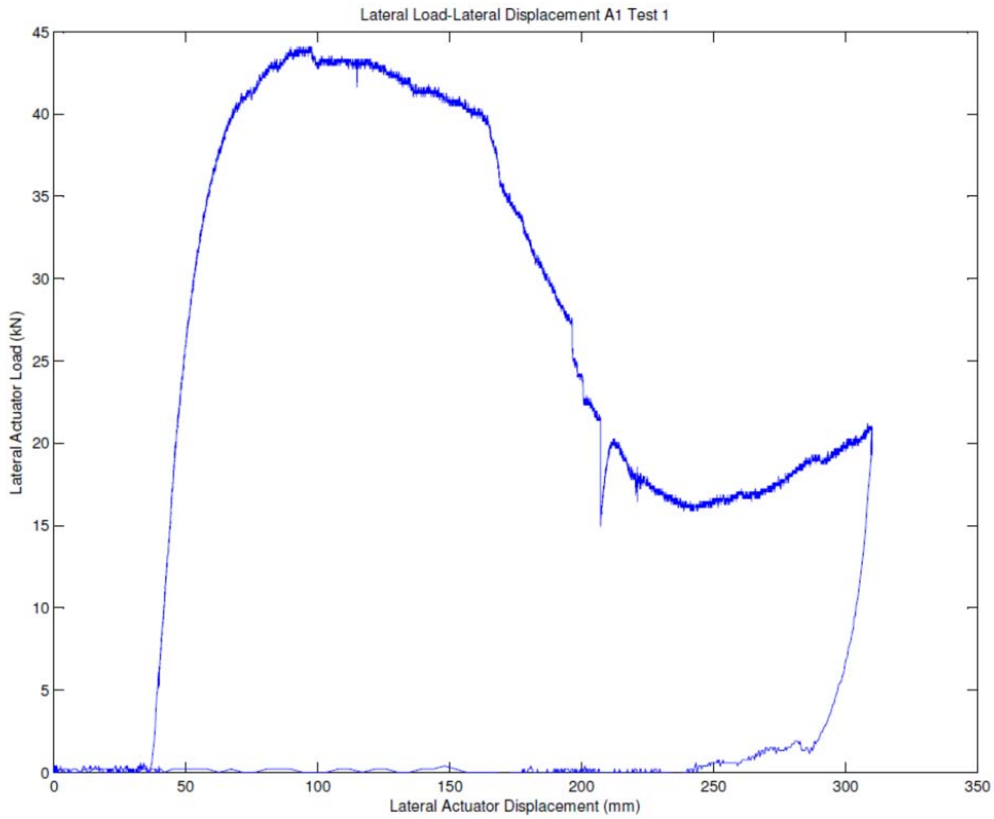


Figure 5: Load/Deflection Curve of Top of Bracket, A-1-1 (Note that initial drop in load corresponded to when the compression concrete on the slab face crushed to the drip groove)

Figure 6: Images from A-1-1

Panel / Bracket Test A-1-2

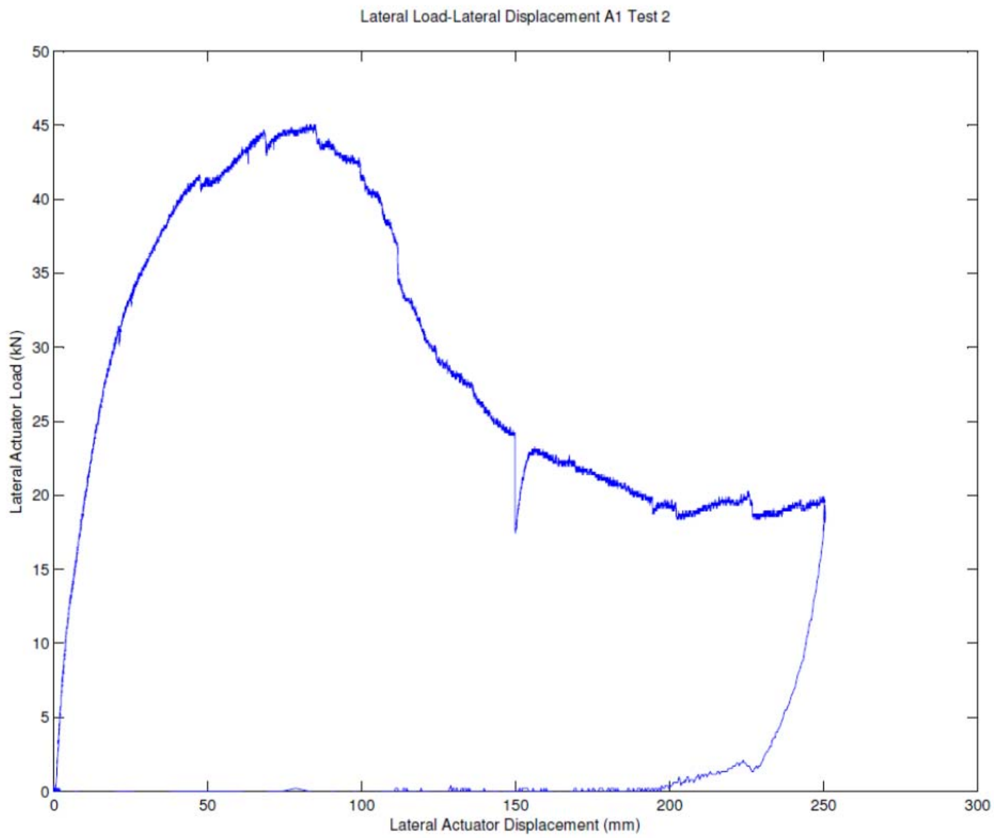


Figure 7: Load/Deflection Curve of Top of Bracket, A-1-2 (Note that initial drop in load corresponded to when the compression concrete on the slab face crushed to the drip groove)

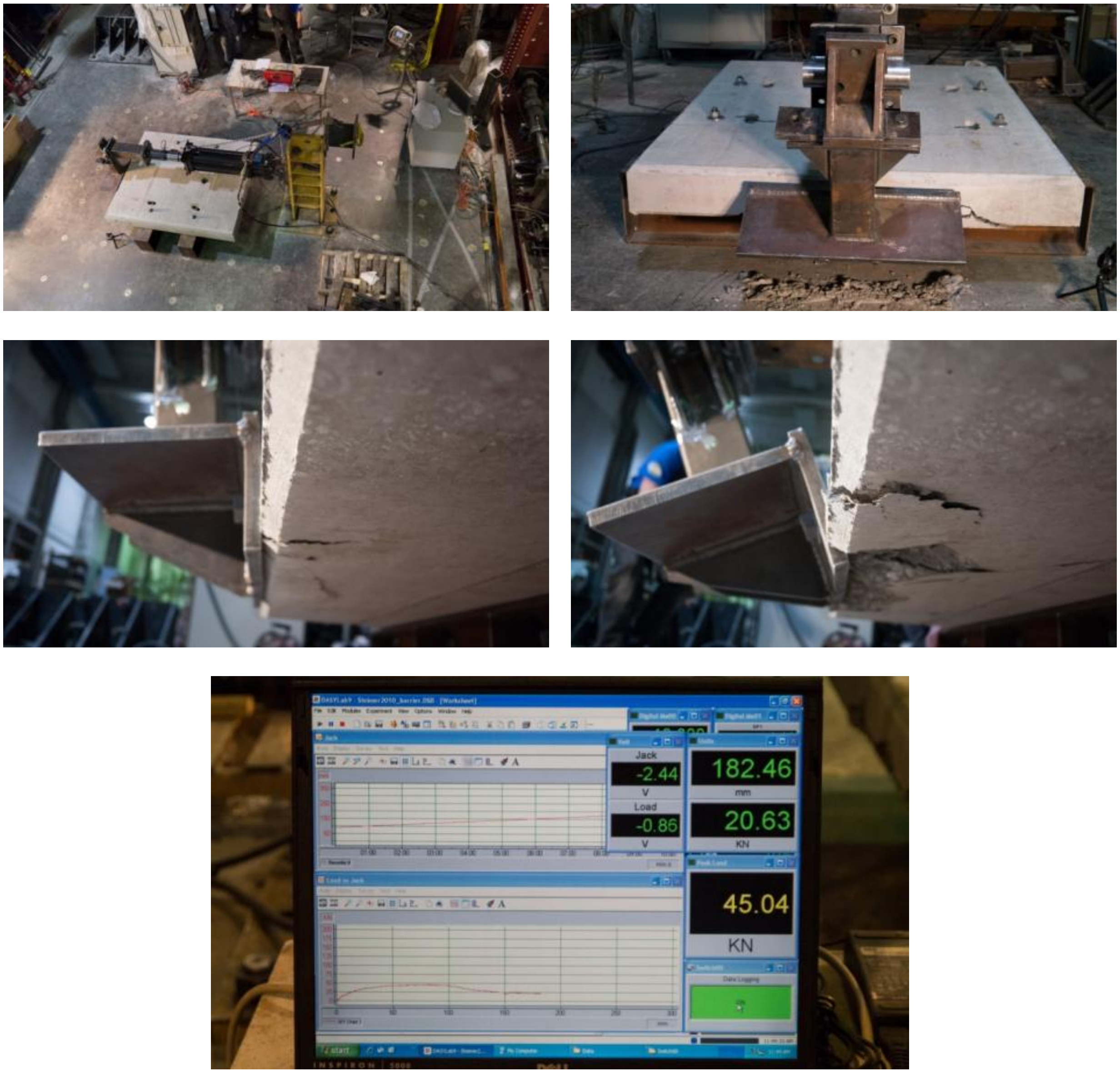


Figure 8: Images from A-1-2

Panel / Bracket Test B-1-1

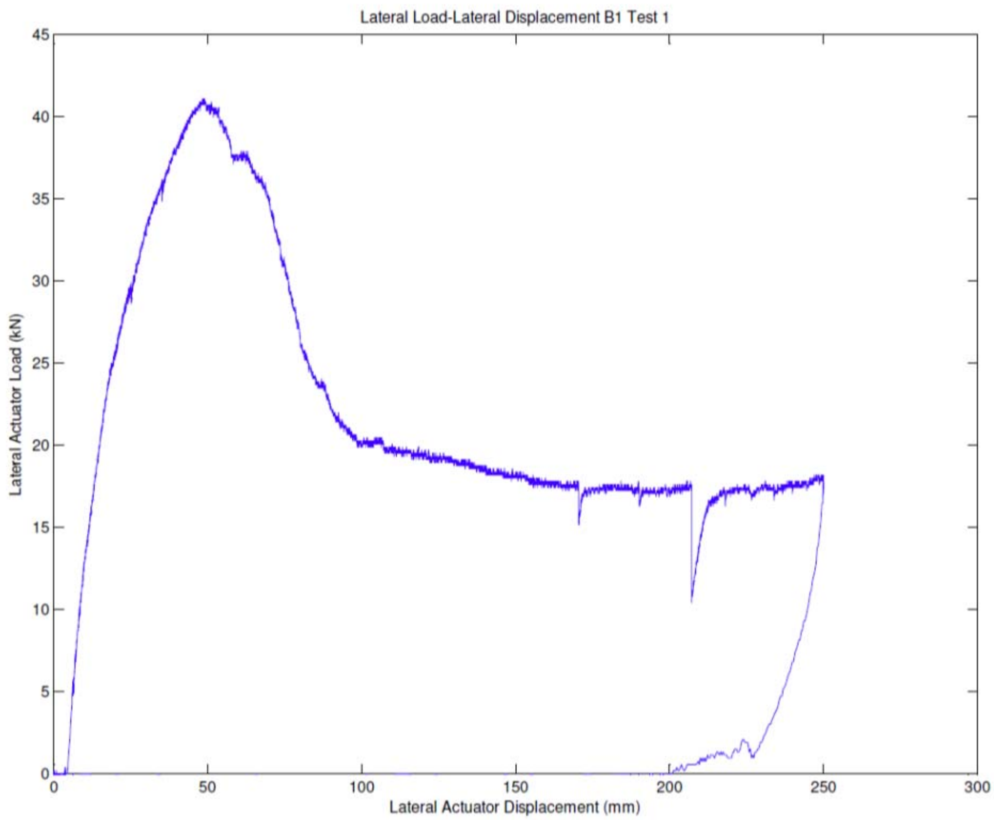


Figure 9: Load/Deflection Curve of Top of Bracket, B-1-2 (Note that initial drop in load corresponded to when the compression concrete on the slab face crushed to the drip groove)

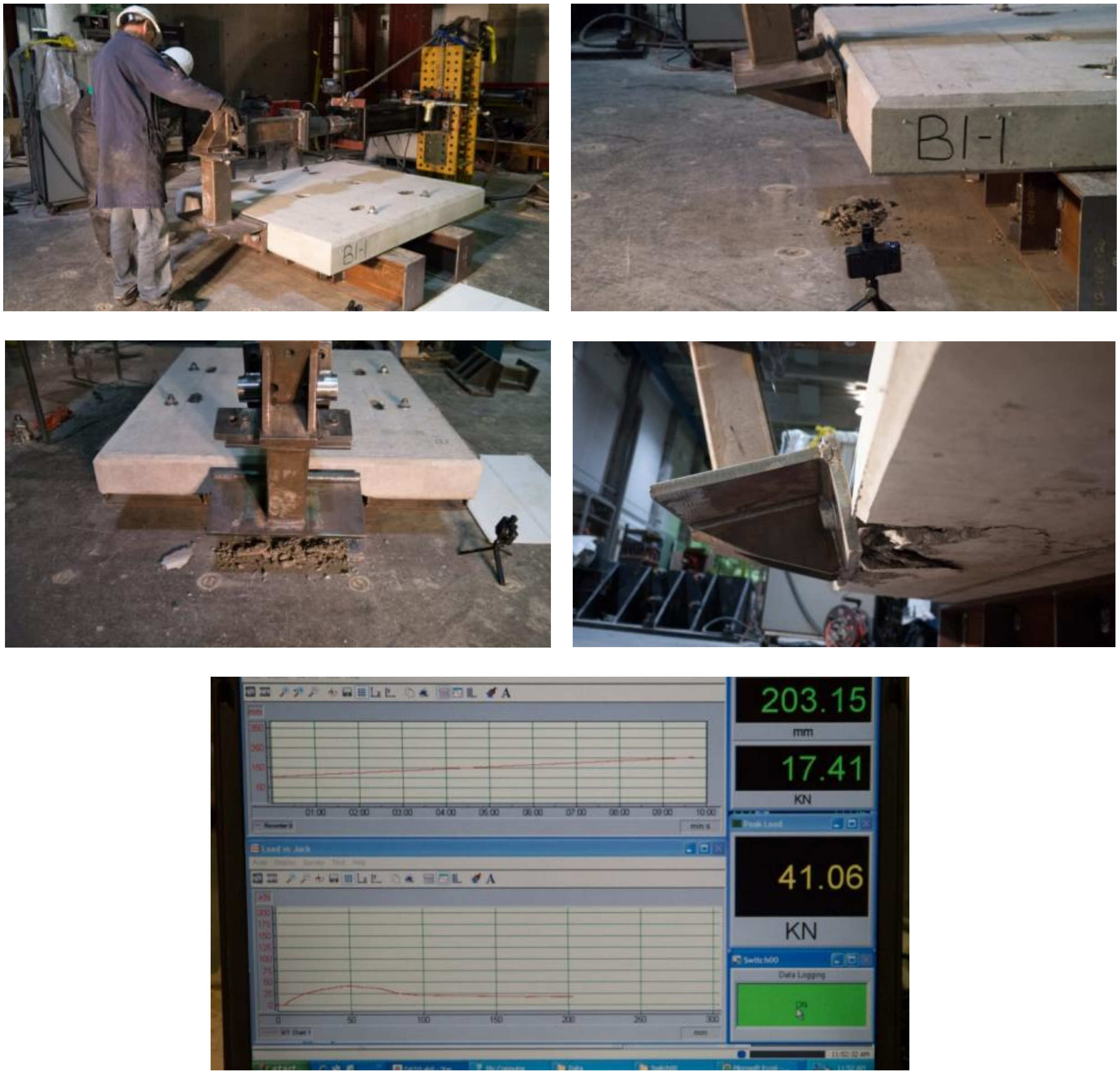


Figure 10: Images from B-1-1

Panel / Bracket Test B-1-2

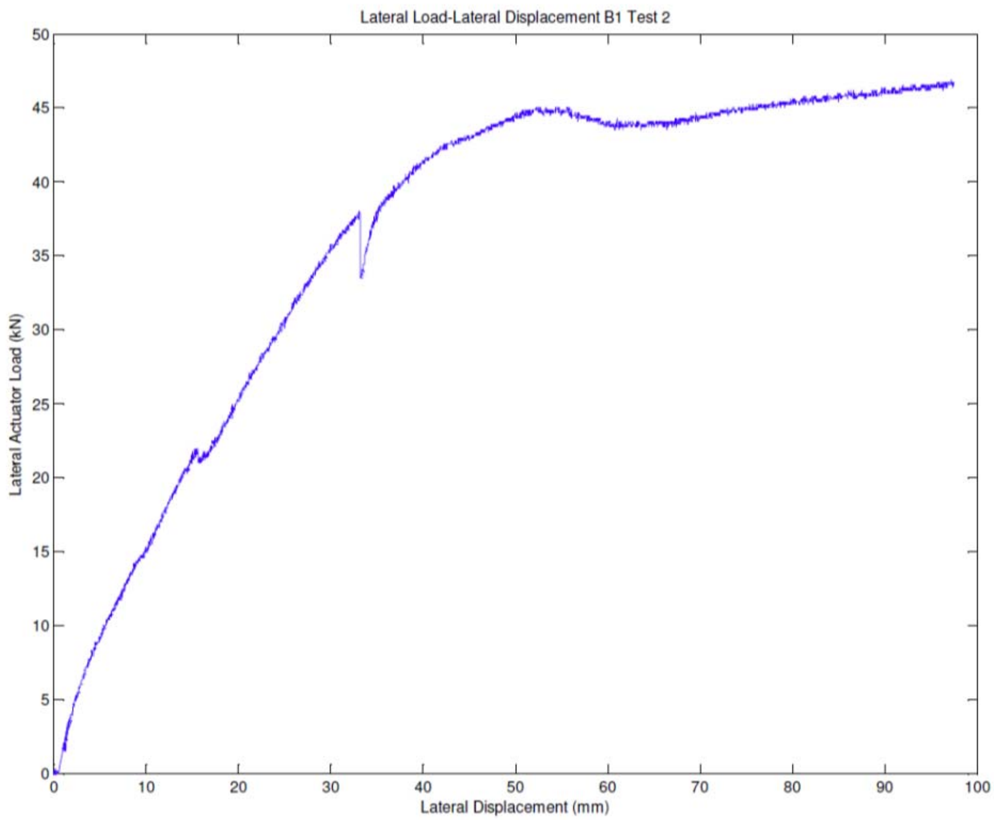


Figure 11: Load/Deflection Curve of Top of Bracket, B-1-2 (Note that drop in stiffness corresponded to when the compression concrete on the slab face crushed to the drip groove, after this point the load was entirely carried by the bending of the bolts. The test was halted as safety became a concern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

Figure 12: Images from B-1-2

Panel / Bracket Test C-1-1

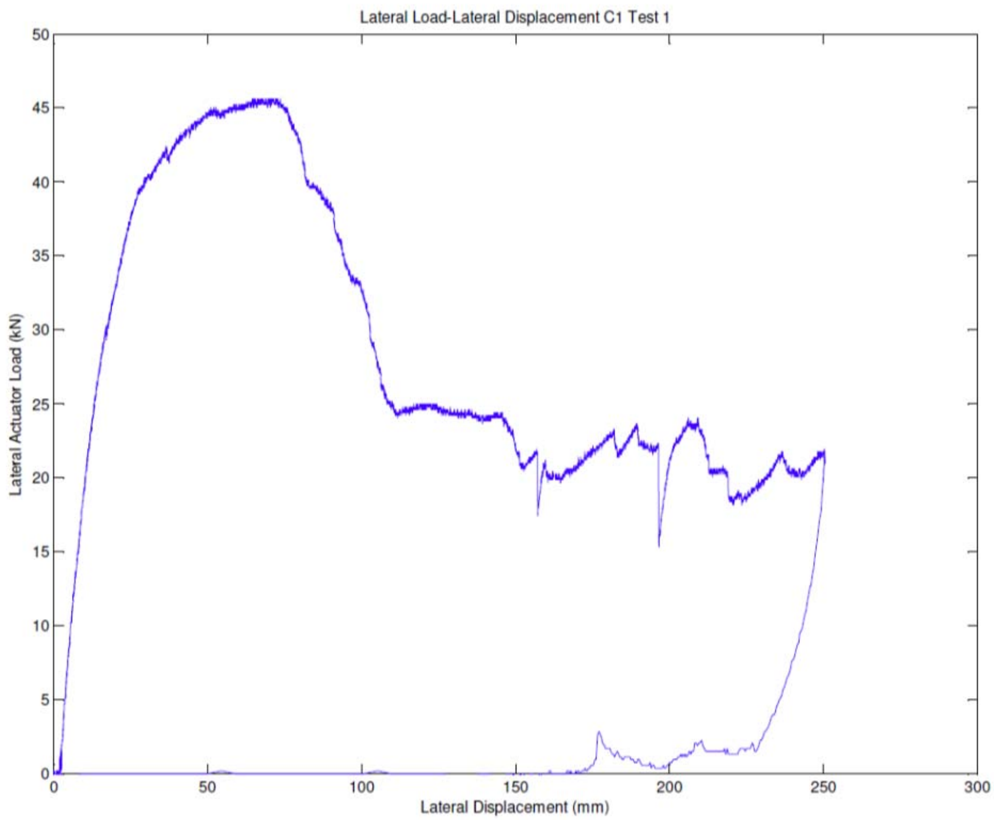


Figure 13: Load/Deflection Curve of Top of Bracket, C-1-1 (Note that initial drop in load corresponded to when the compression concrete on the slab face crushed to the drip groove)

Figure 14: Images from C-1-1

Panel / Bracket Test C-1-2

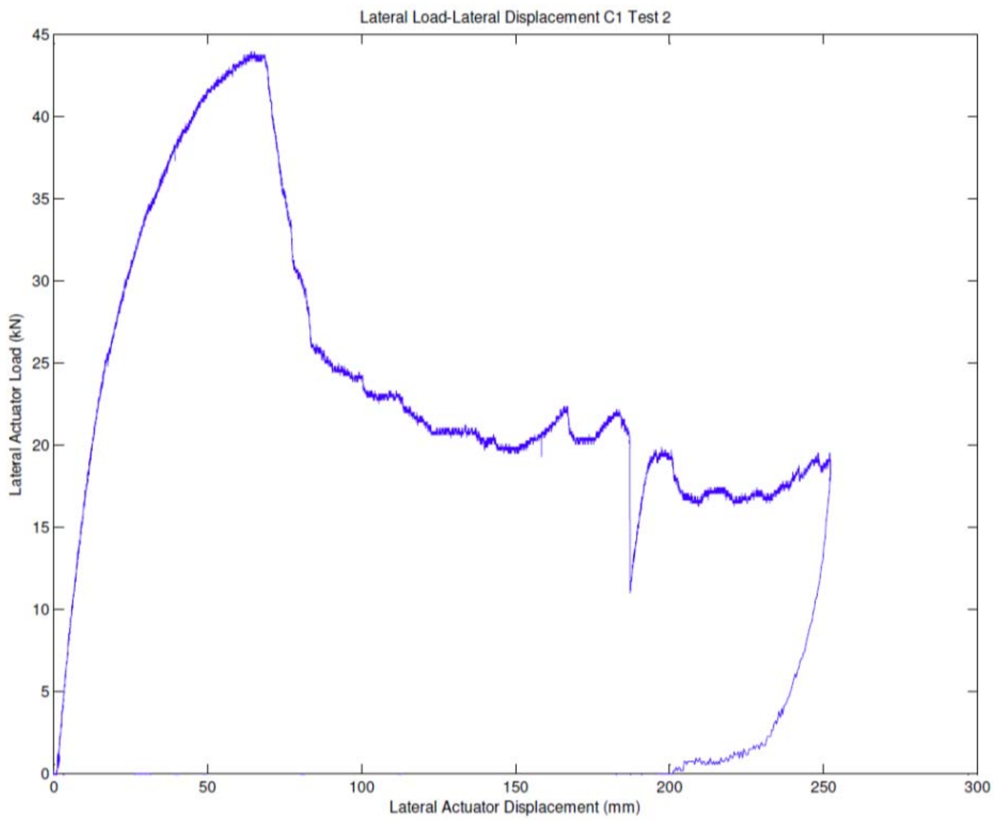


Figure 15: Load/Deflection Curve of Top of Bracket, C-2-1 (Note that initial drop in load corresponded to when the compression concrete on the slab face crushed to the drip groove)


CI-2

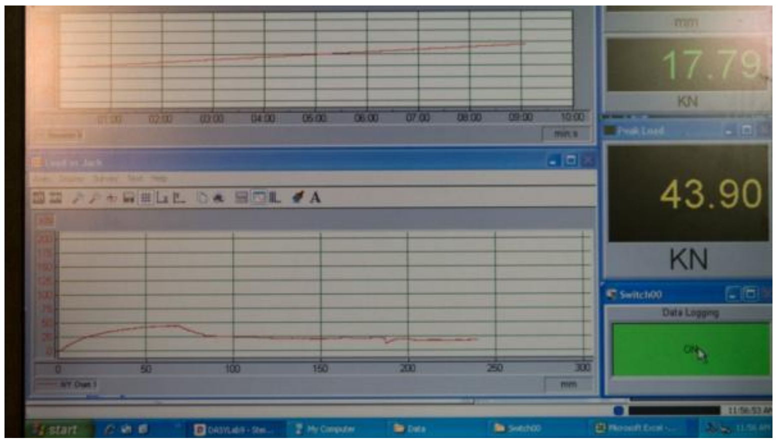


Figure 16: Images from C-2-1

Panel / Bracket Test C-2-1

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

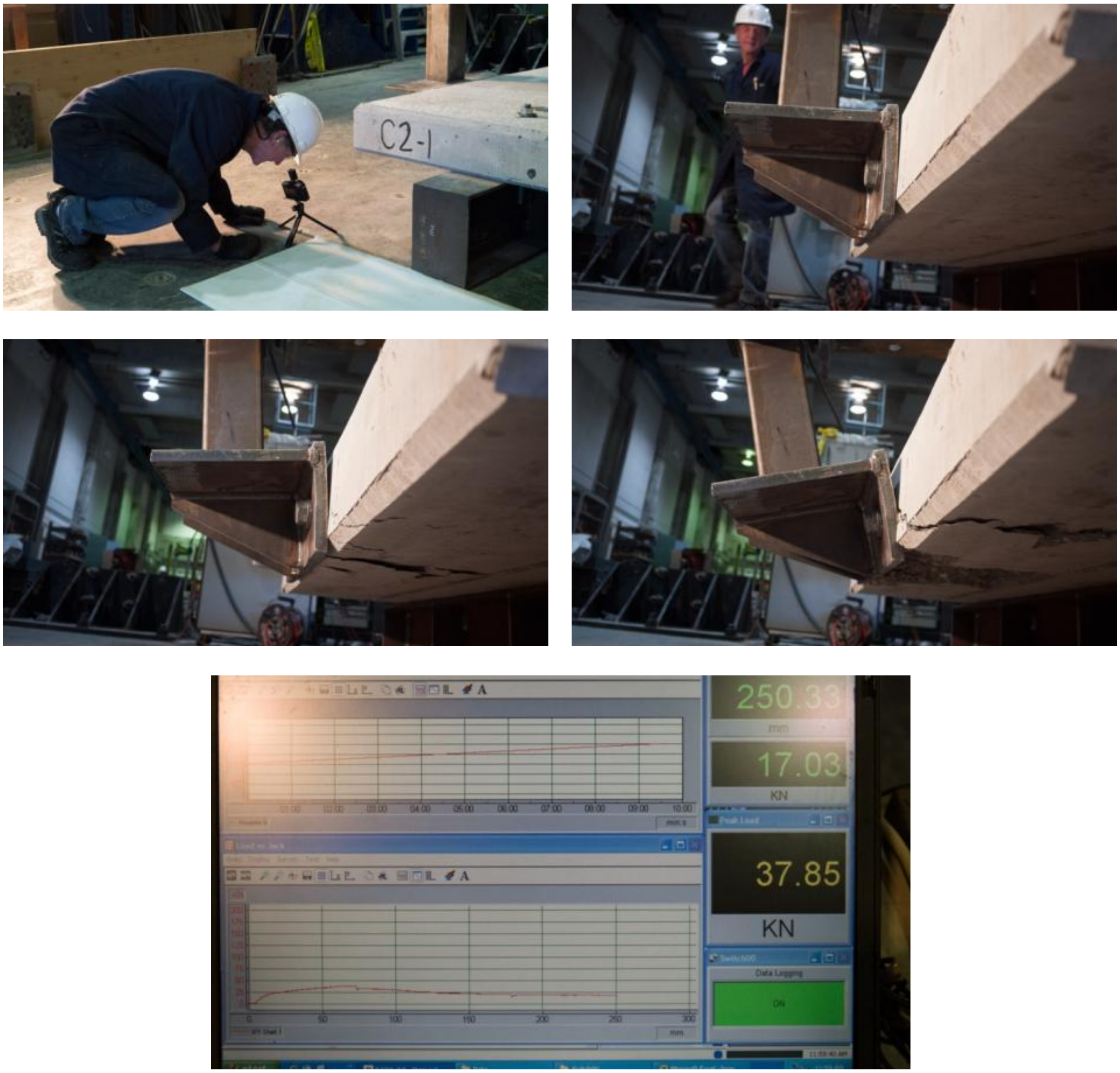
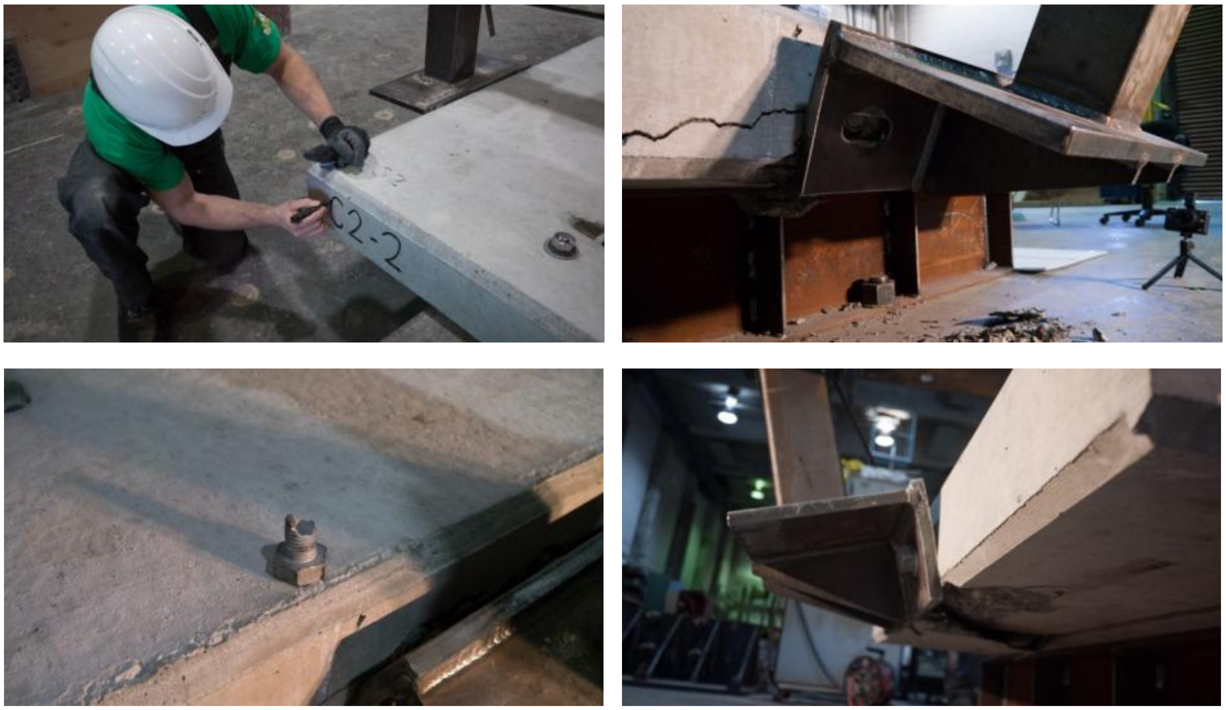


Figure 18: Images from C-2-1

Panel / Bracket Test C-2-2

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



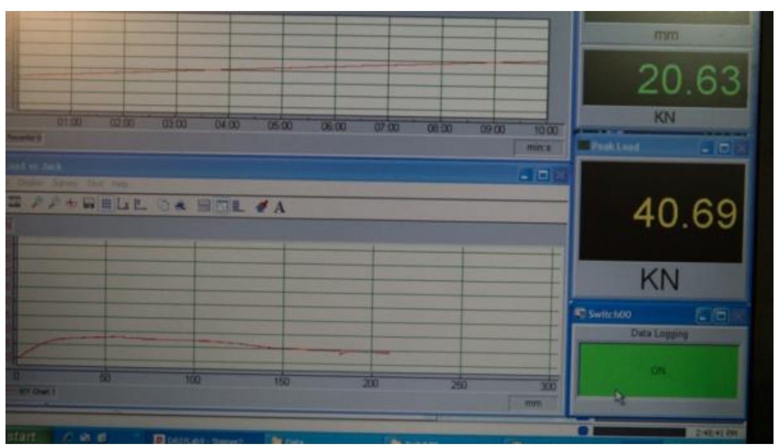


Figure 20: Images from C-2-2

Panel / Bracket Test D-1-1

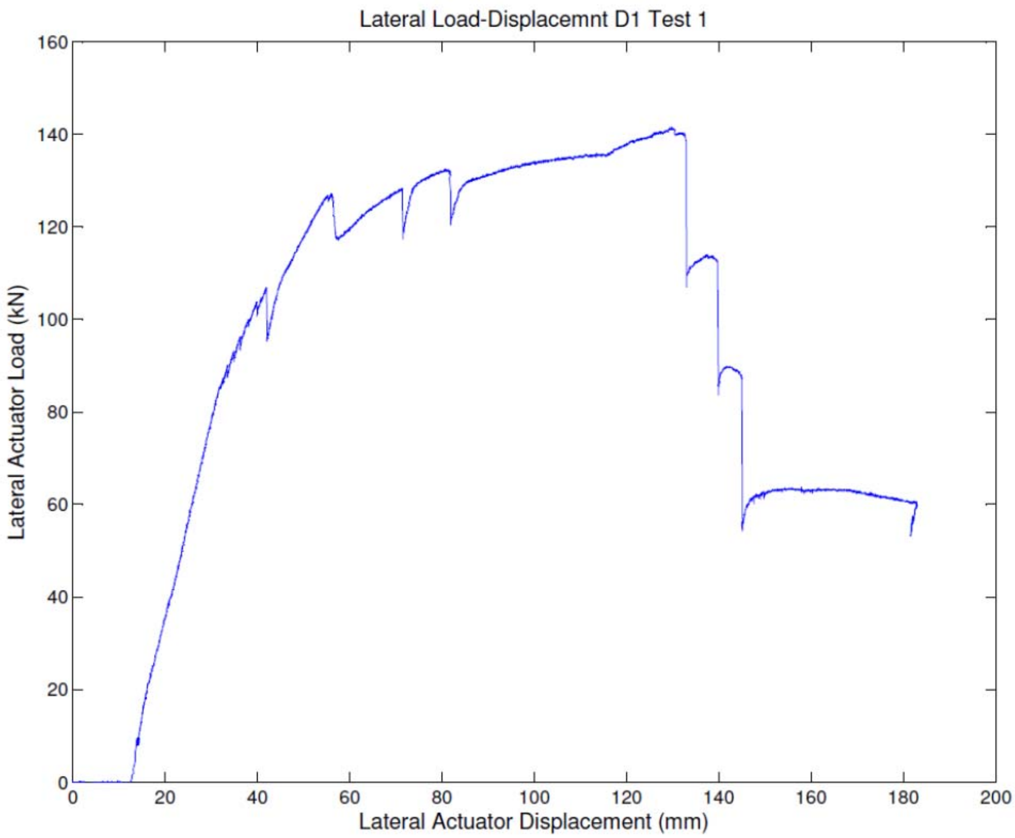


Figure 21: Load/Deflection Curve of Top of Bracket, D-1-1 (Note drops in load correspond to roughly the capacity of one of the four main anchor rods as they fractured in sequence)

Figure 22: Concrete panel attachment detail failing

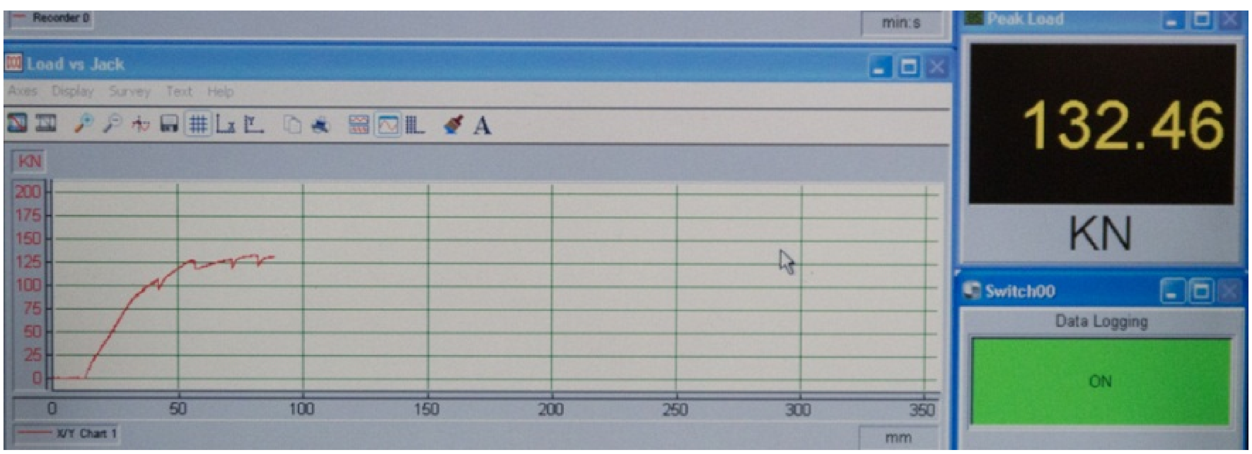


Figure 23: Example of computer screen readout during end phase of test

Panel / Bracket Test D-1-2

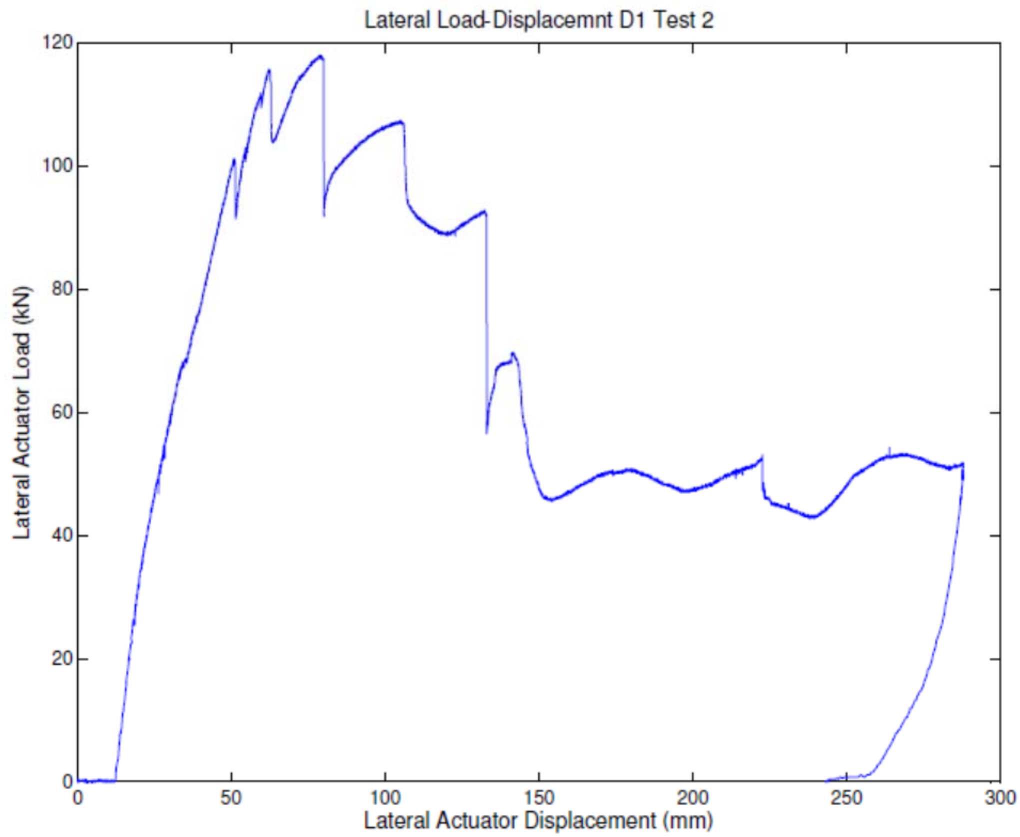


Figure 24: Load/Deflection Curve of Top of Bracket, D-1-2 (Note that the major drops in load correspond to anchor fracture and/or slippage)

Figure 25: Images from D-1-2

Panel / Bracket Test D-2-1

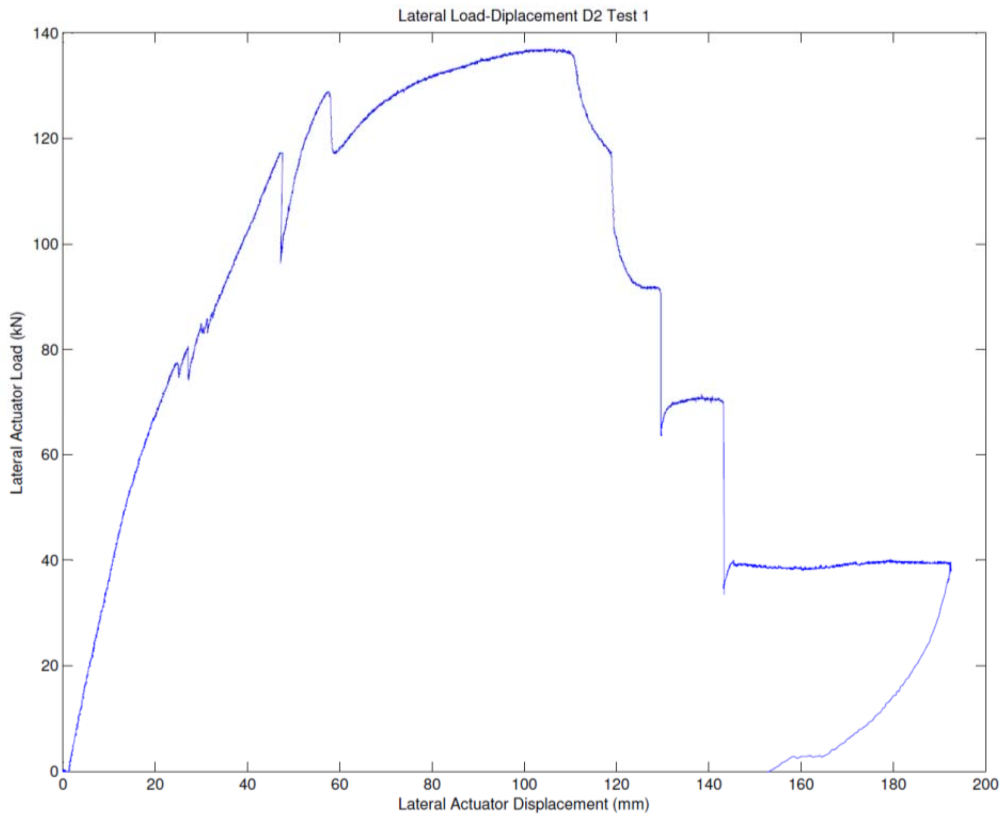


Figure 26: Load/Deflection Curve of Top of Bracket, D-2-1 (Note drops in load correspond to roughly the capacity of one of the four main anchor rods as they fractured in sequence)

Figure 27: Images from D-2-1

Panel / Bracket Test D-2-2

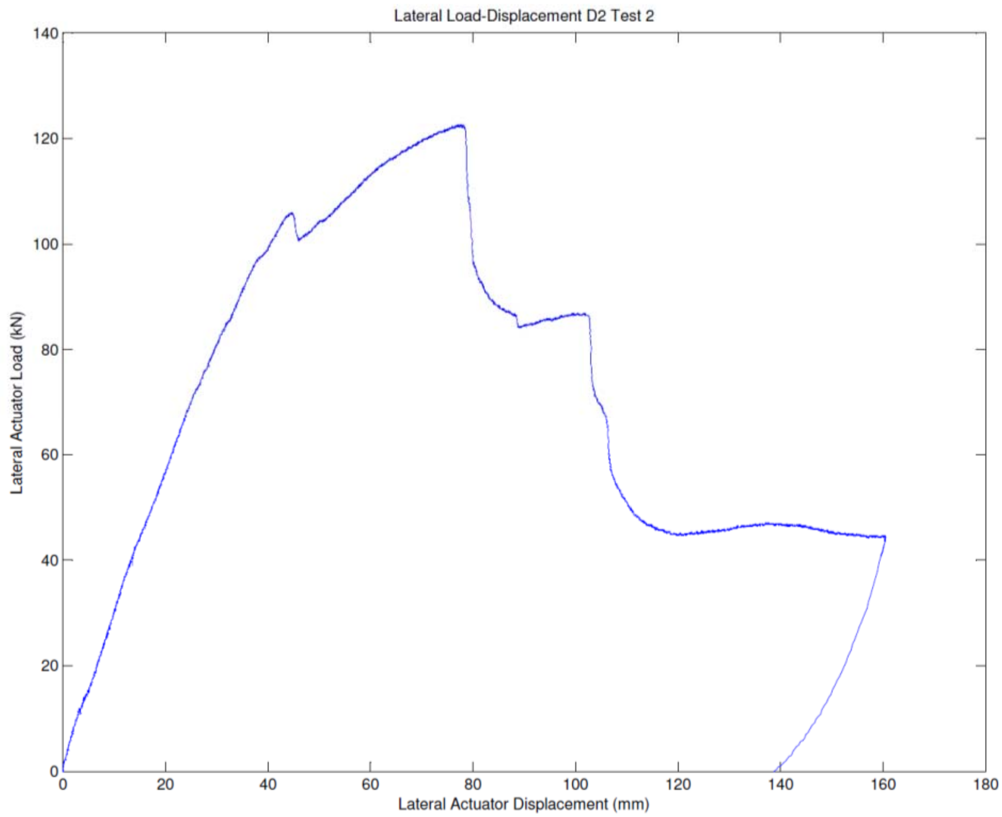


Figure 28: Load/Deflection Curve of Top of Bracket, D-2-2 (Note drops in load correspond to roughly the capacity of one of the four main anchor rods as they fractured in sequence)

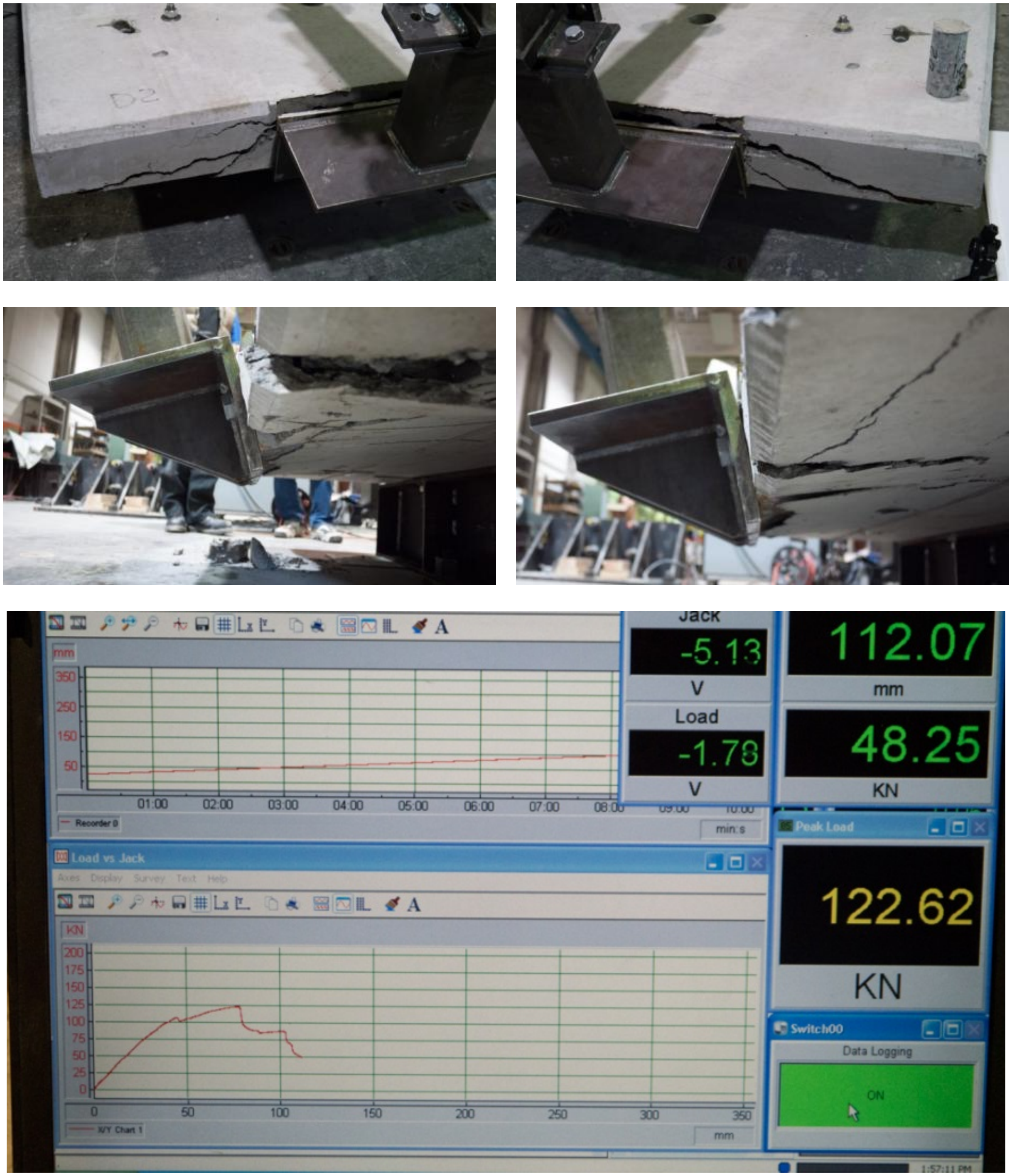


Figure 29: Images from D-2-2

Panel / Bracket Test E-1-1

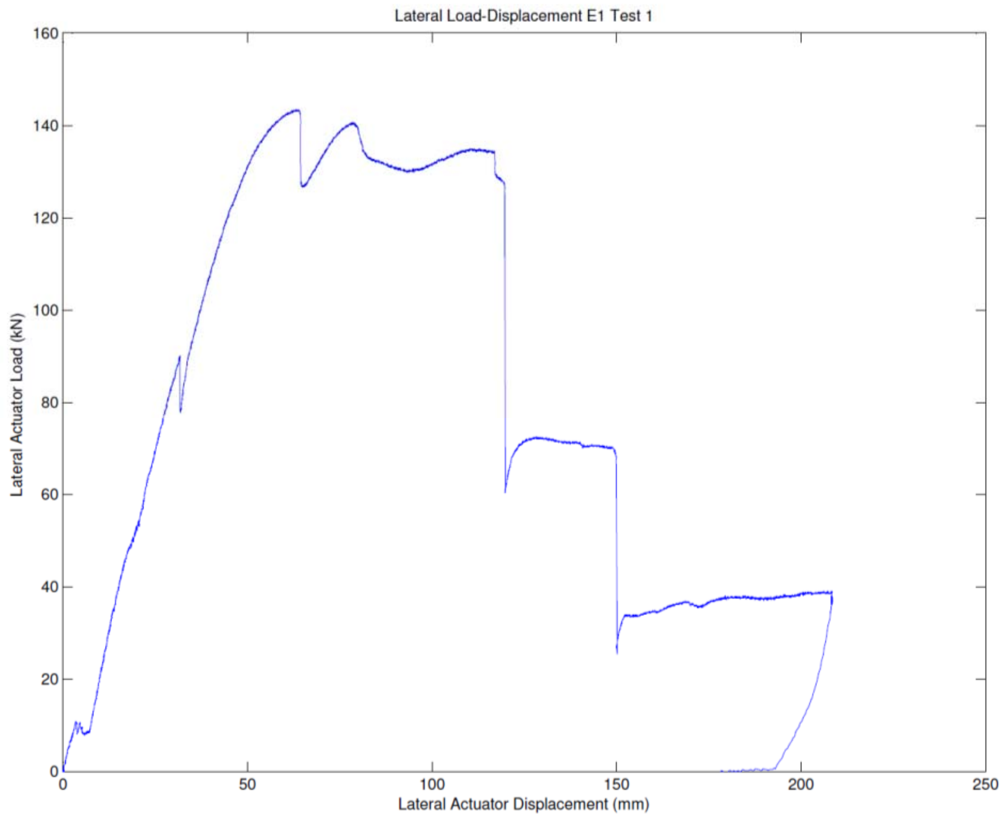


Figure 30: Load/Deflection Curve of Top of Bracket, E-1-1 (Note drops in load correspond to roughly the capacity of one of the four main anchor rods as they fractured in sequence)

Figure 31: Images from E-1-1

Panel / Bracket Test E-1-2

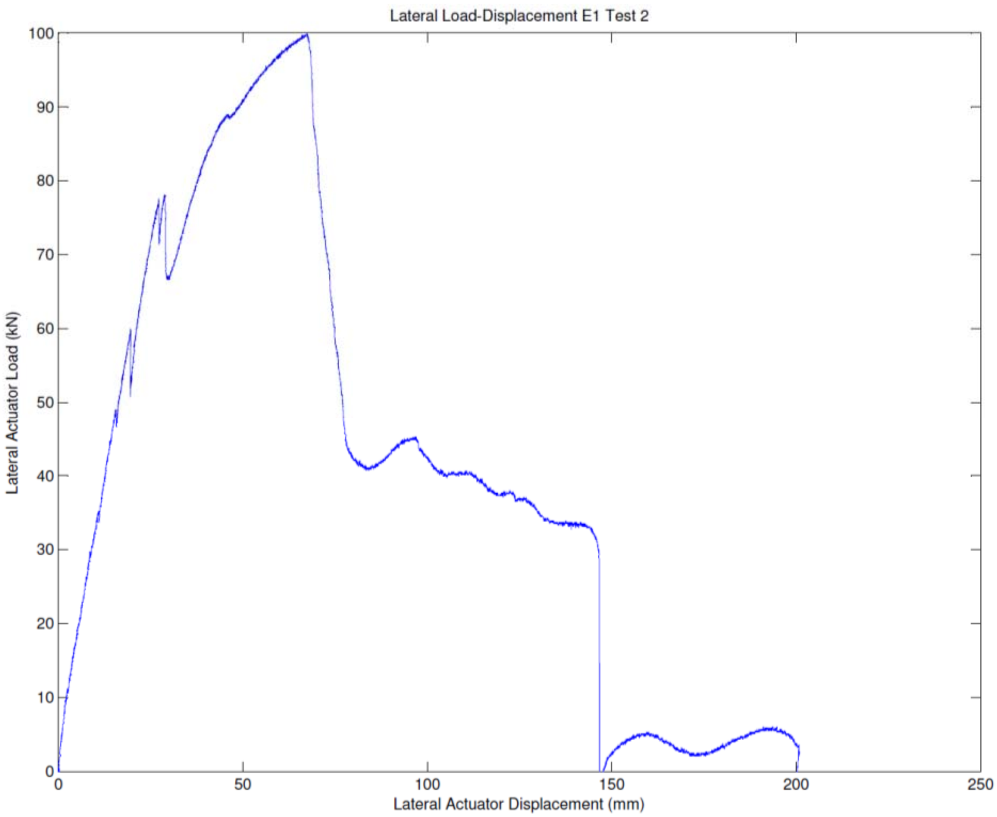


Figure 32: Load/Deflection Curve of Top of Bracket, E-1-2 (Note initial slipping of support apparatus caused initial drops in load during the initial portion of the loading curve. There may have been an initial preload of roughly 50 kN which resulted in the low lateral load capacity of this specimen)

Figure 33: Images from E-1-2

Panel / Bracket Test E-2-1

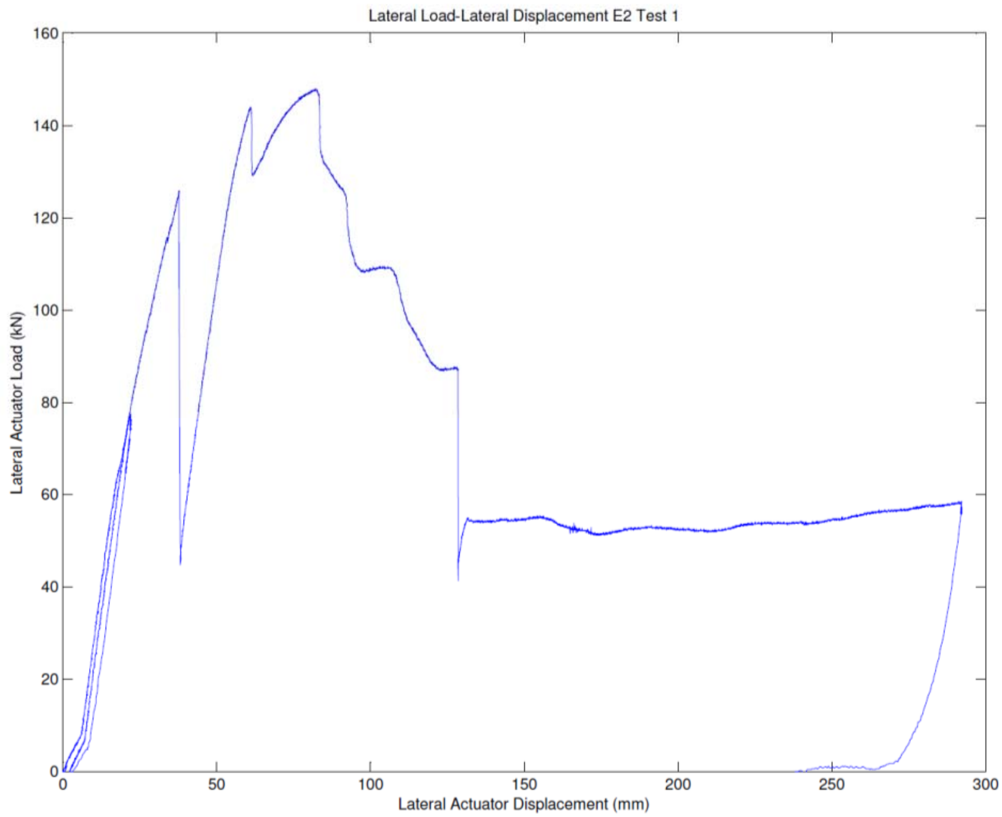


Figure 34: Load/Deflection Curve of Top of Bracket, E-2-1 (Note drop in load of 83 kN at lateral displacement of 39 mm due to panel support apparatus shifting under load)

Figure 35: Images from E-2-1

Panel / Bracket Test E-2-2

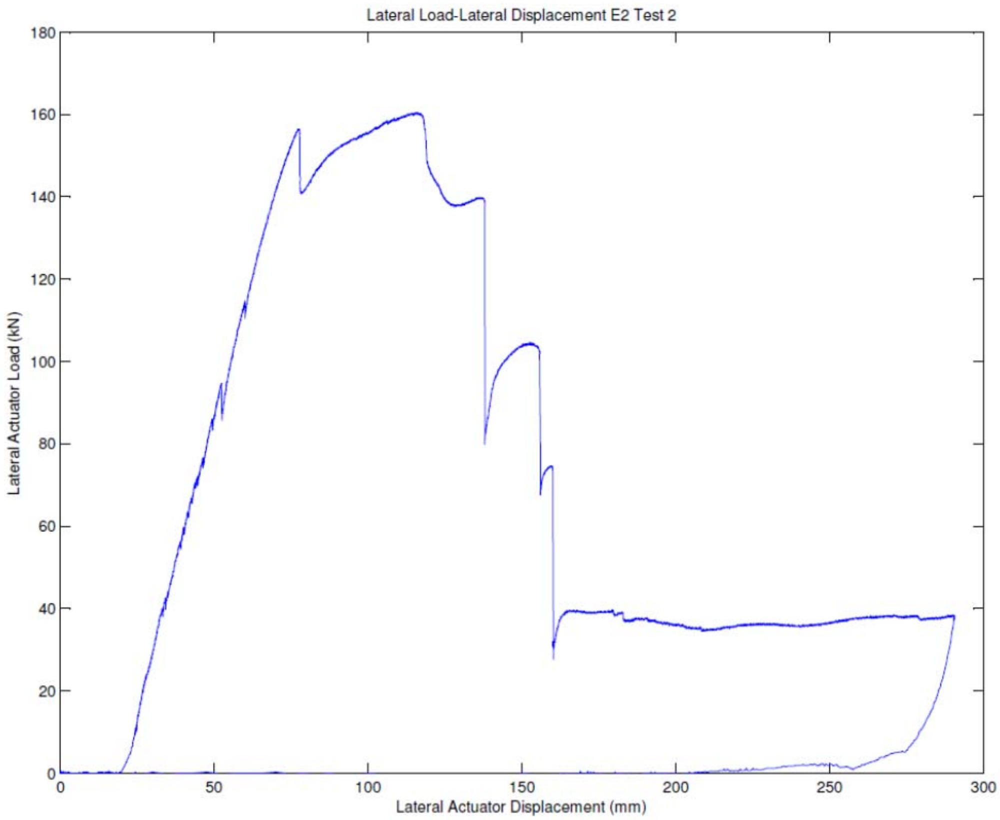


Figure 36: Load/Deflection Curve of Top of Bracket, E-2-2 (Note that the major drops in load correspond to the capacity of an anchor bar as it fractured)

Figure 37: Images from E-2-2

Panel / Bracket Test G-1-1

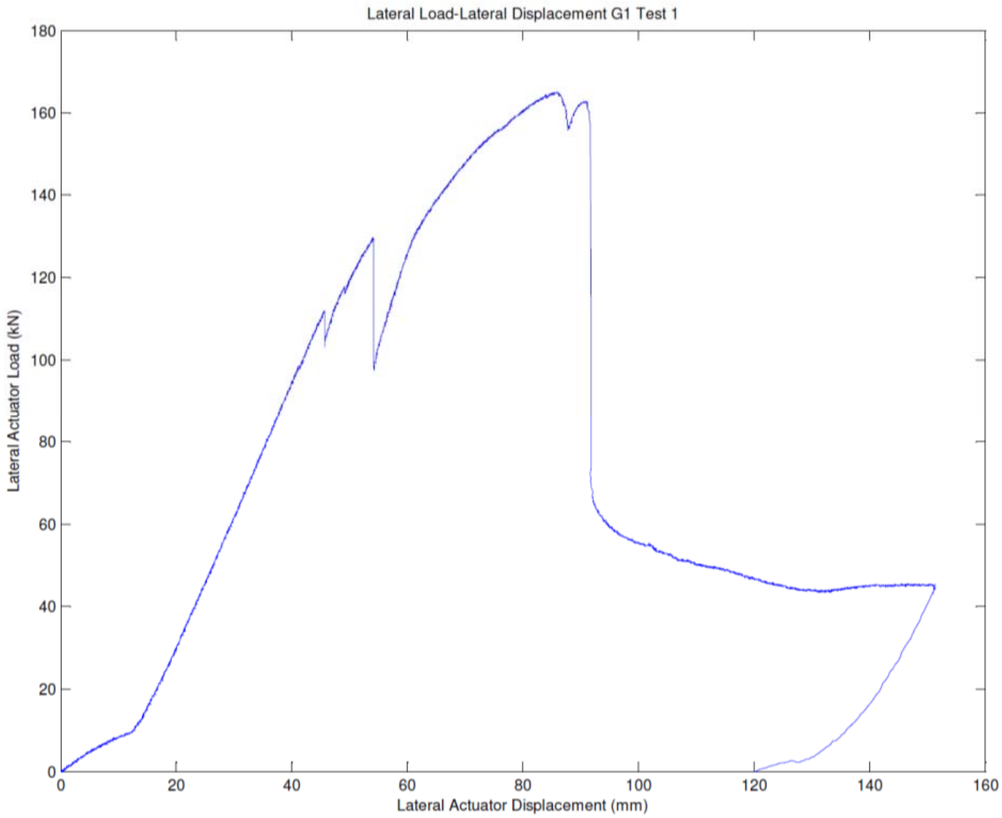


Figure 38: Load/Deflection Curve of Top of Bracket, G-1-1 (Note that the primary failure mechanism was characterized by the concrete cover above the Nelson studs breaking away as the rigid studs reached a critical curvature demand)

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

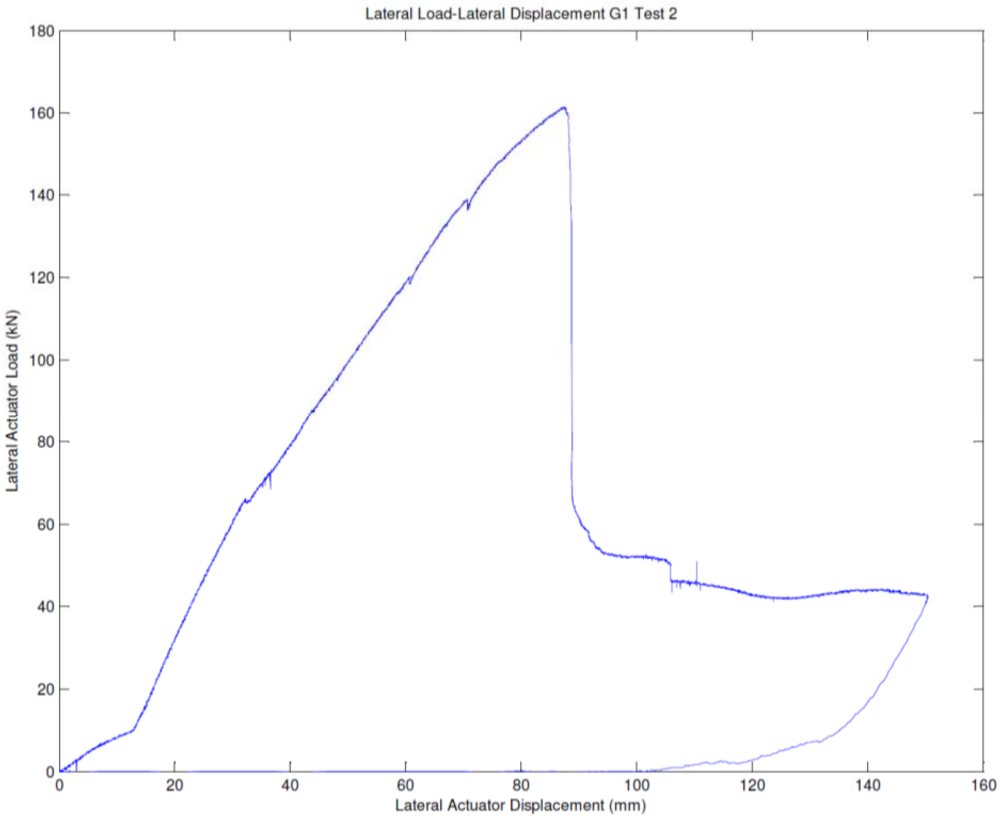


Figure 42: Load/Deflection Curve of Top of Bracket, G-1-2 (Note that the primary failure mechanism was characterized by the concrete cover above the Nelson studs breaking away as the rigid studs reached a critical curvature demand)

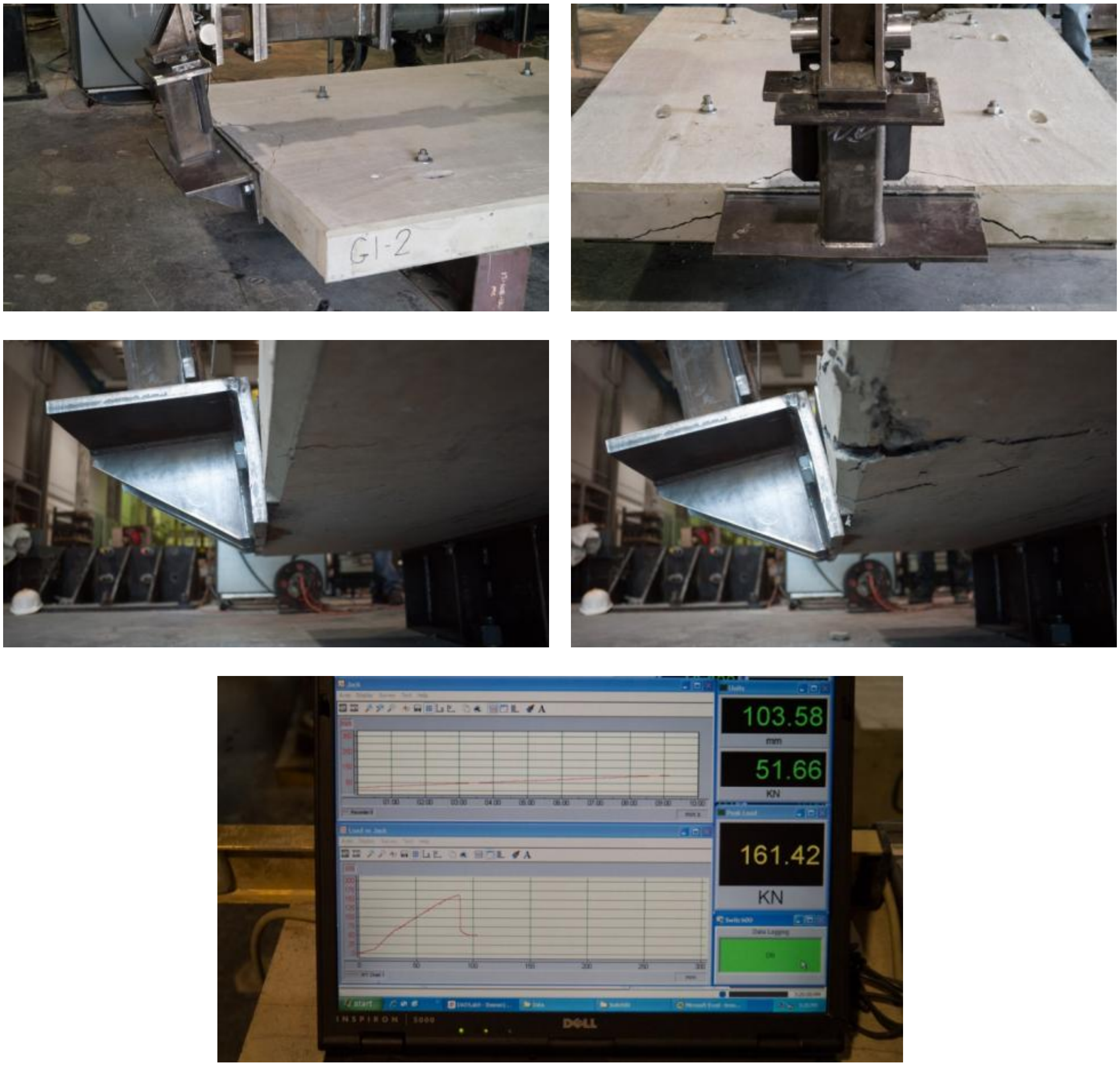


Figure 43: Images from G-1-2

Panel / Bracket Test H-1-1

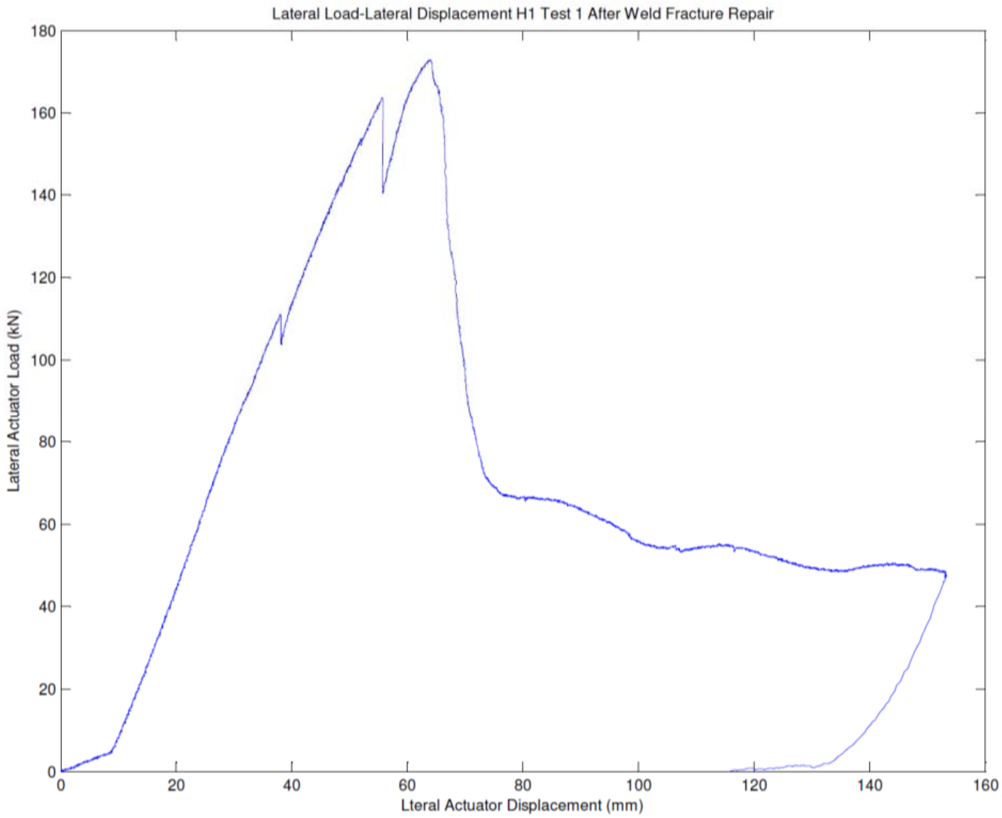


Figure 44: Load/Deflection Curve of Top of Bracket, H-1-1 (Note that the primary failure mechanism was characterized by the concrete cover above the Nelson studs breaking away as the rigid studs reached a critical curvature demand)

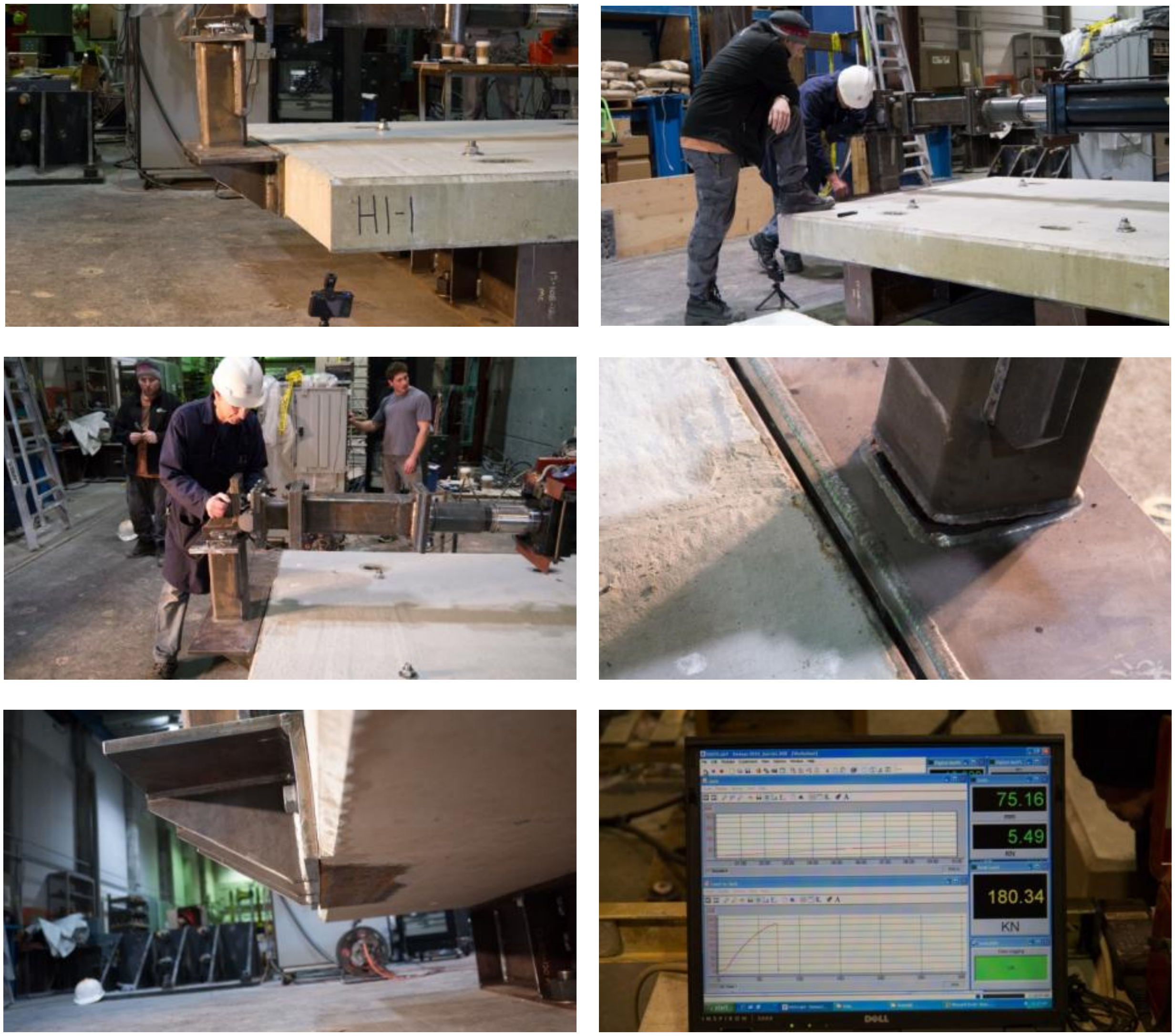


Figure 45: Images from H-1-1 to weld failure at post foot

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

Panel / Bracket Test H-1-2

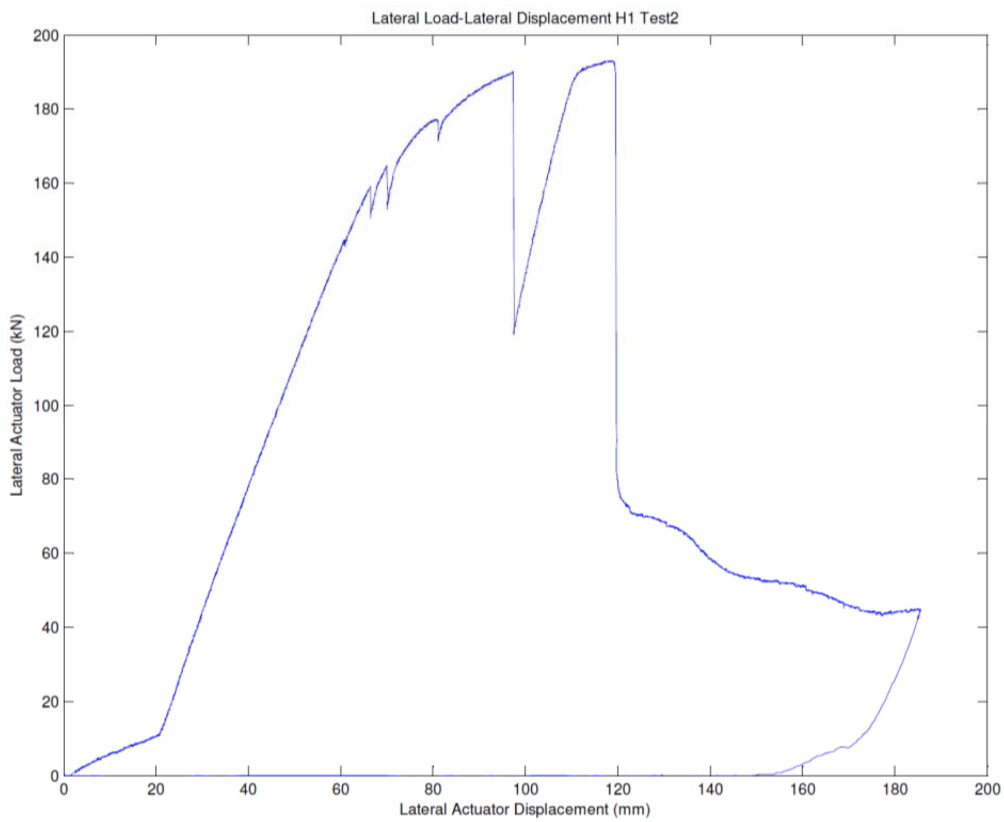


Figure 47: Load/Deflection Curve of Top of Bracket, H-1-2 (Note the major drop in load at 97 mm lateral displacement was caused by the slippage of the support apparatus. Also note that the primary failure mechanism was characterized by the concrete cover above the Nelson studs breaking away as the rigid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


H1-2


H1-2


H1-2


H1-2

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.

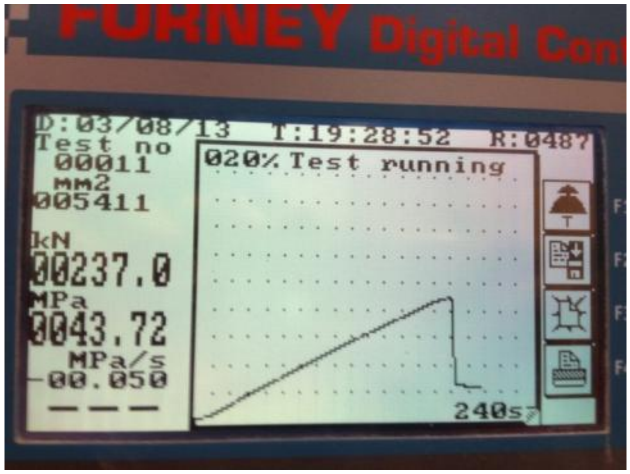
Testing of the specimens should be done according to CSA A23.1/A23.2 *Concrete materials and methods of concrete construction/Test methods and standard practice for concrete*. Grinding (machining) of specimen end face to produce uniform bearing as consistent with the CSA standard is acceptable (according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



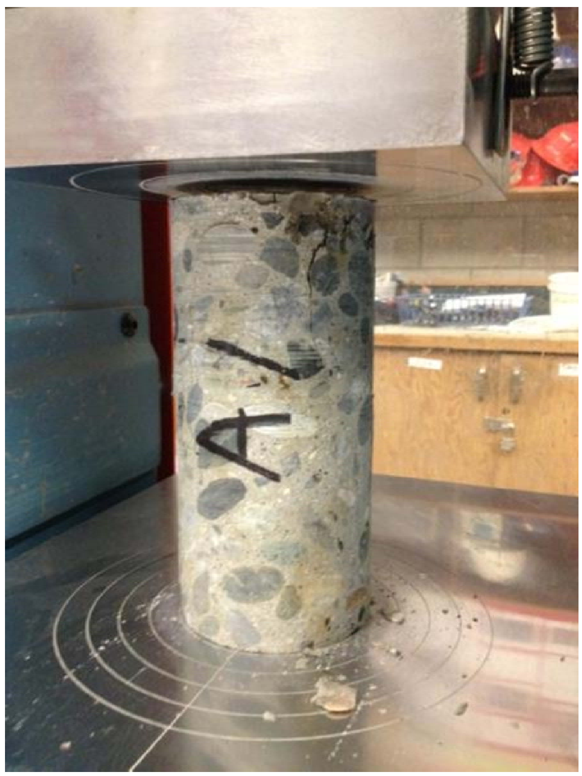


Figure 50: Core Compression Specimen A-1, precision machined compression surfaces

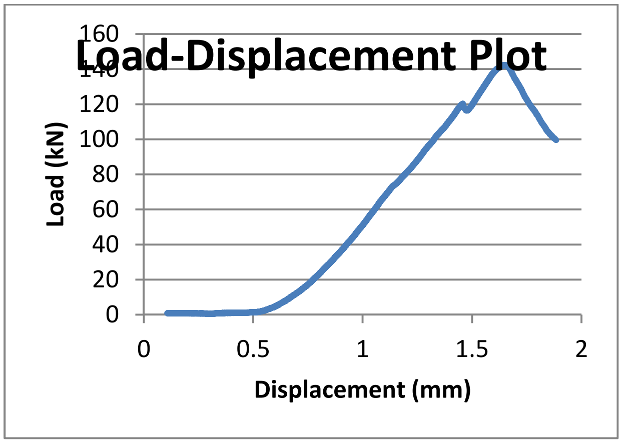


Figure 51: Core Compression Test D-1

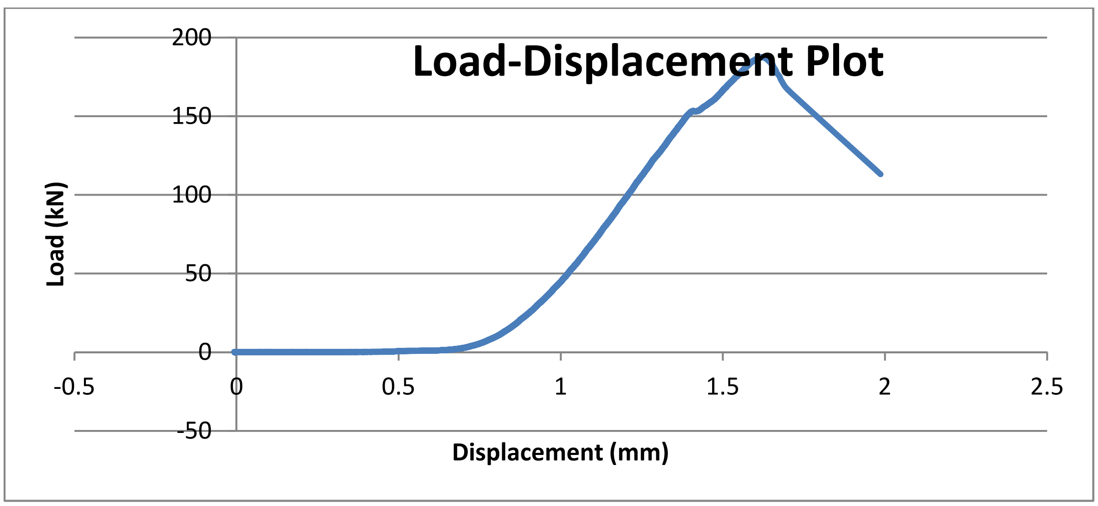


Figure 52: Core Compression Test D-2

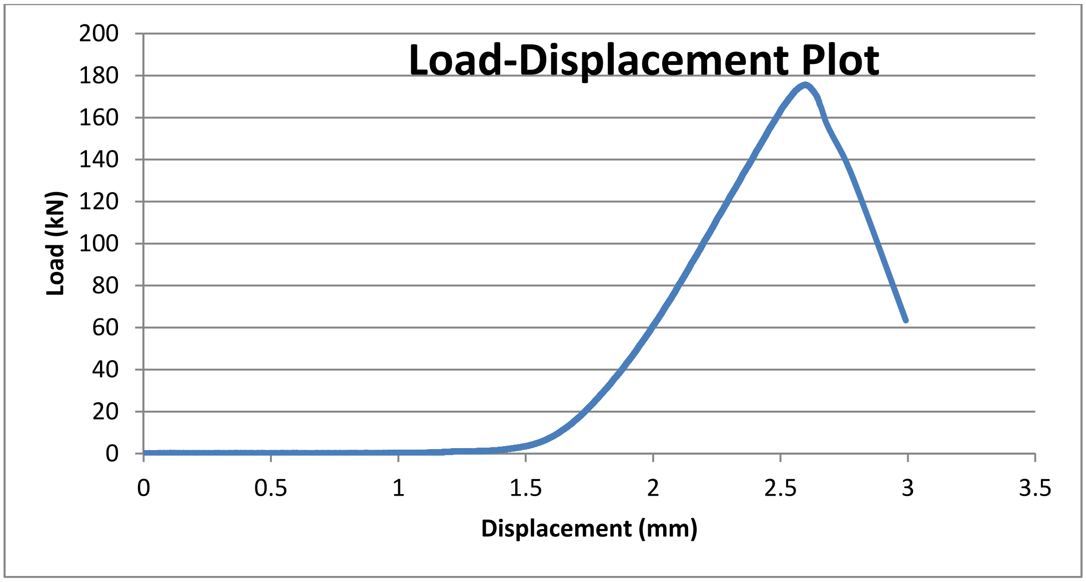


Figure 53: Core Compression Test E-1

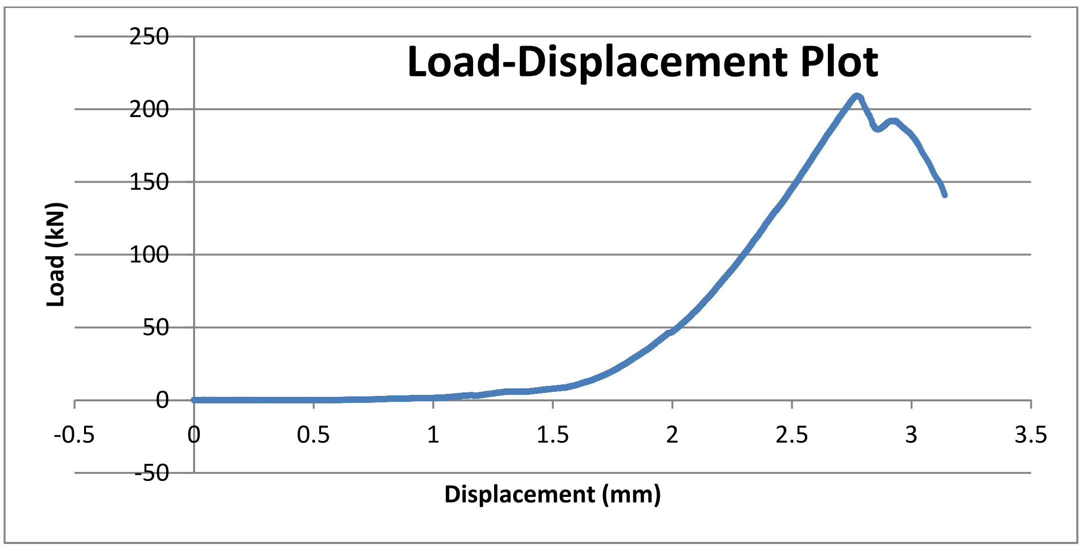


Figure 54: Core Compression Test 675D

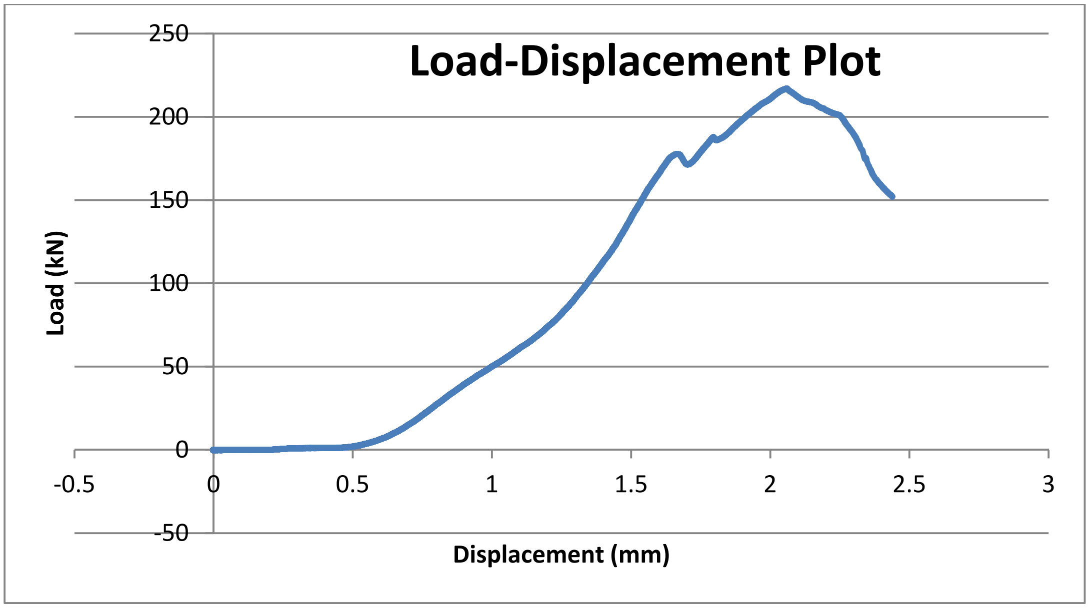


Figure 55: Core Compression Test 676D

Rebar System Photos

Consulting and General Services Contract

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

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. Stiemer, 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 Guardrail Component	Coating
	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 *(Means: Completed materials fabrication, ministry In-plant Inspection, and ministry acceptance of all materials at the fabrication facility)*	January 25, 2013
1.2	Billing Submission *(Latest date billing to be received by the ministry)*	February 15, 2013
1.3	Estimated Delivery Date *(Actual date to be specified by the ministry, with a minimum one week notice prior to required product delivery date/time.)*	Between January 25 and February 15, 2013
1.4	Maximum Storage Period *(Possible storage by fabricator prior to delivery.)*	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

McElhanney

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**

1. 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.

2. 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. *(Name of Fabricator)* during the fabrication of the following components.

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

3. 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

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

CERTIFIED LABORATORY FOR TESTING CONCRETE

CONCRETE TEST REPORT

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

ATTN: MR. NICK EUSTACE

PROJECT NO. V2536
CLIENT PIONEER PRECAST PRODUCTS LTD. a
C.C. REMPEL BROS CONCRETE

PROJECT CONCRETE TESTING 2012 Q.C
CONCRETE TESTING & INSPECTION
8190 AITKEN ROAD
CHILLIWACK

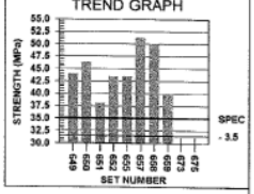
SET NO. 675 NO. OF SPECIMENS 5 DATE RECEIVED 2013.Jan.25 DATE CAST 2013.Jan.23

SPCM NO.	SPECIMEN TYPE	CURE CONDN	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/m³
CEMENT TYPE 10
POZZOLAN CONTENT kg/m³
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/m³
HARDENED DENSITY kg/m³
CAST TIME 08:00
CAST BY VTS AS
CURING CONDITIONS WOODEN

TREND GRAPH



MOULD TYPE PLASTIC

INITIAL CURING TEMP:MAXIMUM 22.0 °C MINIMUM 15.0 °C

SUPPLIER REMPEL BROS CONCRETE
MIX NO. PP35C1
TRUCK NO. 689 TICKET NO. 375885
LOAD VOL. 10 m³ CUM. VOL. 10 m³
WATER ADDED l AUTH. BY
Page 1 of 1 2013.Jan.30

LOCATION
1) 20102541-15-3, 6 PANELS (A,B,C,D,E,F)
2 BALLASTS (G,H) N4-001
2) 20102698-01-3, 8 PANELS Test Panels

COMMENTS
(A,B,C,D,E,F,G,H)
TEST TAKEN @ PANEL A

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.

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

CERTIFIED LABORATORY FOR TESTING CONCRETE

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 CONDN	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			

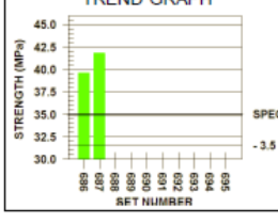
SPECIFIED STRENGTH 35 MPa @ 28 DAYS

CEMENT CONTENT kg/m³
CEMENT TYPE 10
POZZOLAN CONTENT kg/m³
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/m³
HARDENED DENSITY kg/m³
CAST TIME 10:15
CAST BY VTS GM
CURING CONDITIONS WOODEN

TREND GRAPH



MOULD TYPE PLASTIC

INITIAL CURING TEMP:MAXIMUM 22.0 °C MINIMUM 15.0 °C

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.

SUPPLIER REMPEL BROS CONCRETE

MIX NO. PP35C1

TRUCK NO. 1680 TICKET NO. 382181

LOAD VOL. 1.7 m³ CUM. VOL. 1.7 m³

WATER ADDED l AUTH. BY

Page 1 of 1 2013.Mar.21

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.

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

Associated Engineering (B.C.) Ltd.
Suite 300 - 4940 Canada Way
Burnaby, B.C., Canada, V5G 4M5

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.

P:\Proposals\2012\Bur_P_2012.759MOFL\2_Development\Body\Ltr_Mofl_Prp_Testing_Cl2_Cl3_Barr_20120905_Jd.Doc

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

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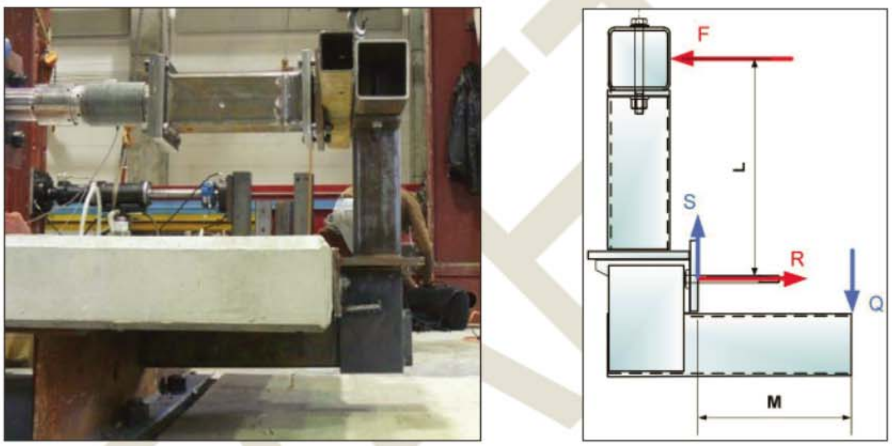
EC-030 Series.

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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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

P:\20102698\00_Eval_Br_Barriers\Engineering\05.00_Design\Development of CL-3 Barrier\DRAFT_mem_mflnro_dev_cl3_barrier_20120315_jh.doc

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March 15, 2012
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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 300-400 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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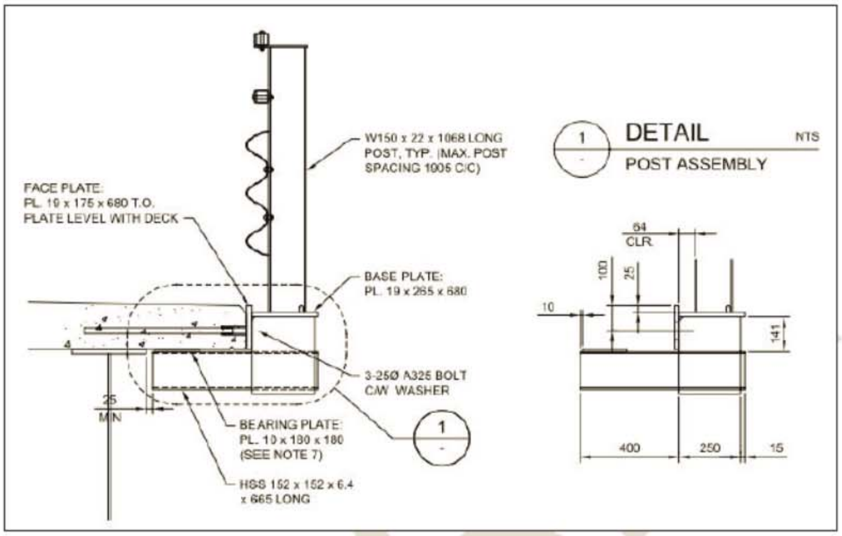
Memo To: Brian Chow, M.A.Sc., P.Eng.
March 15, 2012
- 5 -



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.

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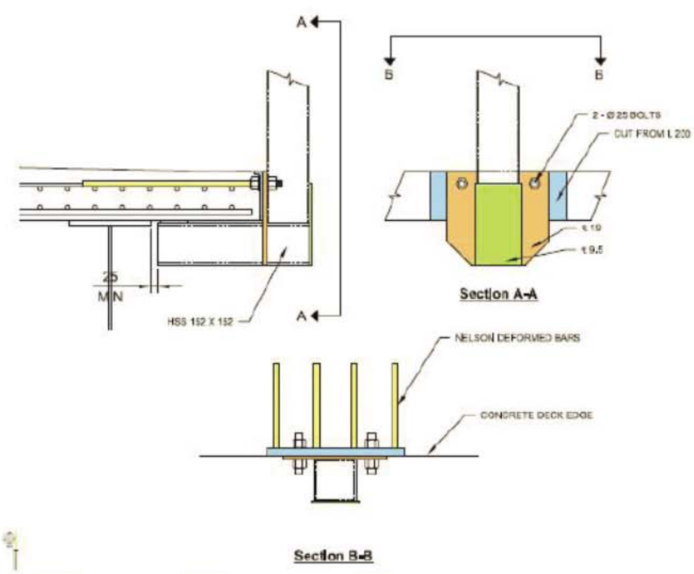
Figure 4

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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Memo To: Brian Chow, M.A.Sc., P.Eng.
March 15, 2012
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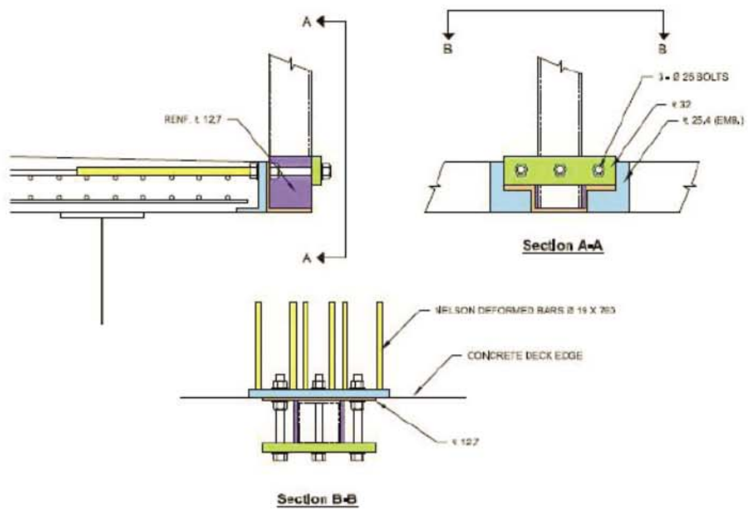
Figure 5
Proposed Alternative Connection Detail



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.

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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:
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

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:
Julien Henley

Associated Engineering (B.C.) Ltd.
300 – 4940 Canada Way
Burnaby, BC V5
G 4M5

Associated Engineering

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Acknowledgement Copy
Please sign and return to Associated Engineering

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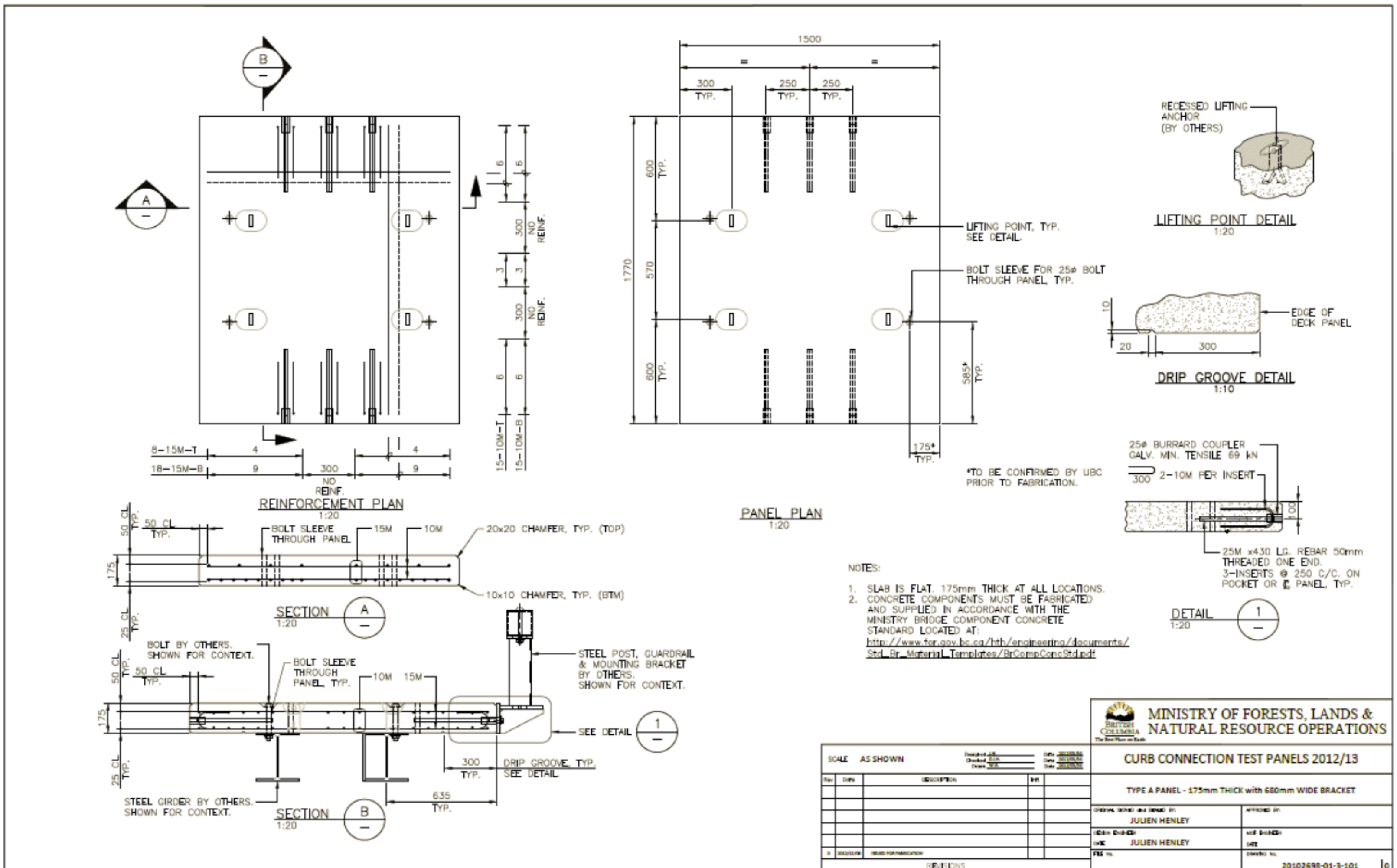
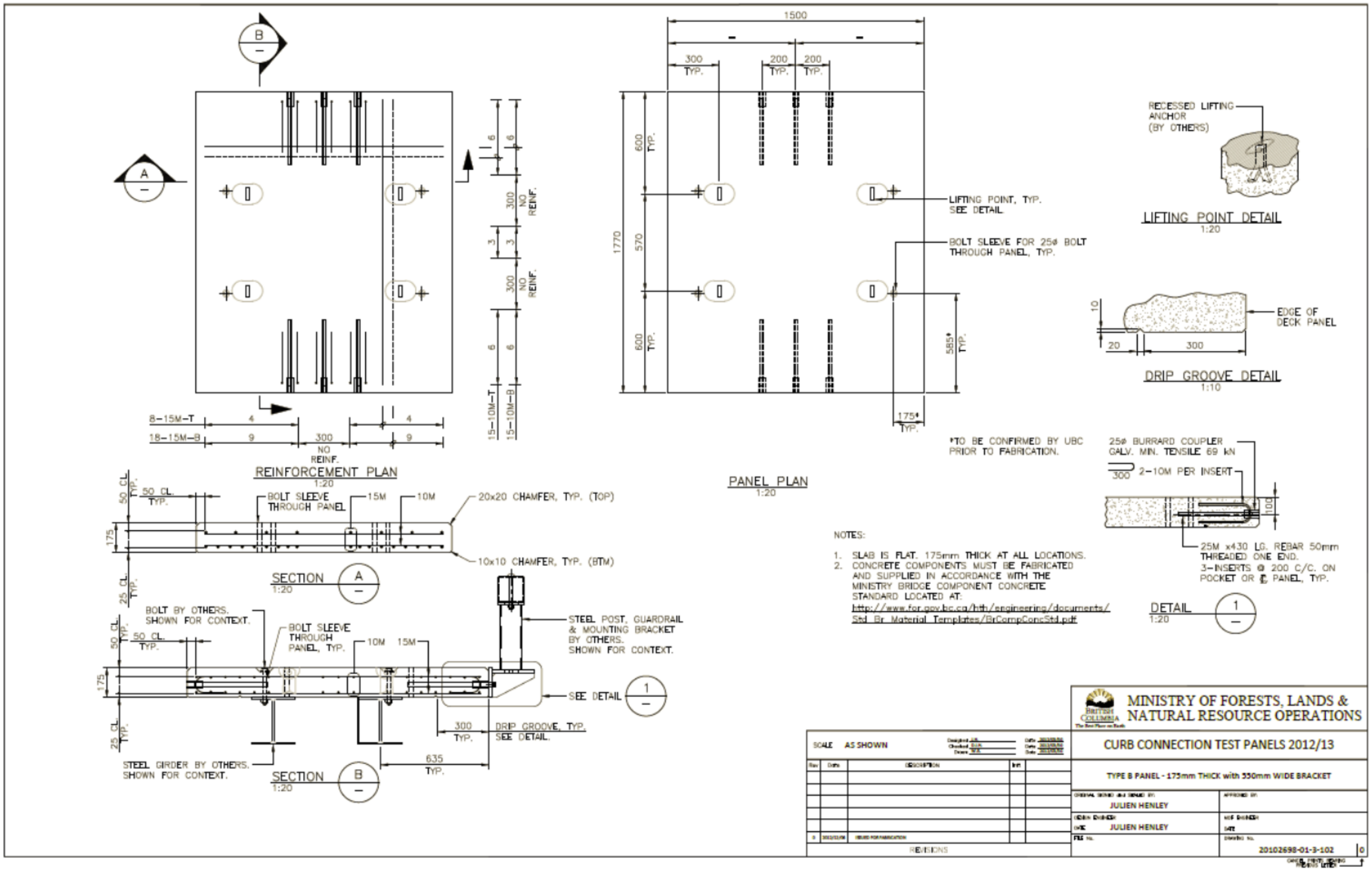
Revised Drawings_Full Set - Jan 10th 2013

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8-15M-T
18-15M-B
NO REINF.
15-10M-T
15-10M-B
PANEL PLAN
1:20
1500
300 TYP.
200 TYP.
1770
600 TYP.
570
585* TYP.
175* TYP.
LIFTING POINT, TYP. SEE DETAIL.
BOLT SLEEVE FOR 25ø BOLT THROUGH PANEL, TYP.
*TO BE CONFIRMED BY UBC PRIOR TO FABRICATION.
RECESSED LIFTING ANCHOR (BY OTHERS)
LIFTING POINT DETAIL
1:20
EDGE OF DECK PANEL
DRIP GROOVE DETAIL
1:10
25ø BURRARD COUPLER GALV. MIN. TENSILE 69 kN
2-10M PER INSERT
25M x430 LG. REBAR 50mm THREADED ONE END.
3-INSERTS @ 200 C/C. ON POCKET OR ℄ PANEL, TYP.
DETAIL 1
1:20
SECTION A
1:20
BOLT SLEEVE THROUGH PANEL
20x20 CHAMFER, TYP. (TOP)
10x10 CHAMFER, TYP. (BTM)
BOLT BY OTHERS. SHOWN FOR CONTEXT.
STEEL POST, GUARDRAIL & MOUNTING BRACKET BY OTHERS. SHOWN FOR CONTEXT.
SEE DETAIL 1
DRIP GROOVE, TYP. SEE DETAIL.
STEEL GIRDER BY OTHERS. SHOWN FOR CONTEXT.
635 TYP.
SECTION B
1:20
NOTES:
1. SLAB IS FLAT. 175mm THICK AT ALL LOCATIONS.
2. 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
MINISTRY OF FORESTS, LANDS & NATURAL RESOURCE OPERATIONS
SCALE AS SHOWN
REVISIONS
CURB CONNECTION TEST PANELS 2012/13
TYPE B PANEL - 175mm THICK with 550mm WIDE BRACKET
JULIEN HENLEY
JULIEN HENLEY
20102698-01-3-102
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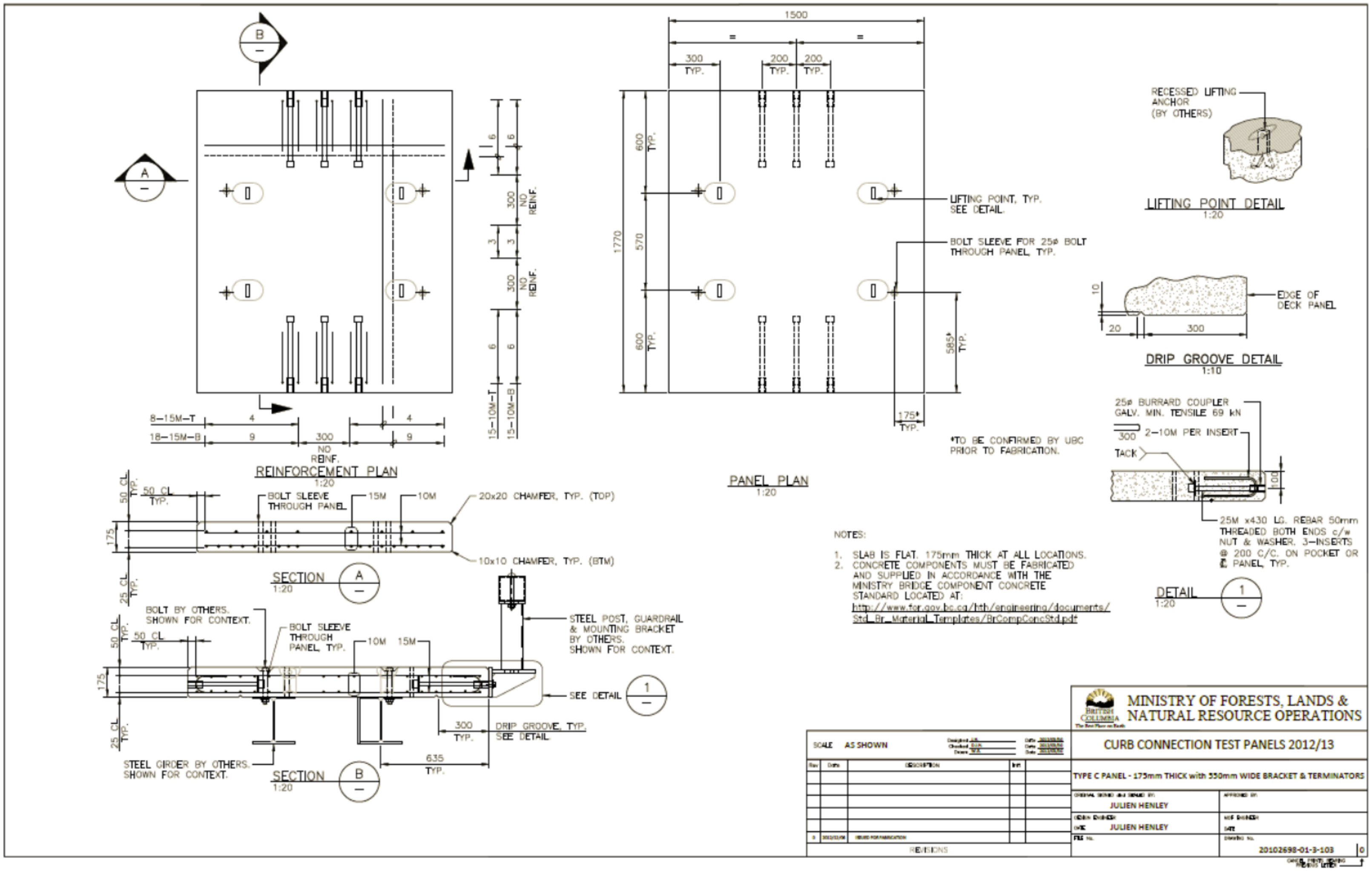
REINFORCEMENT PLAN
1:20
PANEL PLAN
1:20
SECTION A
1:20
SECTION B
1:20
LIFTING POINT DETAIL
1:20
DRIP GROOVE DETAIL
1:10
DETAIL 1
1:20
LIFTING POINT, TYP. SEE DETAIL.
BOLT SLEEVE FOR 25ø BOLT THROUGH PANEL, TYP.
RECESSED LIFTING ANCHOR (BY OTHERS)
EDGE OF DECK PANEL
25ø BURRARD COUPLER GALV. MIN. TENSILE 69 kN
2-10M PER INSERT
TACK
25M x430 LG. REBAR 50mm THREADED BOTH ENDS c/w NUT & WASHER. 3-INSERTS @ 200 C/C. ON POCKET OR ℄ PANEL, TYP.
*TO BE CONFIRMED BY UBC PRIOR TO FABRICATION.
20x20 CHAMFER, TYP. (TOP)
10x10 CHAMFER, TYP. (BTM)
BOLT SLEEVE THROUGH PANEL
BOLT BY OTHERS. SHOWN FOR CONTEXT.
BOLT SLEEVE THROUGH PANEL, TYP.
STEEL POST, GUARDRAIL & MOUNTING BRACKET BY OTHERS. SHOWN FOR CONTEXT.
SEE DETAIL
DRIP GROOVE, TYP. SEE DETAIL.
STEEL GIRDER BY OTHERS. SHOWN FOR CONTEXT.
8-15M-T
18-15M-B
NO REINF.
15-10M-T
15-10M-B
NOTES:
1. SLAB IS FLAT. 175mm THICK AT ALL LOCATIONS.
2. CONCRETE COMPONENTS MUST BE FABRICATED AND SUPPLIED IN ACCORDANCE WITH THE MINISTRY BRIDGE COMPONENT CONCRETE STANDARD LOCATED AT:
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MINISTRY OF FORESTS, LANDS & NATURAL RESOURCE OPERATIONS
CURB CONNECTION TEST PANELS 2012/13
TYPE C PANEL - 175mm THICK with 550mm WIDE BRACKET & TERMINATORS
SCALE AS SHOWN
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20102698-01-3-103
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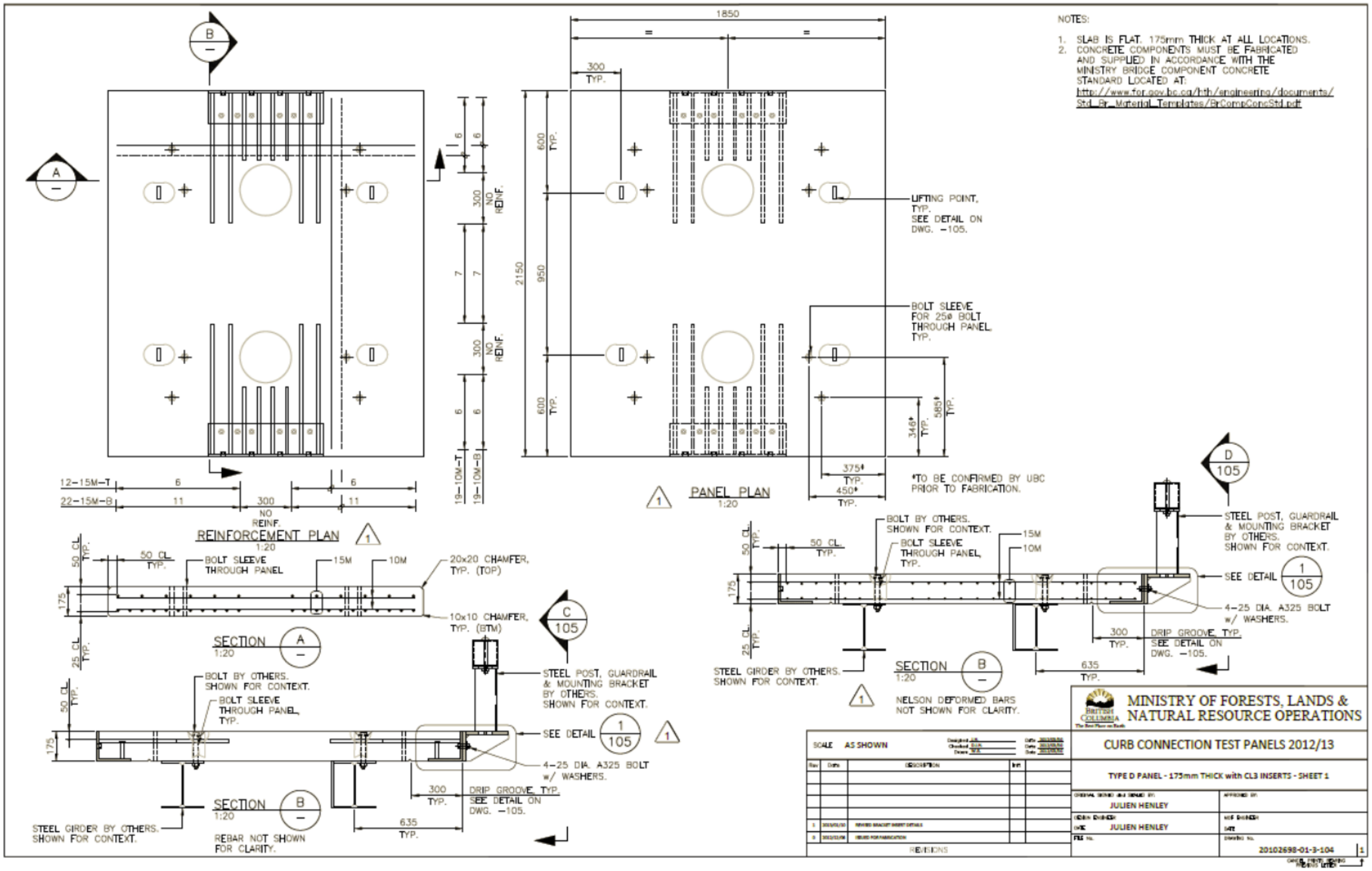
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REINFORCEMENT PLAN
1:20
PANEL PLAN
1:20
SECTION A
1:20
SECTION B
1:20
LIFTING POINT, TYP. SEE DETAIL ON DWG. -105.
BOLT SLEEVE FOR 25ø BOLT THROUGH PANEL. TYP.
*TO BE CONFIRMED BY UBC PRIOR TO FABRICATION.
BOLT BY OTHERS. SHOWN FOR CONTEXT.
BOLT SLEEVE THROUGH PANEL
20x20 CHAMFER, TYP. (TOP)
10x10 CHAMFER, TYP. (BTM)
STEEL POST, GUARDRAIL & MOUNTING BRACKET BY OTHERS. SHOWN FOR CONTEXT.
SEE DETAIL 1/105
4-25 DIA. A325 BOLT w/ WASHERS.
DRIP GROOVE, TYP. SEE DETAIL ON DWG. -105.
STEEL GIRDER BY OTHERS. SHOWN FOR CONTEXT.
REBAR NOT SHOWN FOR CLARITY.
NELSON DEFORMED BARS NOT SHOWN FOR CLARITY.
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
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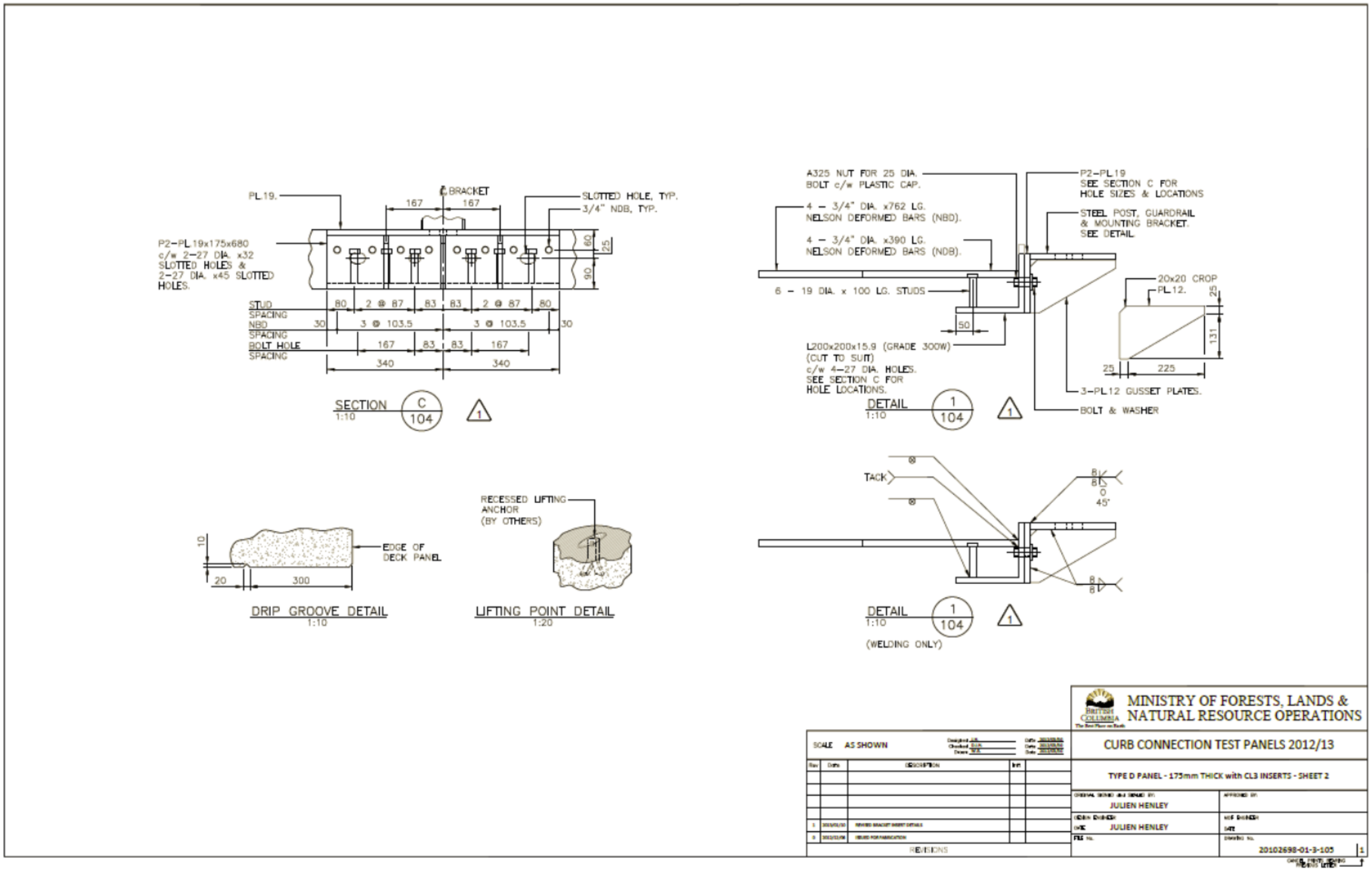
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3/4" NDB, TYP.
P2-PL.19x175x680 c/w 2-27 DIA. x32 SLOTTED HOLES & 2-27 DIA. x45 SLOTTED HOLES.
STUD SPACING
NBD SPACING
BOLT HOLE SPACING
SECTION C 104 1:10
A325 NUT FOR 25 DIA. BOLT c/w PLASTIC CAP.
4 – 3/4" DIA. x762 LG. NELSON DEFORMED BARS (NBD).
4 – 3/4" DIA. x390 LG. NELSON DEFORMED BARS (NDB).
6 – 19 DIA. x 100 LG. STUDS
L200x200x15.9 (GRADE 300W) (CUT TO SUIT) c/w 4-27 DIA. HOLES. SEE SECTION C FOR HOLE LOCATIONS.
P2-PL.19 SEE SECTION C FOR HOLE SIZES & LOCATIONS
STEEL POST, GUARDRAIL & MOUNTING BRACKET. SEE DETAIL
20x20 CROP PL.12.
3-PL.12 GUSSET PLATES.
BOLT & WASHER
DETAIL 1 104 1:10
TACK
DETAIL 1 104 1:10
(WELDING ONLY)
EDGE OF DECK PANEL
DRIP GROOVE DETAIL 1:10
RECESSED LIFTING ANCHOR (BY OTHERS)
LIFTING POINT DETAIL 1:20
MINISTRY OF FORESTS, LANDS & NATURAL RESOURCE OPERATIONS
SCALE AS SHOWN
CURB CONNECTION TEST PANELS 2012/13
TYPE D PANEL - 175mm THICK with CL3 INSERTS - SHEET 2
JULIEN HENLEY
JULIEN HENLEY
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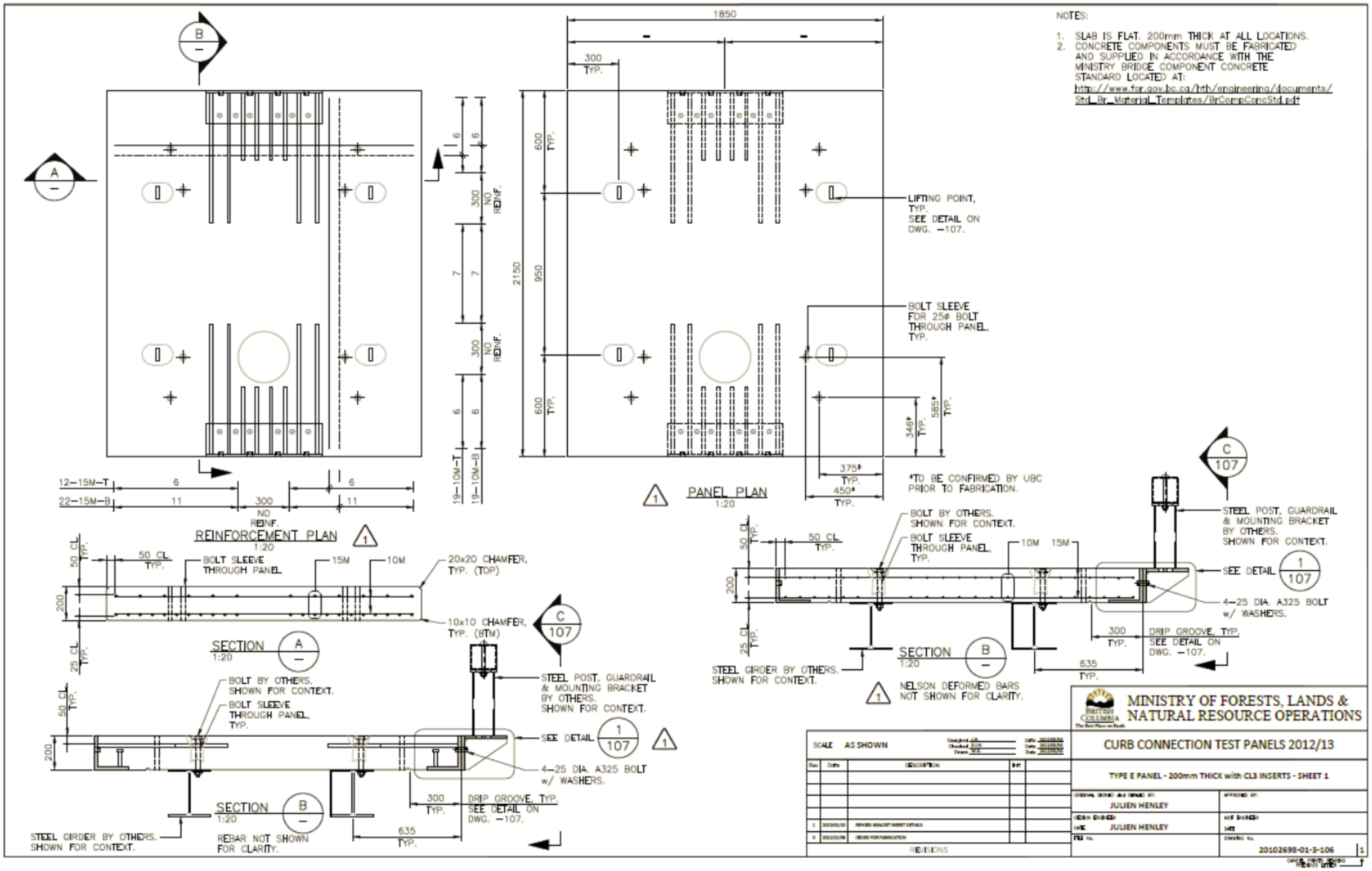
20102698-01-3-105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REINFORCEMENT PLAN
1:20
PANEL PLAN
1:20
LIFTING POINT, TYP. SEE DETAIL ON DWG. -107.
BOLT SLEEVE FOR 25ø BOLT THROUGH PANEL, TYP.
*TO BE CONFIRMED BY UBC PRIOR TO FABRICATION.
SECTION A
1:20
SECTION B
1:20
20x20 CHAMFER, TYP. (TOP)
10x10 CHAMFER, TYP. (BTM)
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BOLT BY OTHERS. SHOWN FOR CONTEXT.
BOLT SLEEVE THROUGH PANEL, TYP.
4-25 DIA. A325 BOLT w/ WASHERS.
DRIP GROOVE, TYP. SEE DETAIL ON DWG. -107.
STEEL GIRDER BY OTHERS. SHOWN FOR CONTEXT.
REBAR NOT SHOWN FOR CLARITY.
NELSON DEFORMED BARS NOT SHOWN FOR CLARITY.
MINISTRY OF FORESTS, LANDS & NATURAL RESOURCE OPERATIONS
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SCALE AS SHOWN
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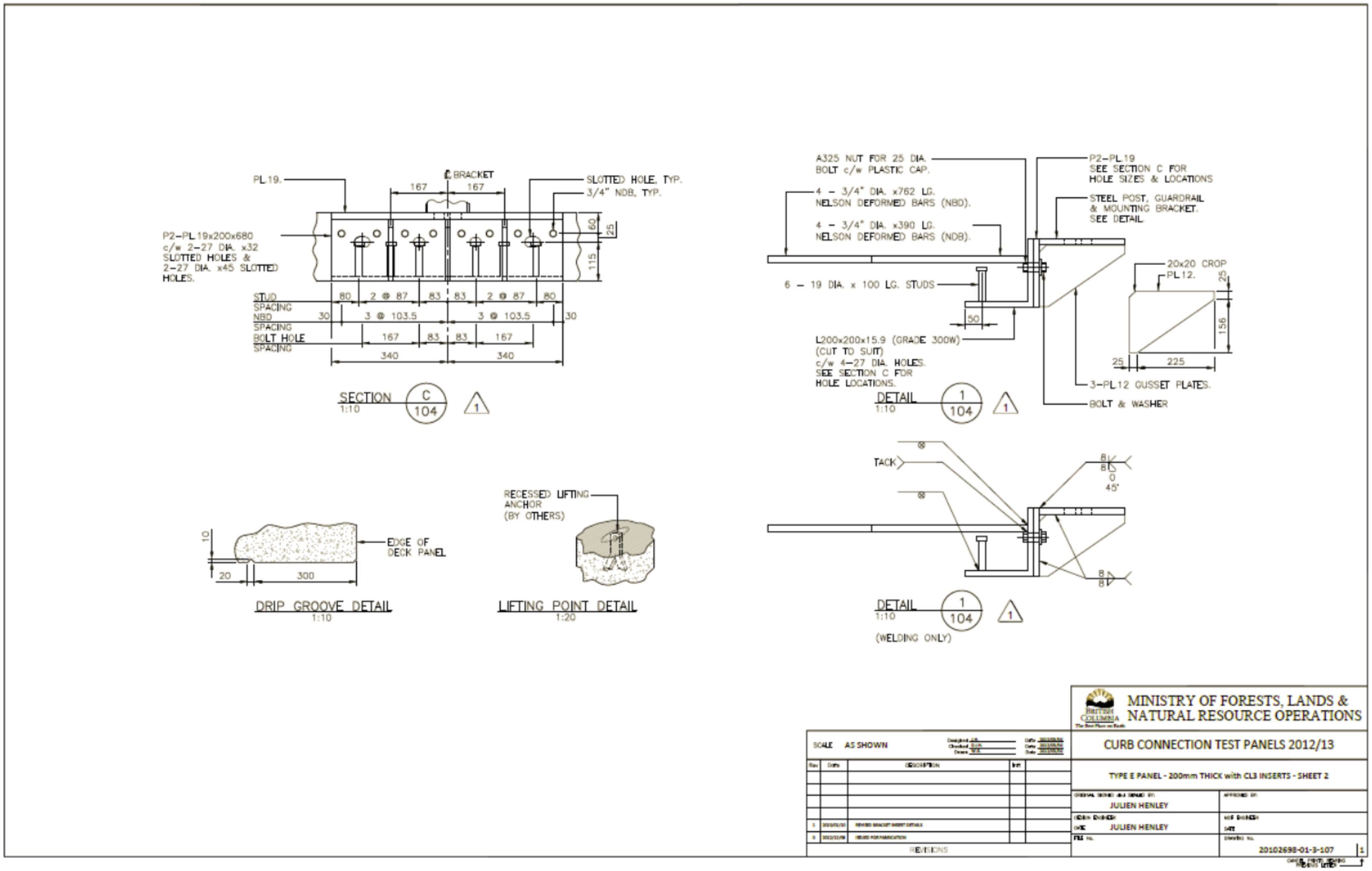
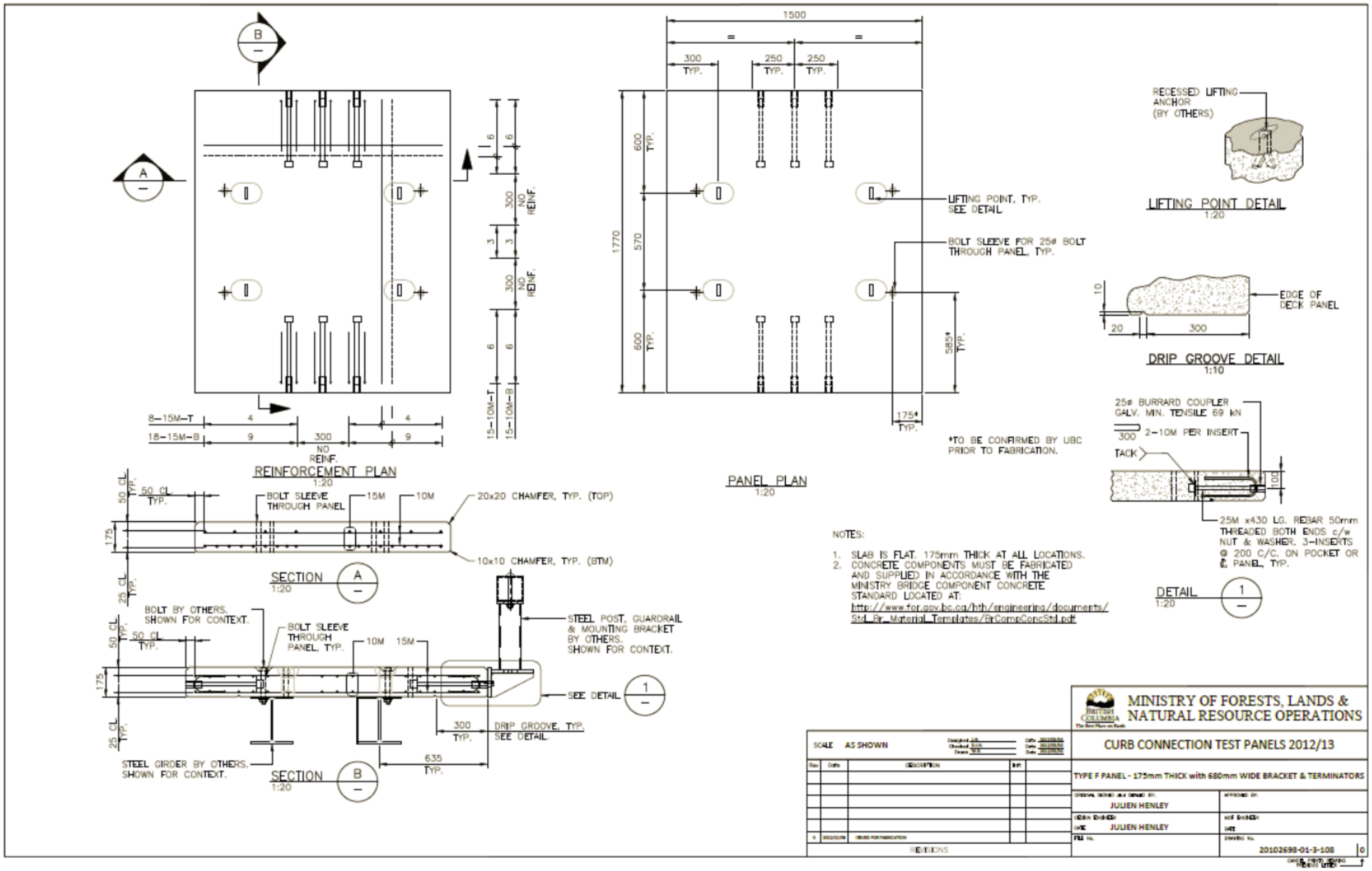

PL 19.
℄ BRACKET
SLOTTED HOLE, TYP.
3/4" NDB, TYP.
P2-PL 19x200x680 c/w 2-27 DIA. x32 SLOTTED HOLES & 2-27 DIA. x45 SLOTTED HOLES.
STUD SPACING
NBD SPACING
BOLT HOLE SPACING
SECTION C 104
1:10
A325 NUT FOR 25 DIA. BOLT c/w PLASTIC CAP.
4 - 3/4" DIA. x762 LG. NELSON DEFORMED BARS (NBD).
4 - 3/4" DIA. x390 LG. NELSON DEFORMED BARS (NDB).
6 - 19 DIA. x 100 LG. STUDS
P2-PL 19 SEE SECTION C FOR HOLE SIZES & LOCATIONS
STEEL POST, GUARDRAIL & MOUNTING BRACKET. SEE DETAIL
20x20 CROP
PL 12.
L200x200x15.9 (GRADE 300W) (CUT TO SUIT) c/w 4-27 DIA. HOLES. SEE SECTION C FOR HOLE LOCATIONS.
3-PL 12 GUSSET PLATES.
BOLT & WASHER
DETAIL 1 104
1:10
TACK
45°
EDGE OF DECK PANEL
DRIP GROOVE DETAIL
1:10
RECESSED LIFTING ANCHOR (BY OTHERS)
LIFTING POINT DETAIL
1:20
DETAIL 1 104
1:10
(WELDING ONLY)
MINISTRY OF FORESTS, LANDS & NATURAL RESOURCE OPERATIONS
SCALE AS SHOWN
CURB CONNECTION TEST PANELS 2012/13
TYPE E PANEL - 200mm THICK with CL3 INSERTS - SHEET 2
JULIEN HENLEY
JULIEN HENLEY
REVISIONS
20102698-01-3-107


REINFORCEMENT PLAN
1:20
PANEL PLAN
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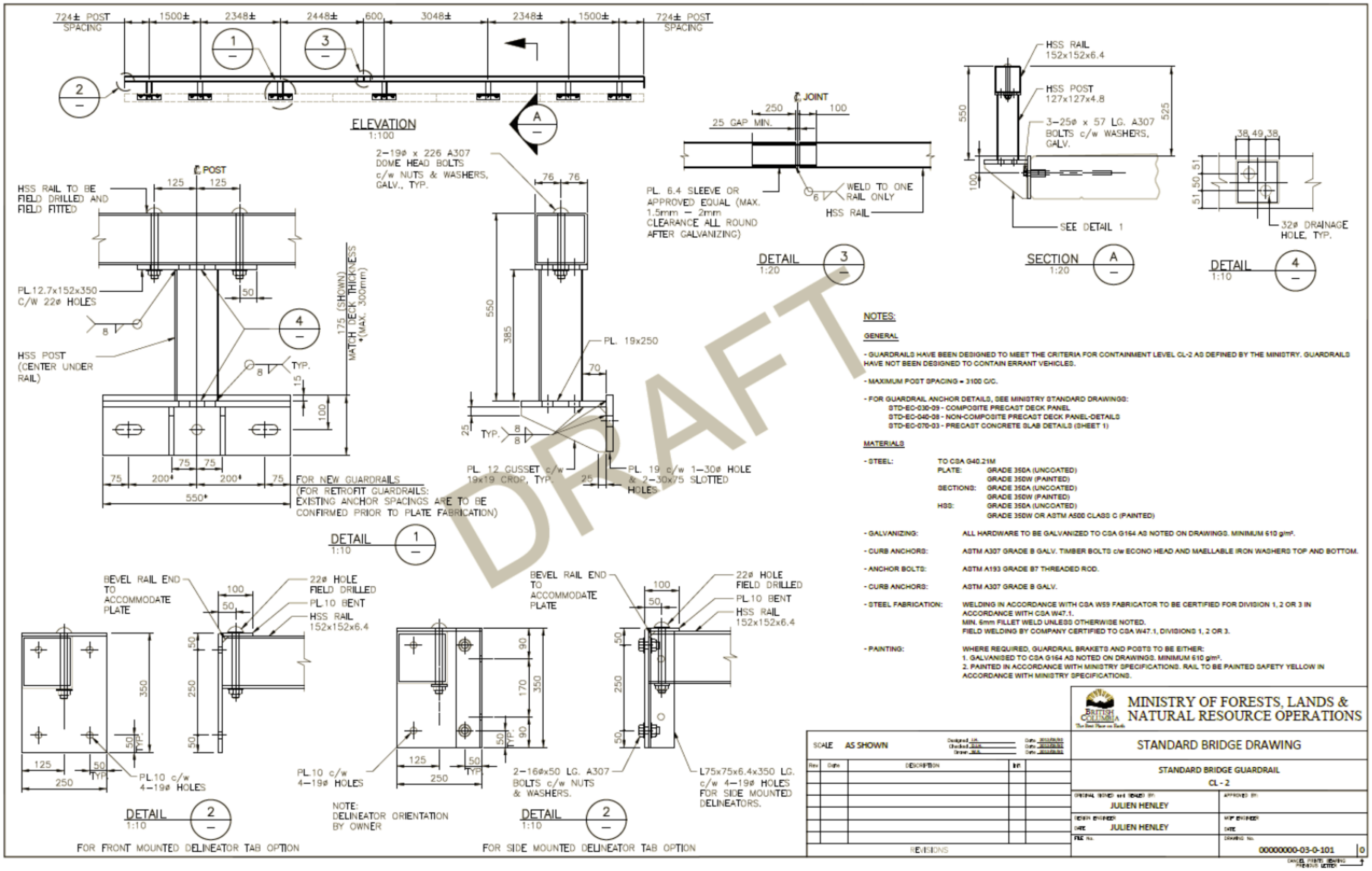
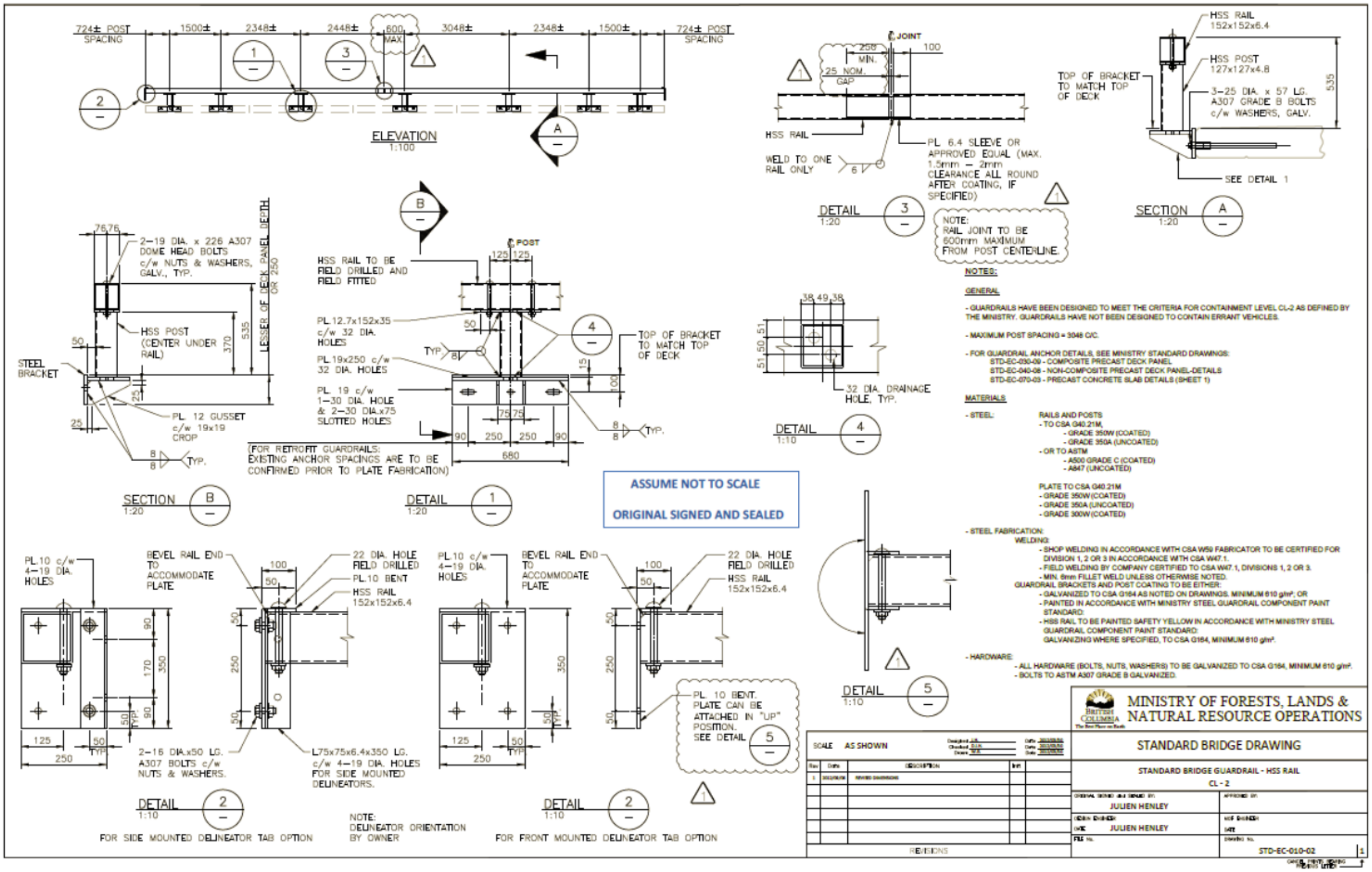
LIFTING POINT DETAIL
1:20
DRIP GROOVE DETAIL
1:10
SECTION A
1:20
SECTION B
1:20
DETAIL 1
1:20
RECESSED LIFTING ANCHOR (BY OTHERS)
LIFTING POINT, TYP. SEE DETAIL
BOLT SLEEVE FOR 25ø BOLT THROUGH PANEL, TYP.
EDGE OF DECK PANEL
25ø BURRARD COUPLER GALV. MIN. TENSILE 69 kN
2-10M PER INSERT
TACK
25M x430 LG. REBAR 50mm THREADED BOTH ENDS c/w NUT & WASHER. 3-INSERTS @ 200 C/C. ON POCKET OR ℄ PANEL, TYP.
*TO BE CONFIRMED BY UBC PRIOR TO FABRICATION.
20x20 CHAMFER, TYP. (TOP)
10x10 CHAMFER, TYP. (BTM)
BOLT BY OTHERS. SHOWN FOR CONTEXT.
BOLT SLEEVE THROUGH PANEL, TYP.
STEEL POST, GUARDRAIL & MOUNTING BRACKET BY OTHERS. SHOWN FOR CONTEXT.
SEE DETAIL
DRIP GROOVE, TYP. SEE DETAIL.
STEEL GIRDER BY OTHERS. SHOWN FOR CONTEXT.
NOTES:
1. SLAB IS FLAT. 175mm THICK AT ALL LOCATIONS.
2. 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
MINISTRY OF FORESTS, LANDS & NATURAL RESOURCE OPERATIONS
CURB CONNECTION TEST PANELS 2012/13
TYPE F PANEL - 175mm THICK with 680mm WIDE BRACKET & TERMINATORS
SCALE AS SHOWN
JULIEN HENLEY
JULIEN HENLEY
20102698-01-3-108
REVISIONS

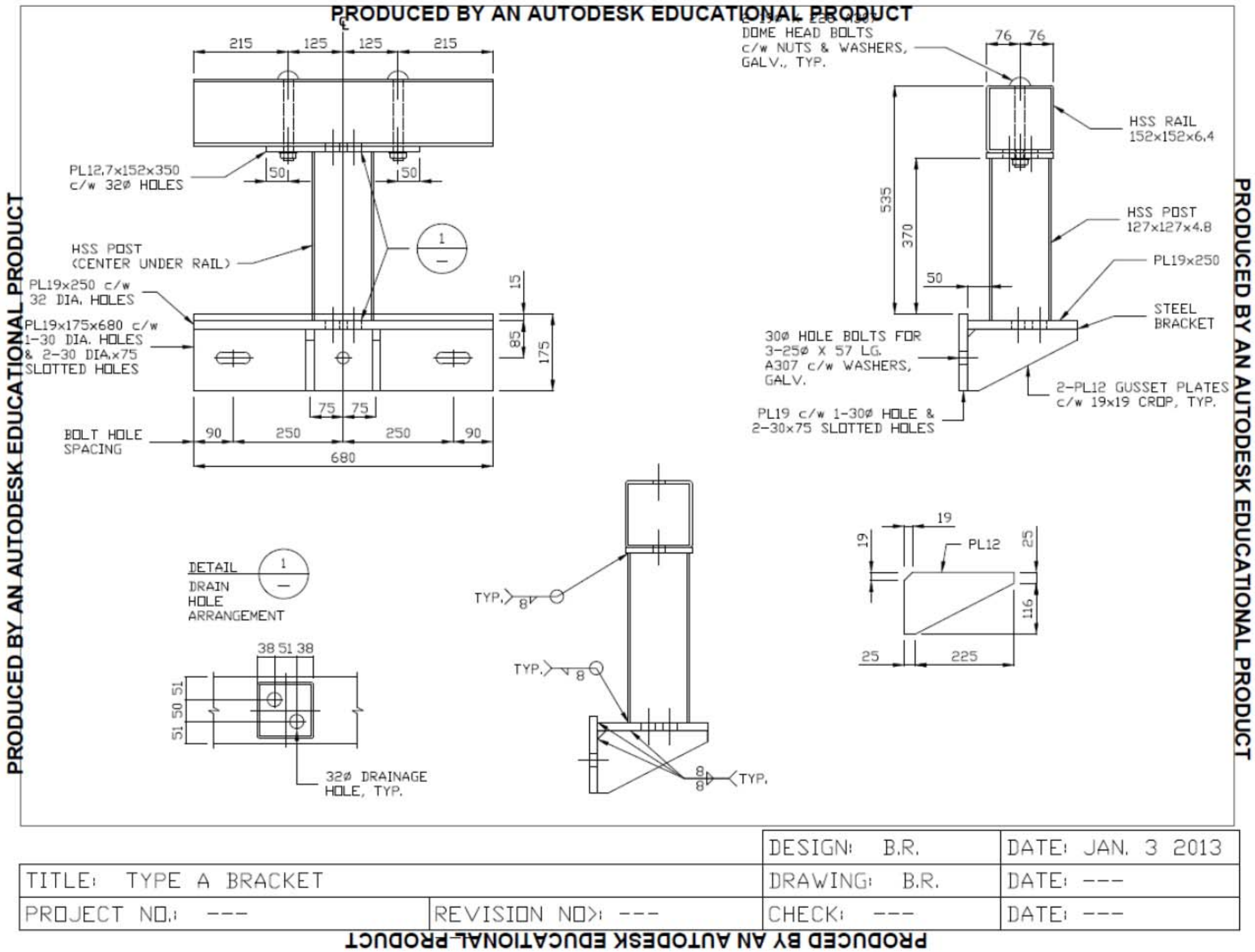
550 mm bracket



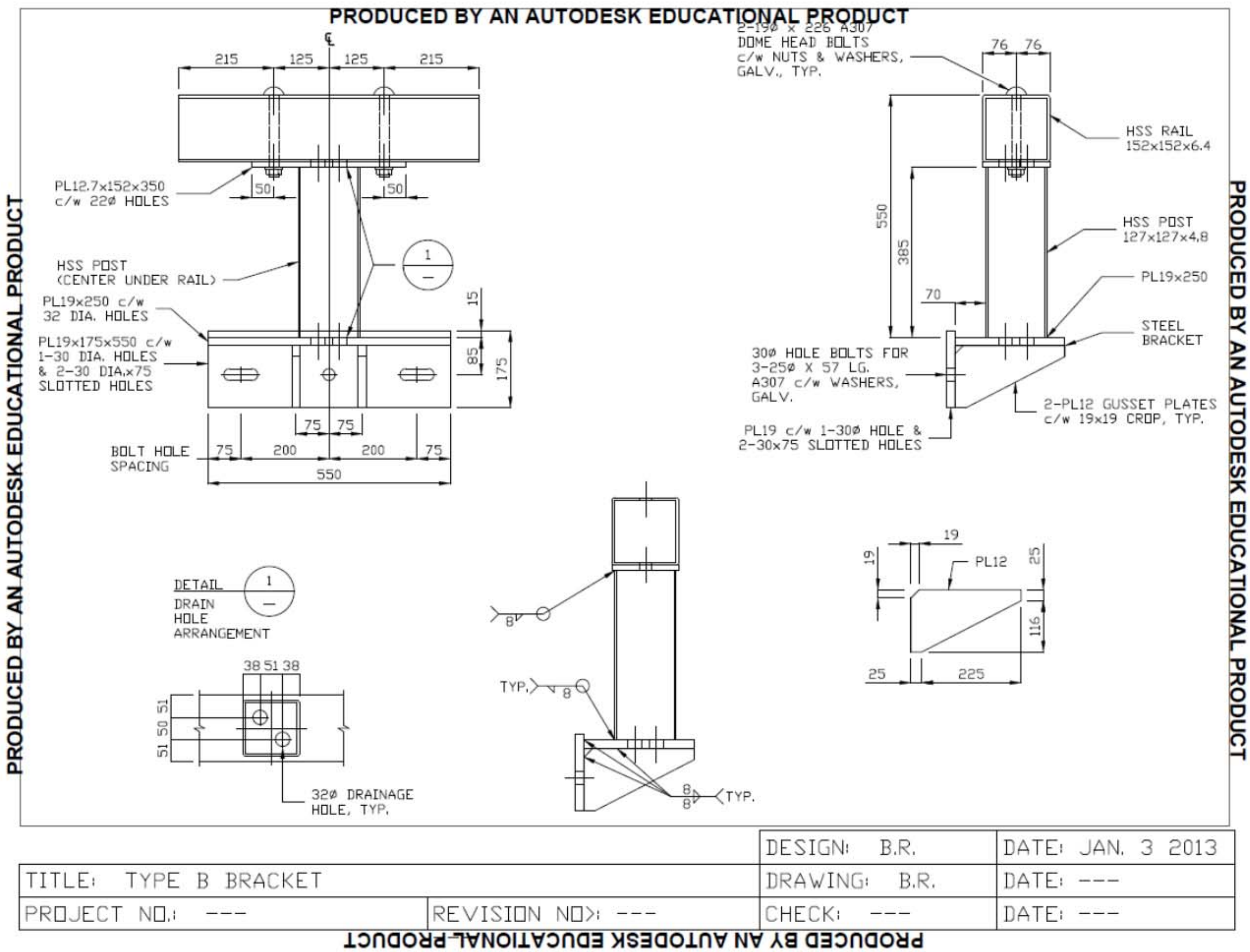
680 mm bracket



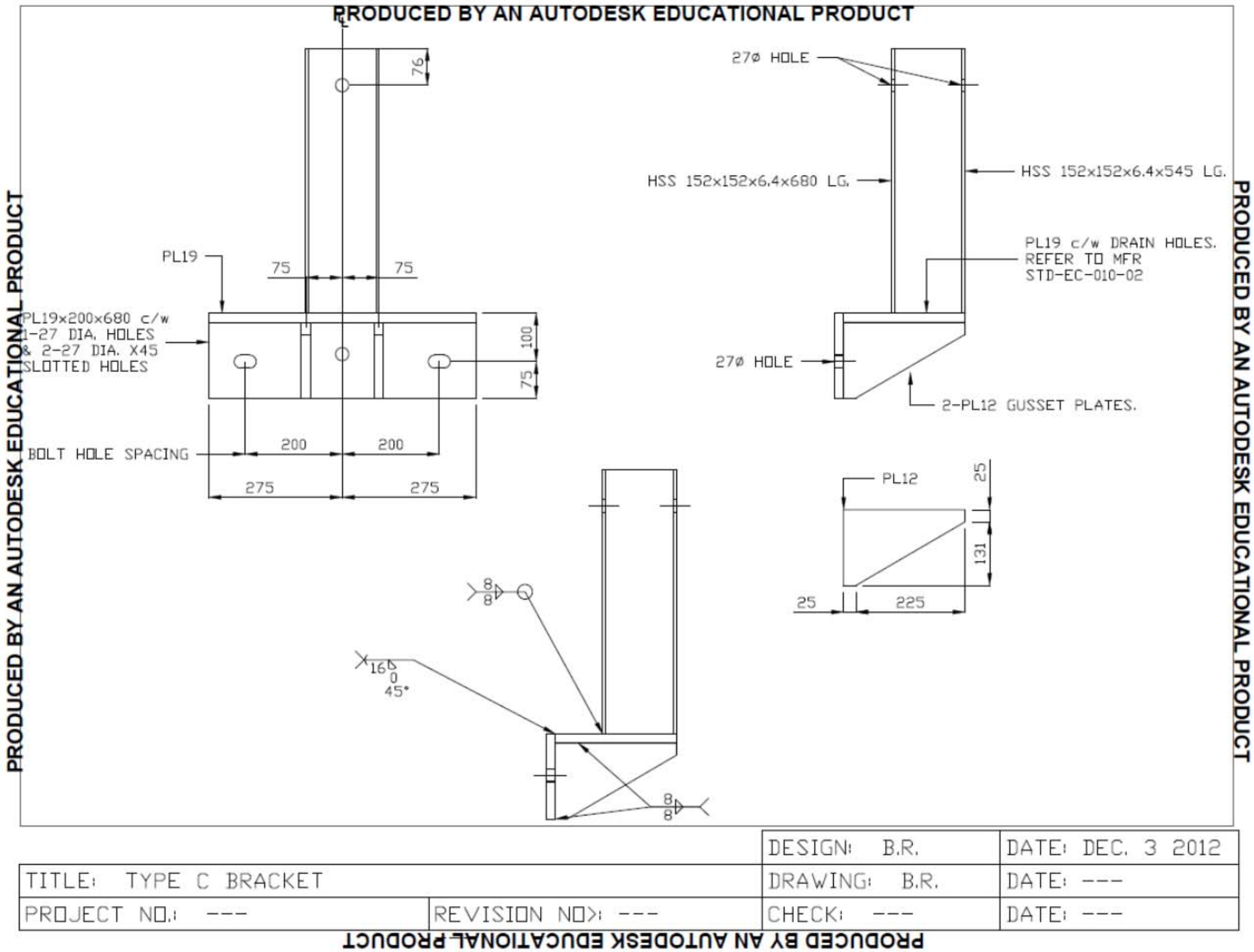
GUARDRAIL DRAWING-Type A Bracket



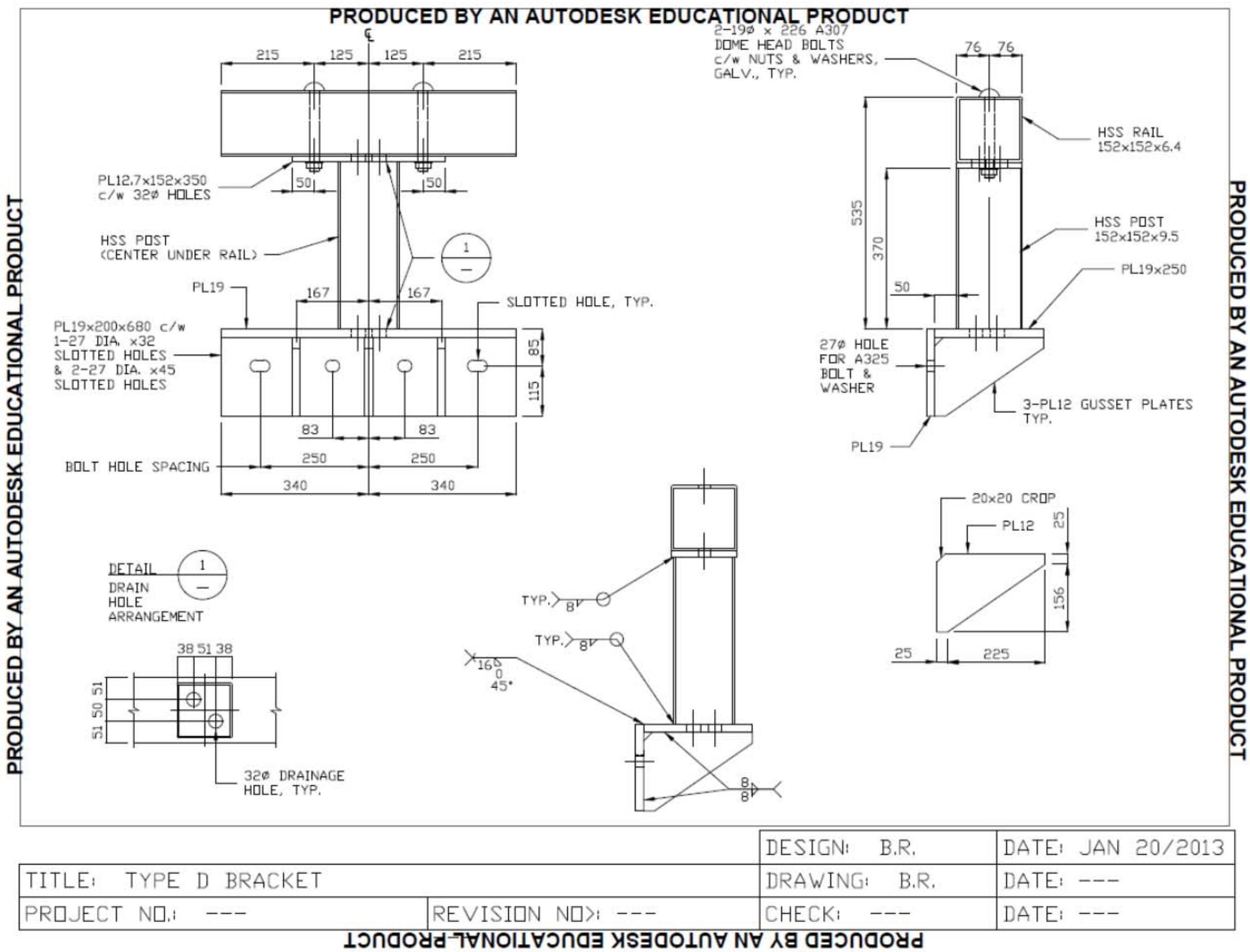
GUARDRAIL DRAWING-Type B Bracket



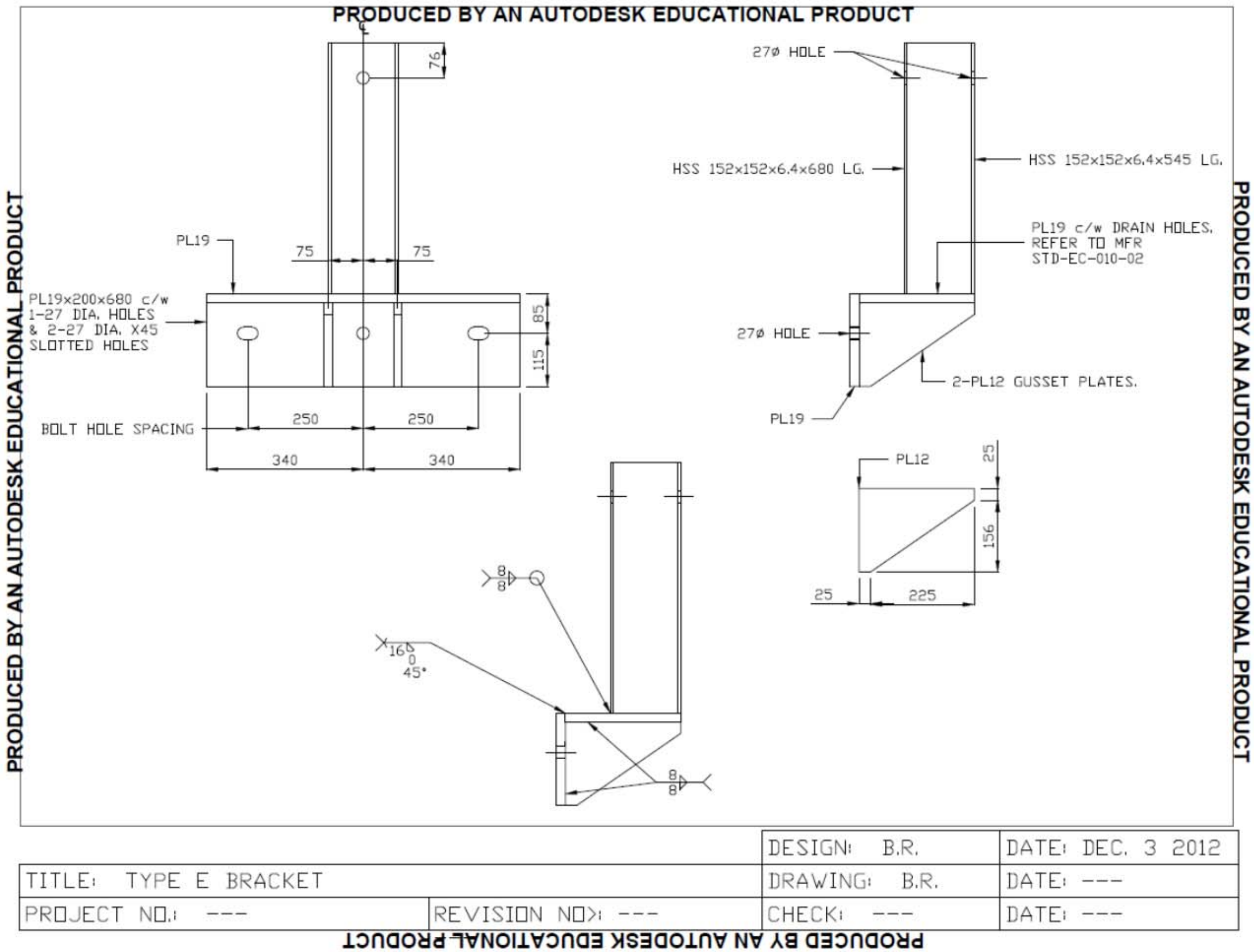
GUARDRAIL DRAWING-Type C Bracket



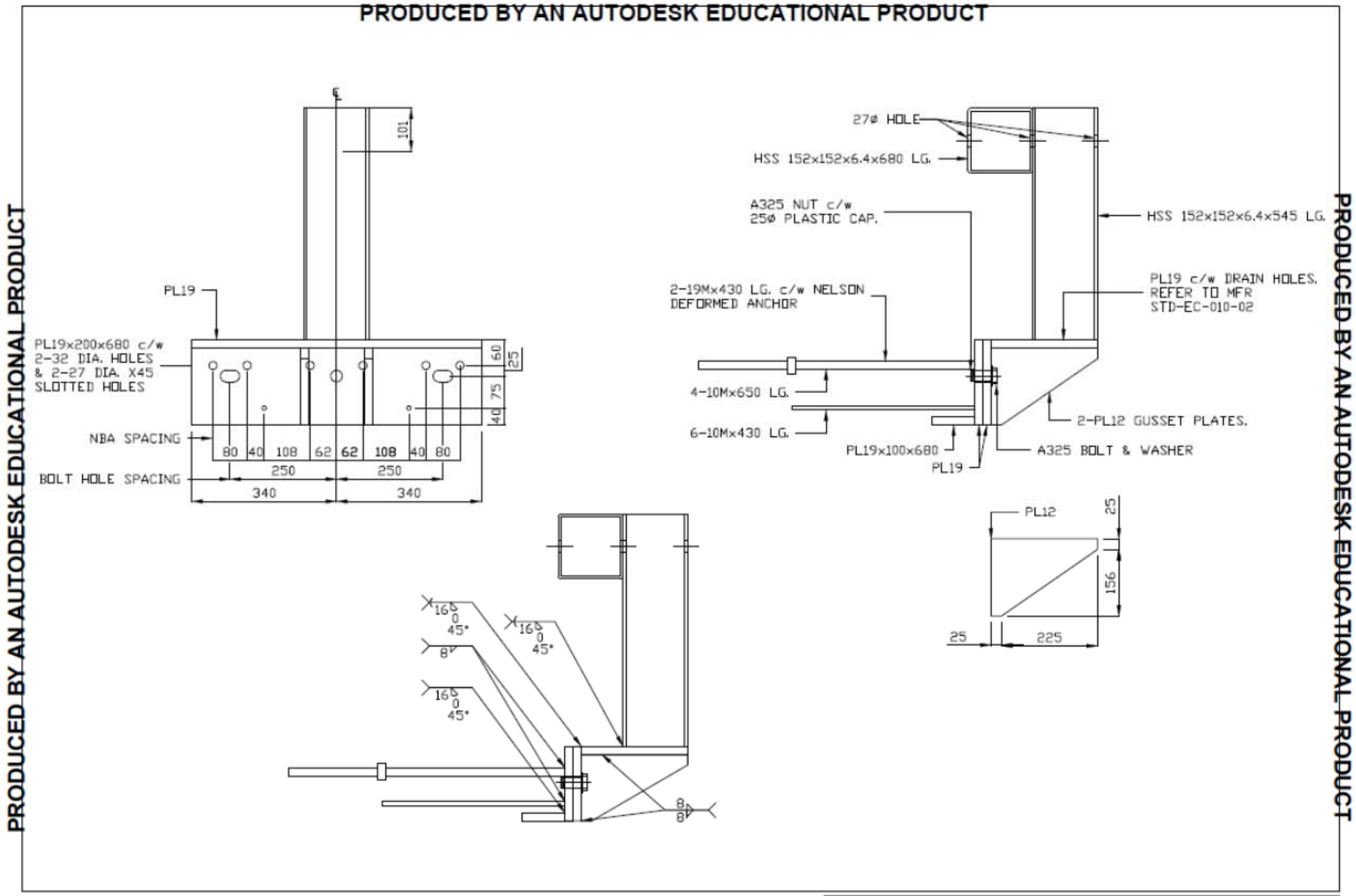
GUARDRAIL DRAWING-Type D Bracket



GUARDRAIL DRAWING-Type E Bracket

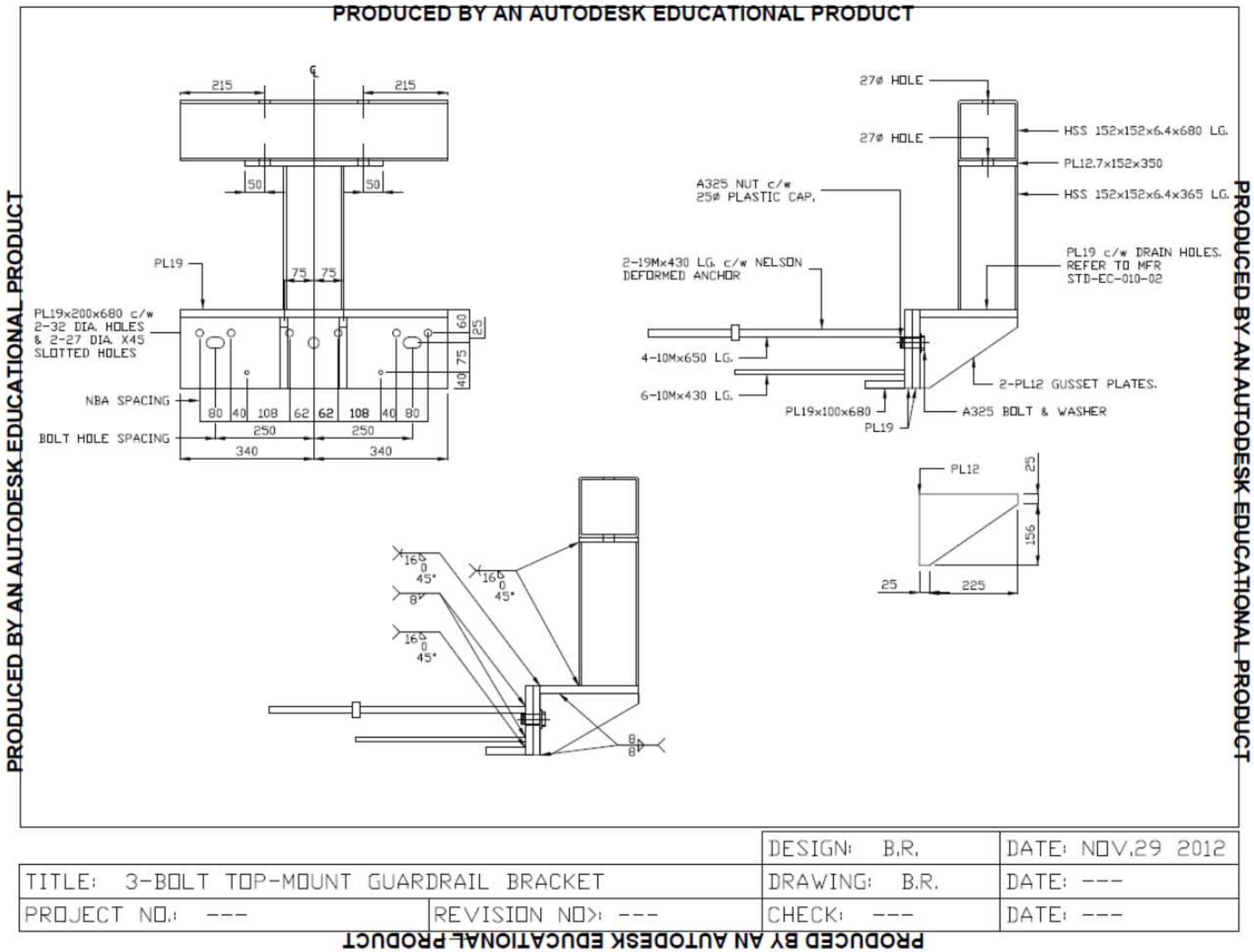


GUARDRAIL DRAWING-3-Bolt Side-Mount Guardrail Bracket

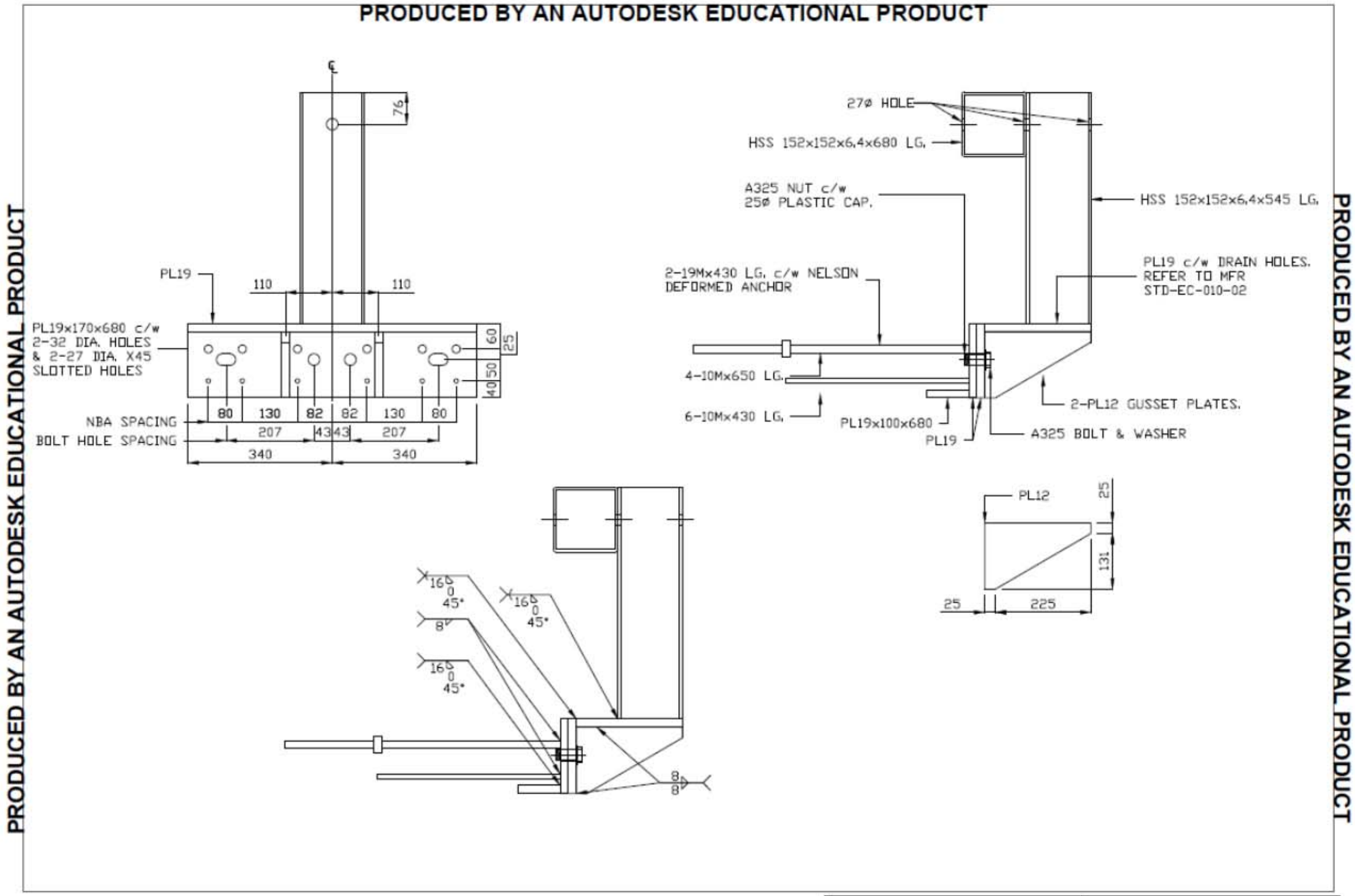


		DESIGN: B.R.	DATE: NOV.29 2012
TITLE: 3-BOLT SIDE-MOUNT GUARDRAIL BRACKET		DRAWING: B.R.	DATE: ---
PROJECT NO.: ---	REVISION NO>: ---	CHECK: ---	DATE: ---

GUARDRAIL DRAWING-3-Bolt Top-Mount Guardrail Bracket



GUARDRAIL DRAWING-4-Bolt Side-Mount Guardrail Bracket



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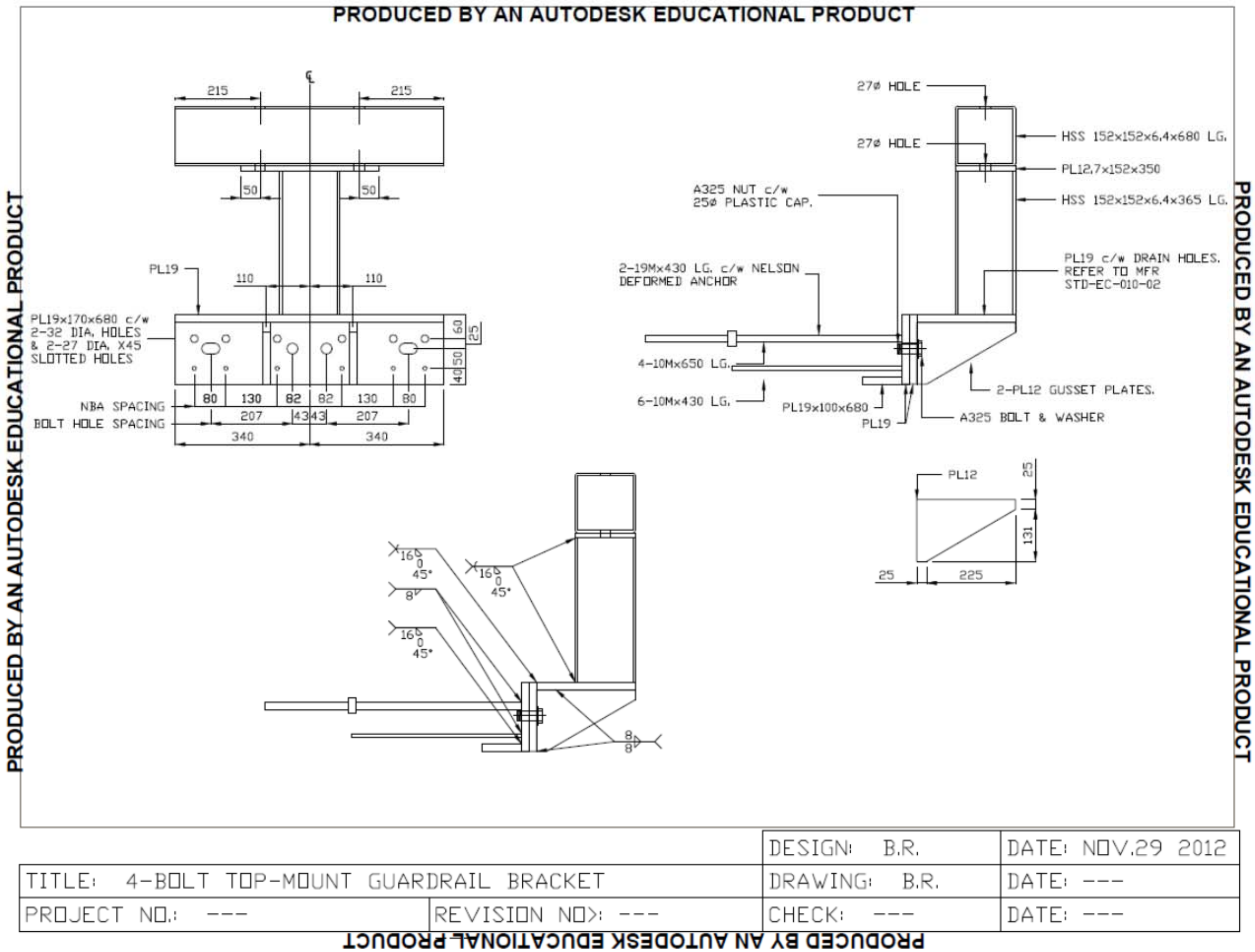
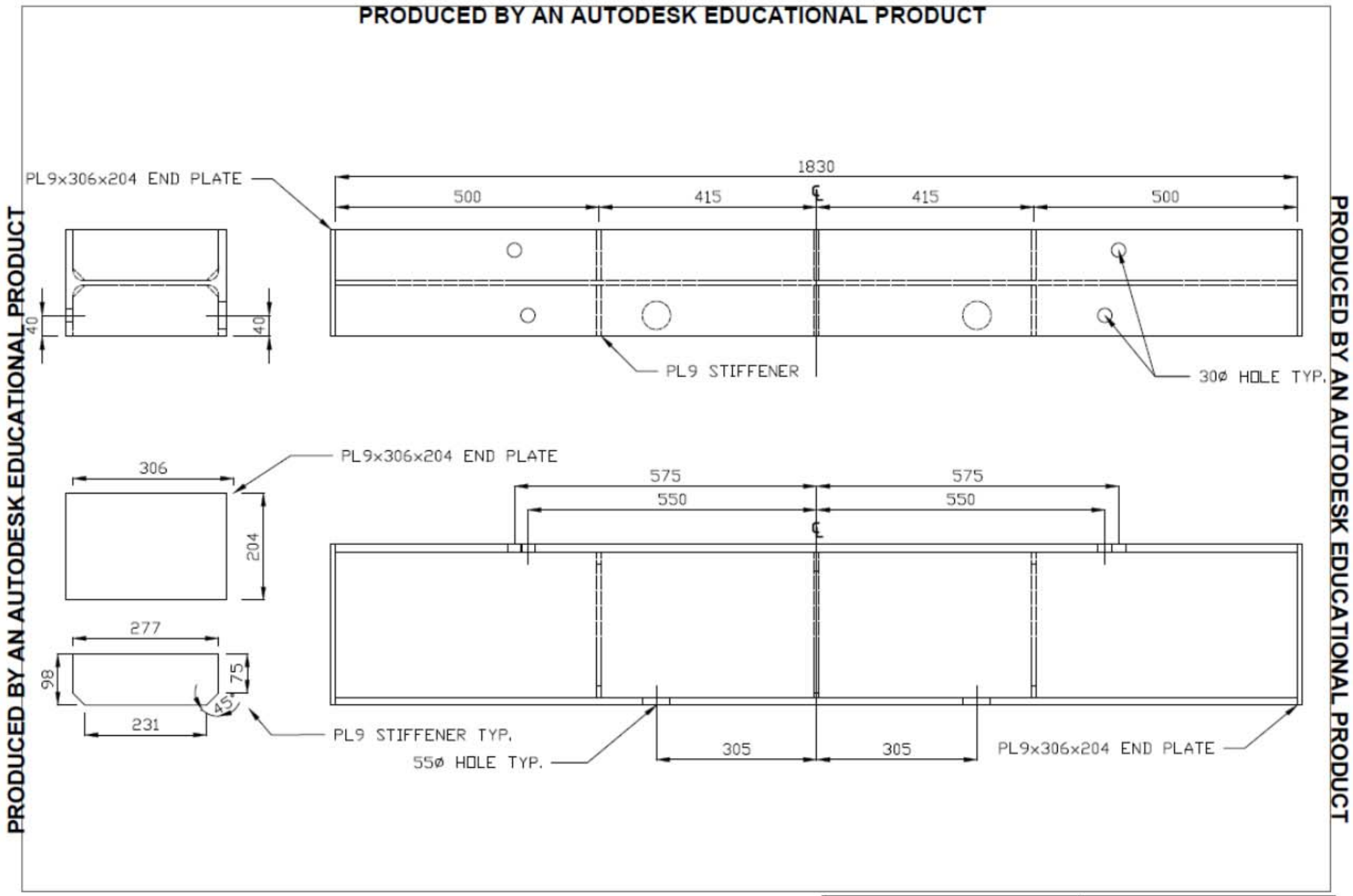
TITLE: 4-BOLT SIDE-MOUNT GUARDRAIL BRACKET	DRAWING: B.R.	DATE: ---
PROJECT NO.: --- REVISION NO>: ---	CHECK: ---	DATE: ---

GUARDRAIL DRAWING-4-Bolt Top-Mount Guardrail Bracket



GUARDRAIL DRAWING-Front Support Beam



		DESIGN: B.R.	DATE: DEC. 3 2012
TITLE: FRONT SUPPORT BEAM		DRAWING: B.R.	DATE: ---
PROJECT NO.: ---	REVISION NO>: ---	CHECK: ---	DATE: ---

GUARDRAIL DRAWING-Back Support Beam