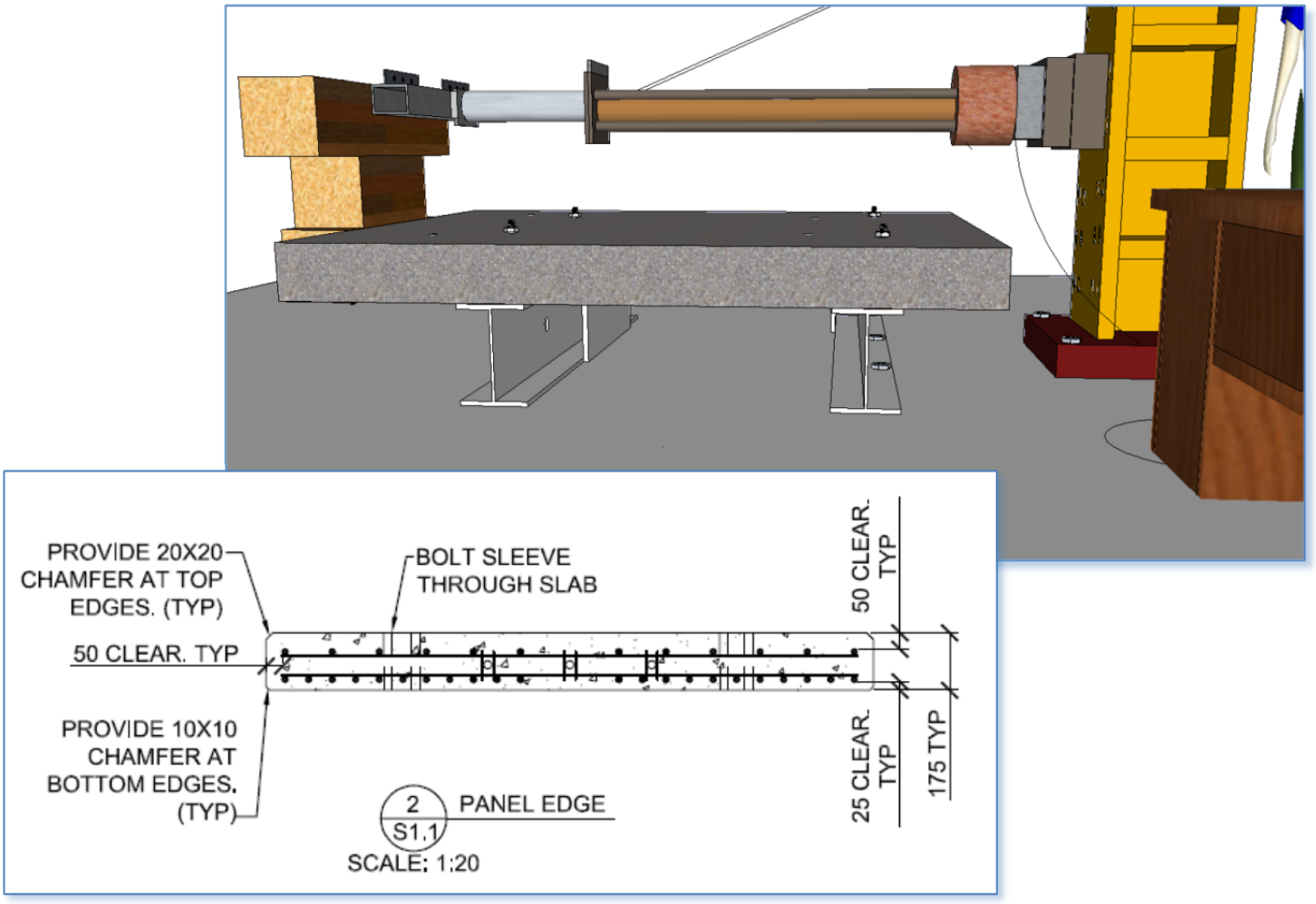
Project "Experimental Evaluation of Concrete Decks with Guard Rail Systems" for Ministry of Forests, Lands and Natural Resource Operations



Project Report by
C. Villiard & C. Dickof & M. Angers & J. Schneider & S.F. Stiemer

UBC

Final Report Vancouver May 2011

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Framework

Objective

In order to establish a baseline on guard rail/connection systems, experimental testing is being performed. The results shall enable the evaluation and development of design parameters for new curb systems in future.

Focal interest of the Phase 2 work, which complements work completed in Phase 1 in fiscal 2009, is determining the governing failure mechanism and ultimate resistance or capacity of the precast concrete deck panel guard rail/connection systems.

Scope of the work

Three different guard rail riser connection systems shall be tested on concrete panels:

Timber guard rail and risers on timber cross ties as per upper left of drawing # STD-E-010-01

All steel retro fit system as per STD-E-010-06 and deck details as per bullet 2 (STD-E-030-12)

All steel system based on a modified STD-E-010-06 which includes a plate on the concrete deck (per attached drawing).

Approach

1. Identification of design elements and components for testing
2. Development of the precast concrete panel test specimen designs to be consistent with current MFR standards
3. Preparation of detailed test plan
4. Theoretical evaluation to predict results
5. Use of available curb system materials from phase 1 and supplementary steel element fabrication of additional test specimens as required
6. Lab testing of riser/connection system
7. Reporting of experimental results

The Ministry of Forests and Range shall be responsible for supply and delivery of the six precast concrete test panels that shall be used in the testing.

Deliverables

1. Precast concrete panel test specimen design drawing(s) for use by MFR in tendering of their supply.
2. Lab testing of curb connection systems
3. Report(s) including a complete set of experimental data and discussion of theoretical verses observed results. A draft report shall be provided as well as a final report. The final report shall provide recommendations on possible improvements to the tested systems.
4. Web/teleconference based presentation of the results to MFR selected group of professional engineers.

Budget

See UILO/MoFR contract.

Timeframe

The projected timeframe for the work will be Oct.18st 2010 to Mar. 31st, 2011.

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Photos from actual failures in the field

Figure 1: Photos from Actual Field Failures of Various Barrier Types, Group 1

Figure 2: Photos from Actual Field Failures of Various Barrier Types, Group 2

Test Set-Up Planning

Original Drawings of timber guard rail attachments to bridge deck

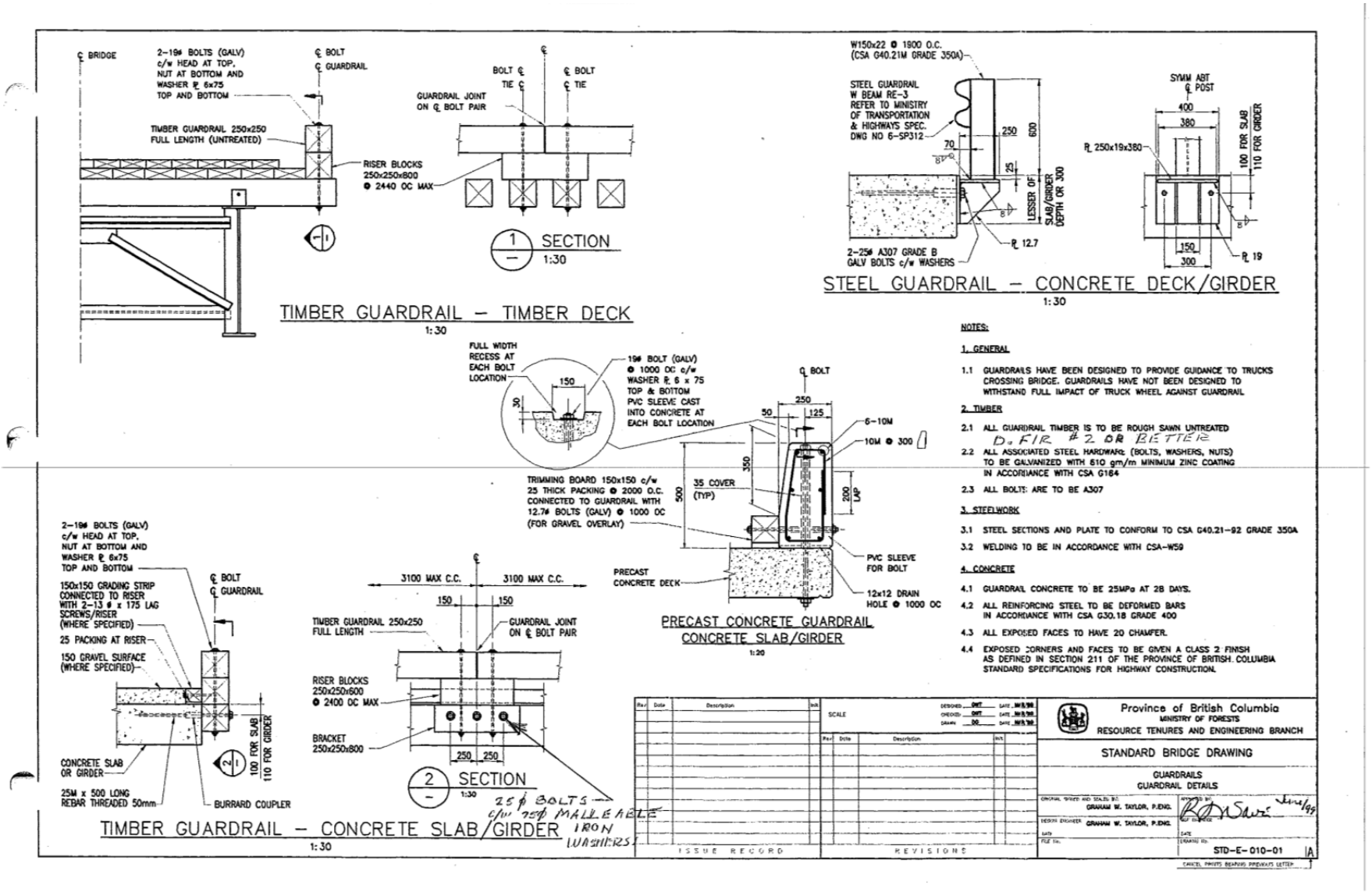

Figure 3: Standard bridge drawing

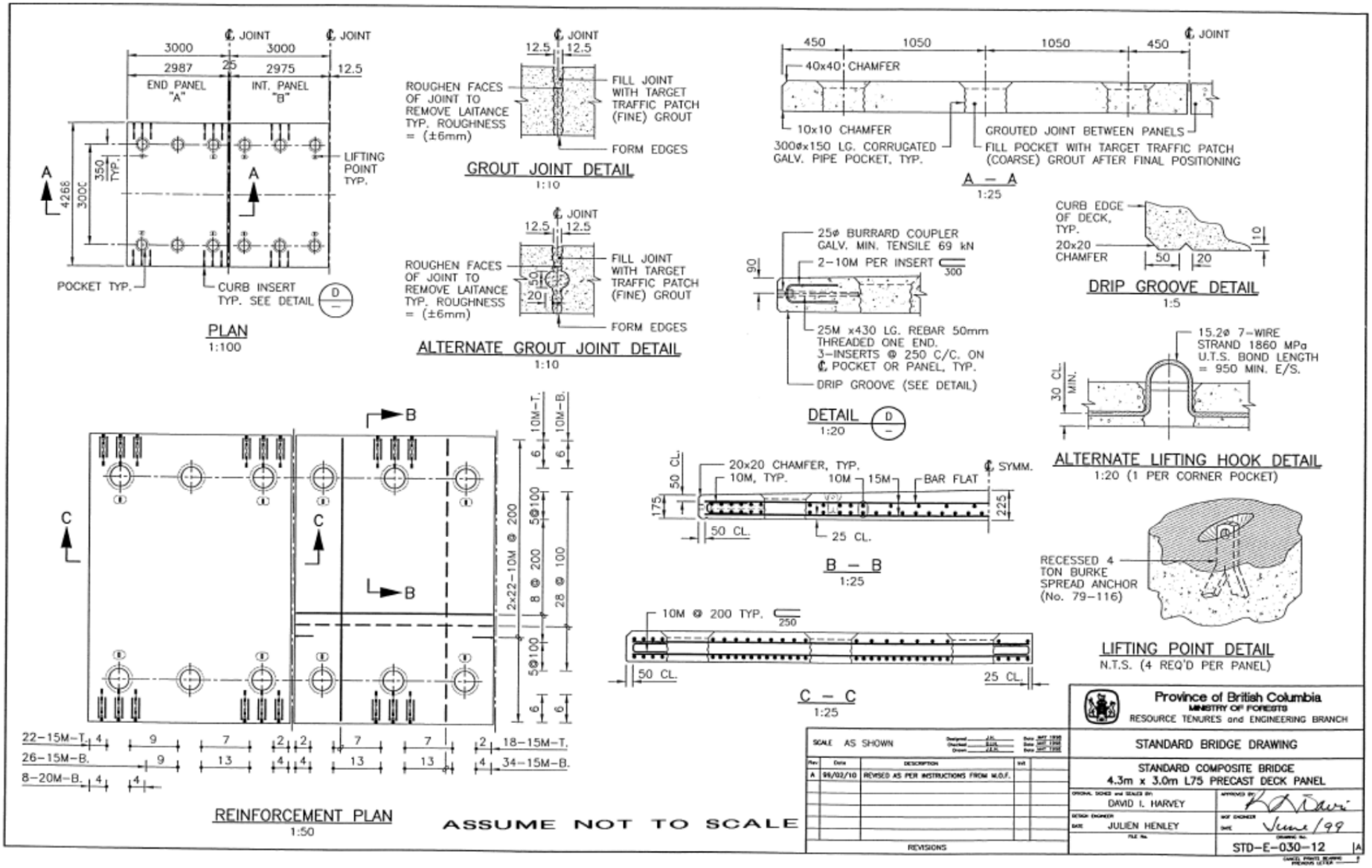

Figure 4: Original drawing of standard composite deck

Concrete Panel for Experimental Investigations

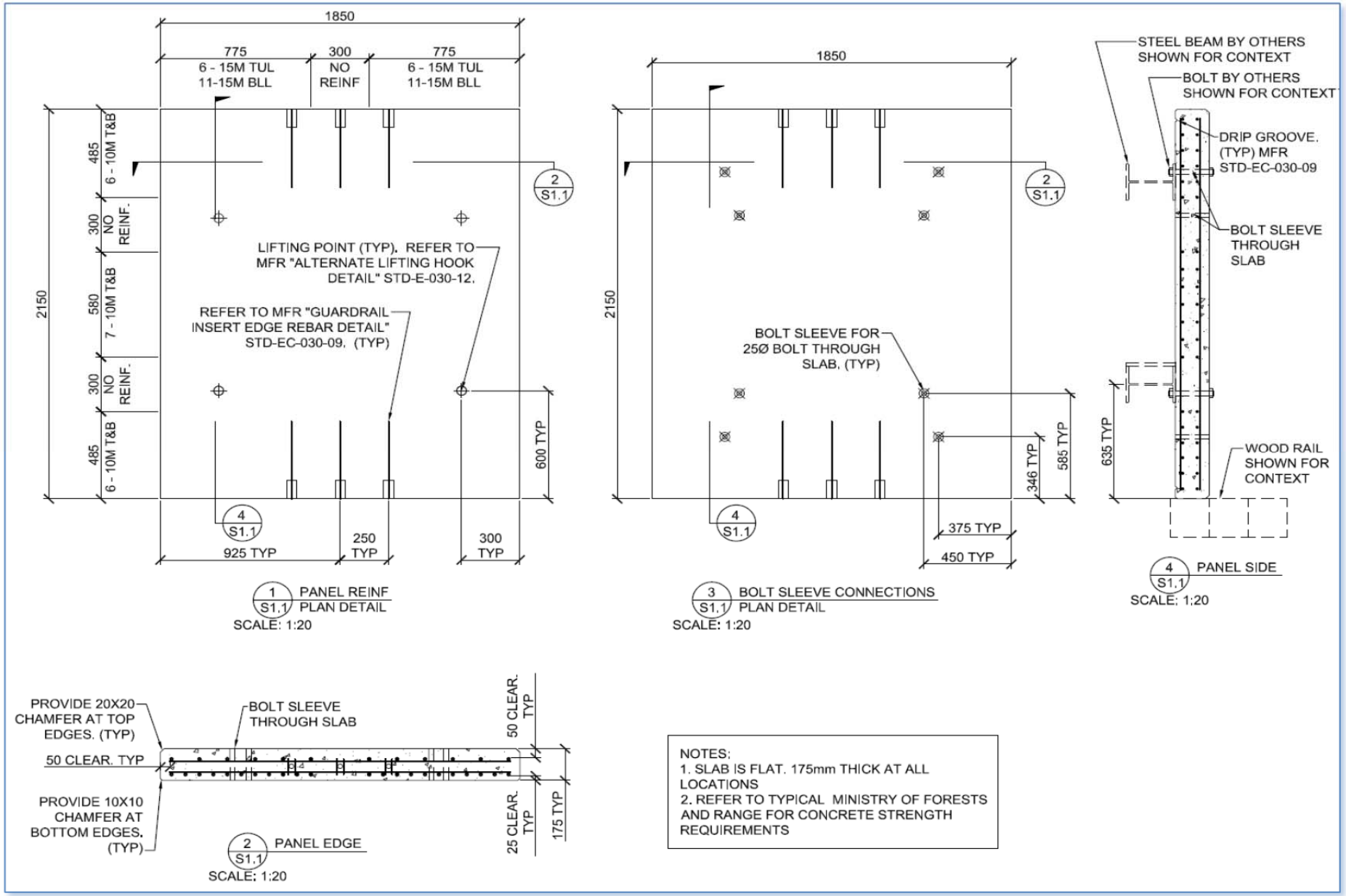

Figure 5: Standard test panel, two edges useful for testing

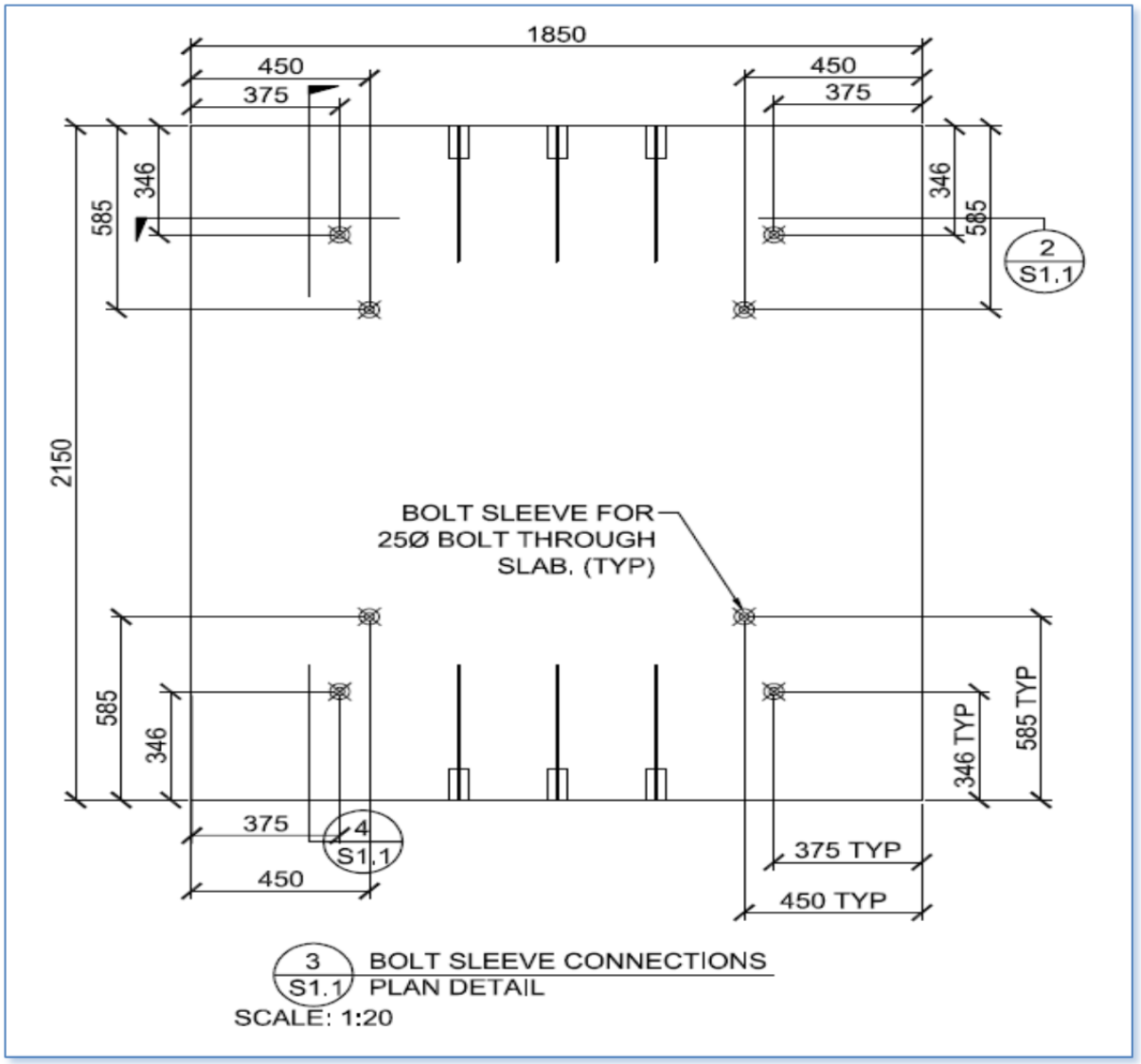

Figure 6: Bolt sleeve locations on concrete test panel

Schematics for test set-up

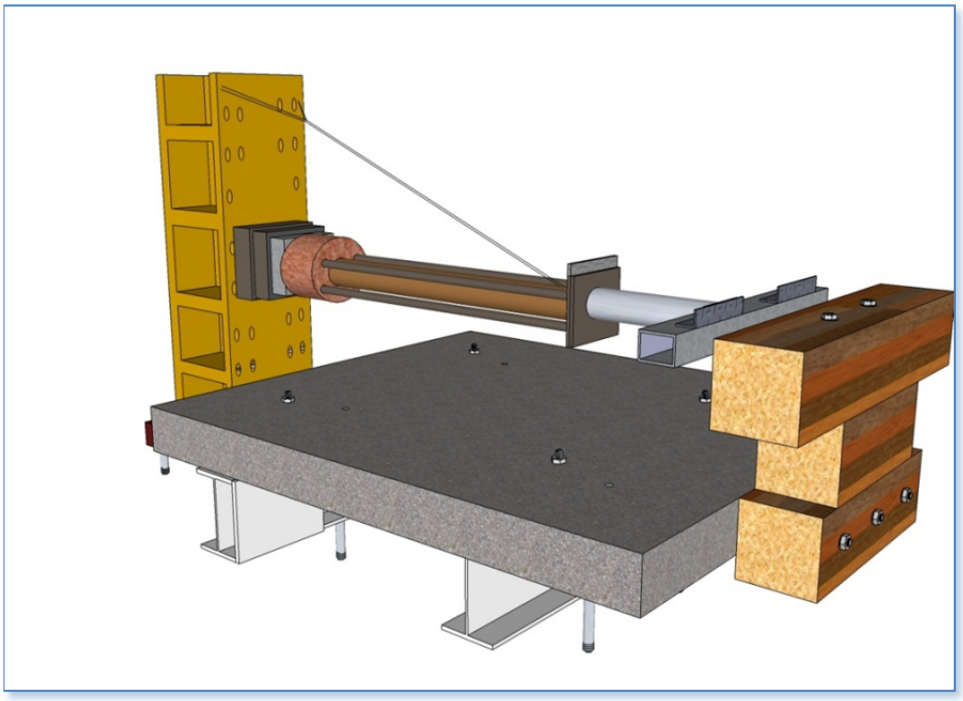

Figure 7: 3-D view of test set-up for standard timber barrier

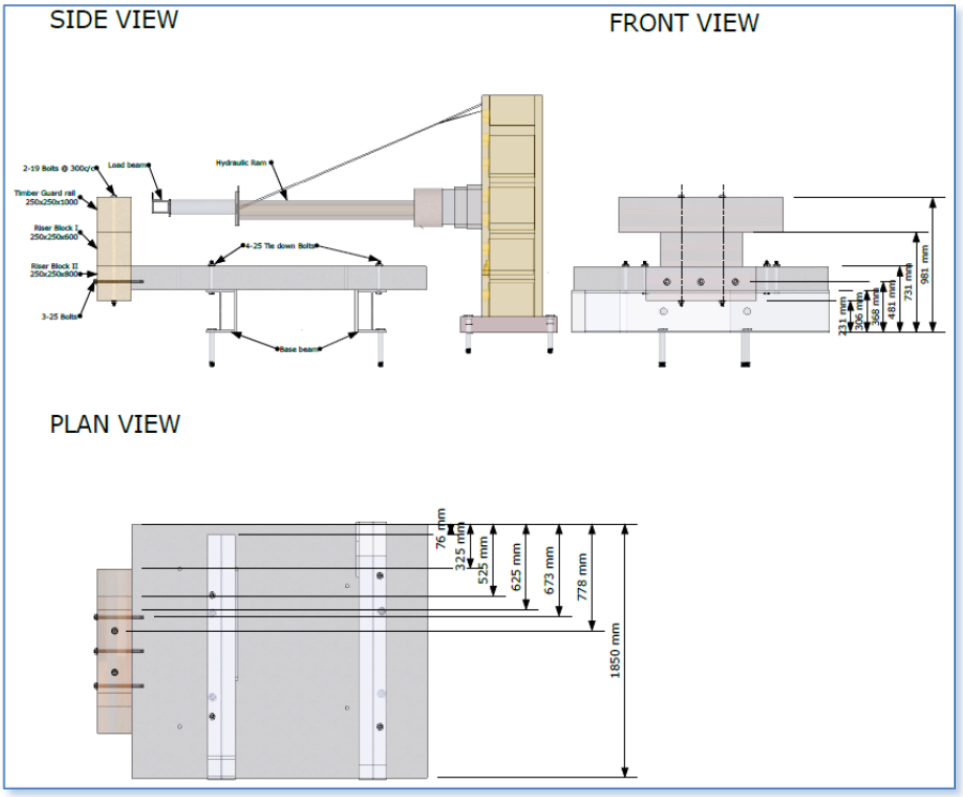

Figure 8: Set-up for testing with standard timber barrier

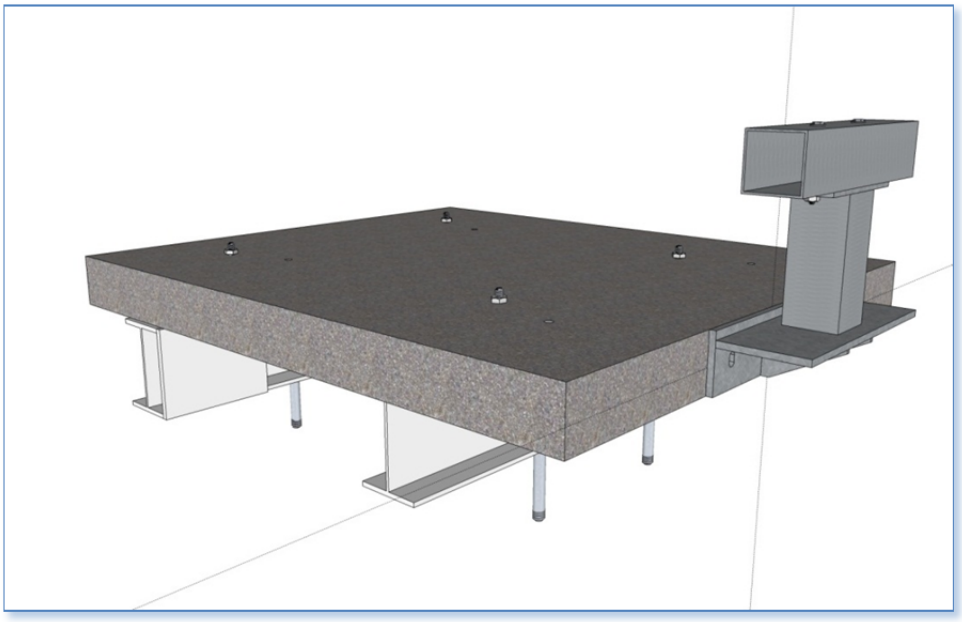

Figure 9: 3-D view of set-up for steel post curb attachment

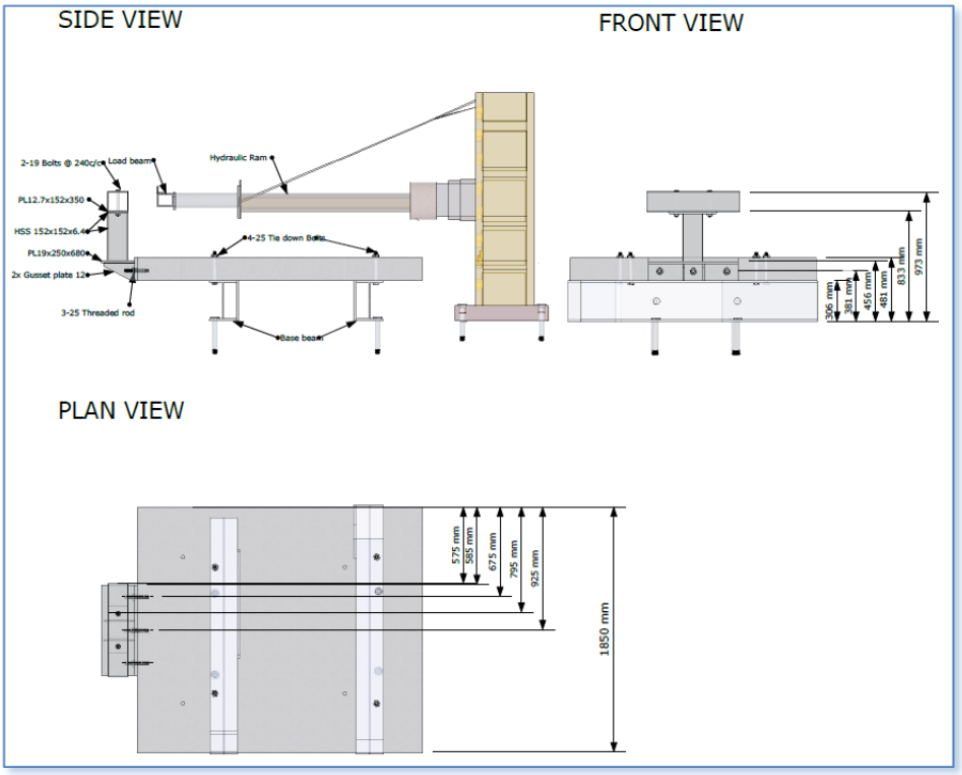

Figure 10: Set-up for testing with steel posts SS

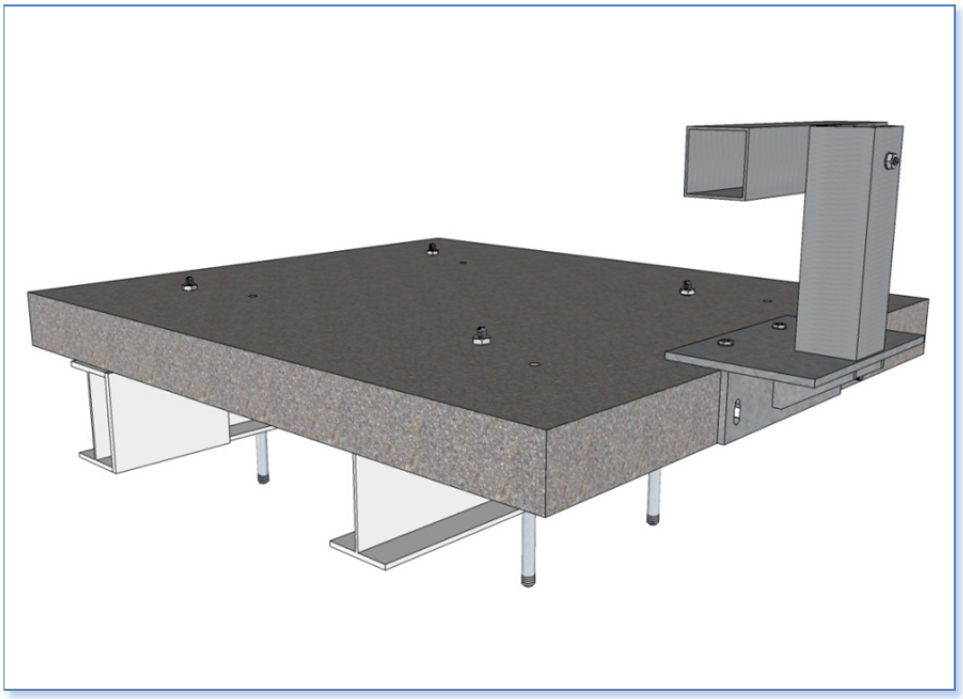

Figure 11: 3-D view of set-up for (modified) steel post curb attachment

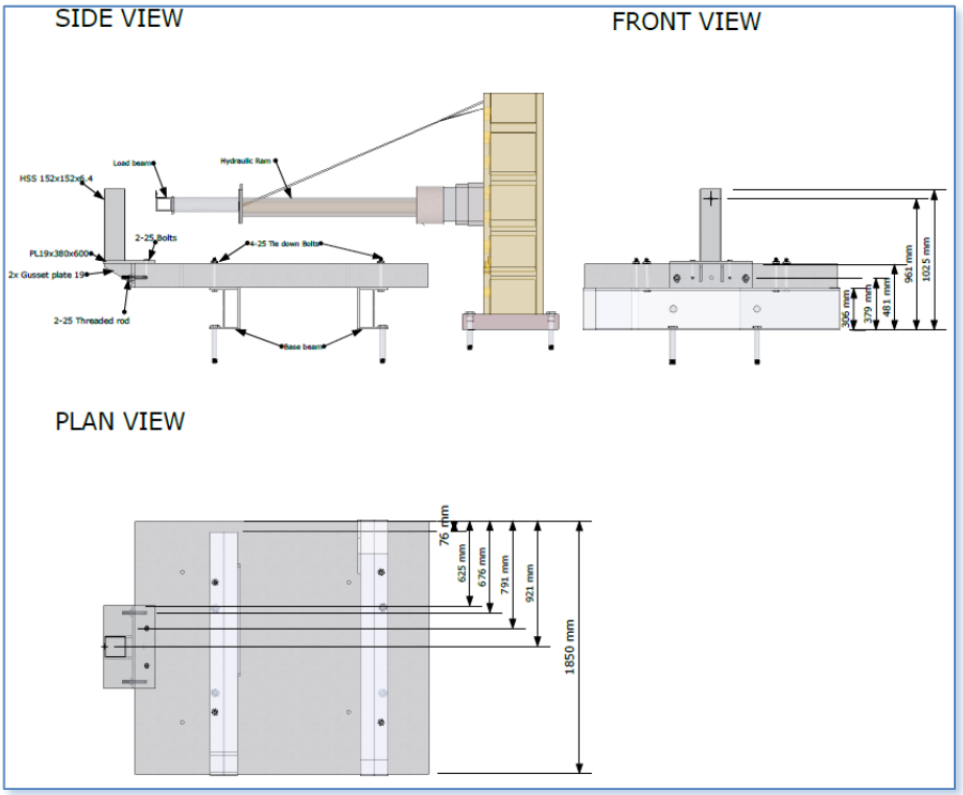

Figure 12: Set-up for testing with (modified) steel posts SM

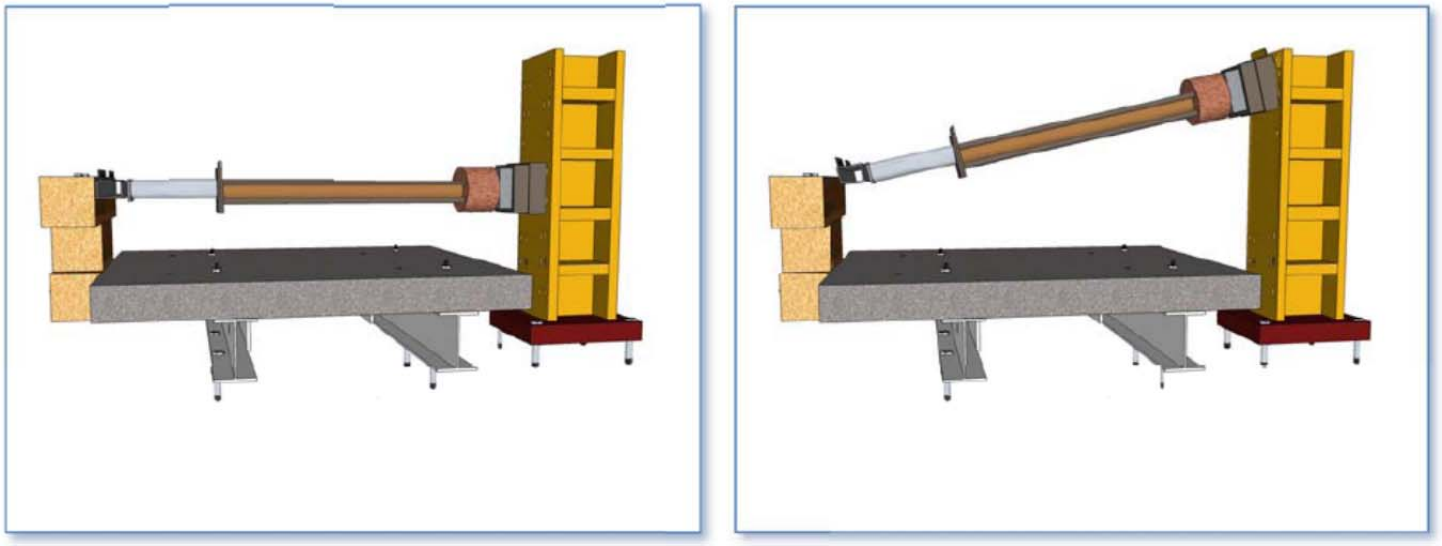

Figure 13: Alteration of Test Set-up (in order to achieve vertical load component) –horizontal and vertical loads are applied progressively and at constant ratio.

Versions of Posts as Tested

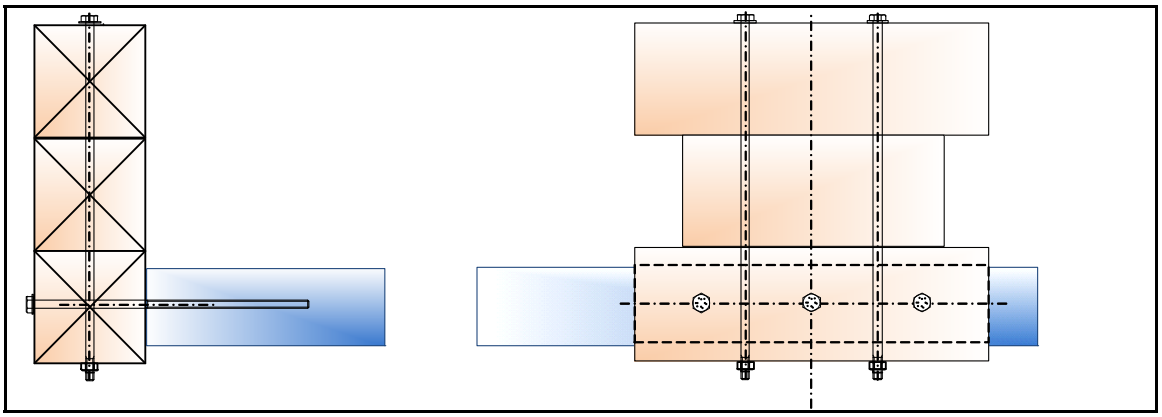

Figure 14: Timber Curb Block

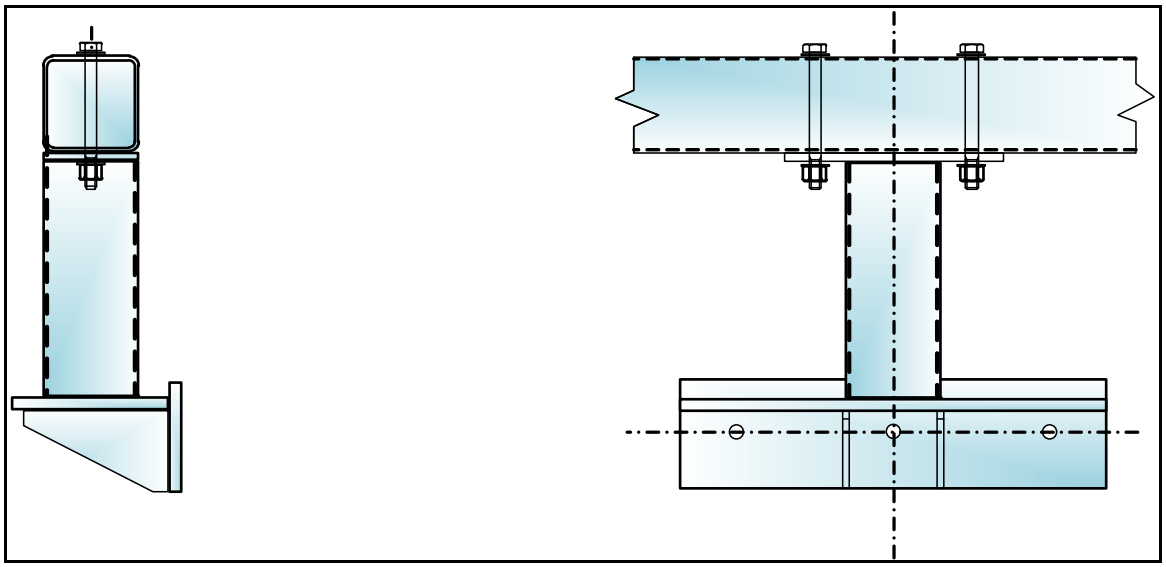

Figure 15: Steel Post System #2 – SS

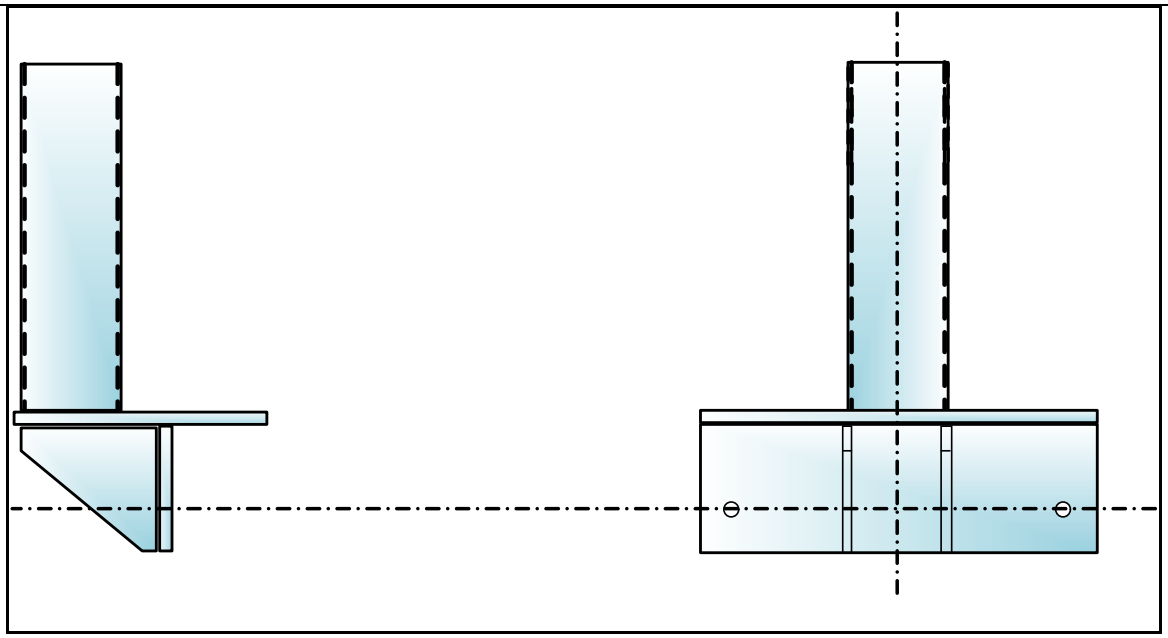

Figure 16: Steel tall post system #3 – SM

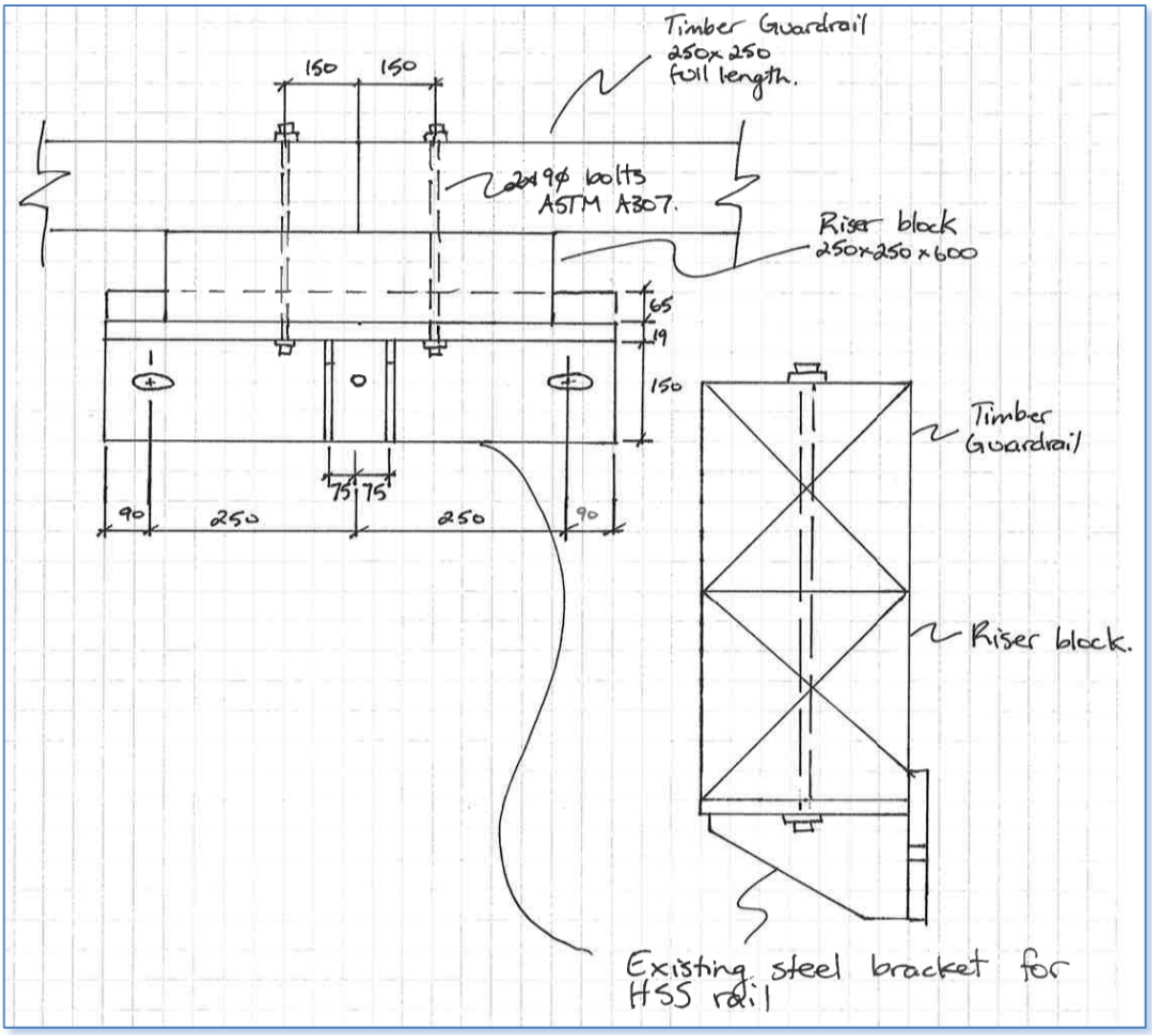

Figure 17: Modification I. as suggested by G. Fraser,
(mail attachment Fri, Mar 4, 2011 at 10:38 AM, Asssociated Engineeering)

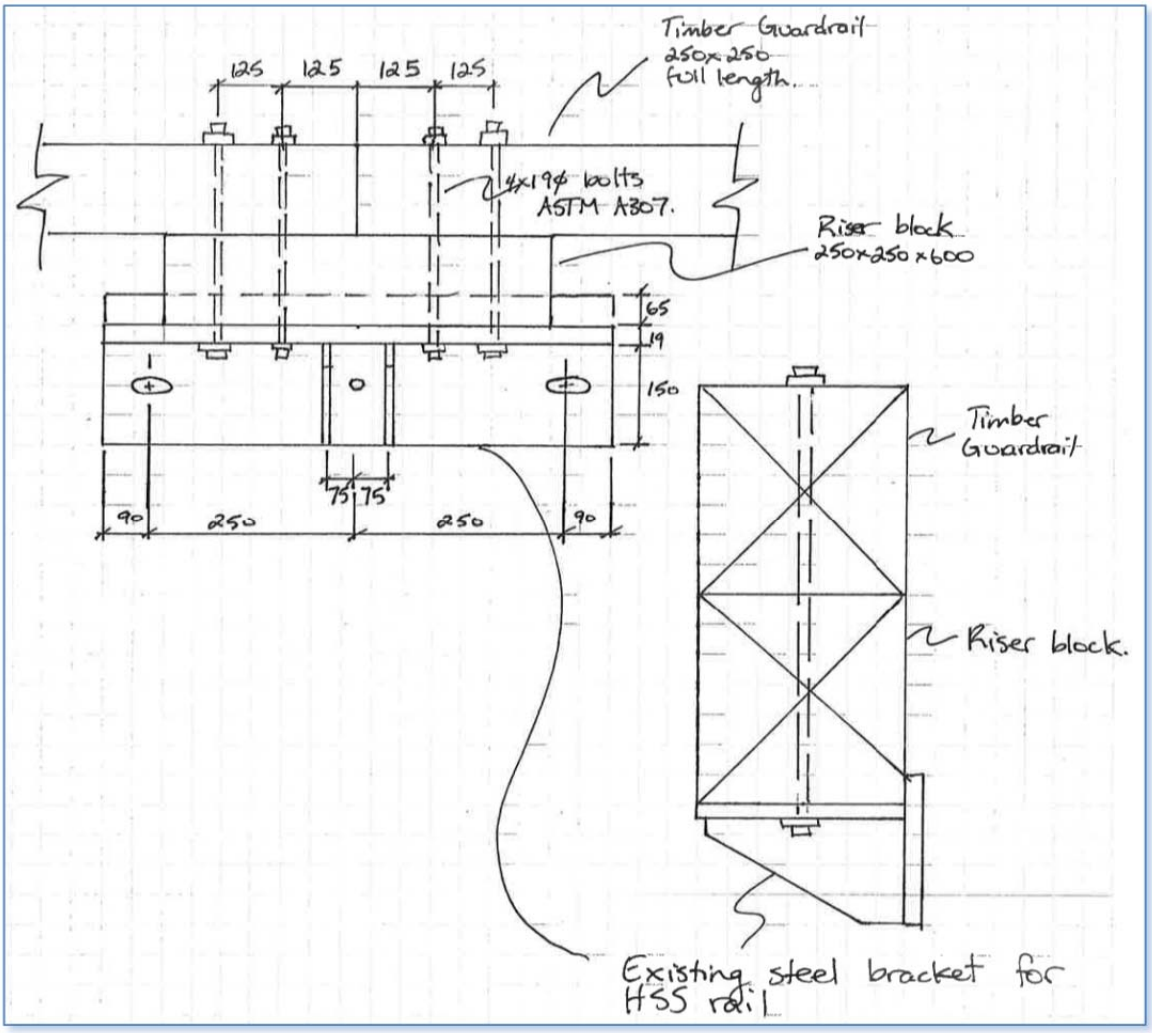

Figure 18: Modification II. as suggested by G. Fraser,
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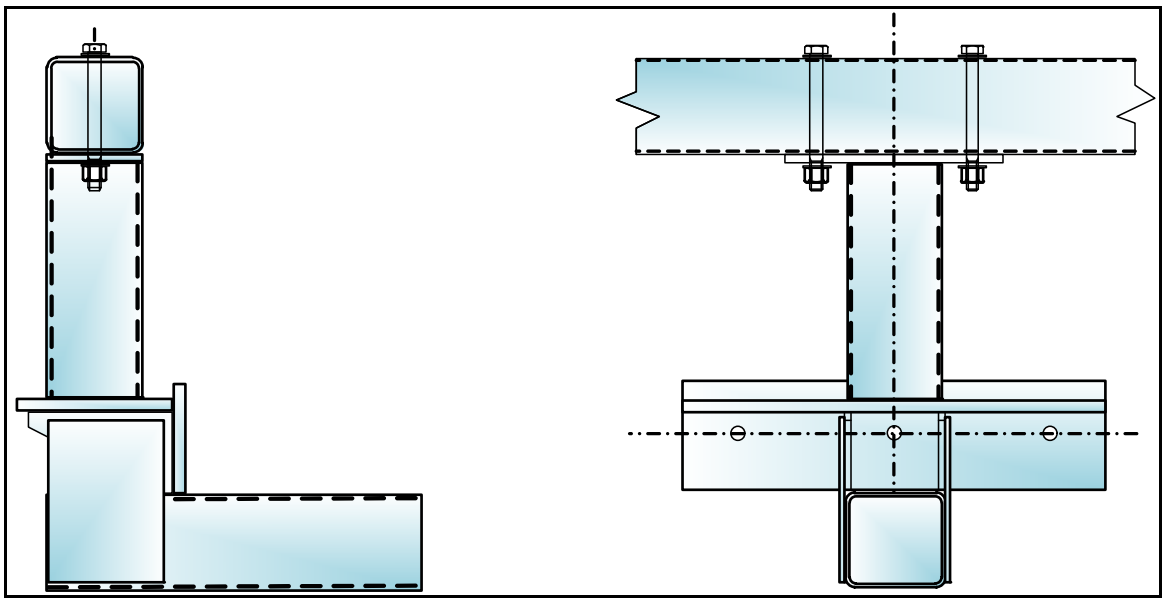


Figure 19: Modified Steel Post System #2- SS with Knee Bracket as used in experiment

Test Results

Loading Considerations

Iterated goal for test series: establish guard rail capacities under boundary conditions close to as prescribed by existing codes, either CHBDC or AASHTO.

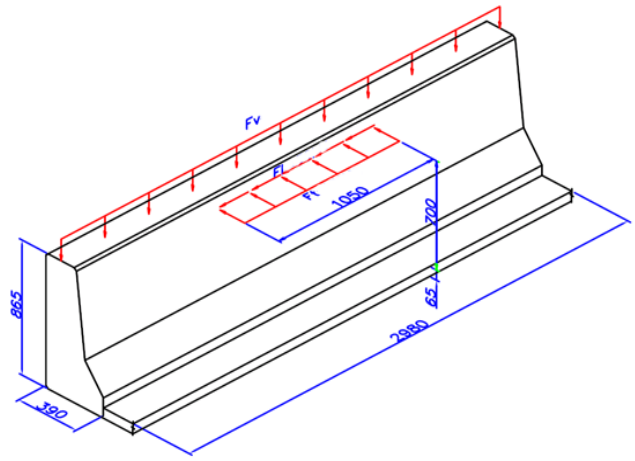


Figure 20: Example Traffic Barrier Loads and Locations as per CHBDC for (PL-2), Clause 3.8.8.1., in Isometric View

The following governing code requirements exist:

AASHTO LRFD Factored Railing Test Level Forces			
Design Forces[2]	TL-1 (kN)[1]	TL-2 (kN)[1]	Application Length (mm)
Transverse Load, kN	60	120	1220
Longitudinal Load, kN	20	40	1220
Vertical Load, kN	20	20	5500
Load Height, mm	460	510	

Figure 21: Table with Design Forces as per AASHTO LRFD Barrier Design Levels (courtesy of Associated Engineering, Vancouver, 28.02.2011) – [1] Based on maximum force applied to barrier

CHBDC Factored Railing Performance Level Forces		
Design Forces	PL-1 (kN)[2]	Application Length (mm)
Transverse Load, kN	85	1200
Longitudinal Load, kN	34	1220
Vertical Load, kN	17	5500
Load Height, mm	600	

Figure 22: Table with Design Forces as per CHBDC Performance Levels (courtesy of Associated Engineering, Vancouver, 28.02.2011) – [2] Based on maximum force transferred to anchorage

The expressions transverse, longitudinal, vertical relate to the deck lane direction. Application height of loading as per CHBDC cannot be achieved. Maximum height of each post system will be utilized (see drawings above).

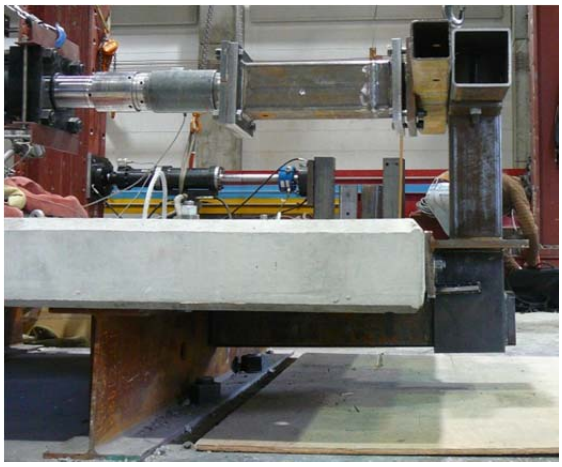
It should be noted that the ratio of transverse load to vertical load for on guard rail post is 6:1.08 for AASHTO and 5:1 for CHDBC.

Test Schedule and Notes

Number	Conc. Panel Number	Sequence	Date	Capacity in [kN]	Notes
General					Concrete decks exceed the required 28 day compressive strength of 35 MPa by more than 50% (tested: 56 MPa). Concrete Cylinder Test Result Sheet (**doc20110210095748.pdf**) is available.
Timber curb block					• Capacities correlate nicely for tests • Failure mode similar to previous tests (two blocks) on steel base • Ultimate capacities similar to previous tests (two blocks) on steel base
1.1	#2	3	17/02/2011	19.7	• Failure in timber • Relatively long sustained maximum load due to large crushing deformation in timber
1.2	#2	4	17/02/2011	23.5	• Failure in timber • Relatively long sustained maximum load due to large crushing deformation in timber • First plateau at 17 kN, then rise to 23.5 kN
1.3	#3	7	01/03/2011	26.3	• Vertical/horizontal loading ratio = 1.08/6 • Horizontal and vertical load component • Significant failure in timber at 23 kN, then increase due to change in geometry to 26 kN (block bends over and is increasingly loaded in tension) • Relatively long sustained maximum load due to large crushing deformation in timber
1.4	#3	8	02/03/2011	23.5	• Vertical/horizontal loading ratio = 1.08/6 • horizontal and vertical load • Failure in timber • Bottom block of timber splits vertically along the three bolt holes

Steel post system #2 - SS					• Capacities correlate nicely for both A307 and A325 bolts respectively • Failure mode similar to previous tests (two blocks) on steel base • Ultimate capacities similar to previous tests (two blocks) on steel base
2.1	#1	1	08/02/2011	64.2	• Use of A307 bolts in horizontal inserts • Bolts rupture in tension • Failure mode different from field observation
2.11	#2 recycled	5	18/02/2011	68.1	• Reuse of concrete panel from test 2.1 – which was practically unscathed, A325 bolts • Failure mode with spalling of concrete in vicinity of bolt inserts • Stress concentration at sleeve-to-rebar interface
2.2	#1	2	08/02/2011	65.7	• Use of A307 bolts in horizontal inserts • Bolts rupture in tension • Failure mode different from field observation
2.21	#2 recycled	6	18/02/2011	57.3	• Reuse of concrete panel from test 2.1 – which was practically unscathed, A325 bolts • Failure mode with spalling of concrete in vicinity of bolt inserts • Stress concentration at sleeve-to-rebar interface, causes larger variability in ultimate capacity
2.3	#3 recycled	9	03/03/2011	57.3	• use of A307 • Vertical/horizontal loading ratio = 1.08/6 • horizontal component variable by hydraulic ram
2.4	#3 recycled	10	03/03/2011	55.8	• use of A307 • Vertical/horizontal loading ratio = 1.08/6 • horizontal component variable by hydraulic ram

Steel tall post system #3 - SM					• Hilti drill did not work due to heavy steel reinforcement, therefore core drilling of holes in deck is underway • These tests will cause the deck to fail
3.1	#4	11	09/03/2011	50.1	• A307 bolts • horizontal loading
3.2	#4	12	10/03/2011	68.7	• A307 bolts • horizontal loading
3.3	#5	13	11/03/2011	67.2	• A307 bolts • horizontal loading
3.4	#5	14	11/03/2011	58.3	• A307 bolts • horizontal loading

Steel base, timber riser – two and four bolts vertical					• These barrier versions were not part of the original schedule and must be seen as purely experimental
4.1	#6	16	17/03/2011	36.1	• steel base, two vertical bolts • Vertical/horizontal loading ratio = 1.08/6
4.2	#6	17	17/03/2011	42.2	• steel base, four vertical bolts • Vertical/horizontal loading ratio = 1.08/6
Steel post system #2 – SS Knee - Bracket					• knee-bracket, three horizontal A307 bolts • horizontal loading
4.3	#6 recyc.	18	17/03/2011	154.8	•
4.4	#6 recyc.	19	17/03/2011	124.1	•
4.5	#3 recyc.	15	11/03/2011	164.4	•

Load Carrying Methods for the Various Systems

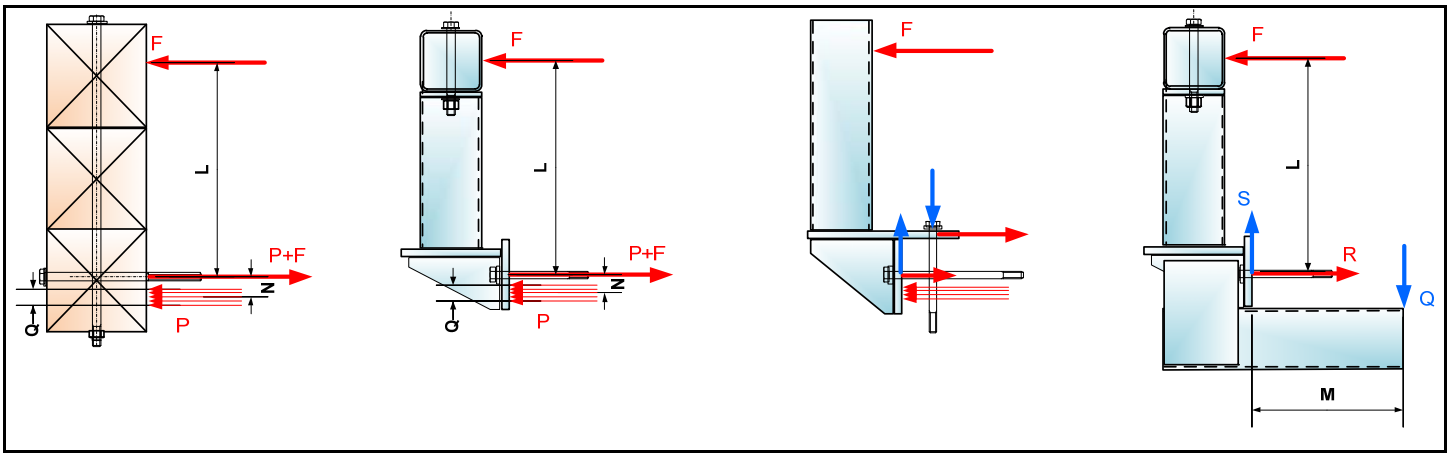


Figure 23: Free Body Diagrammes for the Various Systems
(from left to right: Timber Curb Block, Steel Post, Steel Post with Deck Plate, Steel Post with Knee Bracket)

From the Free Body Diagrammes one can easily see that the compression zone (of height **Q**) on the face of the concrete panel has a relatively small distance (**N**) to the reaction in the horizontal bolt in the concrete. The contact surface height of the concrete deck is reduced by the chamfer and weakened by the drip groove to 175 [mm]. As expected the failures occurred at those location during testing and can be observed in the following.

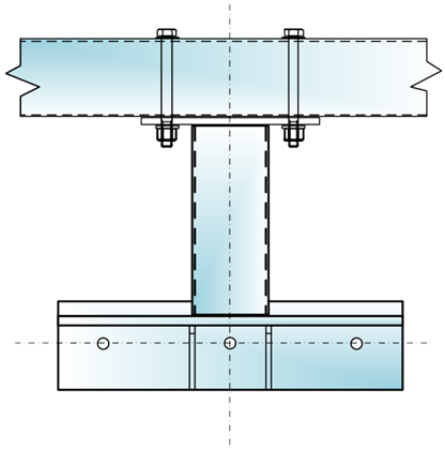
The steel post system with a horizontal deck plate has a more complex free body diagramme, which cannot be fully explained analytically, because it is depending on the individual local stiffnesses. The tests showed that not the posts fail but the concrete deck peel off due to the vertical bolts being too close to the edge of the deck.

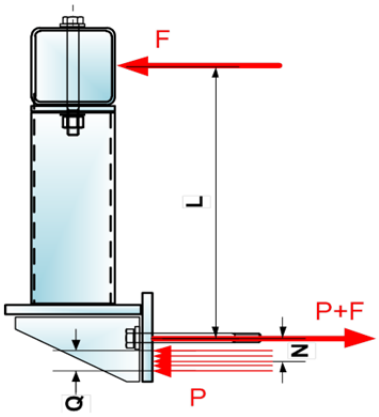
The knee-bracket system uses the horizontal arm to counteract the applied moment (distance **L** x force **F** = distance **M** x reaction **Q**) and thus prevents the magnification of the horizontal reaction in the in the bolt in the deck (**R**). The horizontal bolts experience an addition shear force (**S**).

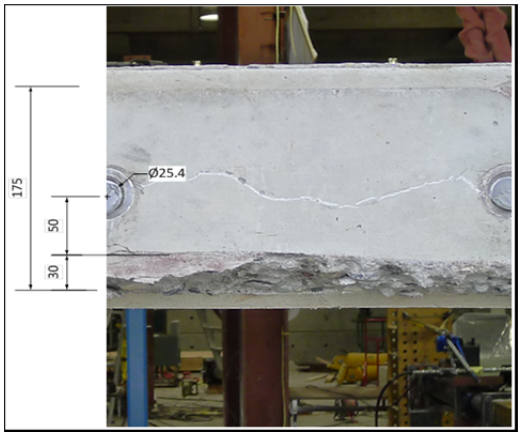
Analysis for Knee Bracket Steel Post

Steel and Timber Bracket	Project	Performance Prediction for Guard Rail System for Bridges	DATE	04/25/11
			TIME	8:01 AM

DESCRIPTION







INPUT

Horizontal Bolt

Bolt length in concrete	bl	=	250	mm
Bolt depth from bottom of concrete surface	d_c	=	90	mm
Bolt Diameter	d_v	=	25	mm
Rebar Diameter	d_r	=	25	mm
lever arm unspalled	l_un	=	65	mm
lever arm spalled	l_sp	=	50	mm

stress concentration factor at rebar to sleeve	str_c	=			1.5	
Plate thickness	t_p	=			19	mm
Specified Minimum tensile strength (A 307)	F_307	=			414	Mpa
Specified Minimum tensile strength (A 325)	F_325	=			830	Mpa
Specified Minimum tensile strength rebar, G30.18-M92 Gr400R or G30.18-M92 Gr400W	F_bar	=			800	Mpa
Number of bolts	n_b	=			3	
Concrete Section Dimensions (crushing area) - see sketch above						
Width	a	=			680	mm
Height Q	b	=			30	mm
Distance Resultant to Reaction in Bolt N	dr	=			65	mm
Reinforcement Ratio (in percent)	RR	=			1	
Concrete Strength	f'_c	=			56	Mpa
Compression Factor	ά	=			0.85	
Post Sizes						
weld size	ws	=			8	mm
post width	pw	=			152	mm
post depth	pl	=			152	mm
post height	ph	=			376	mm
section modulus of HSS152x152x6.4	Sp	=			16600	mm^3
Metal Rail HSS152x152x6.4						
Length	L_r	=			1	m
specified yield strength	Fy	=			350	Mpa
ultimate strength of plate	Fu	=			450	Mpa
ultimate strength of weld metal	Xu	=			480	Mpa
performance factor	phi	=			0.9	
performance factor weld	phiw	=			0.8	

MEASURED PARAMETERS

from Experiment and Free Body (see above)						
applied (failure) load	**F**	=		=	68	kN
horizontal reaction in bolts	**R**	=	P+F	=	616	kN
reaction concrete	**P**	=	F*L/N	=	548	kN
distance between load F and bolt	**L**	=		=	524	mm
distance between load P and bolt	**N**	=	dr	=	65	mm

CALCULATIONS

TRANSVERSE LOAD

initial Failure Mode: Crushing of Concrete at Panel Face (no steel reinforcement)

triangular stress distribution	fmx	=	f'c/2	=	28	Mpa
reduction through drip groove	redd	=			0.50	

effective Concrete Gross Area (1/2 factor)	A_g	=	a*b*redd	=	10200	mm^2
Concrete Crushing load	CC	=	ά*fmx*A_g/1000	=	**243**	kN
Failure Mode: Crushing of Concrete						
Concrete Gross Area	A_gc	=	a*b	=	20400	mm^2
Steel Area	A_st	=	(A_gc*RR)/100	=	204	mm^2
Concrete Crushing load	CCc	=	(ά*f'c*(A_g-A_st)+(A_st*Fy))/1000	=	**547**	kN
Failure Mode: horizontal A307 bolts in tension						
Tensile Resistance of A307 bolts	R_307	=	(n_b*F_307*0.75*PI()*d_v^2/4)/1000	=	**457**	kN
Bending Resistance	M_rb1	=	(R_307*l_un)/1000	=	**30**	kNm
Lateral Load Allowance for three bolts	P_vt1	=	(M_rb1*1000)/(ph+pw/2)	=	**66**	kN
Failure Mode: horizontal A325 bolts in tension						
Tensile Resistance of A325 bolts	R_325	=	(n_b*F_325*0.75*PI()*d_v^2/4)/1000	=	**917**	kN
Bending Resistance	M_rb2	=	(R_325*l_sp)/1000	=	**46**	kNm
Lateral Load Allowance for three bolts	P_vt2	=	(M_rb2*1000)/(ph+pw/2)	=	**101**	kN
Failure Mode: horizontal rebar in tension						
Tensile Resistance of rebar (weld fails)	R_bar	=	(n_b*F_bar*0.75*PI()*d_r^2/4)/1000/str_c	=	**589**	kN
Bending Resistance	M_rb3	=	(R_bar*l_sp)/1000	=	**29**	kNm
Lateral Load Allowance for three bolts	P_vt3	=	(M_rb3*1000)/(ph+pw/2)	=	**65**	kN

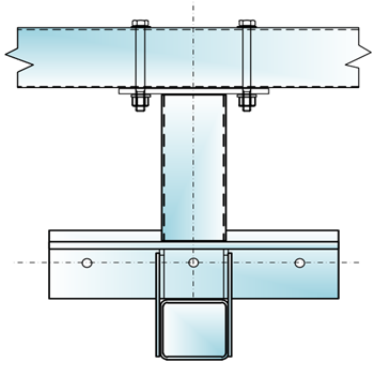
CONCLUSIONS

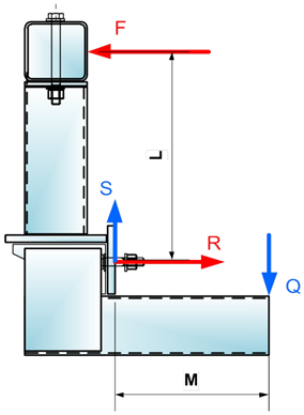
Timber Bracket: Failure occurred in crushing of lower section of base timber block. Subsequently the entire block split vertically.

Steel Bracket: Failure occurred in crushing of the lower section on the face of the concrete panel, crushing area was reduced by influence of drip groove.

Knee Bracket Steel Post	Project	Performance Prediction for Guard Rail System for Bridges	DATE	04/24/11
			TIME	6:22 PM

DESCRIPTION





INPUT

Horizontal Bolt

Bolt length in concrete	bl	=	250	mm
Bolt depth from bottom of concrete surface	d_c	=	90	mm
Bolt Diameter	d_v	=	25	mm
Plate thickness	t_p	=	19	mm
Specified Minimum tensile strength (A 307)	F_ub	=	414	Mpa
Number of bolts	n_b	=	3	

Post

weld size	ws	=	8	mm
post width	pw	=	152	mm
post depth	pl	=	152	mm
post height	ph	=	465	mm
section modulus of HSS152x152x6.4	Sp	=	16600 0	mm^3
specified yield strength	Fy	=	350	Mpa
ultimate strength	Fu	=	450	Mpa
strength of electrode	Xu	=	490	Mpa
performance factor	phi	=	0.9	
performance factor welds	phiw	=	0.67	

MEASURED PARAMETERS

from Experiment and Free Body (see above)

applied (failure) load	**F**	=		=	160	kN
horizontal reaction in bolts	**R**	=	F	=	160	kN

vertical reaction in bolt	**S**	=	Q	=	**140**	kN
reaction on knee bracket	**Q**	=	F*L/M	=	**140**	kN
distance between load P and bolt	**L**	=		=	**524**	mm
length of knee bracket	**M**	=		=	**600**	mm

CALCULATIONS

Failure Mode: Horizontal Bolts in Shear and Tension

Bolt Shear Resistance	V_rb	=	n_b*(F_ub*0.6*3.14*d_v^2/4)/1000	=	**366**	kN
Bolt Tensile Resistance	T_rb	=	n_b*(F_ub*0.75*3.14*d_v^2/4)/1000	=	**457**	kN
check Combined Shear/Tension Resistance	check	=	(S/V_rb)^2+(F/T_rb)^2	=	**26.9%**	

OBSERVATIONS FROM TESTING

Failure occurred at inside knee corner in weld due to tensile stress concentration, a failure mode that would need FE analysis to be computed.

Summary

Test Data Summary

12 planned, 7 added = 19 tests performed, 1200 data per each test record

700 photos

20 videos, 2GB each

10 spreadsheets computation

50 drawings of test and specimen details

Discussion

Failure is defined by **reaching the maximum capacity**. When the specimens are subjected to continued loading beyond the point of maximum capacity; failure modes as seen in the field might occur. This is a **post-failure appearance**, and does not have any meaning for the ultimate capacity of a rail guard. Unfortunately, post-failure appearance often leads to rather wrong conclusions about the structural behaviour and may even direct design decisions into the wrong direction. As a simple example, one can look at a crashed airplane. Surely, everybody will conclude, a plane with those deformed wings won't be able to fly, but the real reason was ....?

Result Summary from 2010

As reported in "Experimental Evaluation of Guard Rail System for Bridges", Final Report for Ministry of Forests and Range, Engineering Branch, Field Operations Division. Description of detailed system characteristics, failure modes, etc. can be found in the above report. An in-depth comparison of both tests done in 2010 and 2011 would be desirable, but would go beyond the scope of this report. A separate study is recommended and current authors would be available to aid in this work.

Summary of Experimental Results 2010

	Experimental results (averaged values)			CHBDC Performance Level 1		
	Resisting load [kN]	Load app. length [m]	Dist. load [kN/m]	Load [kN]	Load app. length [m]	Dist. load [kN/m]
System 1	42.63	1.1	38.75	62.5	1.25	50.0
System 2	37.97	1.1	34.52	62.5	1.25	50.0
System 3	118.15	1.1	107.41	62.5	1.25	50.0
System 4	173.15	1.1	157.41	62.5	1.25	50.0

Summary of Experimental Results 2010

	Experimental results (averaged values)			CHBDC Performance Level 2		
	Resisting load [kN]	Load app. length [m]	Dist. load [kN/m]	Load [kN]	Load app. length [m]	Dist. load [kN/m]
System 1	42.63	1.1	38.75	125	1.1	113.64
System 2	37.97	1.1	34.52	125	1.1	113.64
System 3	118.15	1.1	107.41	125	1.1	113.64
System 4	173.15	1.1	157.41	125	1.1	113.64

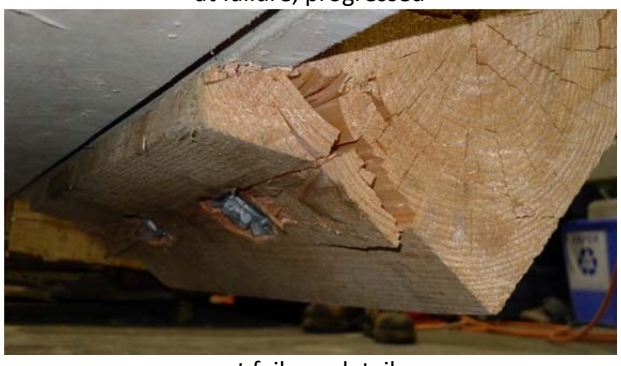
Result Summary from 2011

Note that experimental results cannot directly be compared due to the difference in barrier and deck representation. See commentary after the tables.

AASHTO LRFD Factored Test Level Forces			
Design Forces	Design Levels		
	TL-1 [kN]	**TL-2 [kN]**	**Load Application Length [m]**
Transverse Load	60	120	1.2
Longitudinal Load	20	40	1.22
Vertical Load	20	20	5.5
Load Application Height [m]	0.46	0.51	

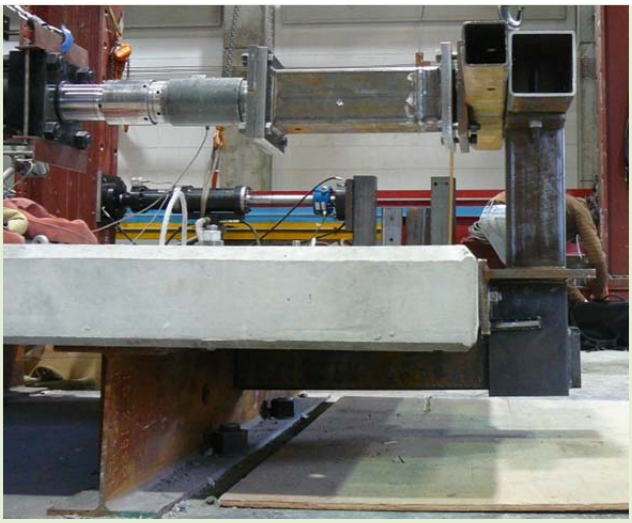
CHBDC Factored Railing Performance Level Forces		
Design Forces	Design Levels	
	PL-1 [kN]	**Load Application Length [m]**
Transverse Load	85	1.2
Longitudinal Load	34	1.22
Vertical Load	17	5.5
Load Application Height [m]	0.6	

Summary of Experimental Results 2011

	Capacities (averages) in [kN]	Failure Description	Photo
Timber Curb Block Tests 1.1 – 1.4	23.25	failure in timber, vertical crushing of blocks at the bolt washers, bottom block of timber splits vertically along the three bolt holes	during testing at failure, begin at failure, progressed  at failure, detail

Steel Post System 2 **Tests 2.1 – 2.4**	61.4	A307: bolts rupture in tension and spalling of concrete in vicinity of bolt inserts (failure shown in second photo to the right) A325: spalling of concrete (failure shown in third photo to the right)	 during testing at failure, A307 bolts at failure, A325 bolts
Steel Tall Post System 3 **Tests 3.1 – 3.4**	61.1	deck fails in block shear	 during testing

			 at failure
Steel Base, Timber Riser with 2 bolts Tests 4.1	36.1	failure in timber, vertical crushing of blocks at the bolt washers, bottom block of timber splits vertically along the three bolt holes	 during testing at failure

Steel Base, Timber Riser with 4 bolts Tests 4.2	42.2	failure in timber, vertical crushing of blocks at the bolt washers, bottom block of timber splits vertically along the three bolt holes	at failure
Steel Post System 2, with Knee – Bracket Tests 4.3 – 4.5	147.1	a) welds in knee-bracket fail in corner of elbow where stresses are concentrated (not shown), b) after reinforcing of knee with gusset plates and larger welds: spalling of large area of deck (shown)	 during testing at failure at failure

In the case of System 1 from 2010, the failure mode was identical to Timber Curb Block from 2011, therefore the numbers can be compared. System 1 involved the failure of two curb blocks (one full and one reduced set), and the capacity of 42.63/2 = 21.3 [kN] compares nicely with the average of 23 [kN] from the equivalent 2011 tests.

In the case of failure involving the concrete deck one can only say that the 2010 tests are upper bound values, in case the deck would have survived.

Conclusions

The experimental investigations were designed to establish a baseline on guard rail/connection systems. Testing set-up and procedures were aiming at reasonable representations of the intentions of existing codes.

Load tests were mostly performed with horizontal load component only. The addition of a vertical load component of the amount prescribed by the codes did not make any considerable difference in the ultimate capacities for the curb post systems. The vertical component can be the governing design parameter for the rails, depending on rail design.

General Comments to Concrete Deck:

- Concrete compressive test results indicated that panel compressive strengths range from 51 to 56 MPa; specified design strength is 35 MPa; the ministry advises that fabricators typically use a mix to allow removing of forms and shipping sooner which results compressive strengths generally exceeding the design requirements.
- The drip groove considerably influenced the capacities when spalling was involved as failure mode. Omission of drip groove seems to be desirable when full deck capacities are required.

Timber curb block:

- Capacities correlate nicely for both A307 and A325 bolts respectively
- Failure mode similar to previous tests (two blocks) on steel base
- Ultimate capacities similar to previous tests (two blocks) on steel base
- Failure of curb blocks did not affect the concrete deck, no damage to deck edge

Steel Post System 2 – SS:

- Tests were conducted with both A307 and A325 bolts
- Capacities correlate nicely for both A307 and A325 bolts respectively
- A307 bolts: Bolts rupture in tension, good repeatability
- A325 bolts: Spalling of concrete in vicinity of bolt inserts, larger variations in capacities
- Failure mode similar to previous tests (two blocks) on steel base
- Ultimate capacities similar to previous tests (two blocks) on steel base

Steel Tall Post System 3 – SM:

- Vertical bolts through deck cause the deck to fail by block shear failure
- Holes for vertical bolts need to be placed at casting of deck, drilling or coring of holes is difficult due to the existing rebar
- Horizontal bolts did not contribute much to capacity of post system
- Test panel fabrication did not include the additional U-bars that should have gone around the inserts for the vertical bolts through the deck.
- These U-bars would likely have provided some additional capacity up to the maximum shear/tensile capacity of the vertical bolts as tested in 2010 (up to 90 [kN] – prorated).

Steel Base, Timber Riser – with two and four vertical bolts through timber:

- This system was not part of the initial scope of the investigations and added on advice of engineers from Associated Engineering, Vancouver
- No particular advantage when compared to all timber curb block
- Capacities similar to all timber curb block
- Doubling of vertical bolts results in 25% increase in capacity

Steel Post System 2 – SS with Knee–Bracket:

- This system was not part of the initial scope of the investigations and added by the investigators using surplus material and own UBC resources
- It was the only system able to comply with and perform above the code requirements of AASHTO LRFD Barrier Design Levels TL-1 and TL-2 as well as CHBDC Performance Levels PL-1
- It can be designed using a theoretical approach (methods of plastic design), because failure occurs in the steel
- Strength, stiffness, and energy absorption capacities can be designed according to the particular need
- Damage to concrete deck can be avoided even at failure of Knee-Bracket (if designed properly)
- Using little additional steel material (little extra costs) the capacity could be tripled
- When properly designed production material and fabrication costs should be the same like for the two other steel post systems

Design Recommendations

- The knee-bracket design as presented above can be optimized to achieve the desirable strength, stiffness, and energy absorption characteristics by plastic hinging. Using the above Free Body Diagramme, such a design can accommodate any requirements as existing in codes or by clients. The knee bracket does not necessarily need to carry steel rails, but can be equipped as well with timber rails.
- In the tests the knee bracket with the lower leg reaching to the main plate girder of the deck has shown superior behaviour and capacities. This way one could design this type to fail without any damage to the concrete deck, however, still achieving very high resistances.
- Timber washers need to be larger, at best of a size 10x10” in order to cover the complete cross beam or guard rail.
- Timber washers could be replaced by perforated plates or segments of structural channels.
- Individual timber guards rails should be connected with plates in order to create catenary action involving all rails and posts of one side of a bridge. Plates ought to be on top and bottom side of rails (sketch below shows top only).
- Vertical bolt should be located eccentrically to provide a larger compression area between the interconnection blocks in order to increase compression capacity.
- Concrete deck thickness should be increased close to the edges to a level to match guard rail capacity. In the current version for timber rail system, an increase from 175 mm to 225 mm (the latter quasi standard in Canada) would achieve this.
- Drip grooves should be omitted or placed differently (more toward plate girder).
- A307 should be continued to be used in order to avoid edge damage.
- Deck edge maybe reinforce by plates or channels in order to avoid premature spalling.

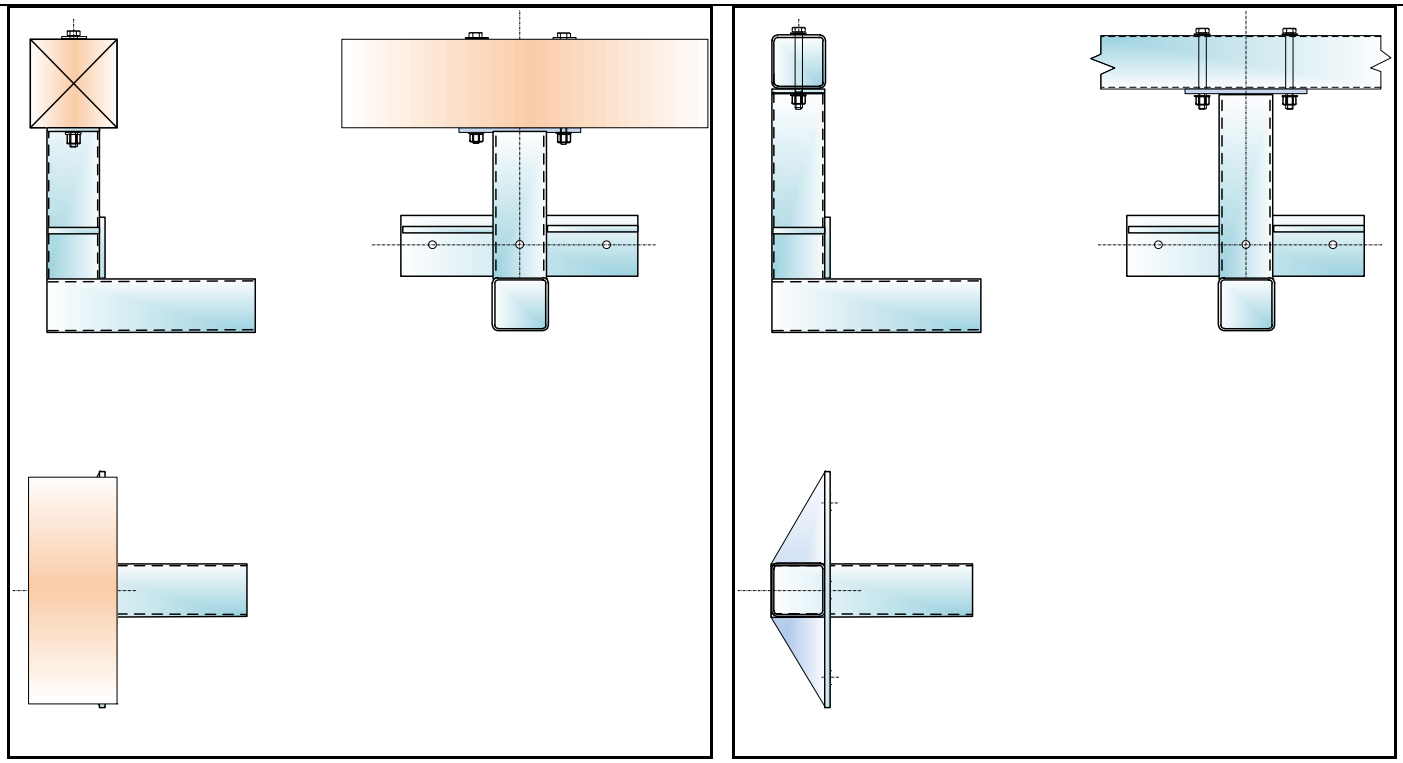


Figure 24: Modified Steel Post System #2- SS with improved Knee Bracket as proposed for Prototype with Timber (left) and Steel Rail (right)

Appendix

Detailed Tests and Diagrammes

Test #1.1

Date: 17/02/2011

Figure 25: Timber curb block, Test #1.1, horizontal load direction

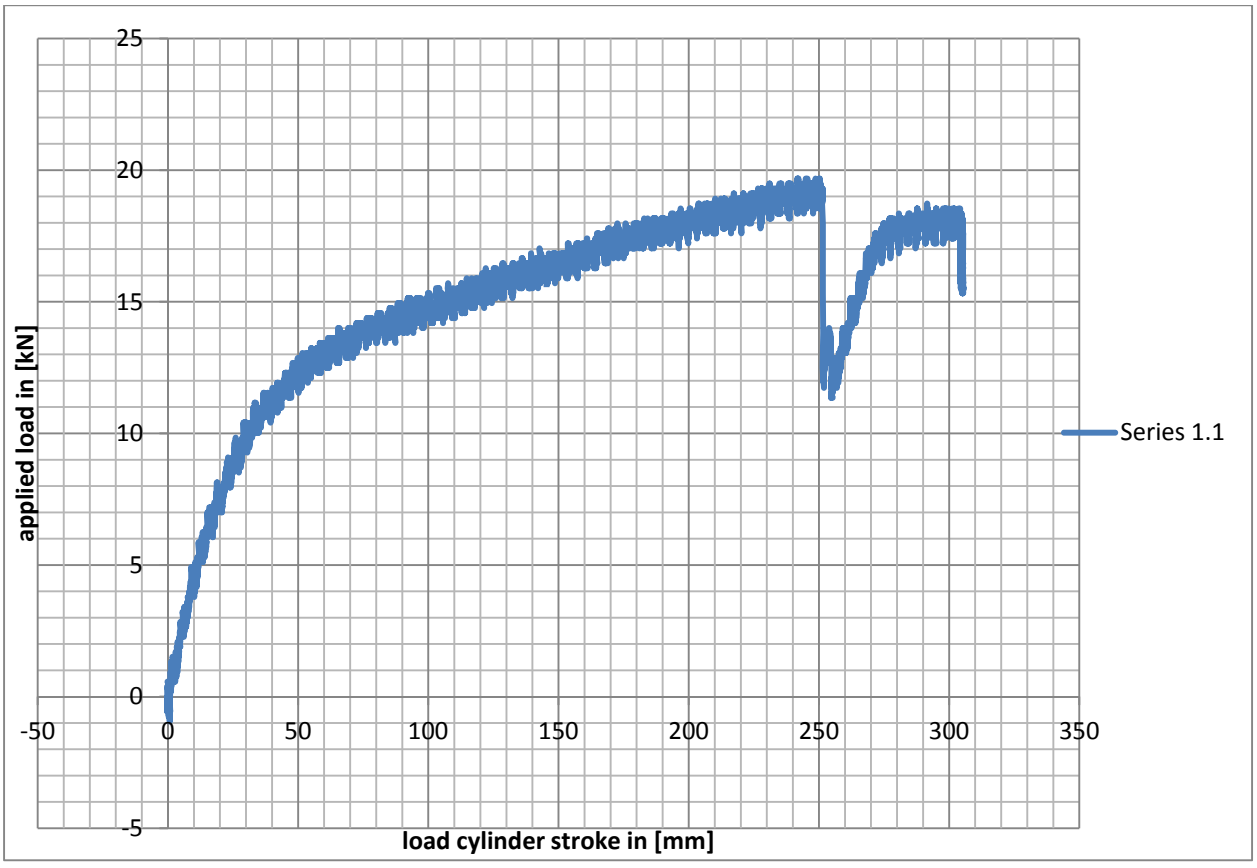


Figure 26: Load Deflection Diagramme of Test 1.1

Figure 27: Typical Images during Test for Series #1.1

Test #1.2

Date: 17/02/2011

Figure 28: Timber curb block, Test #1.2

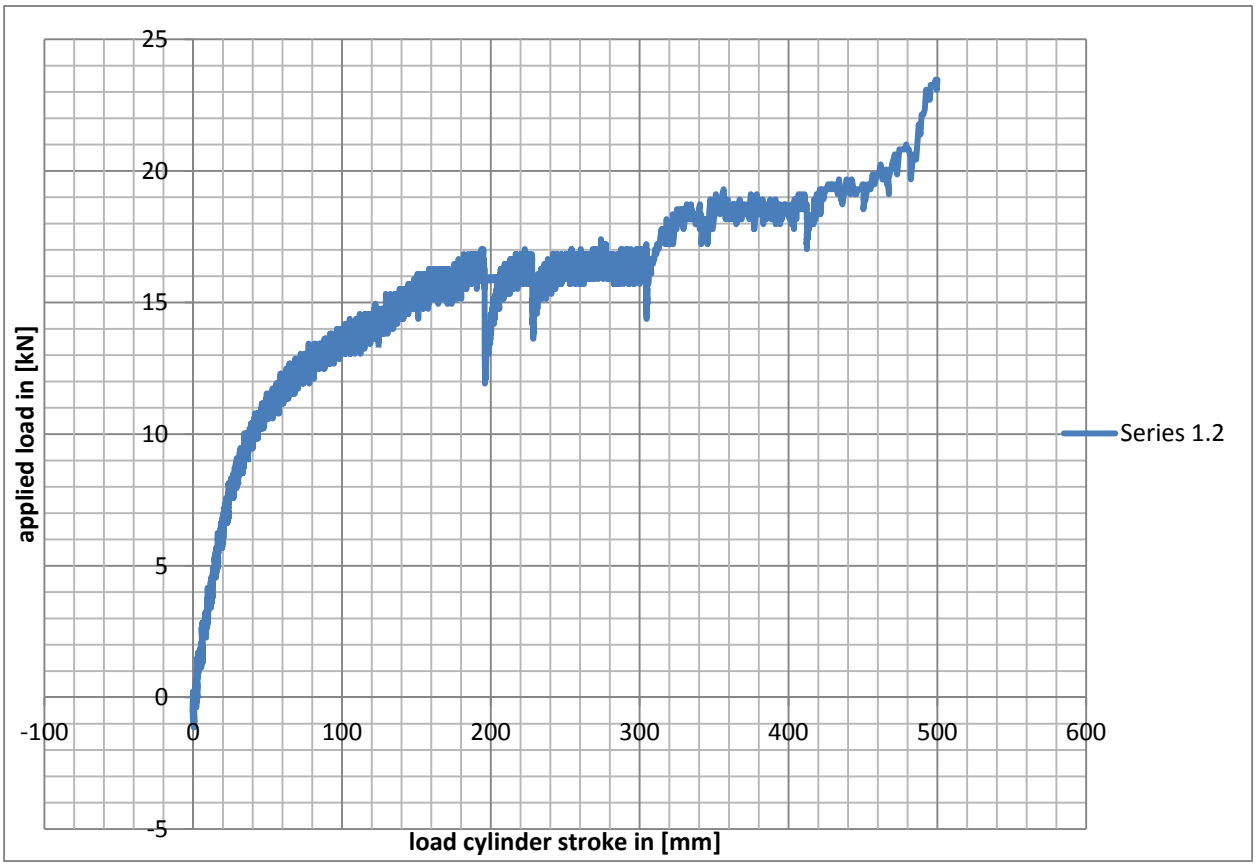


Figure 29: Load Deflection Diagramme of Test 1.2, horizontal load direction

Figure 30: Figure 19: Typical Images during Test for Series #1.2

Test #1.3

Date: 01/03/2011

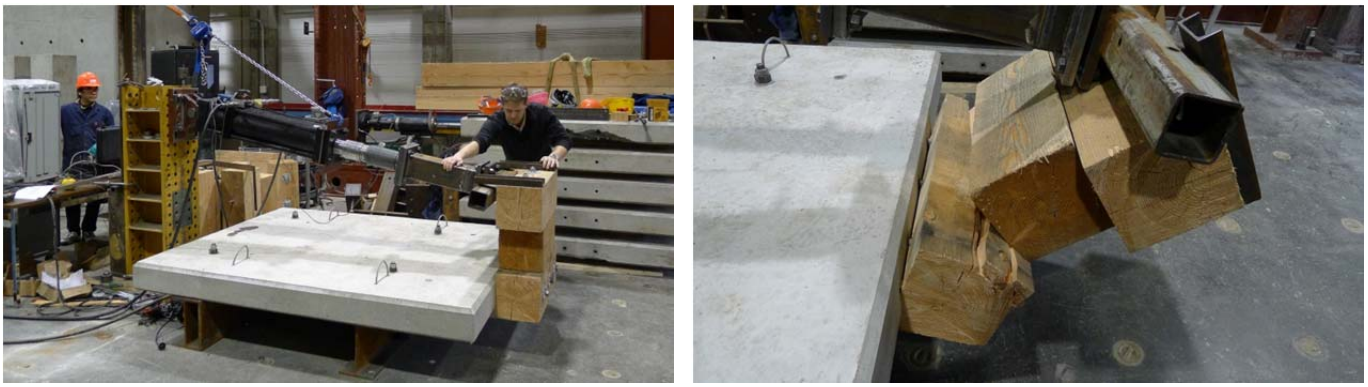


Figure 31: Test Series 1.3, horizontal and vertical load components applied

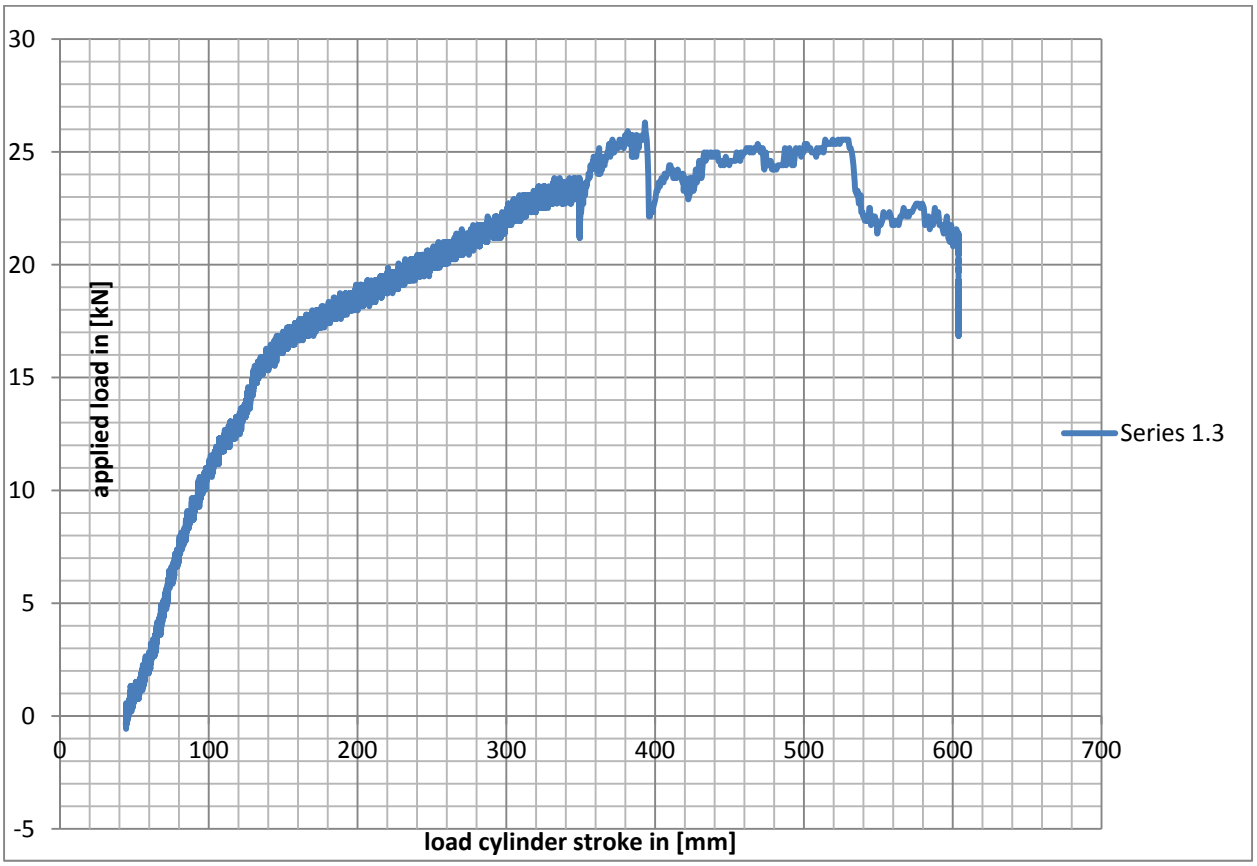


Figure 32: Load Deflection Diagramme of Test 1.3

Test #1.4

Date: 02/03/2011

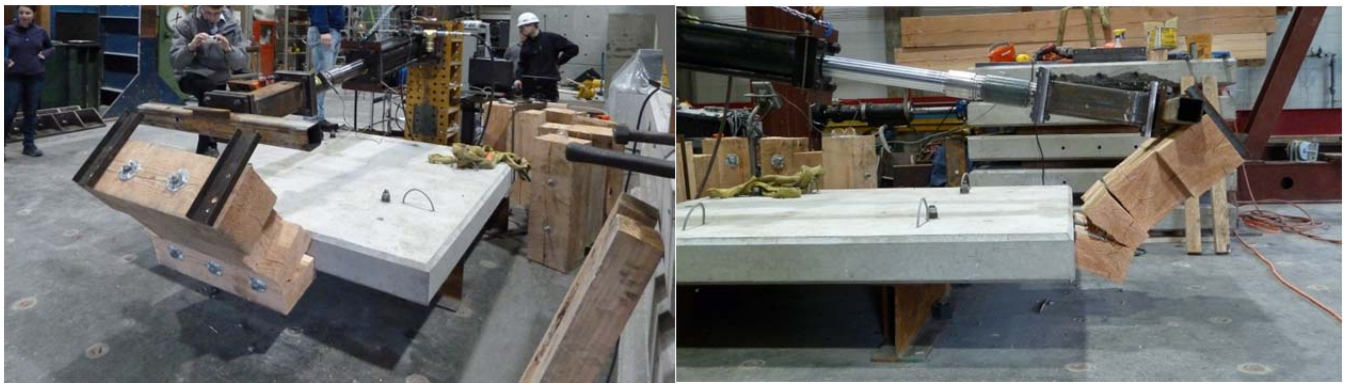


Figure 33: Test Series 1.4, horizontal and vertical load components applied

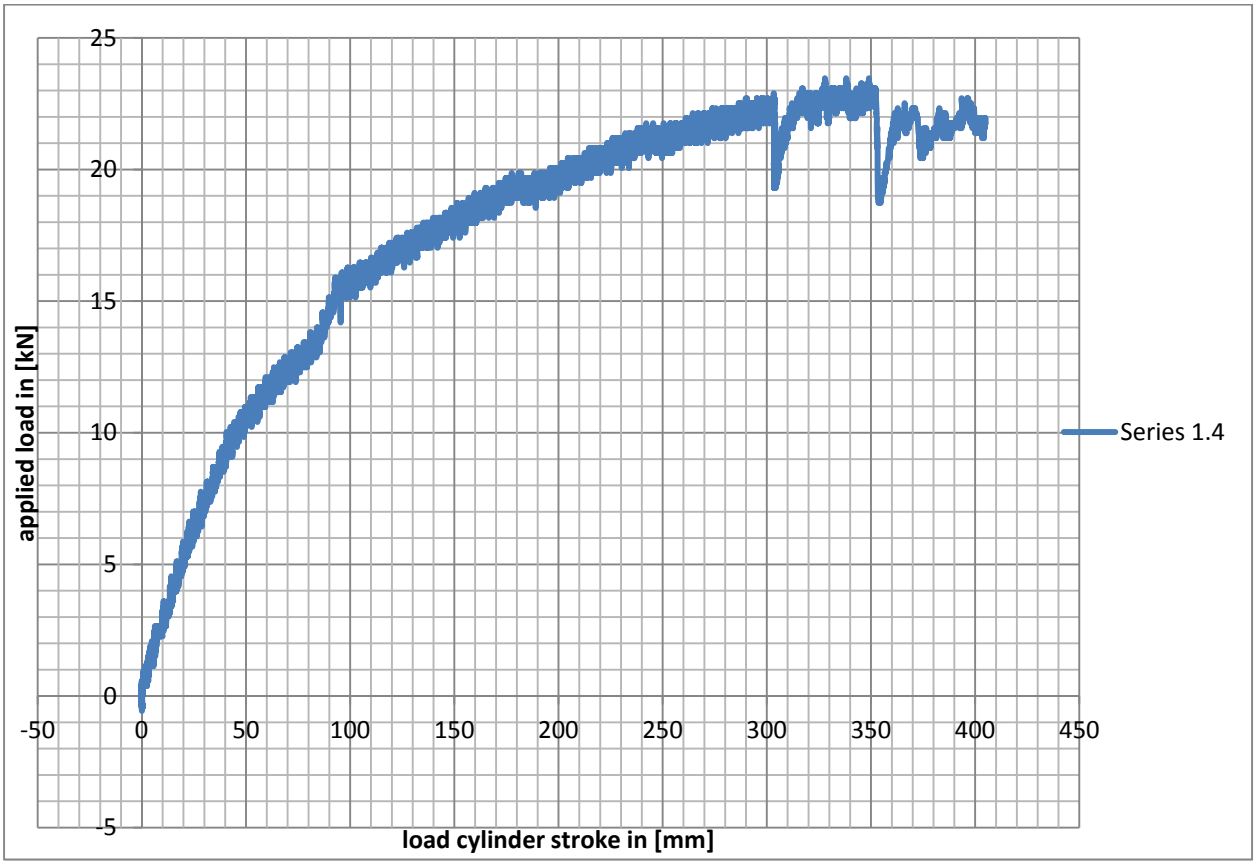


Figure 34: Load Deflection Diagramme of Test 1.4

Test #2.1

Date: 08/02/2011

Figure 35: Steel post, Test #2.1, horizontal load direction,3 x A307 horizontal bolts

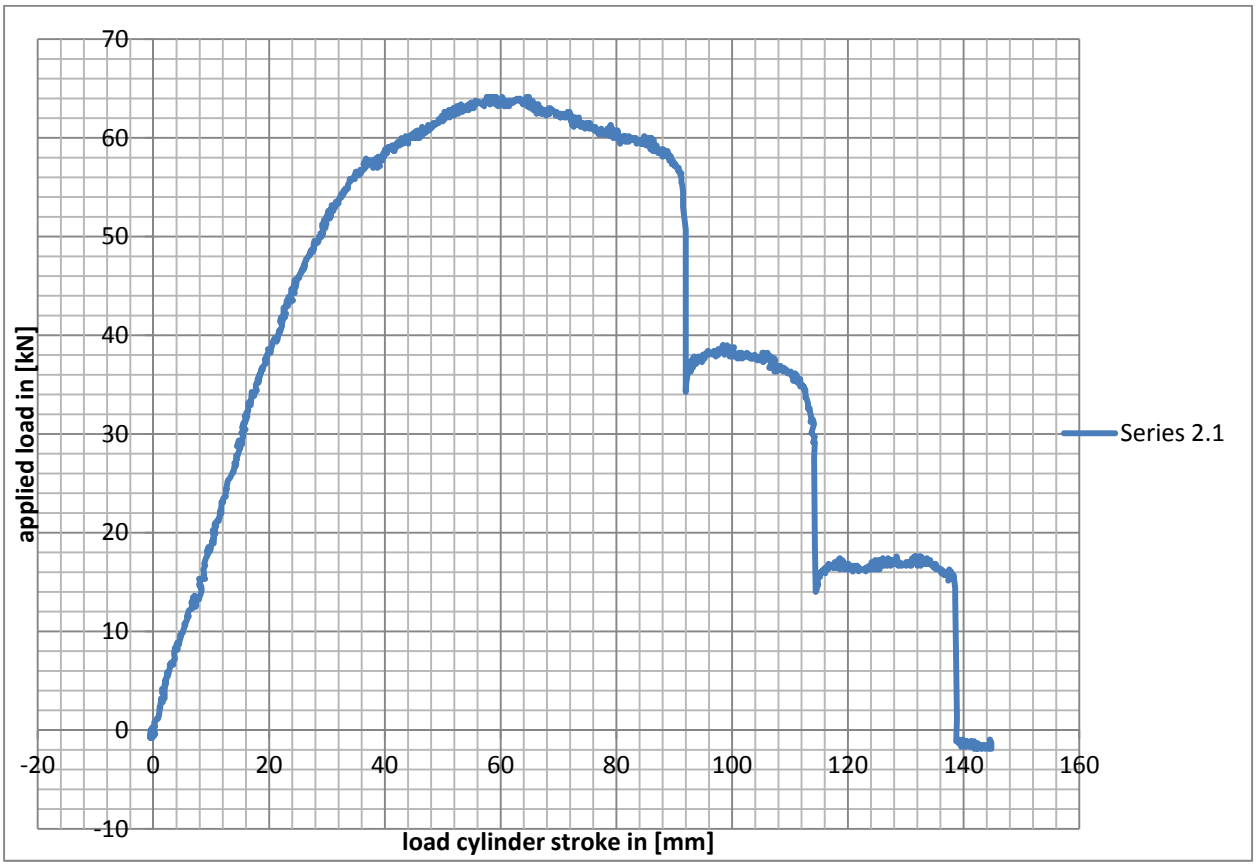


Figure 36: Load Deflection Diagramme of Test 2.1– One bolt failed at a same time / first at 64 kN / second at 38 kN / last at 18 kN

Figure 37: Note: Bolt elongation just before failure

Figure 38: All three bolts stripped in tension, lower concrete face spalled

Figure 39: Concrete deck edge, spalled material limited by drip groove, Note: highlighted crack at midthickness of deck panel

Test #2.2

Date: 08/02/2011

Figure 40: Steel post, Test #2.2, 3 x A307 horizontal bolts

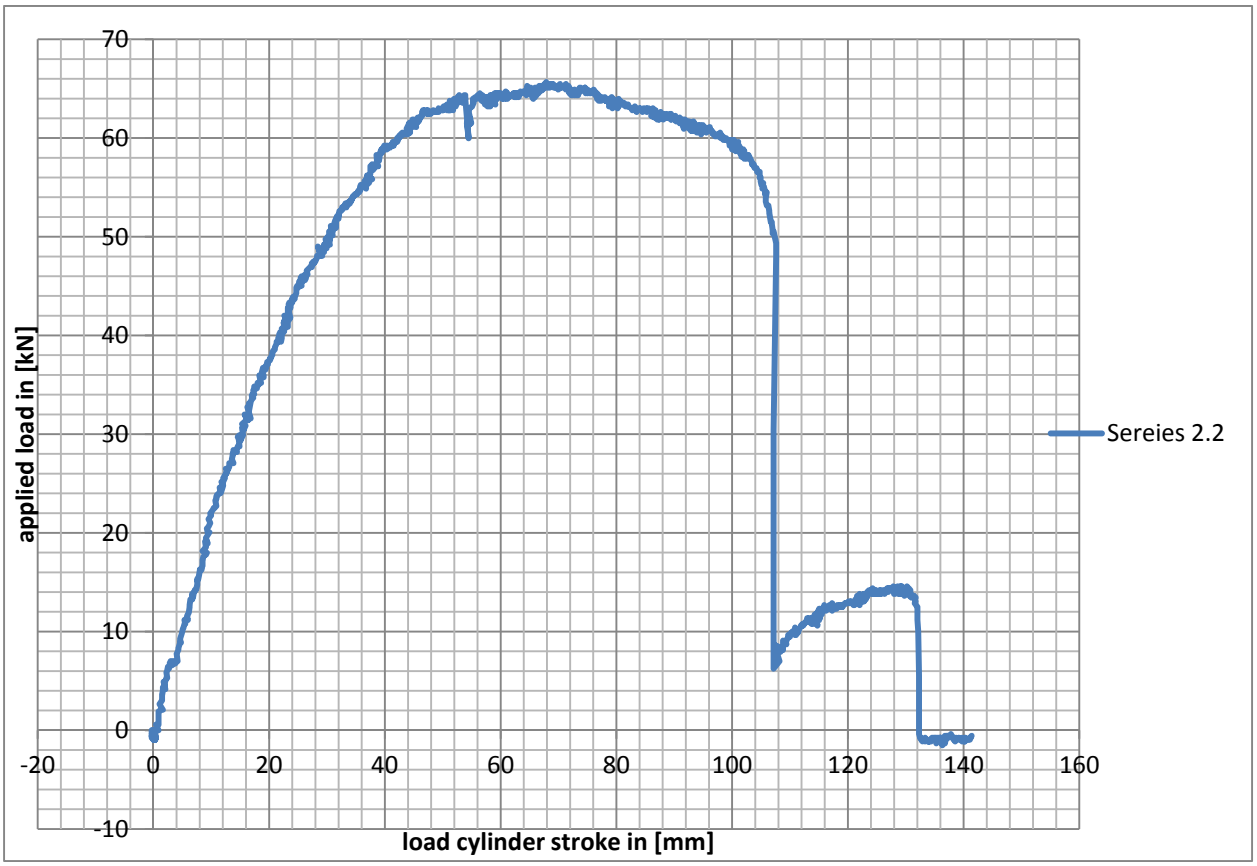


Figure 41: Load Deflection Diagramme of Test 2.2, horizontal load direction, two bolts failed at the same time at max. load

Photos for Test #2.2 are almost identical to #2.1, no nee to show.

Test #2.3

Date: 03/03/2011

Figure 42: Steel post, Test #2.3, vertical and horizontal load components applied, 3 x A307 horizontal bolts

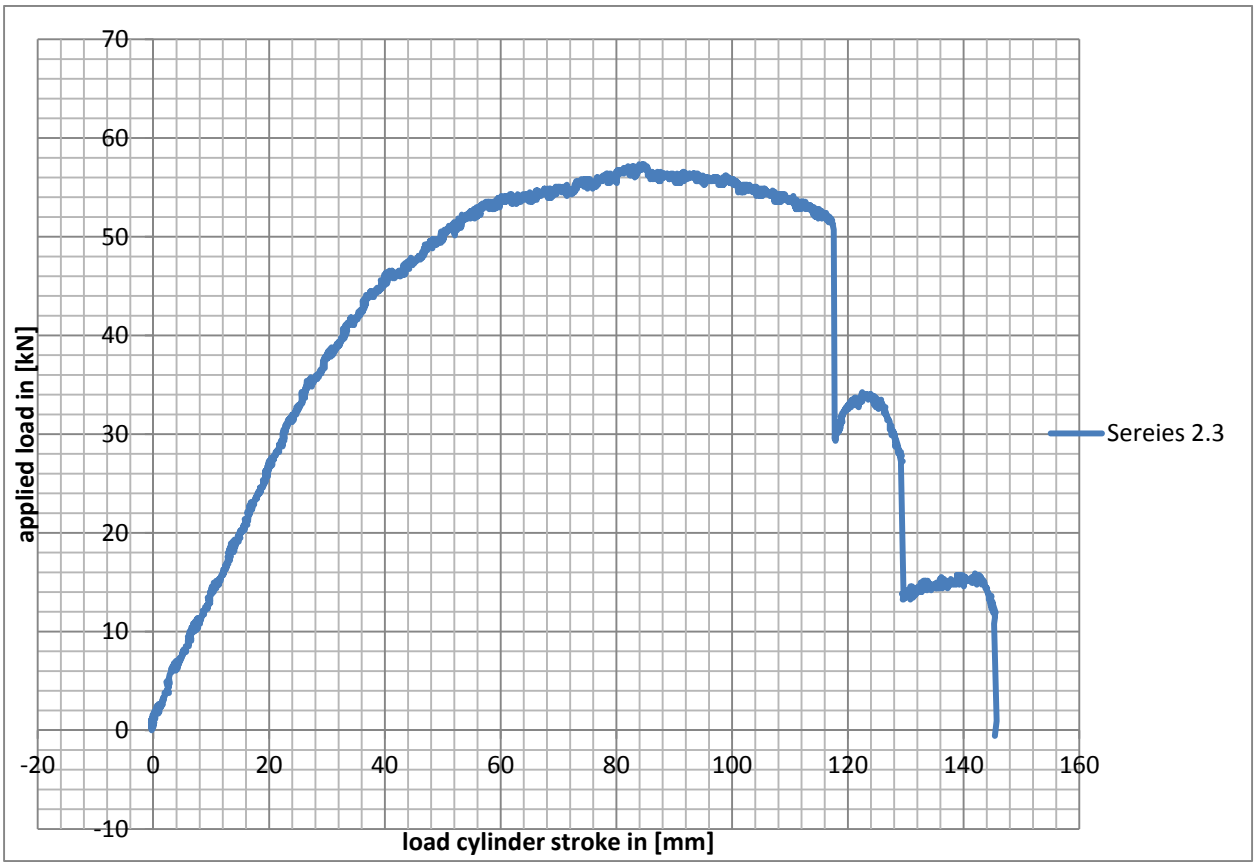


Figure 43: Load Deflection Diagramme of Test 2.3

Test #2.4

Date: 03/03/2011

Figure 44: Steel post, Test #2.4, vertical and horizontal load components applied, 3 x A307 horizontal bolts

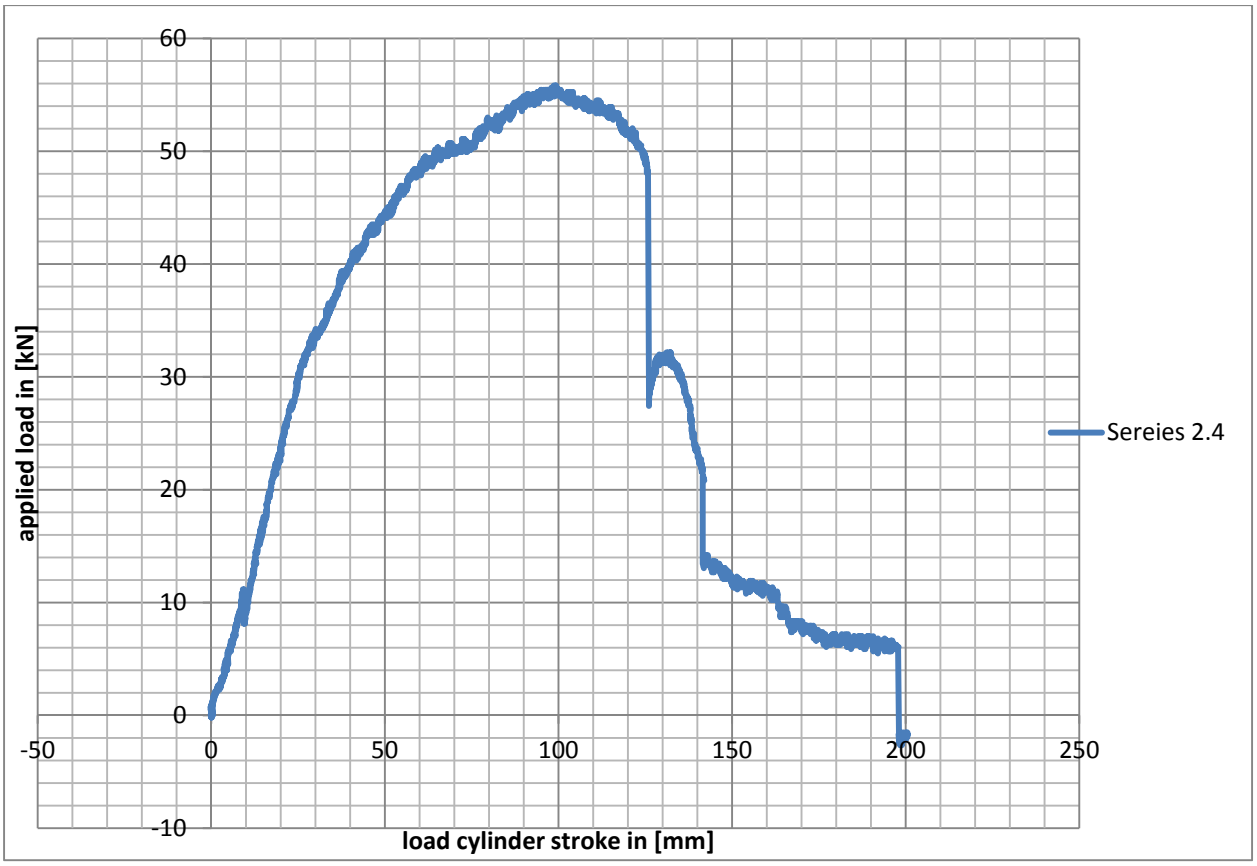


Figure 45: Load Deflection Diagramme of Test 2.4

Test #2.11

Date: 18/02/2011

Figure 46: Steel post, Test #2.11, 3 x A325 horizontal bolts

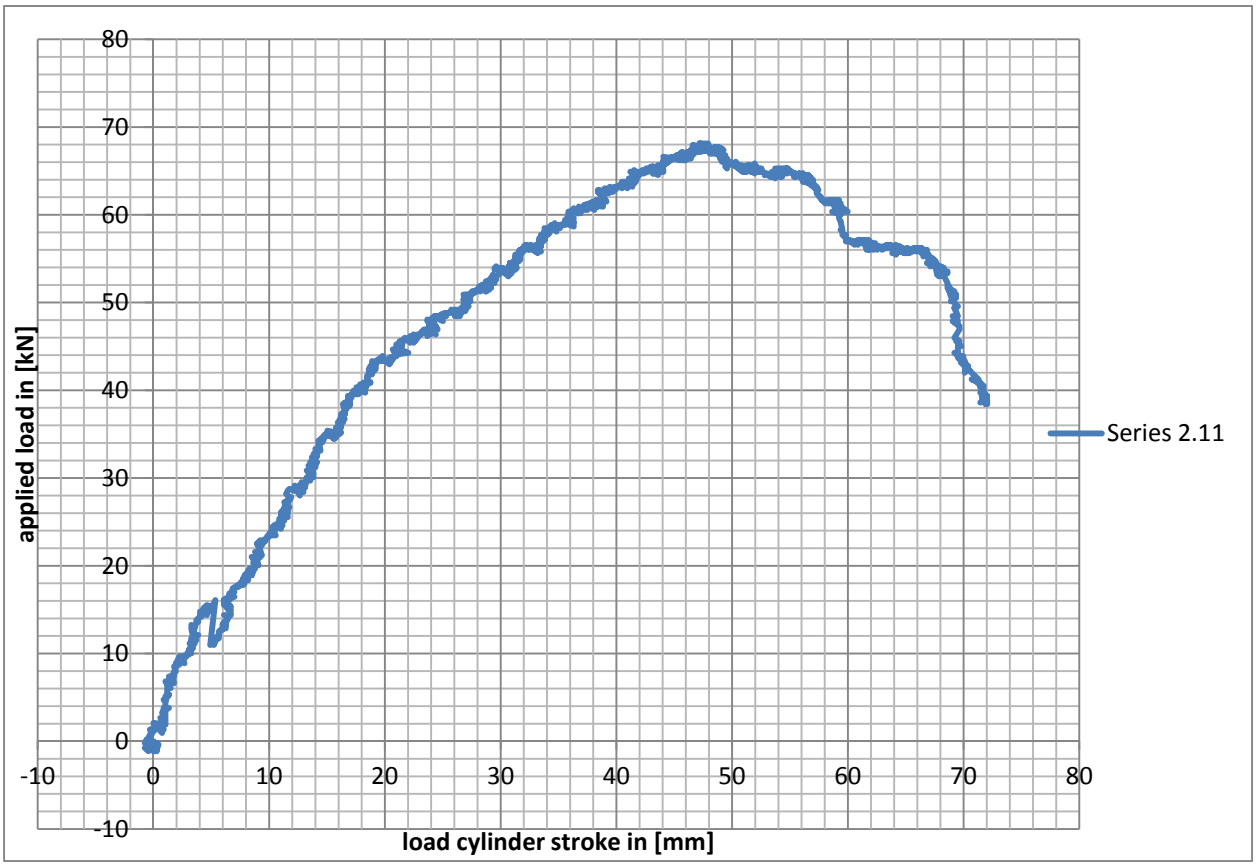


Figure 47: Load Deflection Diagramme of Test 2.11, horizontal load direction

Figure 48: Typical Images during Test for Series #2.11

Test #2.21

Date: 18/02/2011

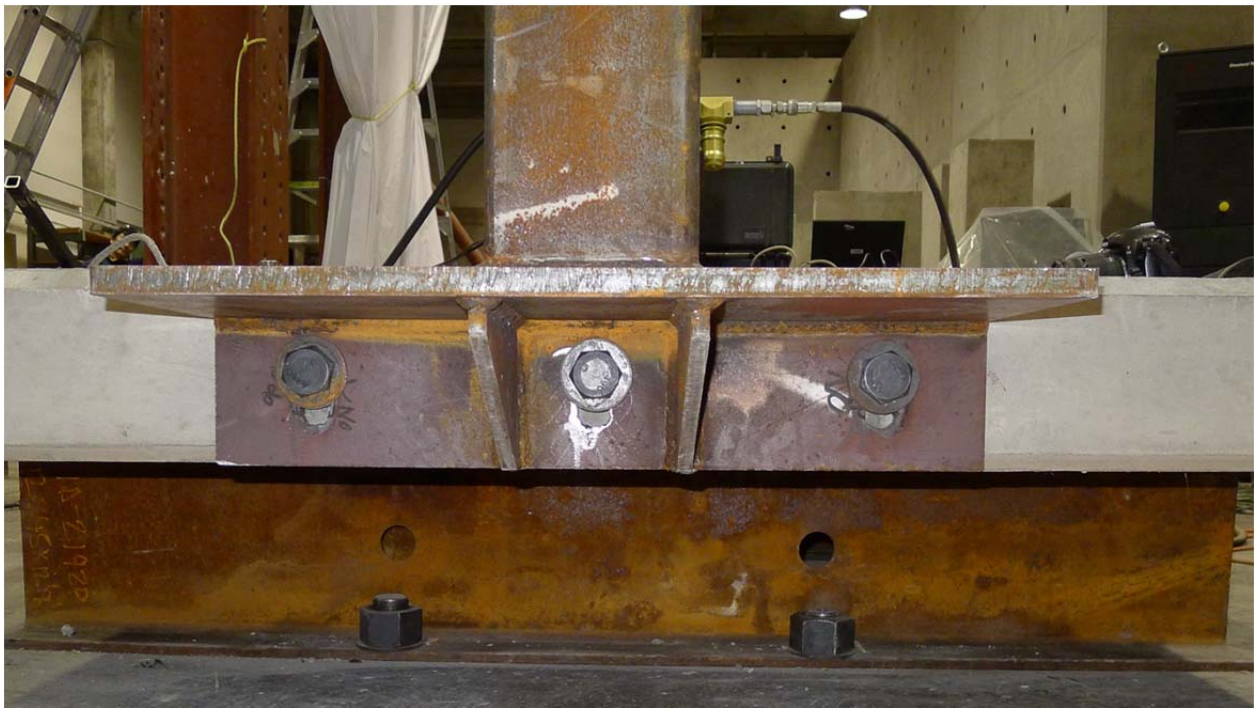


Figure 49: Steel post, Test #2.21, 3 x A325 horizontal bolts

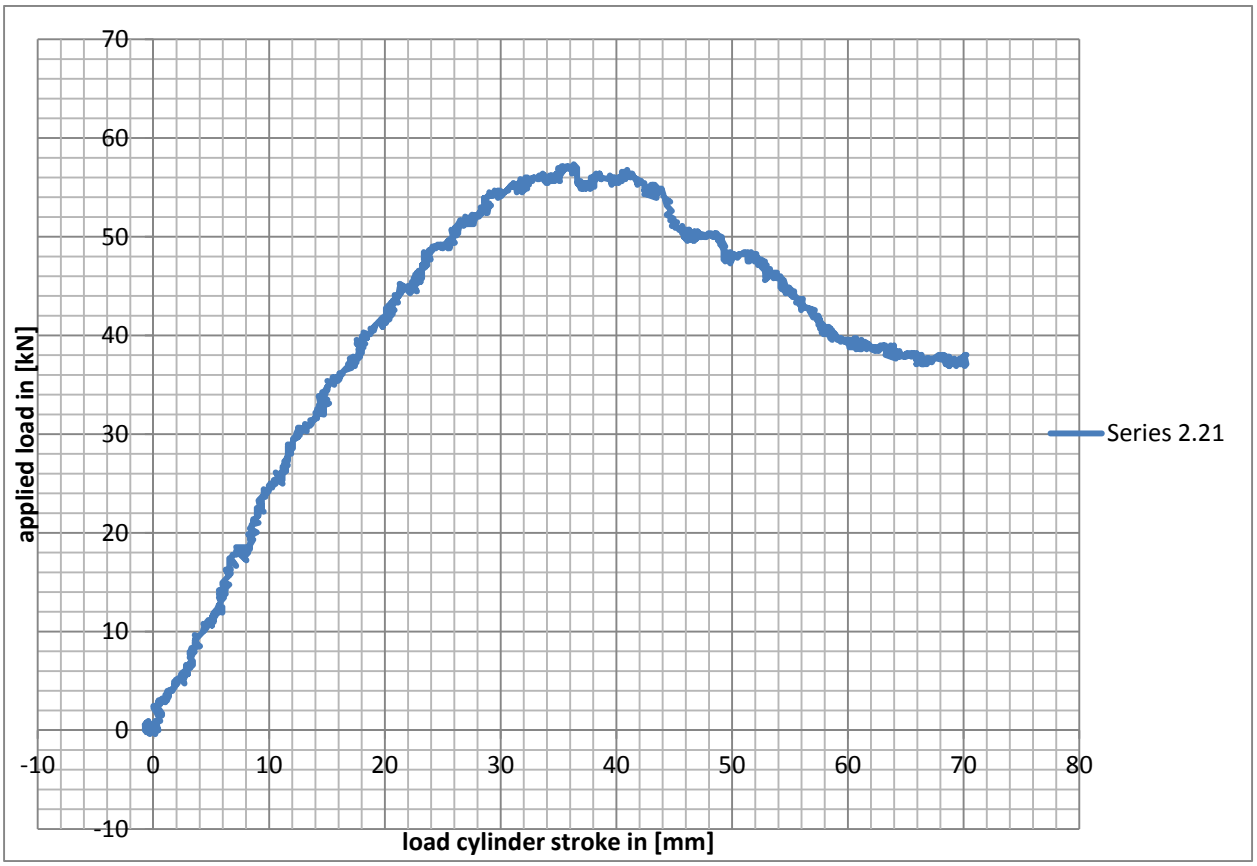


Figure 50: Load Deflection Diagramme of Test 2.21

Figure 51: Typical Images during Test for Series #2.11, horizontal load direction

Test #3.1

Date: 09/03/2011

Figure 52: Steel post, Test #3.1, vertical and horizontal load components applied, 2 x 2 x A307 horizontal bolts

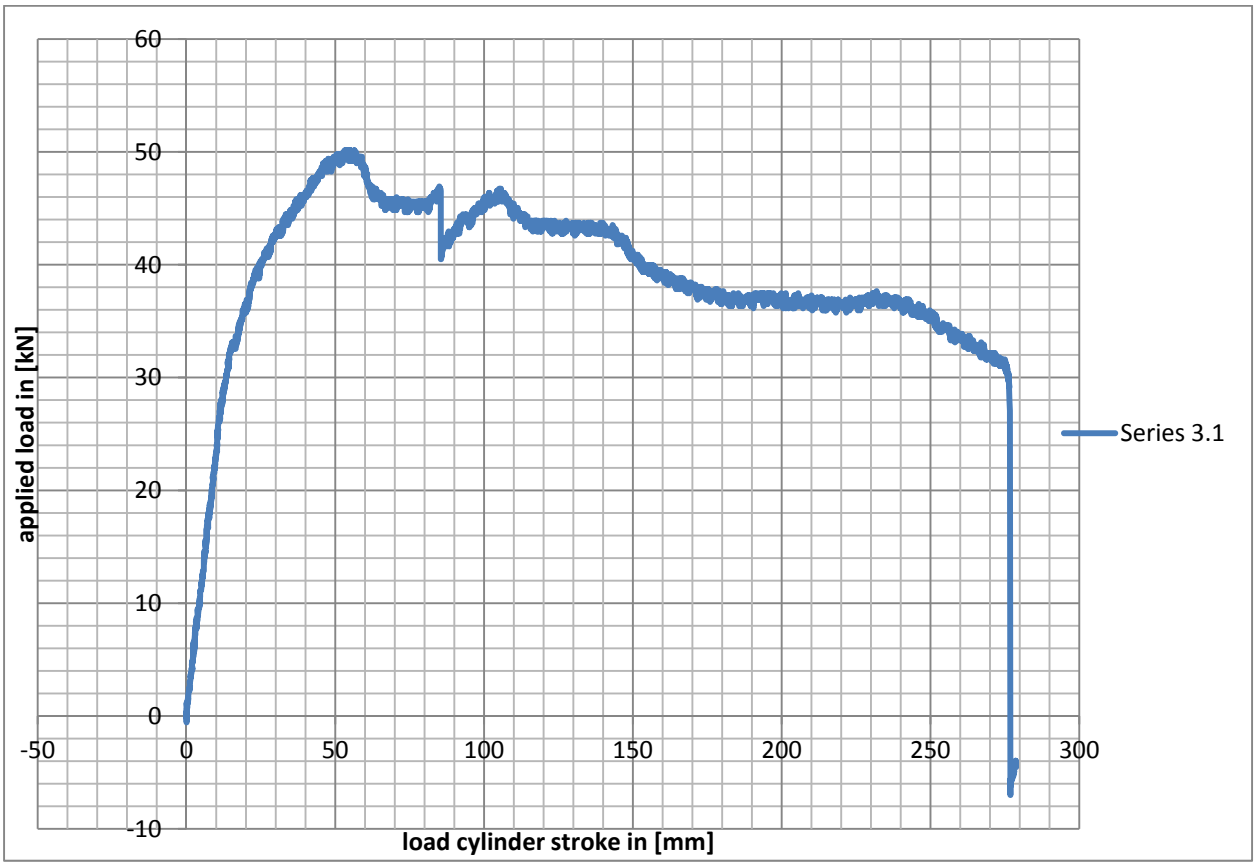


Figure 53: Load Deflection Diagramme of Test 3.1

Test #3.2

Date: 10/03/2011

Figure 54: Steel post, Test #3.2, vertical and horizontal load components applied, 2 x 2 x A307 horizontal bolts

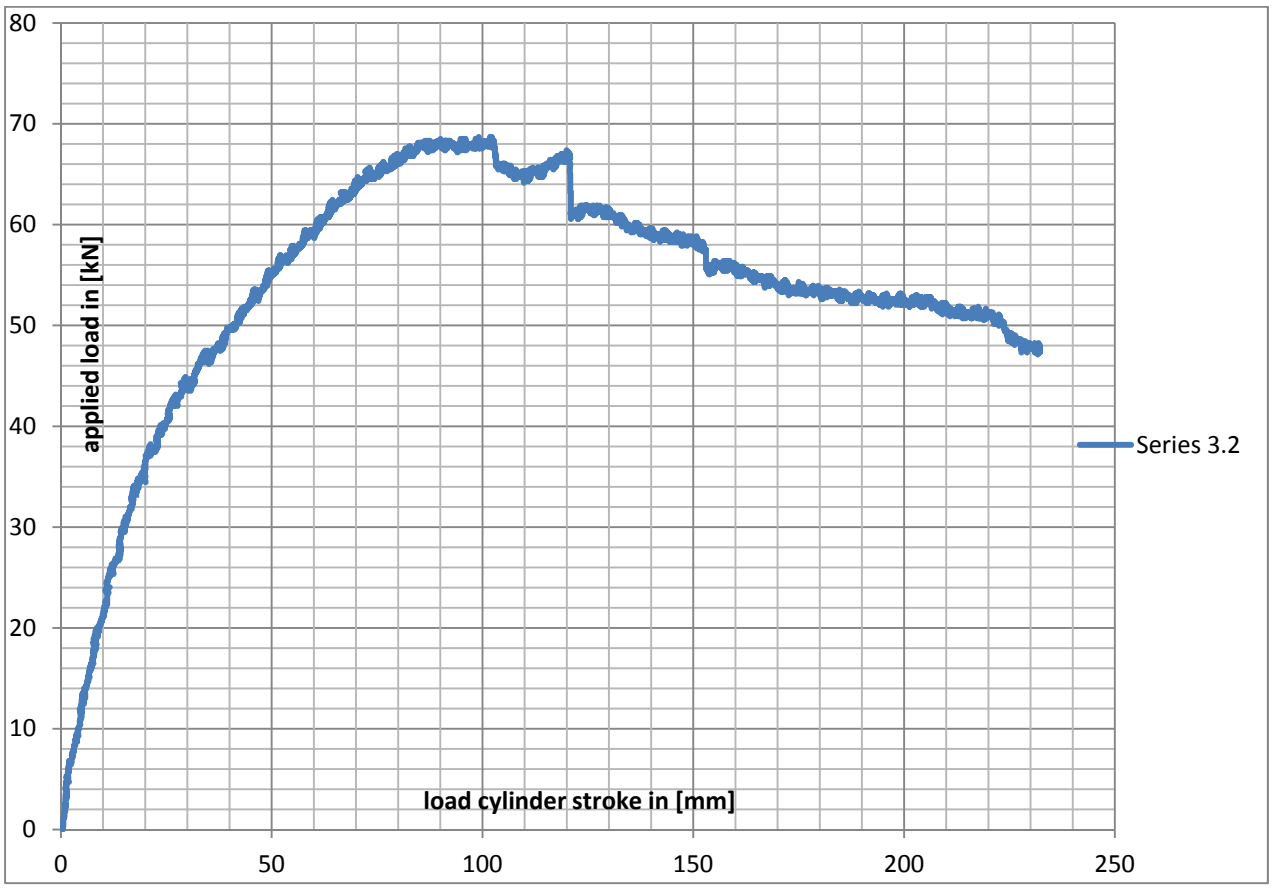


Figure 55: Load Deflection Diagramme of Test 3.2

Test #3.3

Date: 11/03/2011

Figure 56: Steel post, Test #3.3, horizontal load component applied, 2 x 2 x A307 horizontal bolts

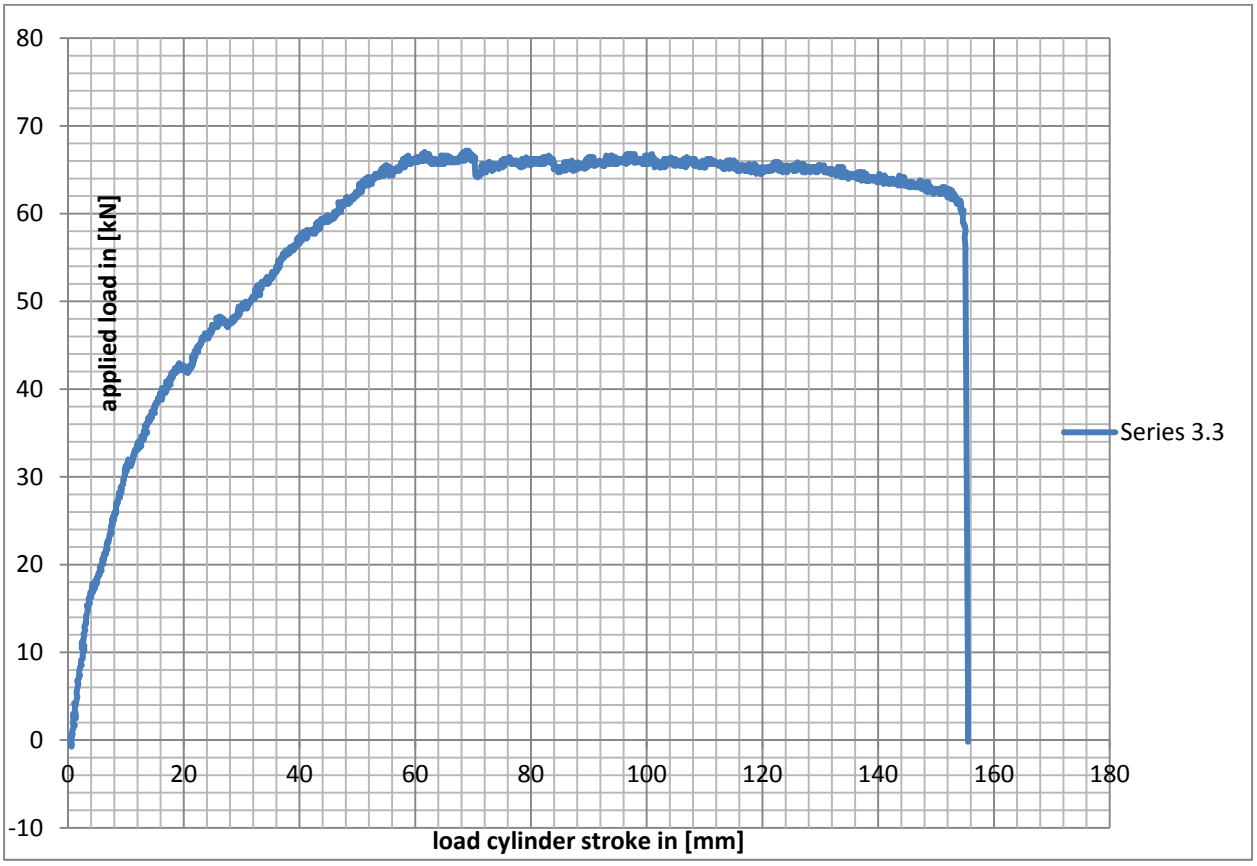


Figure 57: Load Deflection Diagramme of Test 3.3

Test #3.4

Date: 11/03/2011

Figure 58: Steel post, Test #3.4, horizontal load component applied, 2 x 2 x A307 horizontal bolts

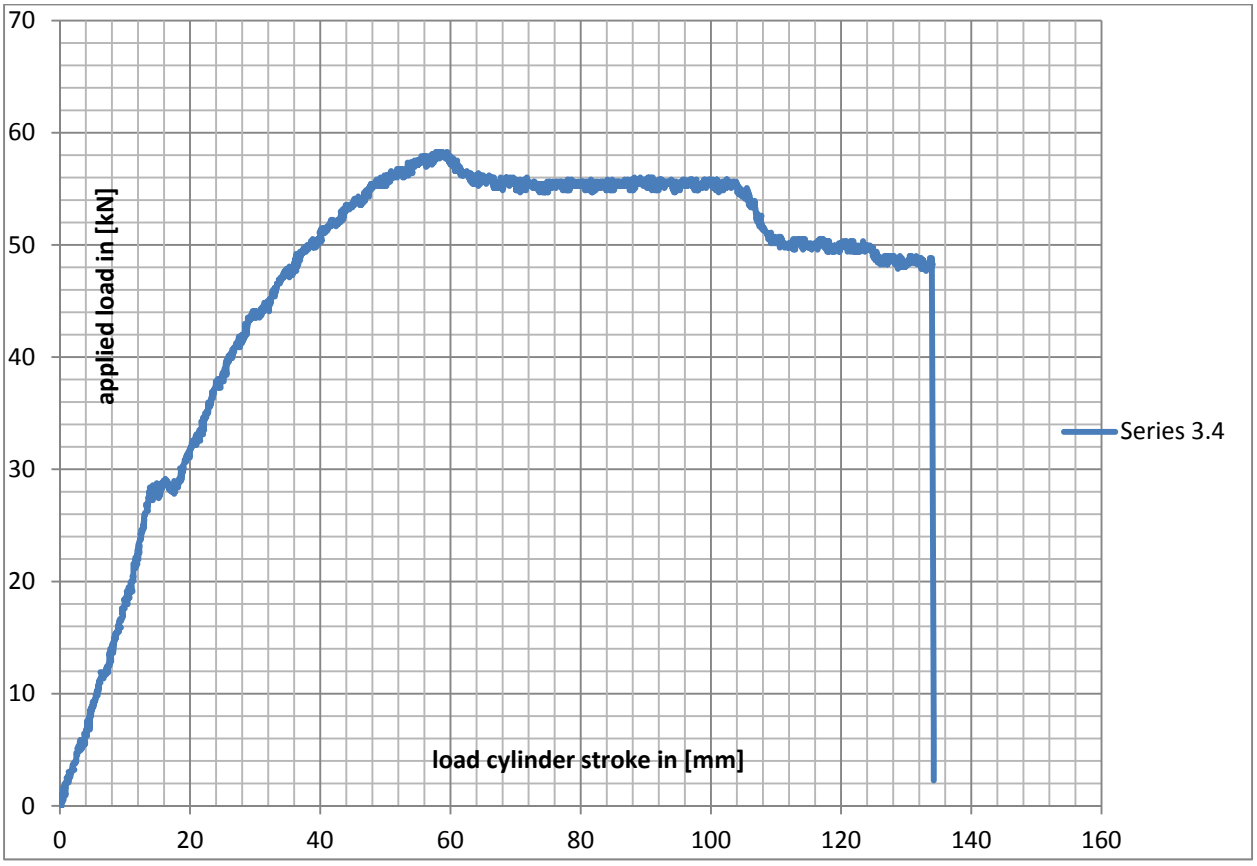


Figure 59: Load Deflection Diagramme of Test 3.4

Test #4.1

Date: 17/03/2011

Figure 60: Steel post, Test #4.1, steel base, timber riser, vert. and hor. components applied, 3 A307 horizontal bolts

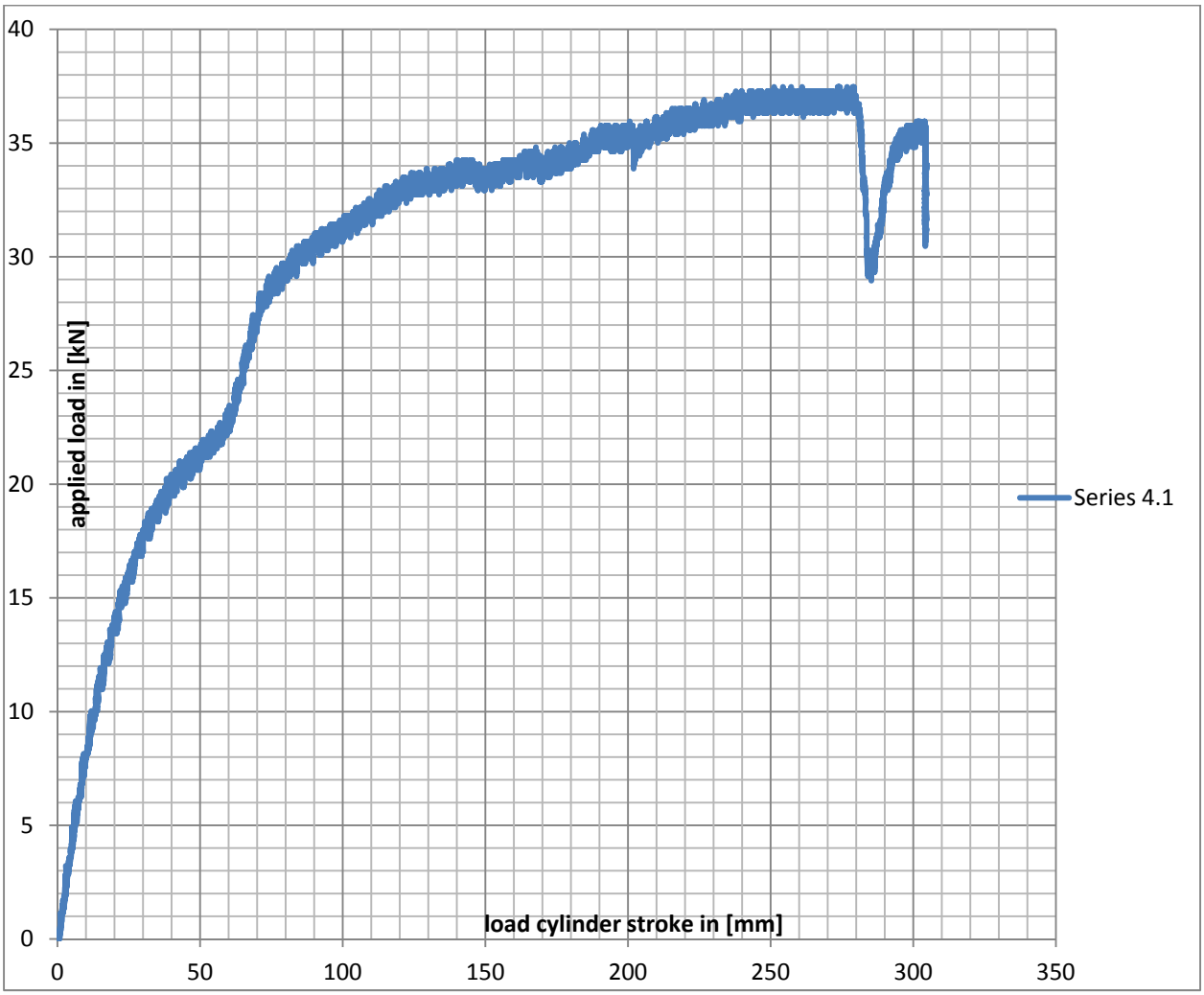


Figure 61: Load Deflection Diagramme of Test 4.1

Test #4.2

Date: 17/03/2011

Figure 62: Steel post, Test #4.2, steel base, timber riser, vertical and horizontal components applied, 3 x A307 horizontal bolts

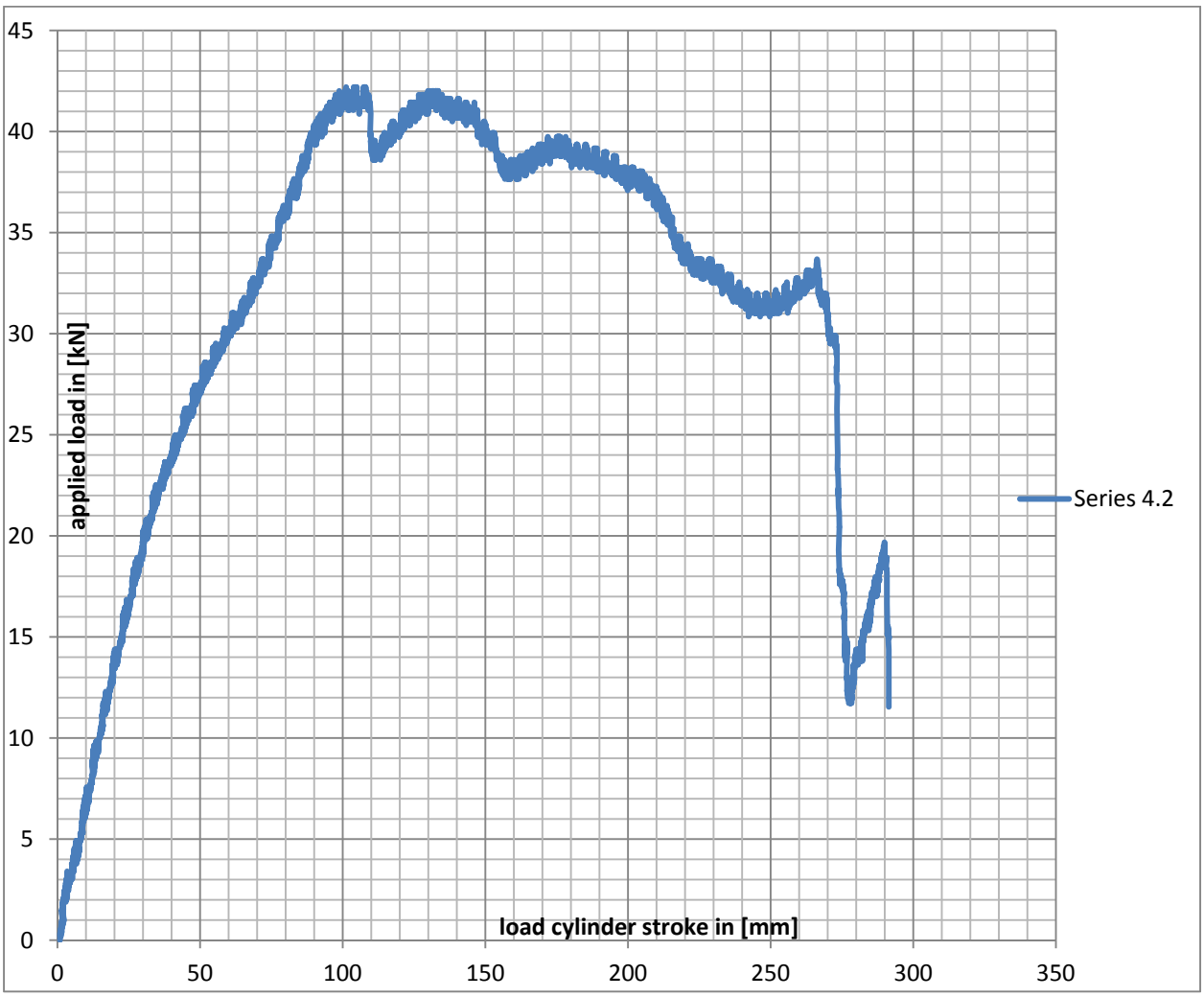


Figure 63: Load Deflection Diagramme of Test 4.2

Test #4.3

Date: 17/03/2011

Figure 64: Steel post, Test #4.3, horizontal load, 3 x A307 horizontal bolts

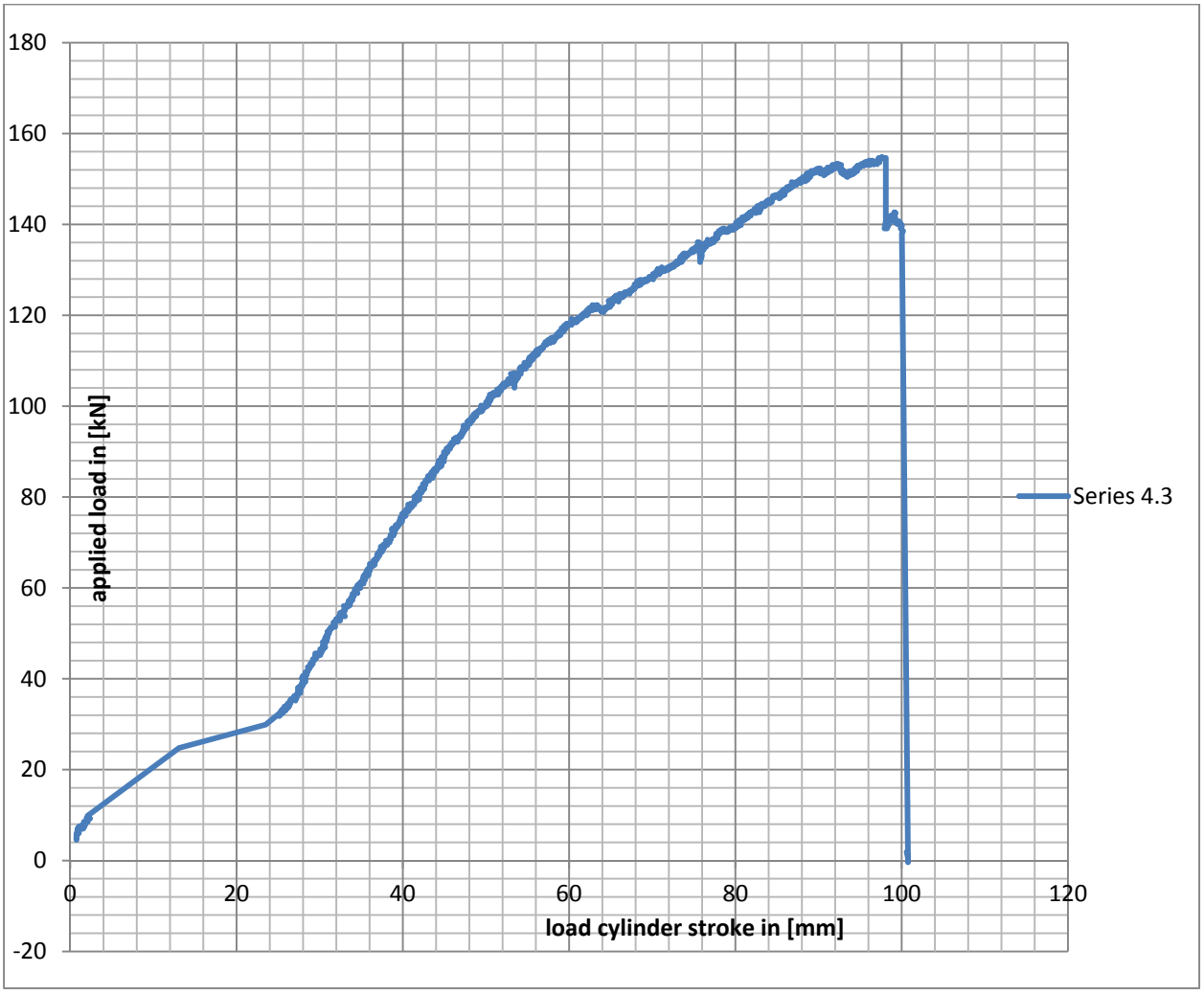


Figure 65: Load Deflection Diagramme of Test 4.3

Test #4.4

Date: 17/03/2011

Figure 66: Steel post, Test #4.4, horizontal load, 3 x A307 horizontal bolts

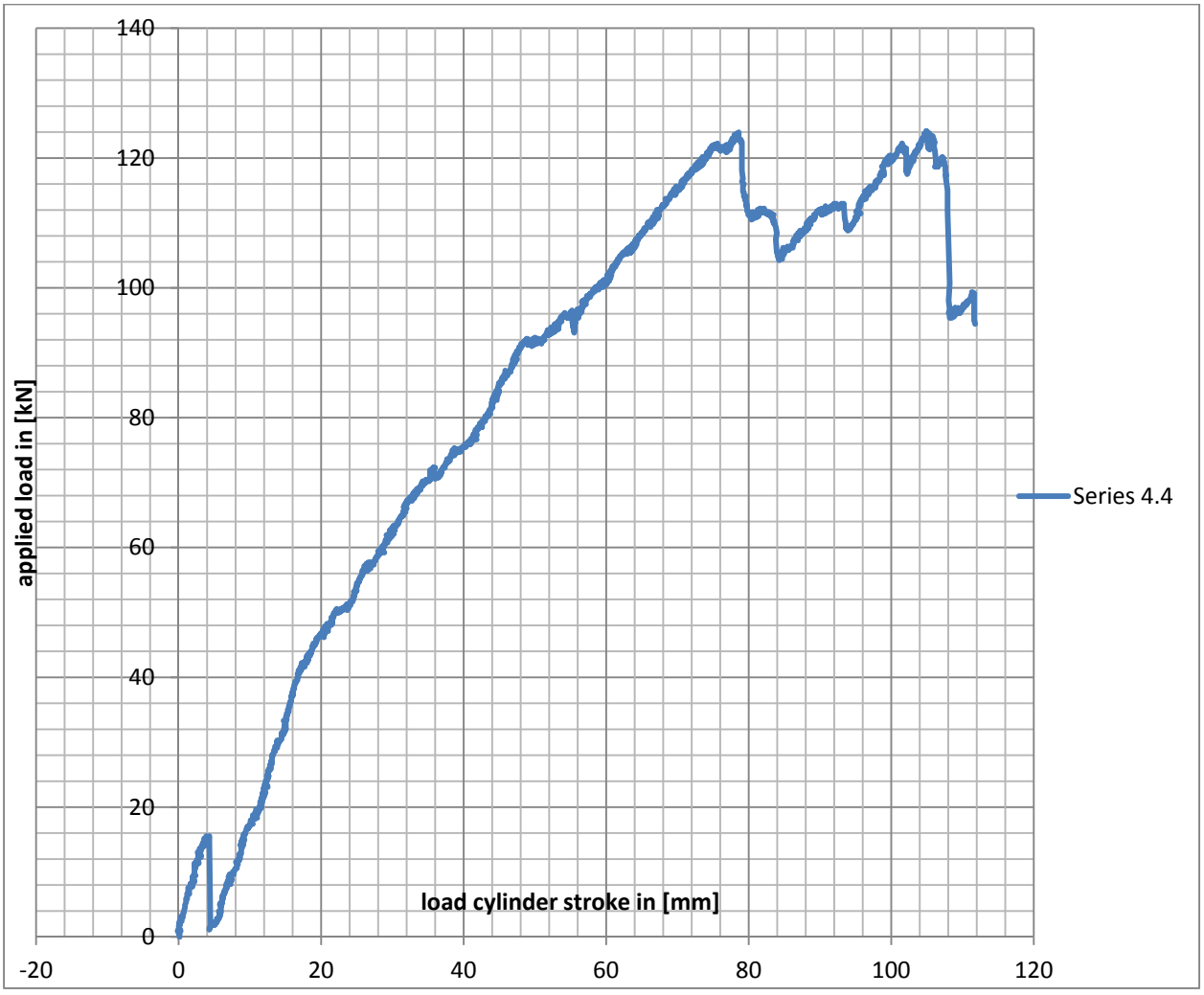


Figure 67: Load Deflection Diagramme of Test 4.4

Test #4.5

Date: 11/03/2011

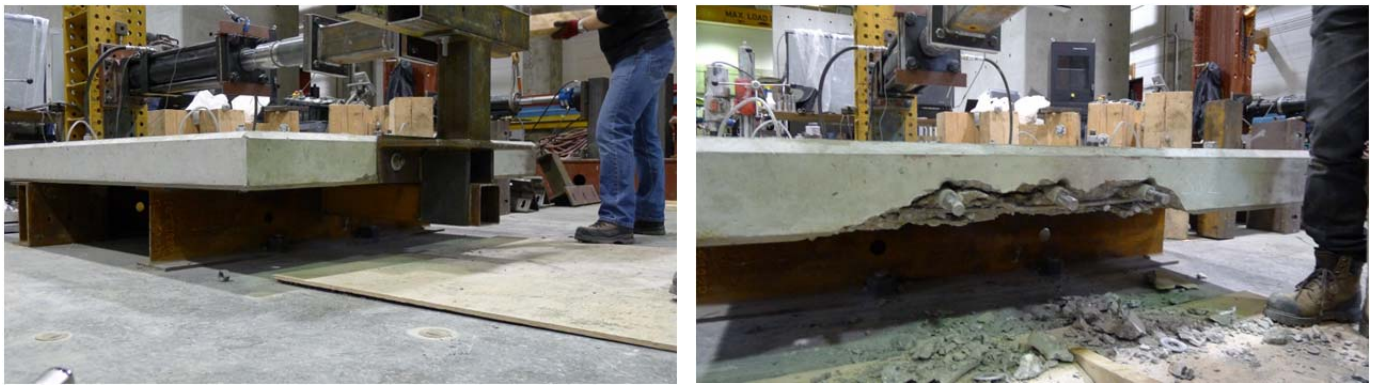


Figure 68: Steel post, Test #4.5, vertical load component applied, 3 x A307 horizontal bolts

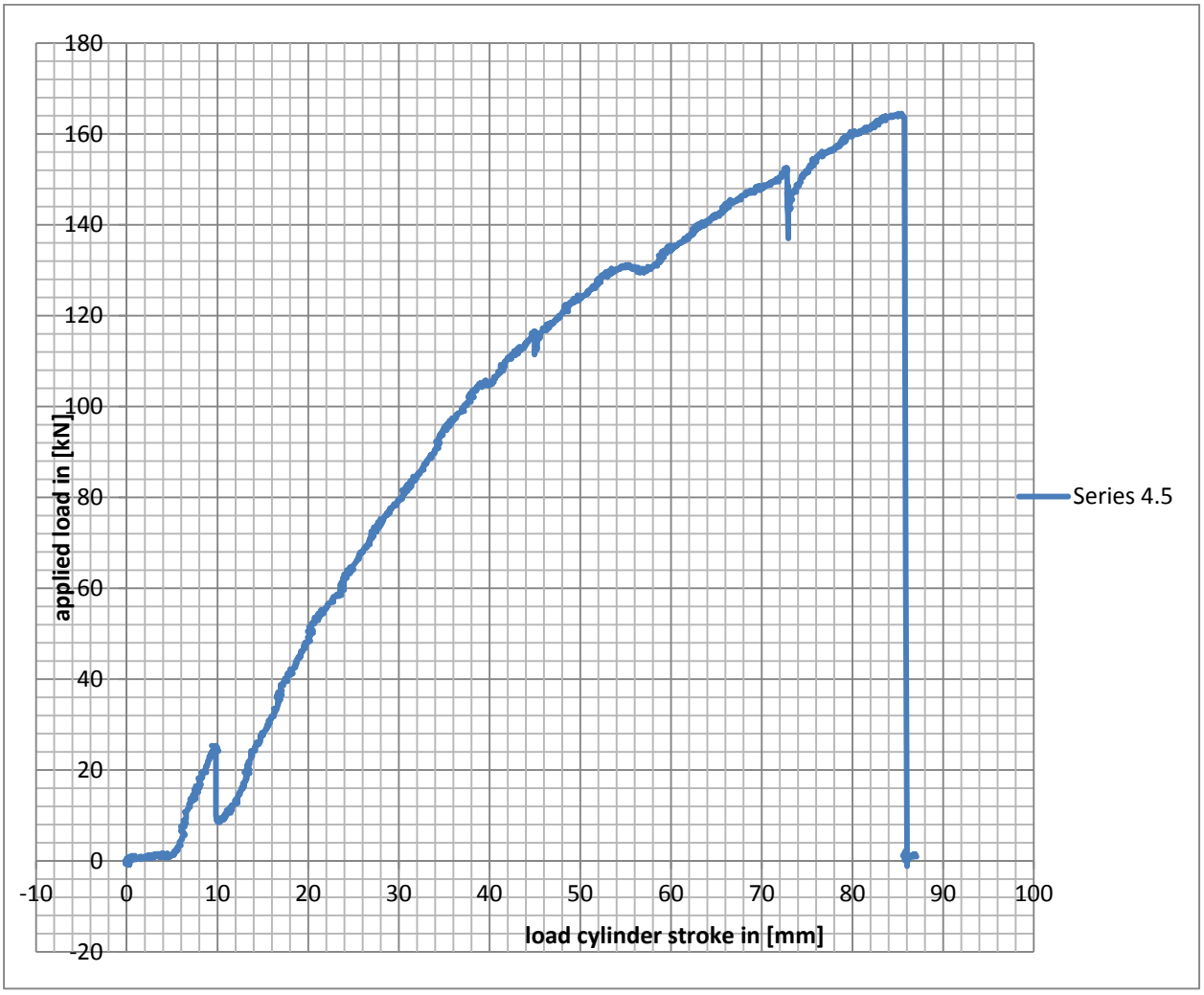


Figure 69: Load Deflection Diagramme of Test 4.5