EFFECT OF SIMULATED CRACKS ON LAP SPLICE STRENGTH OF REINFORCING BARS

By

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Research supported by

FIRSTENERGY NUCLEAR OPERATING COMPANY
(FENOC)

Structural Engineering and Engineering Materials
SL Report 12-2

THE UNIVERSITY OF KANSAS CENTER FOR RESEARCH, INC.
LAWRENCE, KANSAS
June 2012

Abstract

The effect of preexisting cracks, oriented in the plane of and parallel to the reinforcing steel, on the strength of No. 11-bar lap splices was investigated by testing six beams – three with a splice length of 79 in. and three with a splice length of 120 in. One of the beams with a 79-in. splice was cast monolithically and loaded monotonically to failure. To simulate the cracks, the other five beams were cast with a cold joint at the mid-height of the reinforcing steel. Two beams (one with a 79-in. splice and one with a 120-in. splice) with a cold joint were loaded monotonically to failure. The other three beams were preloaded to develop horizontal cracks in the face of the cold joint, unloaded and then loaded to failure; those beams developed horizontal cracks with widths of 20, 30 and 35 mils (0.02, 0.03, 0.035 in.) during the first cycle of loading and just prior to unloading. The nominal concrete compressive strength was 5000 psi.

The methods described in this report provide a viable means of simulating a crack in the plane of flexural reinforcement. In the presence of a simulated crack in the plane of the reinforcing bars, the two specimens with lap-spliced No. 11 bars with a 79-in. splice length achieved bar stresses of 62 and 57 ksi. In the presence of a simulated crack in the plane of the reinforcing bars, the three specimens with lap-spliced No. 11 bars with a 120-in. splice length achieved bar stresses of 72, 67, and 69 ksi.

Personnel

The research described in this report was performed under the direction of David Darwin, Ph.D., P.E., Deane E. Ackers Distinguished Professor of Civil, Environmental, and Architectural Engineering and Director of the Structural Engineering Materials Laboratory and Adolfo Matamoros, Ph.D., Associate Professor of Civil, Environmental, and Architectural Engineering at the University of Kansas. Darwin and Matamoros are joined by post-doctoral researchers, Matthew O'Reilly and Jiqiu Yuan.

David Darwin has extensive experimental and analytical experience in the field of bond and development of reinforcement and has been involved in bond research for over 30 years. He is a member and past chair ACI Committee 408 on Bond and Development of Reinforcement and a member of ACI Subcommittee 318-B Reinforcement and Development (Structural Concrete Building Code). Darwin developed the ACI Committee 408 expression for development and splice lengths, which accurately covers straight reinforcing bars with yield strengths between 40 and 120 ksi and for concrete with compressive strengths ranging from 2,000 to 16,000 psi. Darwin also developed the ASTM A944 Beam-End Test, which is used to evaluate relative bond strength. In addition to Committee 408, Darwin is a member and past chair of ACI on Committee 224 on Cracking, as well as four other ACI technical committees.

Adolfo Matamoros has many years of experience in testing and analysis of reinforced concrete with special expertise in test instrumentation. He is the immediate past chair of ACI Committee 408 and a member of four other ACI technical committees.

Matthew O'Reilly and **Jiqiu Yuan** completed their Ph.D. degrees at the University of Kansas in 2011 and have been serving as senior researchers on a number of experimental projects since receiving their degrees.

Contents

1 Overview / Background

Past research on the strength of lapped bar splices in reinforced concrete has focused on investigating the performance of various lap splice configurations in monolithic members. The research program described in this report investigates the effect of preexisting cracks, oriented in the plane of and parallel to the reinforcing steel, on the strength of lapped bar splices. The research program was conducted in two phases, a pilot study investigating various methods to simulate the preexisting cracks that is described in Appendix A of this report, and a series of beam tests described in the main body of the report.

Beams in the main study had cold joints in the splice region, along the plane of the reinforcement, to facilitate the initiation of a crack prior to failure. Two No. 3-bar hoops (one on each side) crossing the plane of the cold joint, in the center of the specimen and on the exterior of the lap splices, were used to simulate the effects of the continuity of concrete in an actual structure.

The beams contained two spliced No. 11 bars with 79 or 120-in. long lap splices. Some of the beams were loaded until horizontal cracks had developed along the plane of the cold joint with a minimum width of 10 mils (0.01 in.); they were then unloaded and subsequently reloaded to failure. The remainder of the beams were loaded monotonically to failure.

2 Research Program and Test Specimens

2.1 Design of test specimens

A total of six beam-splice specimens were tested in the main study – three specimens with a splice length of 79 in. and three with a splice length of 120 in. For the three specimens with a 79-in. splice length, one was cast with monolithic concrete and the other two were cast with a cold joint in the plane of reinforcing steel. All three specimens with a 120 in. splice length were cast with a cold joint in the plane of reinforcing steel. All specimens with cold joints had two No. 3-bar hoops crossing the plane of the cold joint, outside the spliced bars, at the center of the specimen.

The beams were subjected to four-point loading to provide a constant moment (excluding dead load) in the middle portion of the member, where the splice was located, as shown in Figure 2.1.

The specimens were configured to have a constant moment in the splice region to eliminate the effect of shear forces on splice strength, and also to eliminate the need for shear reinforcement within the splice region. The spacing of the supports was chosen so that the distance from either end of the splice to the central pin and roller supports was equal to or greater than the effective depth of the beam. The span lengths were selected in increments of 3 ft based on the spacing of load points in the Structural Testing Laboratory of University of Kansas.

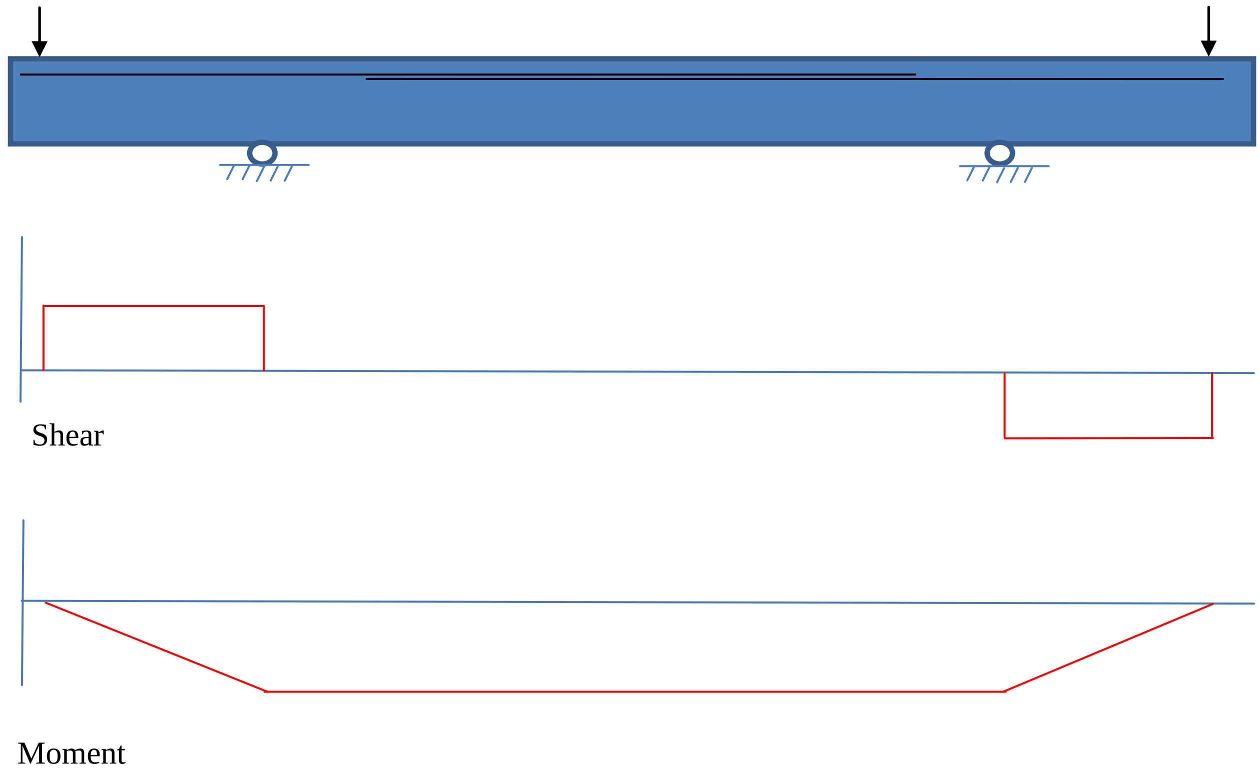


Figure 2.1 – Configuration and shear and moment diagrams for the testing fixture

The reinforcement diagrams for the specimens in the study are shown in Appendix B. The top reinforcement layer of the beams consisted of two No. 11 reinforcing bars, which were spliced at the center of the beam, as shown in Figure 2.1. The No. 11 bars used in the specimens were from a single heat of reinforcement. The bottom layer of reinforcement, placed to maintain the integrity of the beam after failure of the splice and to facilitate placement of shear reinforcement in the constant shear regions, consisted of two Grade 60 No. 3 bars. Beam dimensions and effective depths are summarized in Table 2.1.

The specimens were proportioned to have two splices, each with a nominal side concrete cover of 3 in. to the outermost No. 11 bars and a top concrete cover of 3 in. Grade 60 No. 5 closed hoops spaced at 5 in. on center were placed in the constant shear region (Figure 2.1) of all six beams. Mill certifications for the No. 11, No. 5 and No. 3 bars are reported in Appendix D.

Table 2.1 – Summary of design beam dimensions for beam-splice specimens

Specimen	Splice length (in.)	Simulated crack	Nominal Beam dimensions					
			Support Spacing (ft)	Span, L (ft)	Width, b (in.)	Height, h (in.)	Effective Depth, d (in.)	Depth to A_s', d', (in.)
B1	79	None (monolithic)	11	25	18	24	20.3	2.8
B2	79	Cold joint	11	25	18	24	20.3	2.8
B3	79	Cold joint	11	25	18	24	20.3	2.8
B4	120	Cold joint	14	28	18	24	20.3	2.8
B5	120	Cold joint	14	28	18	24	20.3	2.8
B6	120	Cold joint	14	28	18	24	20.3	2.8

The deformation properties of the No. 11 bars are summarized in Table 2.2. The mean deformation height and spacing of the No. 11 bars meet the requirements of ASTM A615 and the relative rib area, 0.0685, is within the typical range for conventional reinforcement in U.S. practice (0.060 and 0.085) (ACI 408R-03).

Table 2.2 – Summary of design beam dimensions for beam-splice specimens

Bar Size	Mean height* (in.)	Mean height** (in.)	Mean spacing (in.)	Relative rib area
No. 11	0.0811	0.0752	0.869	0.0685

*Per ASTM A 615 **Per ACI 408R-03 and ACI 408.3R-09 for calculation of relative rib area

2.2 Concrete

The concrete used to fabricate the test specimens was supplied by a local ready mix plant. The concrete was non-air-entrained with Type I portland cement, 1½-in. nominal maximum-size crushed coarse aggregate, and a water-cement ratio of 0.42. A trial batch was prepared at the concrete laboratory of the University of Kansas prior to casting the first three beams. The aggregate gradation, mixture proportions, and concrete properties for the trial batch and each of the placements are presented in Appendix E. The dosage of high-range water reducer was adjusted on site when considered necessary to obtain adequate slump for placement.

2.3 Cold joint construction and crack simulation

The specimens with cold joints were cast using two placements, with a cold joint at the mid-height of the top layer of reinforcement, to ensure that a longitudinal crack would develop in the plane of the reinforcing steel before the beam failed. The cold joints spanned the entire length of the spliced region and extended approximately 6.5 ft outside of the spliced region (Figures B.2 and B.3 in Appendix B).

In the first placement, concrete was cast up to the center of the top layer of reinforcement (Figures 2.2, B.2 and B.3). After the concrete was placed, a roughened surface was created to simulate the roughness of a natural crack by introducing indentations in the concrete while it remained plastic (Figure 2.3). The exposed reinforcing steel was cleaned using sponges to facilitate adequate bond between the exposed bars and the concrete cast during the second casting stage. The specimens were moist cured for a day, and the remainder of the concrete was placed no later than 26 hours after the original placement. The concrete for the second placement had the same mixture proportions and was supplied by the same ready-mix plant as the first. Before the second placement, the concrete surface was cleaned using compressed air to remove debris and loose concrete, and maintained in a wet condition until the second placement started (Figures 2.4 and 2.5). After casting, the specimens were moist-cured until the compressive strength of the concrete from the first placement exceeded 3500 psi.

Some beams were loaded in two stages to ensure that the preexisting crack of minimum width had formed in the plane of the reinforcing steel. To do this, beams were loaded monotonically until the width of the horizontal cracks at the cold joint exceeded 10 mils (0.01 in.). After initial loading, the specimens were unloaded and subsequently reloaded monotonically to failure.

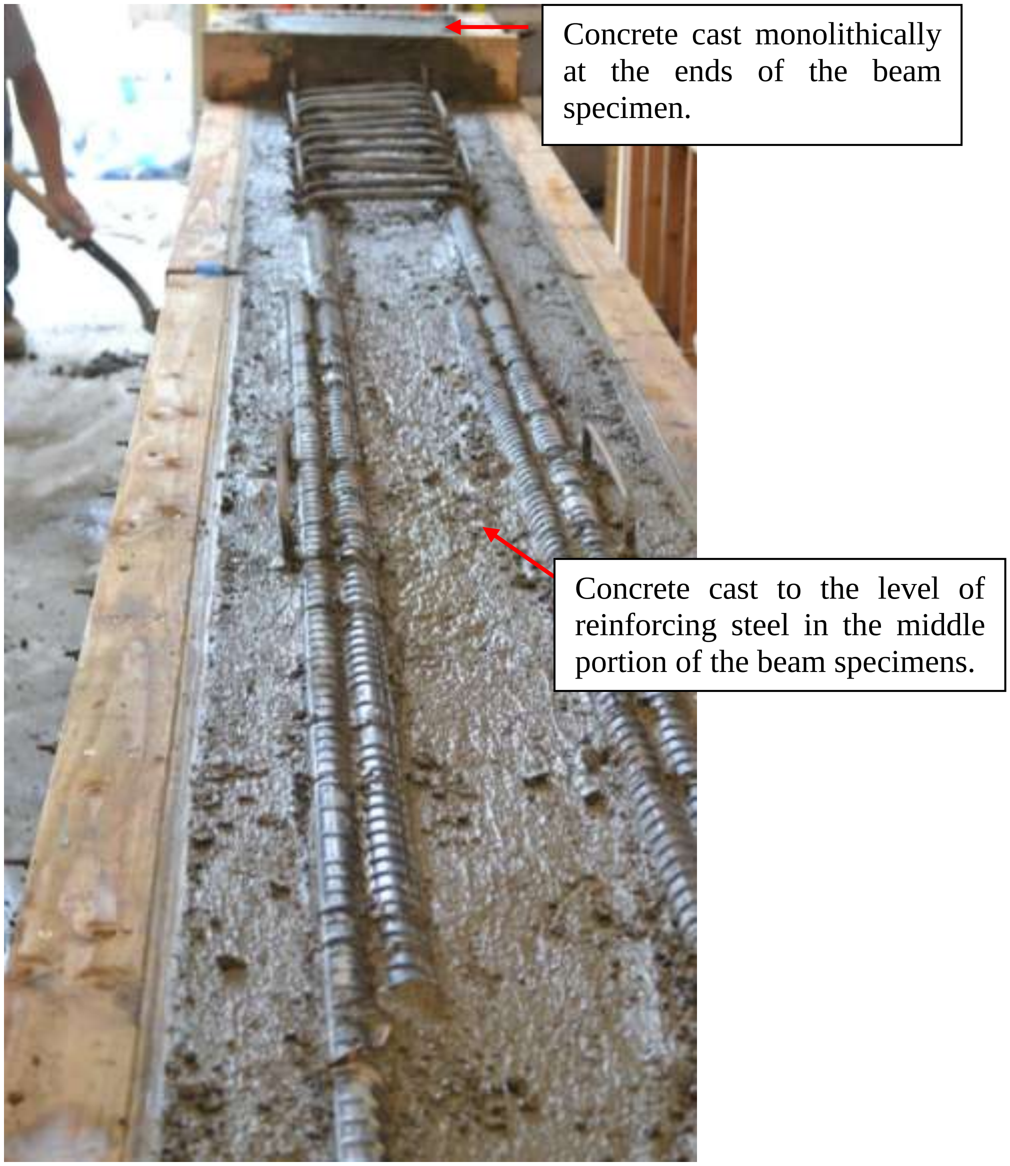


Figure 2.2 – Beam specimen after first stage of casting was completed.

(a) (b)

Figure 2.3 – Roughening of the concrete surface at the cold joint. (a) roughening of the concrete surface while the concrete remains plastic. (b) roughened surface after concrete had set.

Figure 2.4 – Removal of loose concrete using compressed air.

Figure 2.5 – Wetting of concrete surface prior to concrete placement.

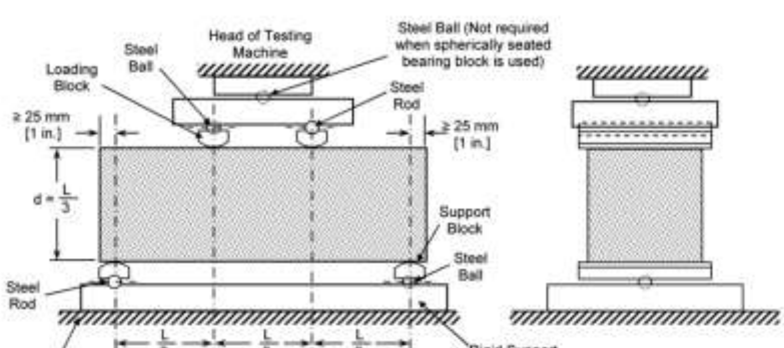
The flexural strength of the concrete (a measure of its tensile strength) was measured in accordance with ASTM C78. For each set of beams, two specimens were cast monolithically with concrete from below the cold joint and two were cast with a cold joint at midspan in the flexure specimen using concrete from both below and above the cold joint in the beam. For Beams 4, 5, and 6, two additional flexure beams were cast monolithically using the concrete from the second placement (above the cold joint). The specimens with the cold joint were cast so that half of the total length was filled with concrete from below the cold joint in the splice specimens; the concrete surface was then roughed (

(c)

Figure 2.6) following the same procedure used for the beam-splice specimens (Figure 2.3 to Figure 2.5). The second half of the "cold joint" flexure specimens was cast using concrete from above the cold joint in the splice specimens. A schematic of the flexure test is shown in Figure 2.6(c). The test results are summarized in Chapter 3 and indicate that the cold joint had significantly lower tensile strength than monolithically-cast concrete, and thus provided a good representation of a preexisting crack.

(a)

(b)



(c)

Figure 2.6 – Flexure beam specimens with cold joint. (a) A flexure beam specimen cast to half of its length. (b) Roughed concrete surface. (c) Schematic of flexure test (ASTM C78).

2.4 Test methodology

2.4.1 Fabrication

Formwork

The formwork was fabricated using plywood and dimension lumber with the tolerances specified in Table 2.. The interior of the forms was coated with a sealant, taped at the seams to prevent leakage, and covered with a thin layer of oil before casting. The dimensions of the formwork were measured and are recorded in Table F.1 (Appendix F).

Table 2.3 – Form tolerances

	Width, in.	Height, in.	Beam Length	
			79-in. Splice Specimens	120-in. Splice Specimens
Nominal	18	24	25 ft	28 ft
Tolerance	± 1/2	±1/2	± 1 in.	± 1 in.

Reinforcement

The steel reinforcement was fabricated to meet the dimensions specified in the drawings shown in Appendix B. In the splice test region, the bar spacing, concrete cover, location of simulated cracks, and splice length satisfied the tolerances specified in Table 2.. Outside of the splice test region, the bar spacing, concrete cover, and longitudinal bar location satisfied the intended tolerance of ± 1/2 in. Inside the forms, the reinforcing steel was supported by chairs tied to the bottom of the hoops outside of the test region (splice region) and to the bottom layer of longitudinal reinforcing steel in the splice region. Spliced bars were supported by small-diameter threaded rods attached to both sides of the forms. The threaded rods were introduced with the objective of achieving the specified tolerance in the cover dimensions and preventing bowing of the forms at the top of the beams. Cover and reinforcement dimensions in the test region were measured and are recorded on Table F.2 (Appendix F).

Table 2.4 – Specified tolerances within the splice region

	Concrete cover, in.	Location of the centroid of the reinforcement, in.	Splice length, in.
Tolerance	± 1/2	± 3/8	± 1/2

2.4.2 Casting

The properties of the plastic concrete were measured in accordance with the ASTM standards cited and are presented in Table F.3. The following properties were recorded:

- Unit Weight (ASTM C138)
- Slump (ASTM C143)
- Concrete Temperature (ASTM C 1064)

The concrete truck operator delivered a ticket with the batched mixture weights. The ticket was examined to verify that the mixture delivered had the specified proportions and that the concrete had arrived less than 45 min. after leaving the batching plant. No water was added to the concrete after the truck left the plant.

The beams were cast in two layers, beginning and ending at the ends of the beams. The bottom and top layers of concrete in the splice regions of all three beams were placed from the middle portion of the batch. The concrete was sampled at two points in the middle portion of the batch in accordance with ASTM C172, the first sample taken immediately after placing the first lift, and the second sample taken immediately after placing the second lift in the splice regions. After placing the second lift, excess concrete was removed from the formwork using a screed. The upper surfaces of the specimens were finished using hand floats.

The samples were consolidated prior to testing the plastic concrete (for slump, unit weight, and concrete temperature) and making the strength test specimens. Ten plastic and six steel 6 × 12-in. cylinder molds were filled in accordance with ASTM C31, along with four flexural beam specimens cast in accordance with ASTM C78. Two of the flexural beam specimens were cast monolithically and two were cast with cold joints. Cylinders cast in plastic molds were used for monitoring the strength of the concrete prior to testing the beam; the cylinders cast in steel molds were used to obtain the compressive strength on the day of test of each beam. All flexural beam specimens were tested on the day of test of the corresponding beam. Test beams and cylinders were labeled with an identifying mark.

For specimens with a cold joint, the concrete above the joint plane was placed no later than 26 hours after the initial placement. The concrete above the cold joint had the same mix proportions as the concrete below the cold joint and was supplied by the same ready-mix plant. The concrete slump, unit weight, and temperature were recorded. A minimum of five 6 × 12-in. cylinders (two in plastic molds and three in steel molds) were prepared. The two cylinders cast using plastic forms were tested on the day of form removal when the concrete below the cold joint had achieved a compressive strength of 3,500 psi. The three cylinders cast using steel molds were used to determine the concrete compressive strength on the day the beams were tested.

2.4.3 Curing

Test cylinders and flexure beam specimens were stored and cured next to the beam-splice specimens and under similar conditions of temperature and humidity. The beams were covered with wet burlap immediately after finishing of the beam surface. The beams, flexure beams, and the 6 × 12-in. cylinder specimens were moist-cured by keeping them covered with wet burlap and plastic until the measured compressive strength of the concrete exceeded 3500 psi. The plastic cylinder molds were sealed with plastic caps during the period in which the beams were wet cured.

The beam formwork and the molds were removed after the 3500 psi threshold was exceeded. After demolding and removal of the forms, the specimens were air-cured until the measured compressive strength reached 5000 ±500 psi.

2.4.4 Test apparatus

The beam-splice specimens were tested using a four-point loading configuration (Figure 2.1 and Figure 2.7). To facilitate inspection of the splice region during the test, the loads were applied in the downward direction (Figure 2.7) so that the main flexural reinforcement would be located at the top of the beam. The splice region was located between the two supports (Figure

2.7) in the central constant moment region of the beam. The final location of the supports was measured (to the nearest $^1/_8$-in.) and is reported in Table F.4 (Appendix F). As-built external dimensions of each test beam were recorded using the same form. The maximum deviation from nominal dimensions in the test region was ½ in.

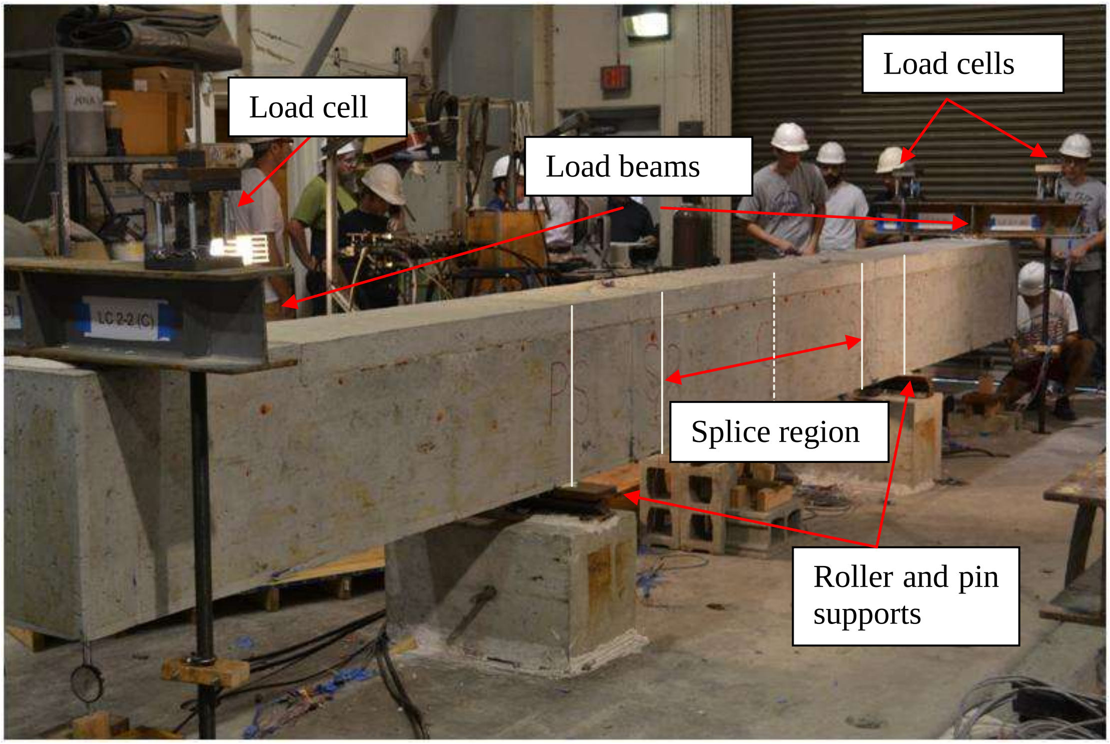


Figure 2.7 – Four-point loading configuration

Loads were applied at the ends of the specimen using two loading frames, as shown in Figure 2.7. Each loading frame consisted of two load rods attached to a loading beam that was placed above the specimen. Loading was imposed through dual-acting center-hole hydraulic rams attached to the lower surface of the reaction floor. At the start of the test, the lower end of the load rods passed through the reaction floor without applying load to the specimen other than the weight of the loading frame and the rods. A total of four rams were used, two for each loading frame. High-pressure hydraulic lines connected the rams to separate pressure and return manifolds, which were connected to the pressure and return lines of a single hydraulic pump. All hoses and other hydraulic hardware were inspected visually before testing began.

The beams were instrumented to measure displacement and load. As shown in Figure 2.8, the applied load was measured with load cells mounted on the load rods, and displacements were recorded using displacement transducers and dial gages (for redundancy) at the center of the beam and at each of the two load points.

Within each specimen, 350-ohm ¼-in. strain gages with attached leads were bonded to the spliced bars, approximately 2 in. outside the edges of the splice. One deformation in each bar was removed using low-heat grinding to provide a smooth surface to attach the strain gages. Strain gages were attached to the bars using epoxy cement and sealed following the recommended procedures by the manufacturer for submersion in concrete. The strain gages were placed so that the coating used to seal the strain gages covered only deformations outside of the splice region. The strain gages provided little useful data.

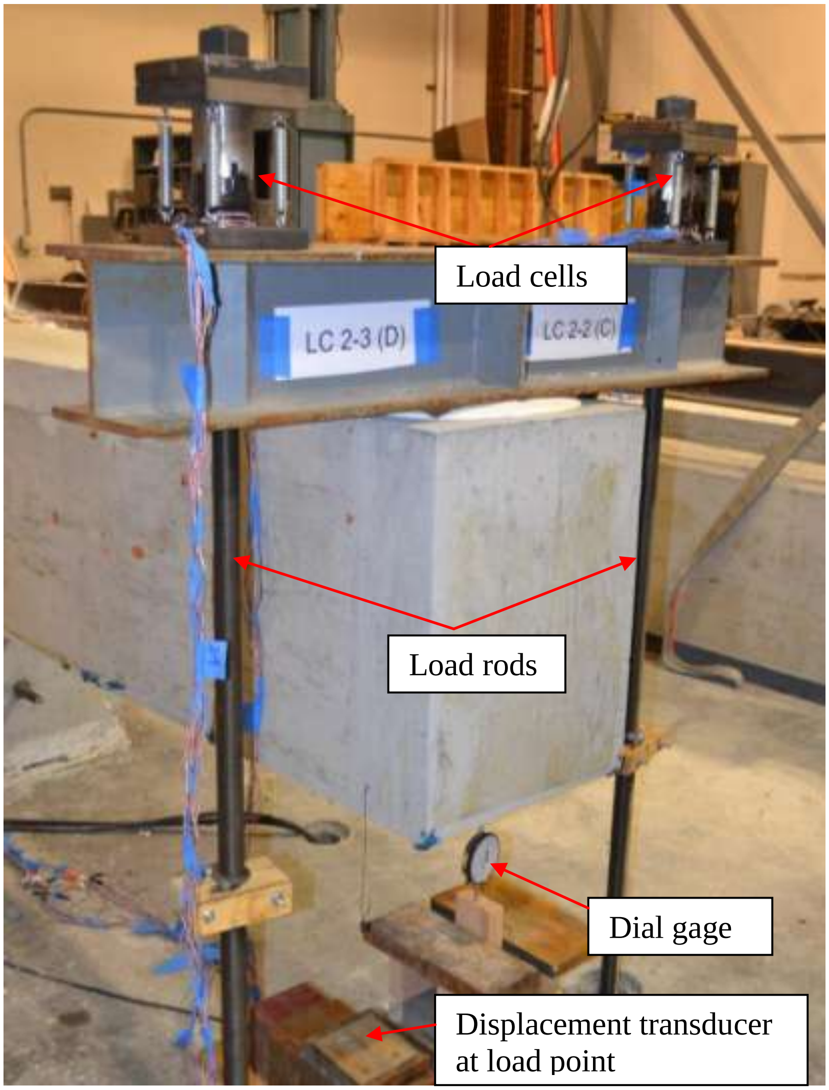


Figure 2.8 – Loading apparatus and instrumentation at each load point

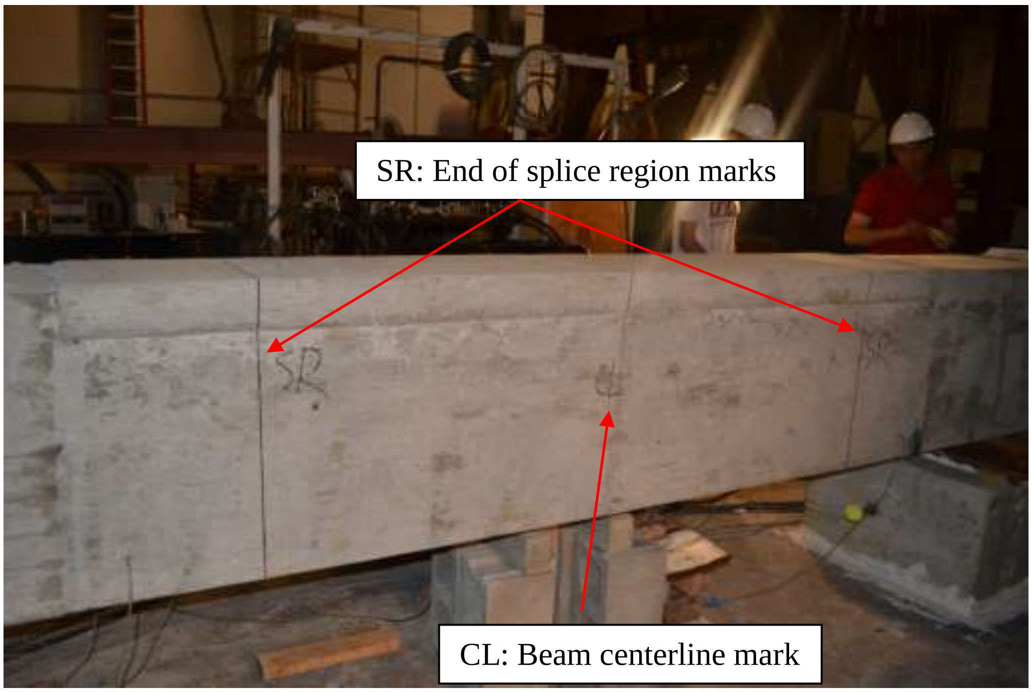
2.4.5 Loading protocol

The double acting rams were fully retracted prior to the start of each test. With the loading rams in the fully retracted position, slack was taken out of the load rods by tightening the nuts until each load rod was nearly engaged with the fully retracted hydraulic jacks, but without applying any load. This procedure was adopted to prevent rotation of the loading beams and consequently maintain even loading across all four rods.

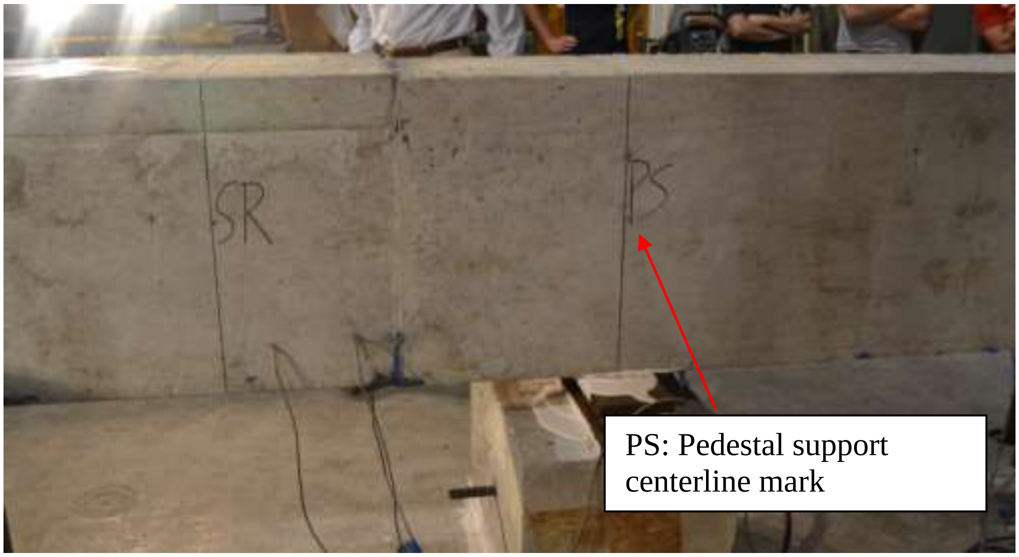
Before load was applied, all displacement transducers, load cells, and strain gages were zeroed and initial readings were recorded for each of the three dial gages. Data were recorded continuously by the data acquisition system with a sampling rate of approximately one sample per second. Recorded data was continuously appended to a data file to prevent any loss of data in case of system failure.

Load was applied using a single manually-controlled hydraulic pump. Loading was stopped at predetermined load levels to visually inspect the beam, mark visible cracks (identified based on the average value of the load applied at one end of the beam, as illustrated in Figure 2.9), measure crack widths using crack comparators, and to record strain and dial gage readings. The specimens were marked before each test to indicate the locations of the ends of the splice region, the beam centerline, the pin and roller (pedestal) supports, and the load apparatus. The markings, shown in Figure 2.10, were 'SR' to indicate the ends of the splice region, 'CL' for the centerline of the beam, and 'PS' for the center of the pedestal support. All longitudinal measurements were taken using the centerline of the beam as a reference point to eliminate any inconsistencies caused by small deviations from the nominal length in the specimens.

Figure 2.9 – Crack inspection and marking during test



(a)



(b)

Figure 2.10 – Beam marks: (a) End of splice region and centerline of the beam; (b) pedestal support centerline

The initial load increment was chosen to be smaller than one half of the calculated flexural cracking load to ensure that all instruments and the hydraulic system were operating properly. From this point forward, loading proceeded in increments of approximately 5 kips at each end of the beam. The final load step at which cracks were marked was approximately two-thirds of the estimated failure load. In some of the specimens, the loading protocol was such that the specimens were unloaded after the formation of a horizontal crack with a width of at least 10 mils in the splice region. After the specimen was fully unloaded, it was loaded to failure following the procedure specified above for monotonically-loaded specimens. The loading protocol used for each beam is presented in Table 2..

A log was maintained to record any meaningful observations during the test, such as load corresponding to flexural cracking, crack widths, file names, and gage readings. The logs are presented in Appendix H.

After failure, cracks were marked on the specimens with each identified using the preliminary value of the average maximum end load (this value typically deviated by a few percent from the recorded value).

The following data were recorded and continuously transferred to disk throughout each test:

- Force applied to each load rod
- Displacement at midspan and each load application point
- Strain in the reinforcing steel

Table 2.5 – Detailed loading protocol for each beam

Beam	Loading Protocol
1	(1)Monotonically-increasing load up to an average end load of 40 kips in increments of 5 kips. At the end of the each increment, the beam was inspected for cracks and dial-gage displacement measurements were recorded. (2) Loading resumed with increasing displacement until failure.
2	(1)Monotonically-increasing load up to an average end load of 25 kips in increments of 5 kips. At the end of the each increment, the beam was inspected for cracks and dial-gage displacement measurements were recorded. (2) Dial-gage measurements were recorded at an average end load of 30 kips. (3) Loading resumed with increasing displacement until failure.
3	(1)Monotonically-increasing load up to an average end load of 30 kips in increments of 5 kips. At the end of the each increment, the beam was inspected for cracks and dial-gage displacement measurements were recorded. (2) The beam was fully unloaded and dial-gage displacement measurements were recorded. (3) The beam was loaded a second time up to an average end load of 35 kips in load increments of 5 kips. At the end of the each increment, dial-gage displacement measurements were recorded. The beam was inspected for cracks at an average end load of 30 kips. (4) Loading resumed with increasing displacement until failure.
4	(1)Monotonically-increasing load up to an average end load of 35 kips in increments of 5 kips. At the end of the each increment, the beam was inspected for cracks and dial-gage displacement measurements were recorded. (2) Loading resumed with increasing displacement until failure.
5	(1)Monotonically-increasing load up to an average end load of 40 kips in increments of 5 kips. The beam was inspected for cracks and dial-gage displacement measurements were recorded at the end of each increment. (2) The beam was fully unloaded and dial-gage displacement measurements were recorded. (3) The beam was loaded a second time up to an average end load of 40 kips in increments of 5 kips. Dial-gage displacement measurements were recorded at the end of each increment. The beam was inspected for cracks at average end loads of 20, 30, 35 and 40 kips. (4) Loading resumed with increasing displacement until failure
6	(1)Monotonically-increasing load up to an average end load of 40 kips in increments of 5 kips. The beam was inspected for cracks and dial-gage displacement measurements were recorded at the end of the each increment. (2) The beam was fully unloaded and dial-gage displacement measurements were recorded. (3) The beam was loaded a second time. The beam was inspected for cracks and dial-gage displacement measurements were recorded at average end loads of 10, 20, 30, 35, and 40 kips. (4) Loading resumed with increasing displacement until failure.

2.4.6 Calibration

Instruments used to measure force and displacement were calibrated following the procedure specified in this section. The applied load was measured using load cells. Displacement transducers (either linear variable differential transformers or string potentiometers depending on availability) were used to record the vertical beam deflections. Load cells and displacement transducers were calibrated using a digitally-controlled hydraulic test frame calibrated annually using NIST-traceable standards. Load cell and displacement transducers were calibrated following the steps listed below:

1) The sensor (load cell or displacement transducer) was connected to the data-acquisition system that was used in the test.
2) The sensor was securely mounted on the testing machine.
3) A series of known force or displacement increments were applied to the sensor. Calibrations were performed exceeding the displacement and load range expected during the tests. In the case of load cells, calibrations were performed between zero and 100 kips. In the case of displacement, calibrations were performed in a range between zero and 4 in.
4) Sensor output was recorded with the data-acquisition system at each known displacement or force increment.
5) A least-squares linear regression analysis was performed on force and displacement versus sensor output to determine the calibration constant.

The load cells and displacement transducers were calibrated before and after testing each three beams and the calibration results are reported in Appendix G. The calibration constant deviated with an average value of 0.28% for all sensors, ranging between 0 to 0.84%.

2.5 Test Facilities

The tests were performed in the Structural Testing Laboratory at the University of Kansas, a facility of the KU Structural Engineering and Materials Laboratory (SEML). The Laboratory has static and servo-hydraulic test equipment for the testing of steel, concrete, and composites. The structural testing bay has 4000 square feet of open laboratory area with a clear height of 30 ft for large-scale structural testing. Loads up to 100,000 lb can be applied on 3-ft centers over a 50 x 80 ft area. The laboratory houses a 600,000-lb universal testing machine for testing steel and concrete. A 450,000-lb MTS Structural Test System supported on a four-column test frame may be used for dynamic and cyclic testing of large scale structural components. 110,000-lb and 55,000-lb MTS Structural Test Systems are also used for cyclic and dynamic testing of full-scale structural components within the test bay. Actuators within the test bay are powered by two hydraulic pumps (total flow rate of 110 gpm), meeting the requirements for demanding cyclic test applications. High-speed Mars Labs, National Instruments (used in the current study), and Hewlett Packard data acquisition systems are available to monitor and record load, strain, and displacement. The structural testing laboratory includes an overhead 20-ton crane with access to the entire lab floor area. Over 500 beam-end tests and over 200 splice tests have been performed in the KU Structural Testing Lab since 1990.

Material tests were performed in the Concrete Laboratory, another SEML facility, which is equipped to run standard tests on cement, aggregates, and concrete. Equipment is available to test concrete aggregate for deleterious behavior, including alkali silica reactivity, and to measure aggregate properties as they affect mixture proportioning. Freeze-thaw equipment is available for running tests under both Procedures A and B of ASTM C666. A walk-in freezer is used for scaling tests. Concrete is cured under controlled temperature and humidity in the lab's curing room. Two hydraulic testing machines, with load capacities of 180 tons (400,000 lb), are used for concrete strength determination.

Certificates of calibration for the equipment used in this study, including for the test frame used to calibrate the sensors, are presented in Appendix I.

2.6 Section Analysis

Splice strength was evaluated based the calculated moment in the splice region at failure (ACI 408R-03). Loads, moments, and stresses for the beams were calculated using a two-dimensional analysis in which loads and reactions were assumed to act along the longitudinal centerline of the beam. Reactions and moments were based on load cell readings and the weight of the loading assemblies. The self-weight of the beam was included in the calculations based on average beam dimensions and an assumed concrete density of 150 pcf.

The test specimens were evaluated using cracked section theory assuming a linear strain distribution through the height of the cross-section. The beams were analyzed using an equivalent rectangular stress block and moment-curvature analyses for comparison. The moment-curvature relationship was derived using the nonlinear stress-strain relationship for concrete proposed by Hognestad (1951) and follows the procedure described by Nilson, Darwin, and Dolan (2010). Figure 2.11 shows the assumed stress distribution in the compression zone for the moment-curvature and the equivalent rectangular stress block analyses. Good agreement in the calculated bar stress at failure was typically noted between results obtained with the two methods.

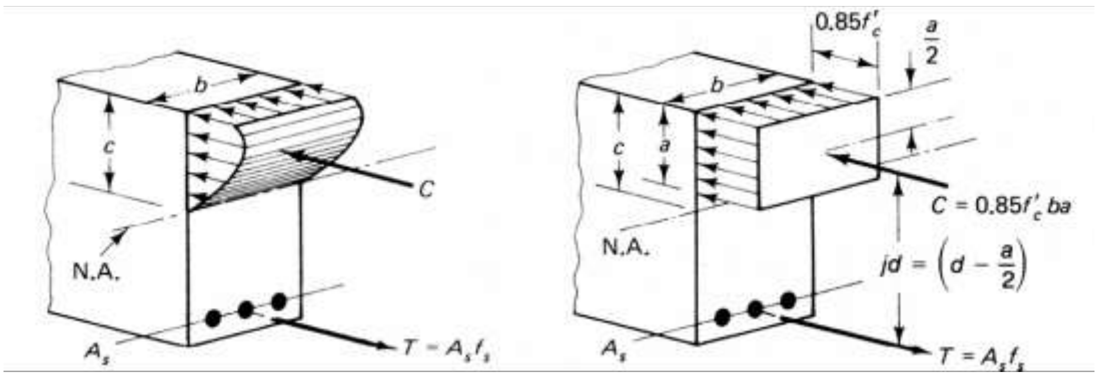


Figure 2.11 – Assumed stress distribution in the compression zone used in moment-curvature and equivalent rectangular stress block analyses [after Nawy (2003)]

In calculating splice strength, the tensile stress in the steel f_s (ksi) was calculated as following the procedures used by ACI Committee 408 (2003):

$$f_s = E_s \times \varepsilon_s = 29000 \times \varepsilon_s \text{ for } f_s \leq \text{ measured yield strength } f_y \quad (1)$$

For $\varepsilon_s > f_y/E_s$, $f_s = f_y$ for $\varepsilon_s \leq \varepsilon_{sh}$, where $\varepsilon_{sh} = 0.0086$ for $f_y = 60$ ksi and 0.0035 for $f_y = 75$ ksi and above. There is no flat portion of the stress-strain curve for $f_y \geq 101.5$ ksi. The modulus of strain hardening $E_{sh} = 614$ ksi for $f_y = 60$ ksi, 713 ksi for $f_y = 75$ ksi, and 1212 ksi for $f_y \geq 90$ ksi. The values of ε_{sh} and E_{sh} for f_y between 60 and 90 ksi are obtained using linear interpolation.

The equivalent rectangular stress block used in the calculations was proposed by Whitney with the values of the parameter β_1 specified in ACI 318-11. The moment-curvature relationship was calculated using the concrete model proposed by Hognestad (1951).

$$f_c = \begin{cases} f_c''\left[2\left(\frac{\varepsilon_c}{\varepsilon_0}\right)-\left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2\right] & \text{for} \quad \varepsilon_c \le \varepsilon_0 \\ f_c''\left[0.15\left(\frac{\varepsilon_0-\varepsilon_c}{\varepsilon_{cu}-\varepsilon_0}\right)-1\right] & \text{for} \quad \varepsilon_c \ge \varepsilon_0 \end{cases} \tag{2}$$

$$f_c'' = 0.85 f_c' \tag{3a}$$

$$\varepsilon_0 = \frac{1.7 f_c'}{E_c} \tag{3b}$$

$$\varepsilon_{cu} = 0.0038 \tag{3c}$$

$$E_c = 1.8\times 10^6 + 460 f_c' \tag{3d}$$

where:

f_c = concrete stress, psi

f_c' = concrete compressive strength, psi

f_c'' = peak concrete stress, psi

ε_c = concrete strain

ε_0 = concrete strain at peak stress

ε_{cu} = ultimate concrete strain at crushing

E_c = approximate concrete modulus of elasticity, psi

Tensile stresses carried by the concrete were neglected in both analyses.

The calculations using both equivalent rectangular stress block and moment-curvature analyses proceed as follows:

1. Select top face concrete strain ε_c in the inelastic range.
2. Assume the neutral axis depth, at distance c below the top face.
3. Assuming a linear variation in strain throughout the depth of the member, determine the tensile strain in the steel ε_s (equal to the tensile strain in the concrete at the level of the steel ε_{sc}).
4. Compute the stress in the reinforcing steel f_s in accordance with the defined stress-strain relationships (above). The tensile force in the steel $T = f_s \times A_s$ (see Figure 2.11).

5. Determine the compressive force C, which equals to 0.85 $f'_c ba$ (Figure 2.11b) for the equivalent rectangular stress block method, or by numerically integrating the concrete stresses as defined by Eq. (2) and (3) for the moment curvature method.
6. If $C = T$, go to step 7. If not, adjust the neutral axis depth c in step 2 and repeats steps 3 – 5.
7. Using the internal lever arm z from the centroid of the concrete stress distribution to the tensile resultant, the calculated bending moment $M = Cz = Tz$.
8. If the calculated bending moment M equals the applied bending moment (from test), f_s equals the force in the reinforcing steel. If the calculated bending moment does not equal the applied bending moment, modify ε_c and c in steps 1 and 2, respectively, and repeat steps 3 – 7 until the calculated bending moment M equals the applied bending moment.

3 Test Results

3.1 General

The testing program consisted of six beam-splice specimens. Three of the specimens had a lap lap splice length of 79 in., and three had a lap splice length of 120 in. The measured loads and calculated bar stresses at failure are presented in

Table 3.1. In addition to failure loads, Table 3.1 includes measured material properties and bar cover dimensions. Bar stresses at failure listed in Table 3.1 include those calculated using the equivalent rectangular stress block and moment-curvature analysis. Measured specimen dimensions and other details of the beam tests are presented in Appendix H.

Moment-curvature analyses consistently produced calculated higher bar stresses than did the analysis using the equivalent rectangular stress block. This is to be expected because the parameters of the equivalent stress block were calibrated to reflect the characteristics of the compression zone when the peak strain in the concrete exceeds 0.003 and the concrete in the compression zone is well into the nonlinear range. Under these conditions, the depth of the compression zone is reduced, resulting in a slightly larger distance between the tension and compression resultants. With the exception of Beam 1, the splices failed prior to crushing of the concrete in the bottom surface of the beam, so it was to be expected that the equivalent rectangular stress block would slightly overestimate the distance between tension and compression resultants and consequently underestimate the stress in the reinforcing bars. The difference, on average, between the bar stresses at failure calculated by the two methods was 1.5 ksi for the six beams tested in this study, with moment-curvature analysis producing the greater value. All bar stress values discussed subsequently are those calculated using moment-curvature analysis, which is considered to be more accurate method for the reasons stated above.

Table 3.1 – Bar stresses at failure for beam-splice specimens

Beam ID – Splice length	Concrete strength, psi	Concrete cover, in.[a]	Total load at splice failure, kips	Calculated moment at splice failure, kip-ft	Calculated bar stress at failure, ksi		Failure mode
					Equiv. stress block	Moment-curvature	
1 – 79 in. (monolithic)	5330/ 4330[+]	3/3/3	103	344	70	70	Flexural Failure [*]
2 – 79 in. (cold joint, loaded monotonically)		3/3/3	85	292	59	62	Splice failure[**]
3 – 79 in. (cold joint, unloaded and reloaded)		3.25/3.35/2.9	80	270	53	57	Splice failure[**]
4 – 120 in. (cold joint, loaded monotonically)	5230/ 5490[+]	3/2.8/3.4	105	350	71	72	Splice failure and secondary flexural failure[***]
5 – 120 in. (cold joint, unloaded and reloaded)		3.15/3/15/3.15	96	325	66	67	Splice failure and secondary flexural failure[***]
6 – 120 in. (cold joint, unloaded and reloaded)		3.15/3.15/2.9	100	338	69	69	Splice failure and secondary flexural failure[***]

[a] Top cover/north side cover/south side cover
[+] Compressive strength of concrete below and above the cold joint.
[*] Test was stopped after reinforcing steel yielded, when crushing of the concrete in the compression zone was observed.
[**] Splice failed prior to yielding of the flexure reinforcement.
[***] Splice failed after yielding of the flexure reinforcement

3.2 Beams 1, 2, and 3 with 79-in. splice length

3.2.1 Concrete strength

The concrete strengths for Beams 1, 2 and 3 are summarized in Table 3.2. Beam 1 was cast monolithically, while Beams 2 and 3 were cast in two stages to accommodate the presence of a cold joint at the level of the flexure reinforcement. Beam 1 and the concrete below the cold joint for Beams 2 and 3 were placed on May 24, 2012 and the concrete above the cold joint was placed on May 25, 2012. The forms were removed on May 28, 2012, when the average concrete compressive strength for both placements exceeded 3500 psi. All three beams were tested on May 31, 2012. On that date the concrete from the first placement had an average compressive strength of 5330 psi, and the concrete from the second placement had an average compressive strength of 4330 psi (Table 3.2). The average split cylinder strength and the average modulus rupture were 435 and 570 psi for the concrete below the cold joint in accordance with ASTM C496 and ASTM C78, respectively. The tensile strength for the concrete above the cold joint was not recorded for the first three beams. The flexural beam specimens with cold joints were also tested and had an average modulus of rupture of 140 psi, significantly lower than that of specimens cast monolithically. The fact that the tensile strength of the flexural beam specimens with cold joints was significantly lower than the strength of monolithic specimens indicates that the presence of a cold joint did in fact introduce a weak plane at the level of reinforcing steel. The proportions of the concrete mixture and the properties of the concrete for each placement are reported in Table E.2 of Appendix E.

Table 3.2 Concrete strengths for Beams 1, 2, and 3

	Concrete below cold joint	Concrete above cold joint
Average Compressive Strength when forms were removed	4010[a]	3640[b]
Average Compressive Strength at test date, psi	5330[c]	4330[d]
Split Cylinder Strength (ASTM C496), psi	435[c]	--
Modulus of Rupture (ASTM C78), psi	570[c]	--
Modulus of Rupture for specimens with cold joint, psi	140[d]	--

[a]Tested at 4 days; [b]tested at 3 days; [c]tested at 7 days; [d]tested at 6 days

A segment of the No. 11 bars used in the splice-beam specimens was tested in tension and the bar strains were recorded using a linear variable differential transformer (LVDT) used as the extensometer (gage length = 8.0 in.). The measured stress-strain curve for the No. 11 bar is shown in Figure 3.1. The yield stress calculated using the 0.2% offset method was 67 ksi and the measured elastic modulus was 28,990 ksi. The maximum measured steel stress was 105 ksi.

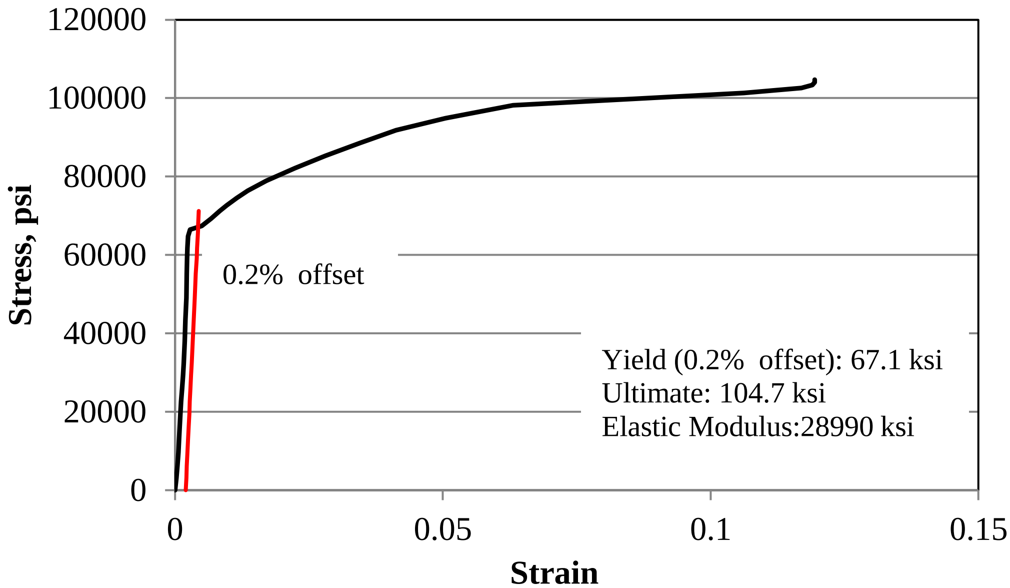


Figure 3.1 – Measured stress-strain curve for No. 11 bar

3.2.2 Beam 1 (monolithic concrete)

3.2.2.1 Beam 1 load-deflection curve

Beam 1 was cast monolithically with a splice length of 79 in. It was loaded monotonically to failure (the load protocol is presented in Table 2.). The load-deflection curve for Beam 1 is shown in Figure 3.2. The displacement shown in the figure was calculated by adding the average of the displacement at the two load points to the displacement at the beam centerline. The load shown in the figure corresponds to the total load applied to the beam (the sum of the two end loads). The load-deflection relationship shows that there was a significant reduction in the stiffness of the beam at a total load of approximately 20 kips, which coincided with the first observation of flexural cracks. Another significant reduction in flexural stiffness was observed at a total load of 94 kips and a total displacement of approximately 2.8 in. In this case the reduction in stiffness is attributed to the yielding of the flexural reinforcement. The calculated bar stress corresponding to the total load of 94 kips is 68 ksi based on moment-curvature analysis (

Table 3.1). The positive slope of the load-deflection relationship after a total load of 94 kips is attributed to the strain hardening of the reinforcing steel. Loading continued until a flexural failure occurred, which was accompanied by crushing of the concrete in the compression zone, near the supports, at a total load of 103 kips, corresponding to a bar stress of 70 ksi, and a total deflection of approximately 5 in. (Figure 3.3).

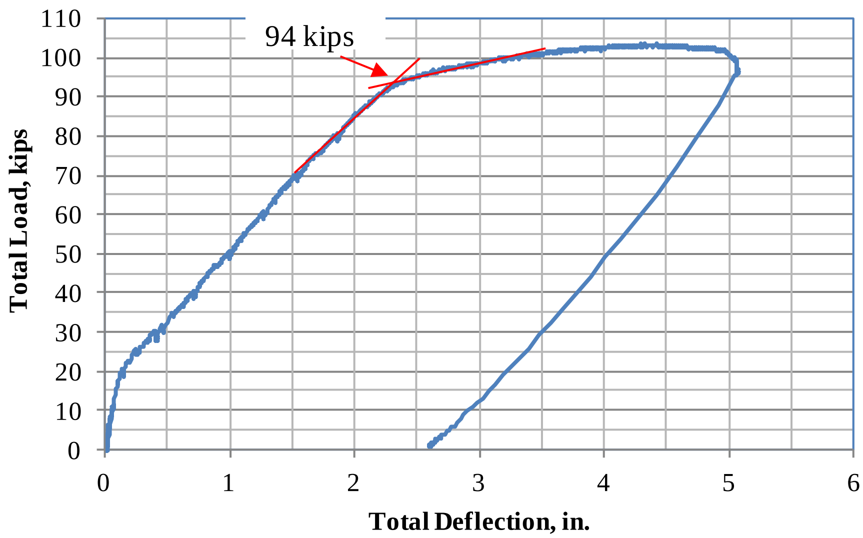


Figure 3.2 – Total load vs. total deflection for Beam 1 (cast monolithically) (Total load calculated as the summation of the two end loads and total deflection calculated defined by adding the average deflection at two ends and the deflection in the beam centerline).

Figure 3.3 – Flexural failure in the compression region for Beam 1. Numbers indicate maximum average end load when cracks marked.

3.2.2.2 Crack progression-Beam 1

Maximum measured crack width versus average end load for Beam 1 is shown in Figure 3.4; the crack map for Beam 1 is presented in Figure 3.5 (see figures in Appendix C for greater detail). The first flexural cracks formed near the east support at the end of the east splice region, at an average end load of 10 kips (total load of approximately 20 kips). The flexural cracks grew progressively wider and more numerous as the load increased. The first horizontal crack formed near the support at an average end load of 25 kips (Figure 3.6). Both longitudinal and flexural cracks continued to increase in width and number as the load increased. At the last crack marking prior to failure (average end load of 40 kips), the widest flexural crack had a width of 25 mils (0.025 in.) and the widest horizontal (bond) crack had a width of 18 mils (0.018 in.).

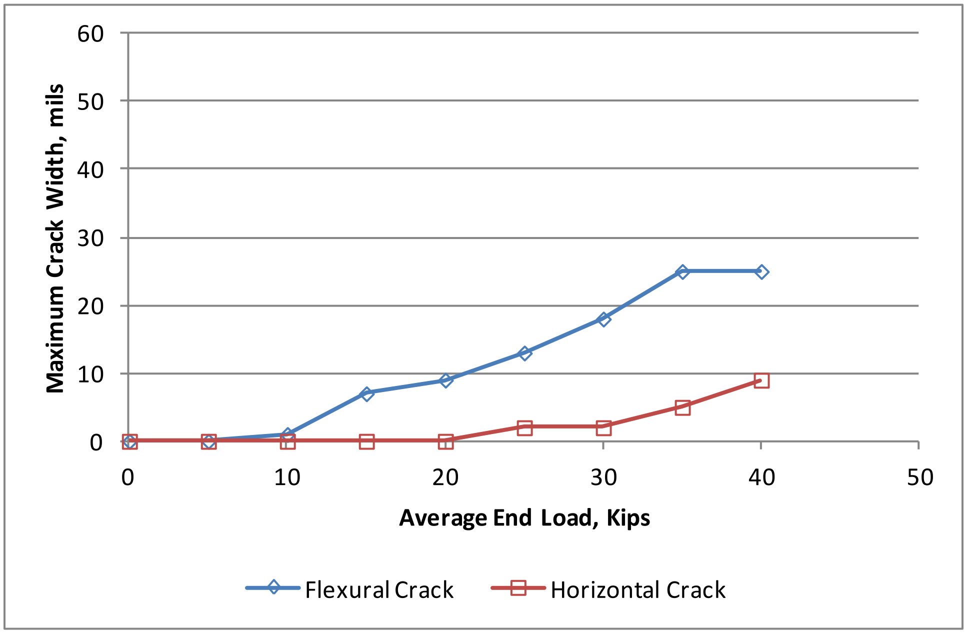


Figure 3.4 – Maximum crack width vs. average end load (one-half of total load) for Beam 1.

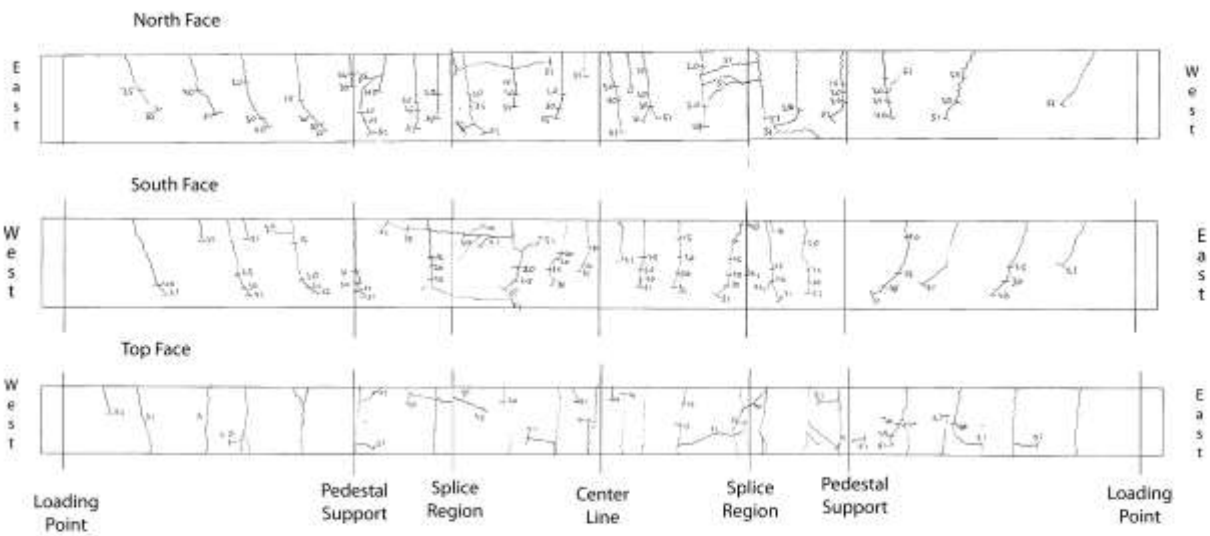


Figure 3.5 – Crack map for Beam 1. Numbers indicate maximum average end load when cracks marked. See Figure C.1 in Appendix C for greater detail.

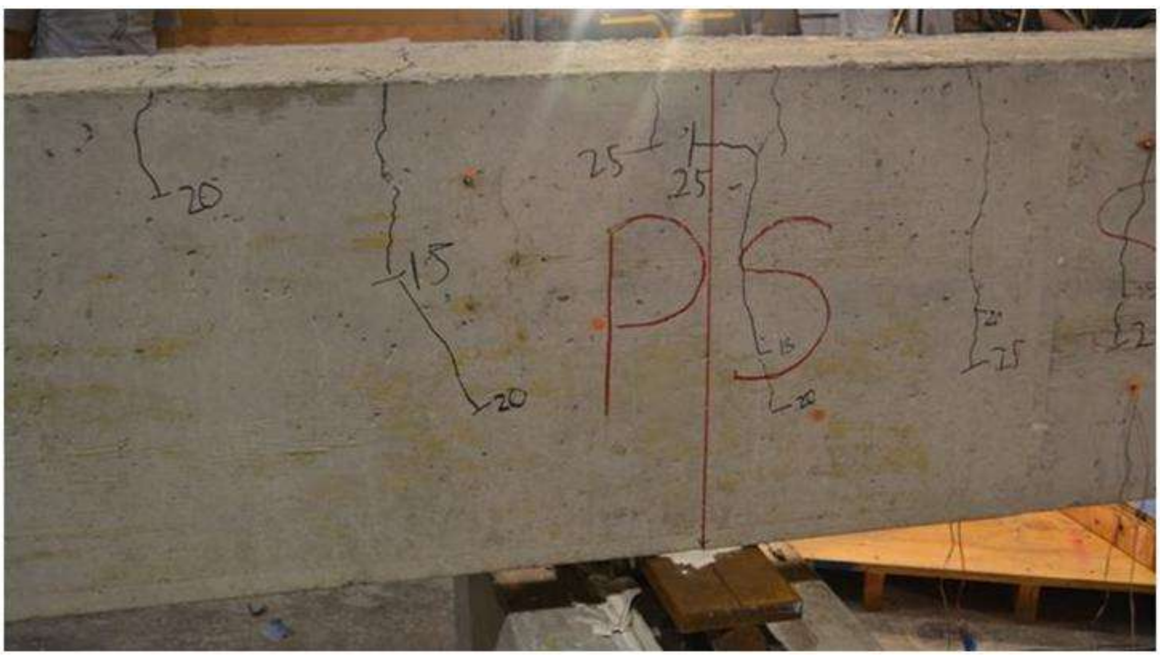


Figure 3.6 – Beam 1, north side of east support with horizontal crack, 25 kip end load.

Failure occurred at an average end load of 51 kips (total load of 103 kips). The failure mode was yielding of the bars followed by crushing of the concrete near the supports (Figure 3.7). Both flexural and horizontal cracks were present near the splice region (Figure 3.8). At the support (Figure 3.9), flexural cracks extended most of the depth of the beam; no horizontal cracks were present.

A detailed autopsy was not performed on Beam 1. Concrete was removed in selected regions to verify the concrete cover to the splice was within tolerances. Top cover was 3 in. to the outer bar in the splice and 3-1/4 in. to the inner bar in the splice for both splices.

Figure 3.7 – Beam 1, underside near support, failure.

Figure 3.8 – Beam 1, north side of west splice region, failure.

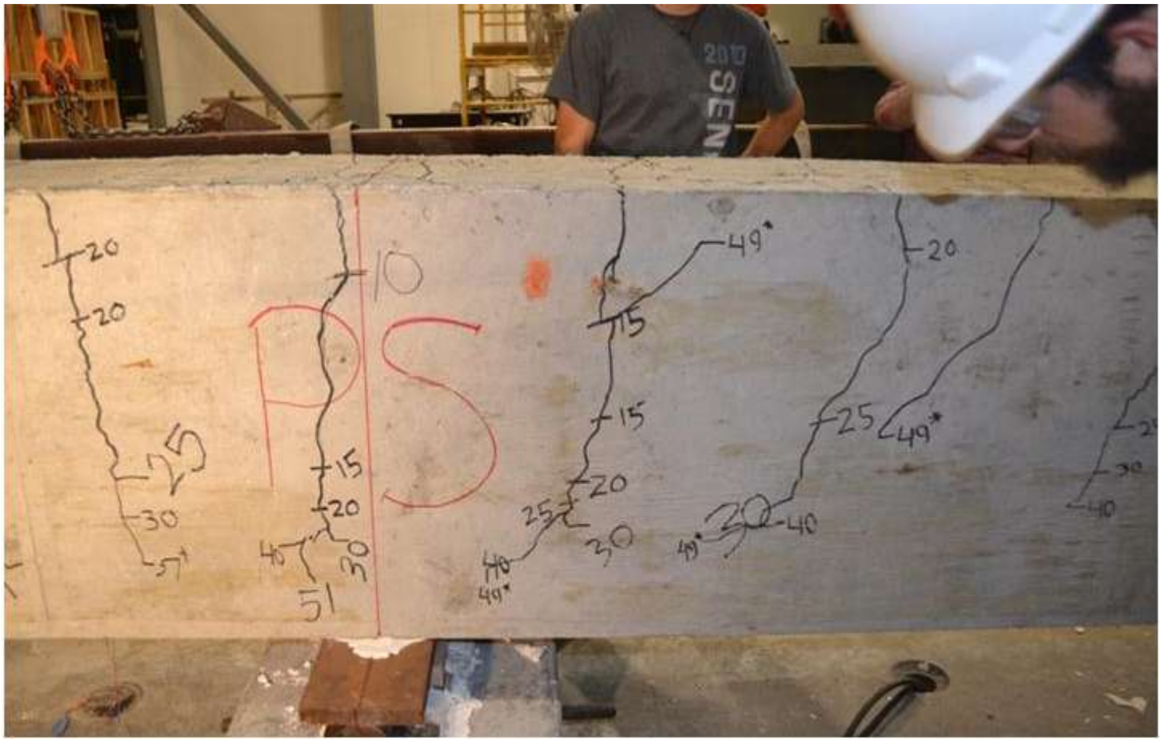


Figure 3.9 – Beam 1, south side of east support, failure.

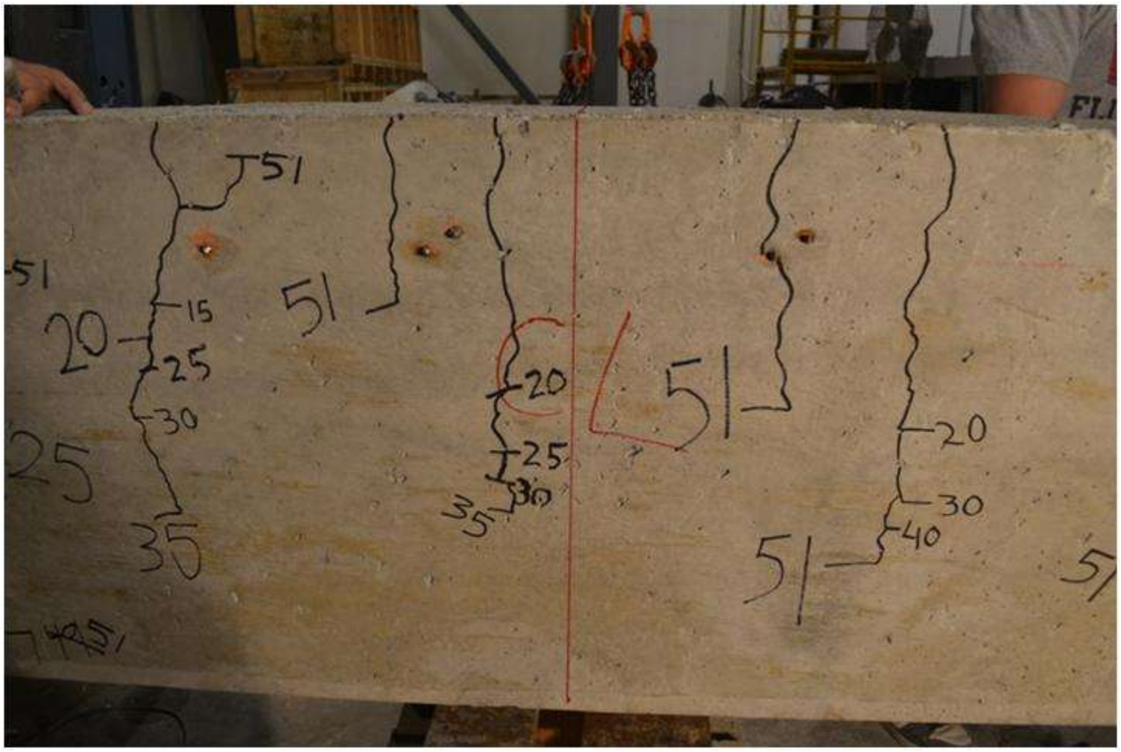


Figure 3.10 – Beam 1, centerline, failure.

3.2.3 Beam 2 (cold joint, monotonically-loaded)

3.2.3.1 Beam 2 load-deflection curve

Beam 2 was cast with a cold joint in the plane of reinforcing steel. It was monotonically loaded loaded with a load increment of approximately 5 kips (average end load, the load protocol is presented in Table 2.). The load-deflection curve for Beam 2 is shown in Figure 3.11. The total displacement and total load shown in the figure were calculated in the same manner as for Beam 1. The total load corresponding to cracking was very similar to that of Beam 1, approximately 20 kips. The beam was loaded to a maximum total load of 85 kips, with a corresponding total displacement of 2.25 in. At this point the beam failed with a sudden splitting of the concrete along the cold joint. Wide horizontal cracks were observed in the plane of the cold joint within the splice region (Figure 3.12). The widest horizontal crack was measured to be ½ in. wide after failure. It is concluded that the beam failed due to failure of the splice at a total load of 85 kips. The calculated bar stress corresponding to the total load of 86 kips is 62 ksi based on moment-curvature analysis (

Table 3.1), above the minimum specified yield strength of 60 ksi for Grade 60 reinforcement but 5 ksi below the actual yield strength of 67 ksi.

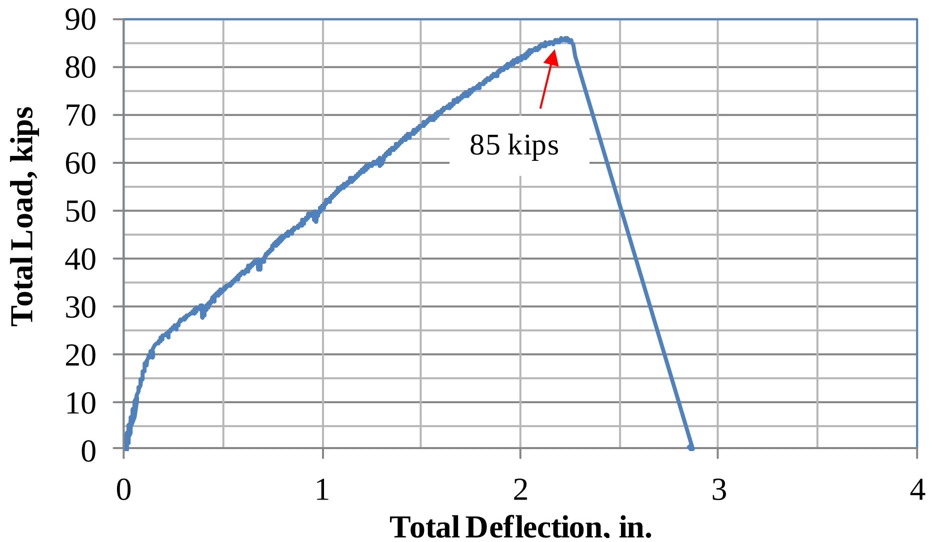


Figure 3.11 – Total load vs. total deflection for Beam 2 (with a cold joint)

Figure 3.12 – Beam 2 (with a cold joint) failed with wide horizontal crack

3.2.3.2 Crack progression-Beam 2

Maximum measured crack width versus load for Beam 2 is shown in Figure 3.13; the crack map for Beam 2 is presented in Figure 3.14. The first flexural cracks formed near the supports and ends of both splice regions at an average end load of 15 kips (total load of 30 kips). Horizontal cracks first formed at an average end load of 20 kips at both ends of the splice region along the cold joint (Figure 3.15). Both longitudinal and flexural cracks continued to increase in width and number as the load increased, with horizontal cracks propagating along the cold joint. When the last cracks were marked prior to failure (conducted at an average end load of 30 kips), the widest flexural crack had a width of 20 mils (0.02 in.) and the widest horizontal crack had a width of 13 mils (0.013 in.).

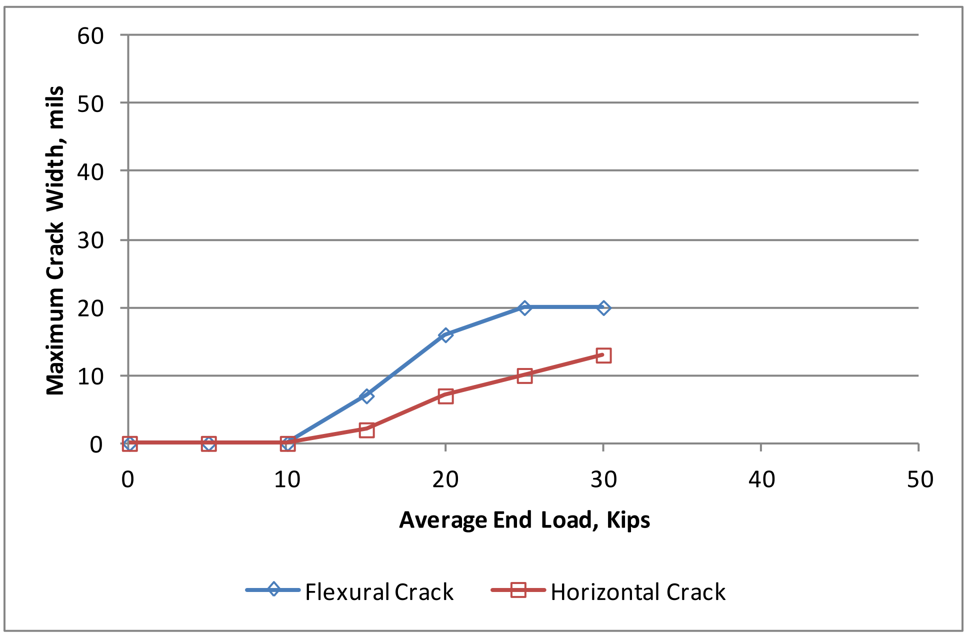


Figure 3.13 – Maximum crack width vs. average end load for Beam 2.

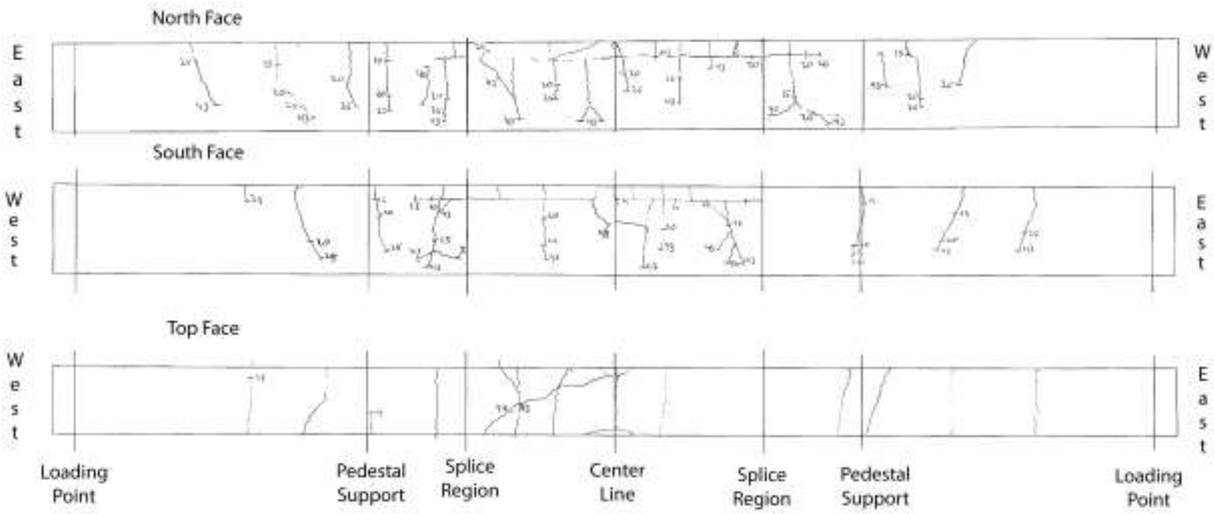


Figure 3.14 – Crack map for Beam 2. Numbers indicate maximum average end load when cracks marked. See Figure C.2 in Appendix C for greater detail.

Figure 3.15 – Beam 2, northeast support with horizontal crack, 20 kip end load.

Failure occurred at an average end load of approximately 43 kips (total load of 85 kips). At failure, the concrete above the cold joint separated from the remainder of the beam (Figure 3.16). Near the splice region, a large flexural crack was also present (Figure 3.16). The horizontal crack progressed approximately 12 in. past both ends of the splice region, and with the exception of near the centerline, continued along the cold joint. At the centerline, the crack split through the cover and around the single hoop present at the centerline (Figure 3.17), indicating the hoop was effective in preventing the crack from growing near the centerline. As shown in Figure 3.17, the region affected by the hoop was small.

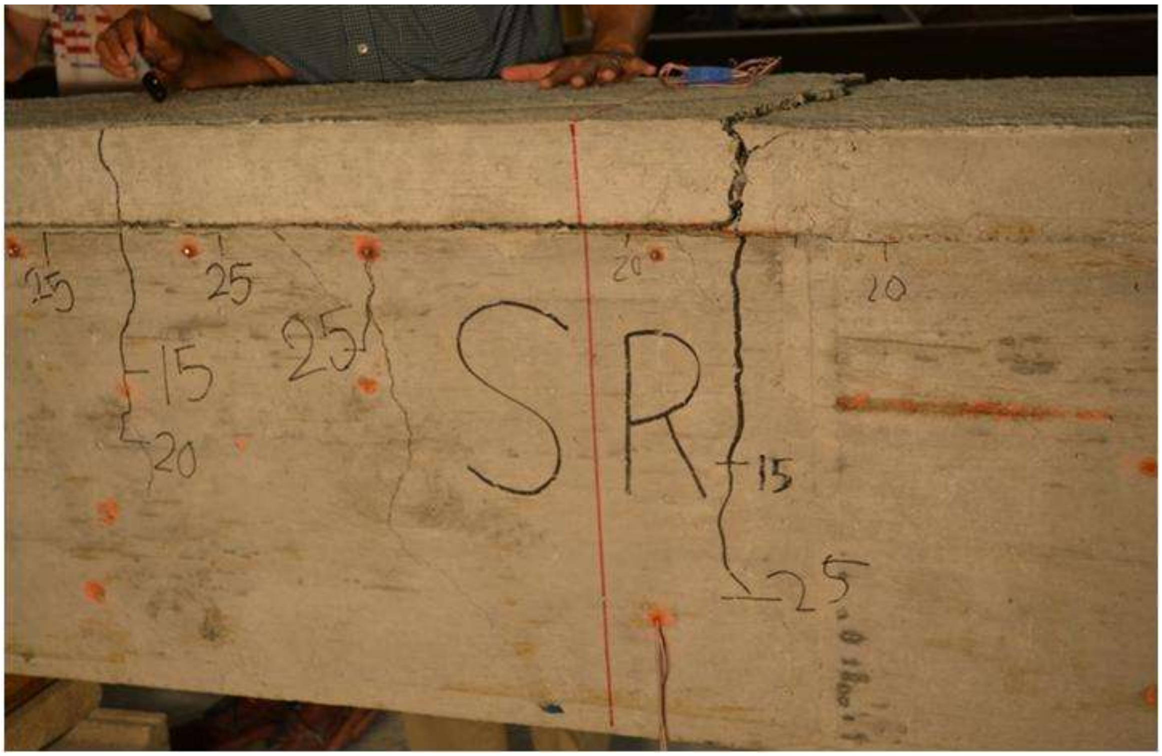


Figure 3.16 – Beam 2, southwest splice region showing separation of concrete, 43 kip end load.

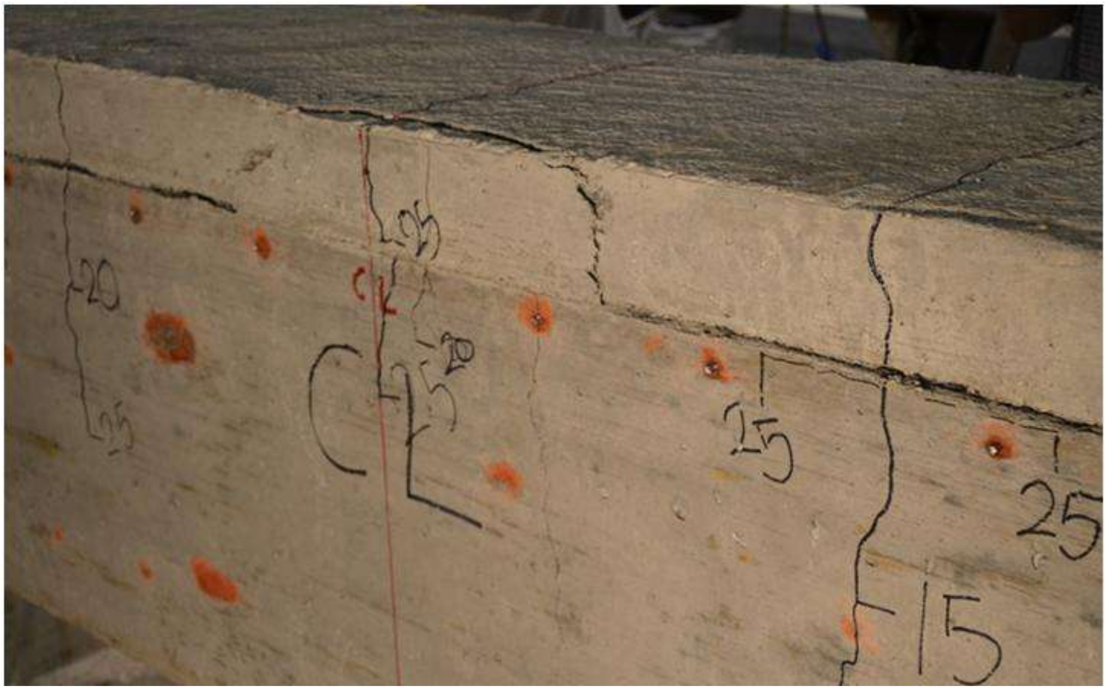


Figure 3.17 – Beam 2, centerline at failure.

3.2.4 Beam 3 (cold joint, cycled)

3.2.4.1 Beam 3 load-deflection curve

Beam 3 was cast in the same manner and at the same time as Beam 2, with a cold joint in the plane of reinforcing steel. Instead of loading the beams to failure monotonically, Beam 3 was first loaded to a total load of 60 kips, unloaded to zero, and then re-loaded monotonically to failure (the load protocol is presented in Table 2.). When the beam was first loaded to a total load of 60 kips (average end load of 30 kips), the average end load was increased in increments of approximately 5 kips. The specimen was inspected for cracks, which were marked at each load step. At a total load of 60 kips, the maximum horizontal crack width was 20 mils (0.02 in.). When the beam was loaded for the second time, it was loaded up to a total load of 60 kips without inspecting for cracks. The only visual measurement conducted during the second loading was the recording of dial gage readings at approximately 5-kip increments (average end load). The beam was inspected for cracks again when the total load reached 60 kips for the second time. At this point some of the horizontal cracks widened to a maximum width of 35 mils (0.035 in.)

The load-deflection curve for beam 3 is shown in Figure 3.1818. Overall, Beam 3 performed very similar to Beam 2, except for the peak load. The beam failed at a total load of 80 kips (compared with a total load of 85 kips for Beam 2), in the same manner as observed for Beam 2. A wide horizontal crack in the plane of the cold joint, within the splice region, was observed after failure (Figure 3.19), with the widest portion of the crack being 3/8-in. It is concluded that the beam failed due to a splice failure. The calculated bar stress corresponding to the total load of 80 kips is 57 ksi based on moment-curvature analysis (

Table 3.1).

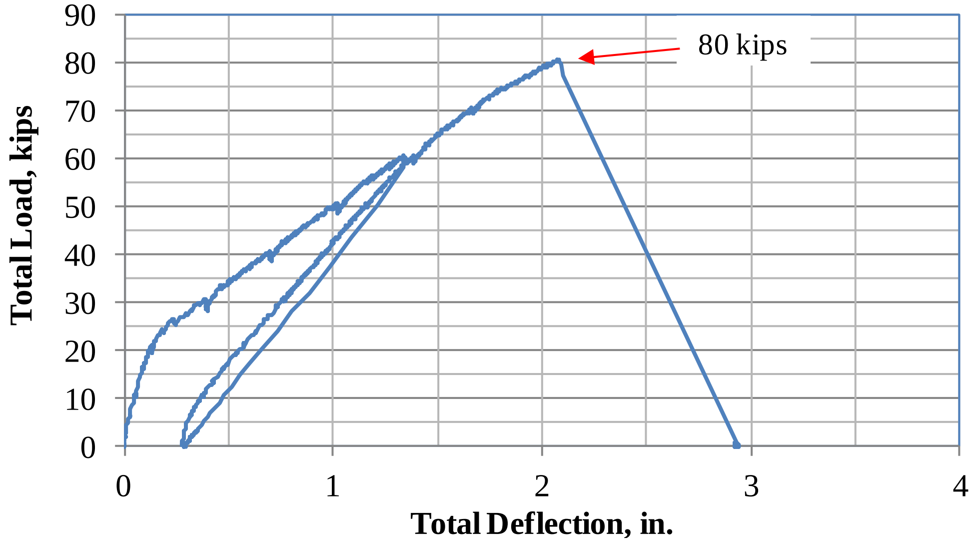


Figure 3.18 – Total load vs. total deflection for Beam 3 (with a cold joint)

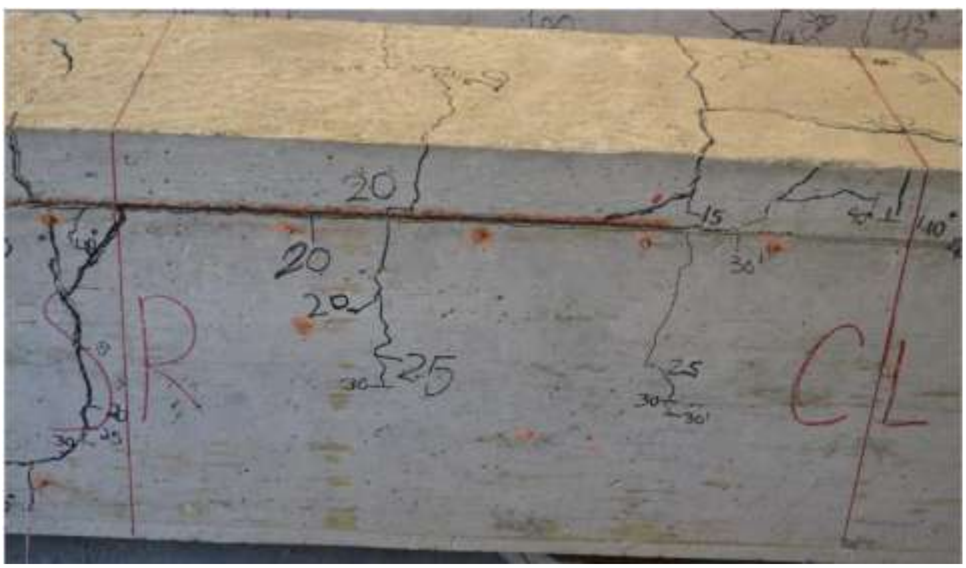


Figure 3.19 –Beam 3 failure with wide horizontal cracks along cold joint

3.2.4.2 Crack progression-Beam 3

Maximum measured crack width versus load for Beam 3 is shown in Figure 3.20; the crack map for Beam 3 is presented in Figure 3.21. As seen in both figures, the first flexural cracks formed near end of the east splice region at an average end load of 10 kips (total load of 20 kips). At an average end load of 15 kips, flexural cracks were present at both ends of the splice region and both supports. A horizontal crack first formed at an average end load of 15 kips at the west end of the splice region along the cold joint, with additional horizontal cracks forming and reaching a 9-mil (0.009 in.) width at an average end load of 20 kips (Figure 3.22). At an average end load of 30 kips, a 40-mil (0.04-in.) width flexural crack and 20-mil width horizontal crack were recorded. At this point, the beam was unloaded. With zero load, the maximum flexural and horizontal crack widths decreased to 13 and 7 mils (0.013 and 0.007 in.), respectively. The load was reapplied, and at the last crack mapping (average end load of 30 kips), the widest flexural crack had a width of 55 mils (0.055 in.) and the widest horizontal crack had a width of 35 mils (0.035 in.), much wider than the cracks noted at the first loading to a 30-kip average end load.

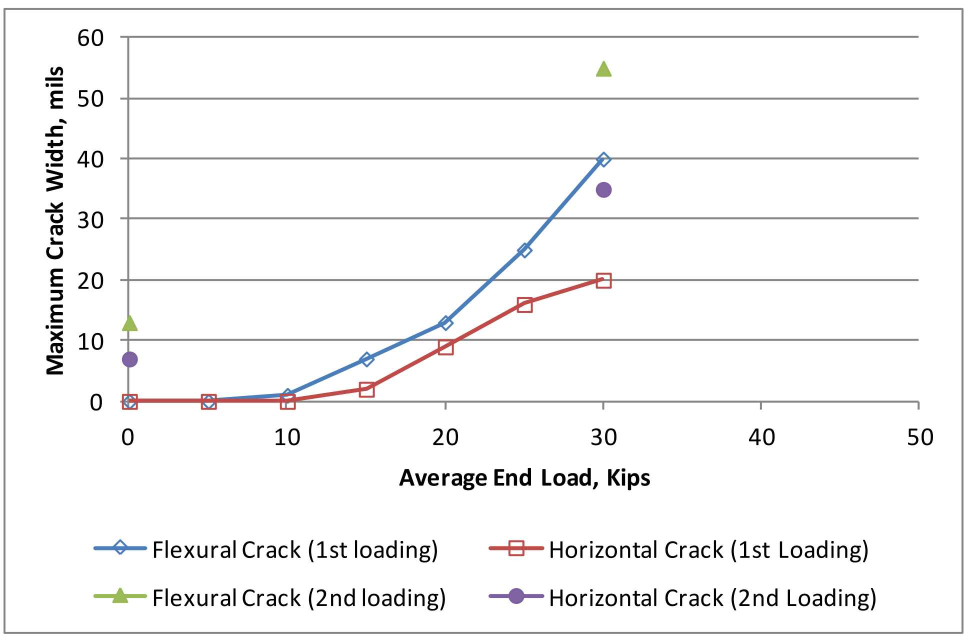


Figure 3.20 – Maximum crack width vs. average end load for Beam 3.

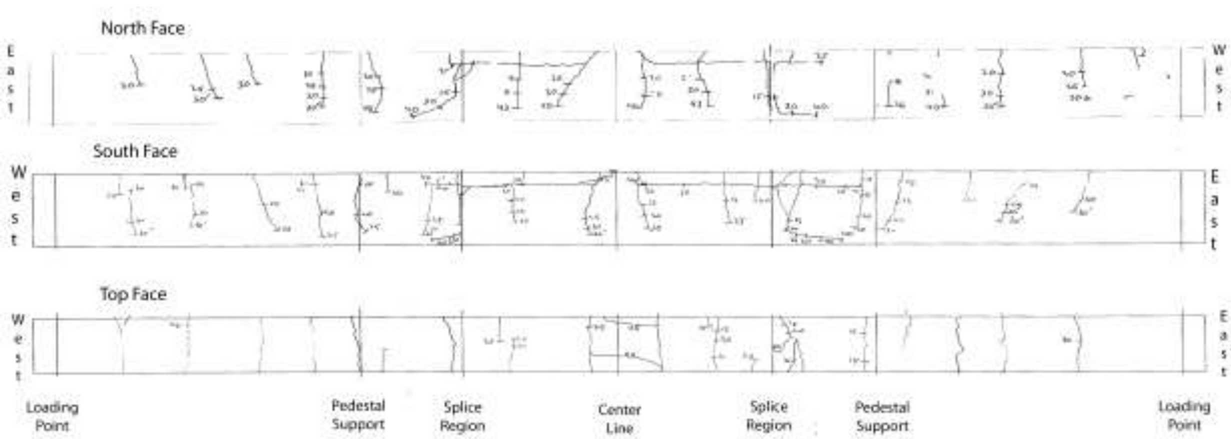


Figure 3.21 – Crack map for Beam 3. Numbers indicate maximum average end load when cracks marked. See Figure C.3 in Appendix C for greater detail.

Figure 3.18 – Beam 3, northwest splice region with horizontal crack, 20 kip end load.

Failure occurred at an average end load of 40 kips (total load of 80 kips), a slightly lower load than the monotonically loaded Beam 2 (total load of 85 kips). At failure, the concrete above the cold joint separated from the remainder of the beam, with the horizontal crack propagating along the cold joint in a region that was somewhat larger than the splice region except for a small region near the centerline, which was restrained by the No. 3-bar hoop (Figure 3.23). Large flexural cracks were also present near both ends of the splice region.

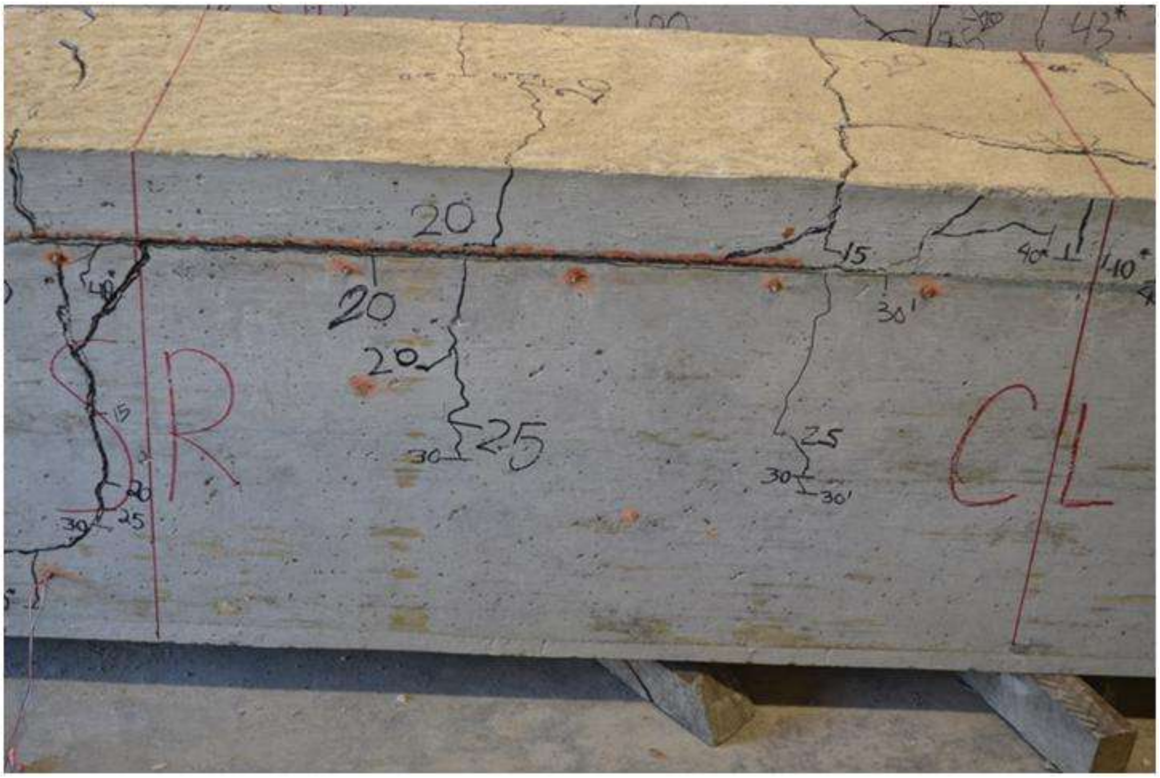


Figure 3.23 – Beam 3, splice region and centerline showing separation of concrete, 40 kip end load.

3.3 Beams 4, 5, and 6 with 120-in. splice length

3.3.1 Concrete strength

The concrete strengths for Beams 4, 5 and 6 are summarized in Table 3.3 3. The three beams were cast in two stages to accommodate the presence of a cold joint at the level of the flexural reinforcement. The concrete below the cold joint was placed on June 13, 2012, and the concrete above the cold joint was placed on June 14, 2012. The forms were removed on June 17, 2012 when the average concrete compressive strength for both placements exceeded 3500 psi. The beams were tested on June 20, 2012. On that date, the concrete from the first placement had an average compressive strength of 5230 psi, and the concrete from the second placement had an average compressive strength of 5490 psi (Table 3.3). The higher strength for the second placement was likely due to the slightly lower water-cement ratio of the concrete, as shown on the batch ticket (Appendix H). The average split cylinder strength and average modulus rupture were, respectively, 370 and 600 psi for the concrete below the cold joint and 470 and 700 psi for the concrete above the cold joint. The flexural beam specimens with cold joints were also tested and had an average modulus of rupture of 274 psi, significantly below that of specimens cast monolithically. The proportions of the concrete mixture and the properties of the concrete for each placement are reported in Table E.2 of Appendix E.

Table 3.3 – Concrete strengths for Beams 4, 5, and 6

	Concrete below cold joint	Concrete above cold joint
Average Compressive Strength when Forms were removed	4310[a]	4520[b]
Average Compressive Strength at test date, psi	5230[c]	5490[d]
Split Cylinder Strength (ASTM C496), psi	370[c]	470[d]
Modulus of Rupture (ASTM C78), psi	600[c]	700[d]
Modulus of Rupture for specimens with cold joint, psi	274[d]	--

[a]Tested at 4 days; [b]tested at 3 days; [c]tested at 7 days; [d]tested at 6 days

The same reinforcing steel was used for Beams 4, 5, and 6 as for Beams 1, 2, and 3. The measured stress-strain curve for the No. 11 bar is shown in Figure 3.1.

3.3.2 Beam 4 (cold joint, monotonically-loaded)

3.3.2.1 Beam 4 load-deflection curve

Beam 4 was cast with a cold joint in the plane of reinforcing steel. It was subjected to monotonically-increasing load in increments of approximately 5 kips (average end load, the loading protocol is presented in Table 2.). The load-deflection curve for Beam 4 is shown in Figure 3.194. The total load and deflection were determined in the same manner as for Beams 1, 2 and 3. The flexural stiffness of the beam decreased once the total load exceeded 20 kips, coinciding with the formation of flexural cracks. A sharp decrease in the slope of the load-deflection curve was observed at a total load of about 94 kips and corresponding deflection of approximately 2.8 in. The stress at the end of the spliced bars for a total load of 94 kips was 68 ksi. The decrease in the slope of the load-deflection curve at a total load of 94 kips indicates that the reinforcing steel yielded. After yielding of the reinforcing steel, the total load continued to increase but at a lower rate, which is attributed to the strain hardening of the reinforcing steel. The beam was loaded to a total load of 105 kips (and a displacement of 5.5 in.) and at that point failed with the sudden splitting of the concrete along the cold joint. Wide horizontal cracks in the plane of the cold joint were observed within the splice region. Wide flexural cracks were also observed near the support (Figure 3.205). It is concluded that the reinforcing steel yielded at a total load of approximately 94 kips and beam failed at a total load of 105 kips due to failure of the splice, the latter corresponding to a bar stress of 72 ksi (Table 3.1).

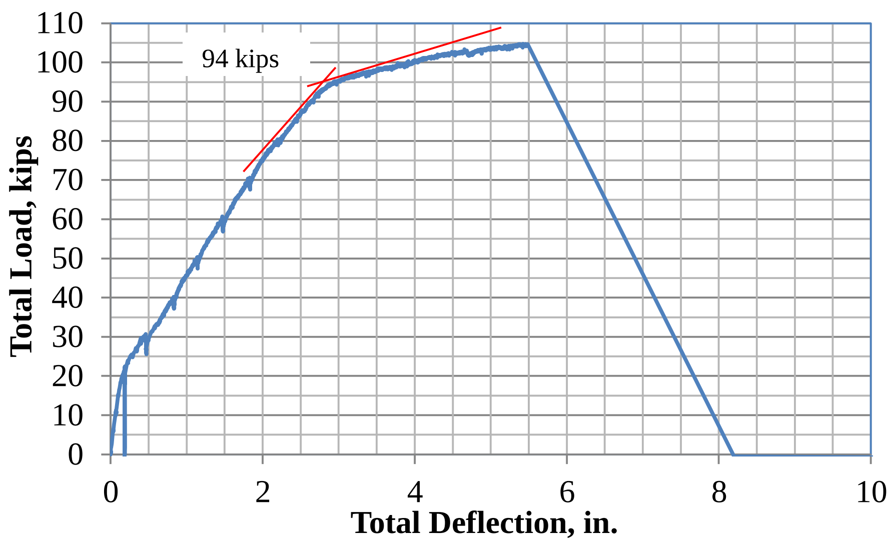


Figure 3.194 – Total load vs. total deflection for Beam 4 (with a cold joint)

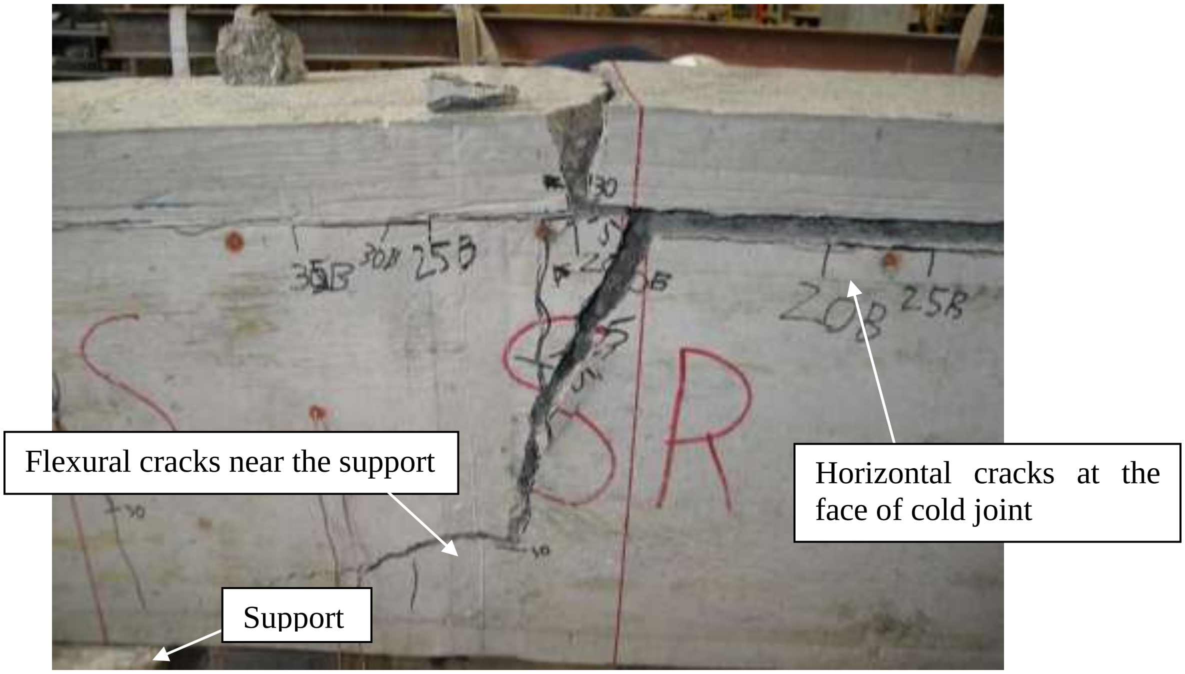


Figure 3.20 – Beam 4 (with a cold joint) at failure

3.3.2.2 Crack progression-Beam 4

Maximum measured crack width versus load for Beam 4 is shown in Figure 3.26; the crack map for Beam 4 is presented in Figure 3.27. The first flexural cracks formed near end of the west support at an average end load of 10 kips (total load of 20 kips). At an average end load of 15 kips, flexural cracks were present at both ends of the splice region and both supports. Horizontal cracks first formed at an average end load of 20 kips, at the both ends of the splice region along the cold joint. Both longitudinal and flexural cracks continued to increase in width and number as the load increased, with horizontal cracks propagating along the cold joint. At the last load prior to failure at which cracks were marked (average end load of 35 kips), the widest flexural crack had a width of 30 mils and the widest horizontal crack had a width of 16 mils. At this point, the horizontal cracks extended along most of the length of the splice region (Figure 3.28), with some of the horizontal cracks that formed at earlier stages merging together.

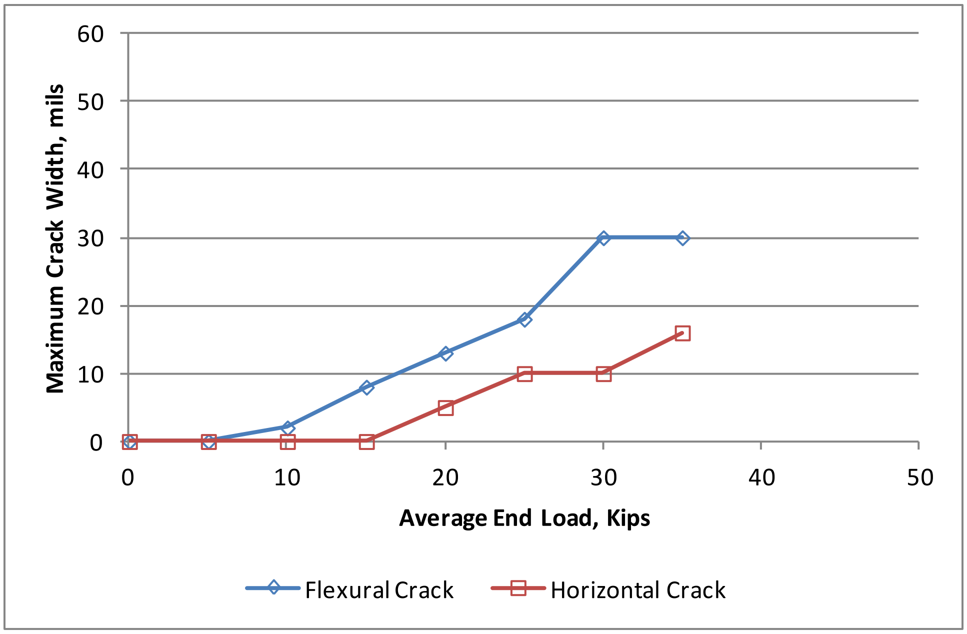


Figure 3.21 – Maximum crack width vs. average end load for Beam 4.

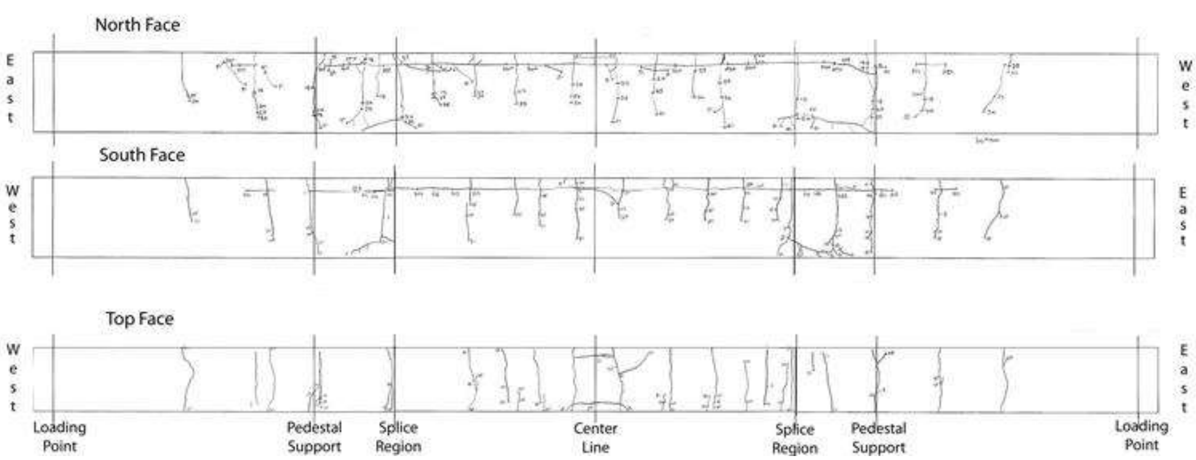


Figure 3.27 – Crack map for Beam 4. Numbers indicate maximum average end load when cracks marked. See Figure C.4 in Appendix C for greater detail.

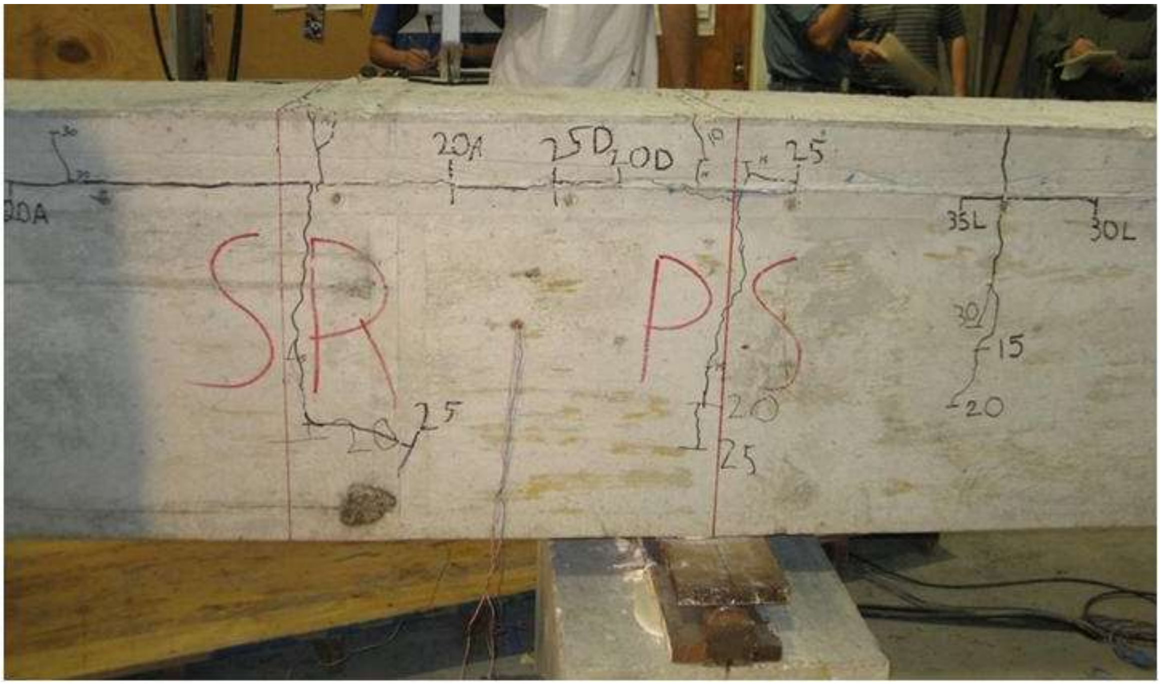


Figure 3.28 – Beam 4, south side of west splice region with horizontal cracks, 35-kip end load.

At failure, the concrete above the cold joint separated from the remainder of the beam, with the horizontal crack propagating along the cold joint between the pedestal supports except for a small region near the centerline that was restrained by the No. 3-bar hoop (Figure 3.29). Large flexural cracks were also present near both ends of the splice region (Figure 3.30).

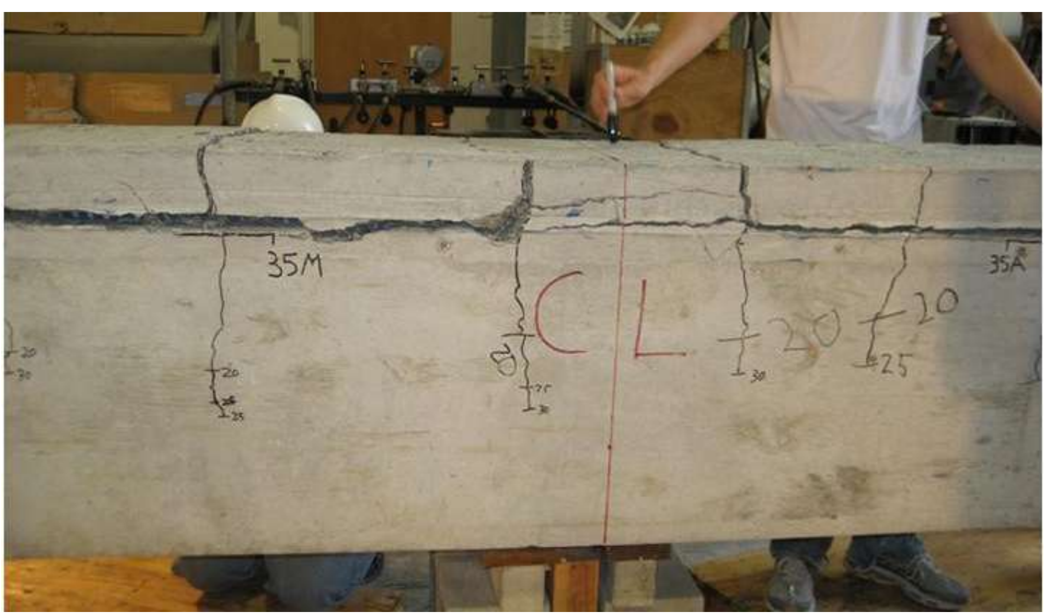


Figure 3.29 – Beam 4, centerline showing separation of concrete, 52-kip end load.

Figure 3.30 – Beam 4, end of splice region at 52-kip end load.

3.3.3 Beam 5 (cold joint, cycled)

3.3.3.1 Beam 5 load-deflection curve

Beams 5 and 6 were cast in the same manner and at the same time as Beam 4, with a cold joint at the plane of reinforcing steel. Instead of monotonically loading the beams to failure, Beam 5 was first loaded to a total load of 80 kips, and subsequently unloaded to zero, and then re-loaded to failure (the load protocol is presented in Table 2.). When the beam was first loaded to a total load of 80 kips, the average end load was increased in increments of approximately 5 kips. The specimen was inspected for cracks and marked at each load step. Horizontal cracks on the plane of the cold joint within the splice region were observed when the beam was subjected to a total load of 80 kips. The maximum horizontal crack width at this load was 35 mils (0.035 in.). It should be noted that the beam was unloaded in a rapid manner and that one of the load cells had large fluctuations after that point (load cell C in Figure 3.221). Although there were clear problems with the load readings from load cell C for the remainder of this test, the rams were at all times subjected to uniform pressure, and load readings from the other 5 beam tests show that the load was evenly applied to the four different load rods at all times. Furthermore, the load beam remained level and the displacement readings were similar at both ends of the beam, strong indicators that although the load cell readings were not accurate, the load was uniformly applied to the four load rods. Based on these observations, the total load was calculated based on the readings from load cells A and B. When the beam was loaded for the second time, it was loaded up to a total load of 80 kips at an increment of 5 kips (average end load). At the end of the each increment, dial-gage displacement measurements were recorded. The beam was inspected for cracks at total loads of 40, 60, 70, and 80 kips. When the beam was inspected for crack during the second loading, some of the horizontal cracks elongated or widened and some new horizontal cracks were noticed. The maximum horizontal crack width was still 35 mils (0.035 in.)

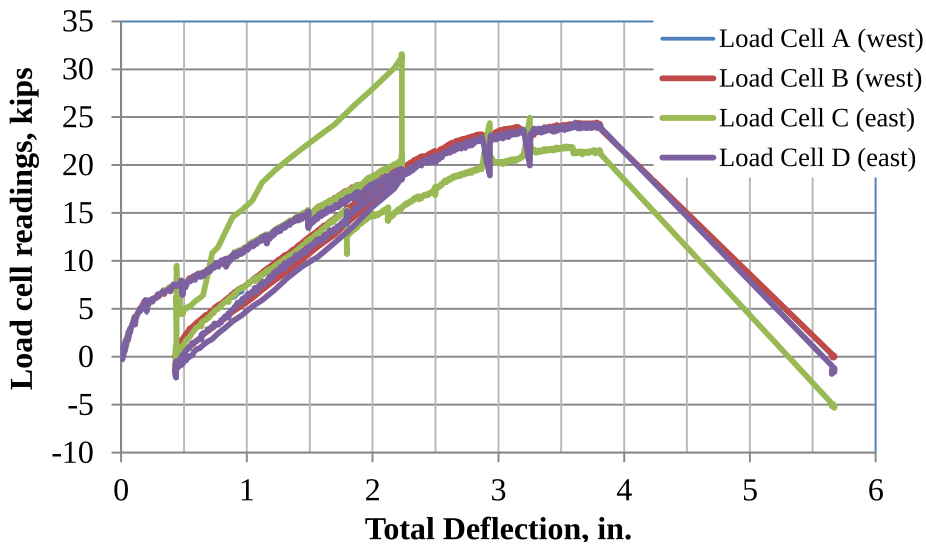


Figure 3.22 – Load cell readings for Beam 5

The load-deflection curve for Beam 5 is shown in Figure 3.32 . Due to the problem documented for load cell C, the total load is calculated as twice the summation of load cells A and B, located at the West loading point. Overall, Beam 5 performed very similar to Beam 4. The slope of the load-deflection curve first decreased at a total load of 20 kips, which coincides with the first observation of flexural cracks. Another decrease in the slope of the load-deflection curve was observed at a total load 91 kips, with a corresponding total displacement of approximately 2.7 in, which is attributed to the yielding of the flexural reinforcement. The calculated bar stress corresponding to the total load of 91 kips is 66 ksi based on moment-curvature analysis. The positive slope of the load-deflection relationship after a total load of 91 kips is attributed to the strain hardening of the reinforcing steel. The beam was loaded to a total load of 96 kips, with a corresponding total displacement of 3.6 in., at which point the beam failed suddenly. Wide flexural cracks near the support and horizontal cracks in the plane of cold joint were observed within the splice region (Figure 3.2033). It is concluded that the reinforcing steel yielded at a total load of 91 kips and beam failed at a total load of 96 kips due to failure of the splice, the latter corresponding to a bar stress of 67 ksi (Table 3.1).

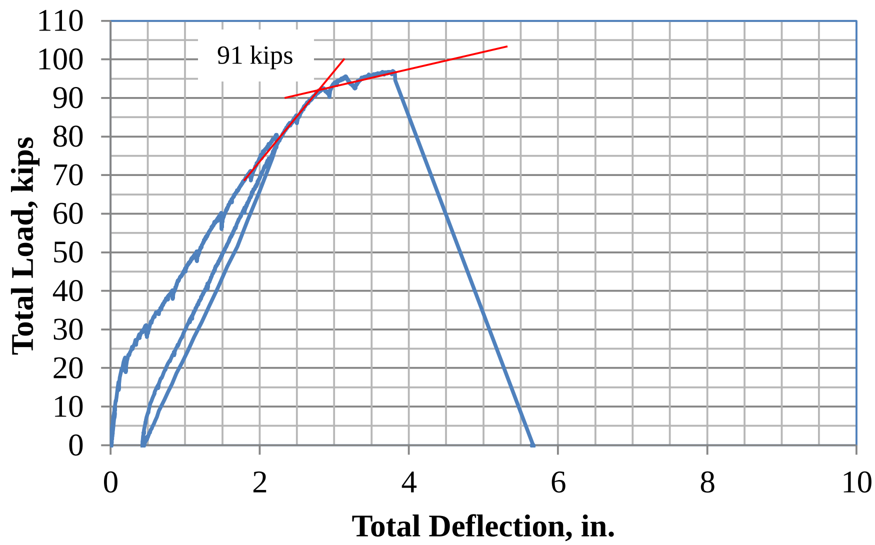


Figure 3.32 – Total load vs. total deflection for Beam 5 (with a cold joint)

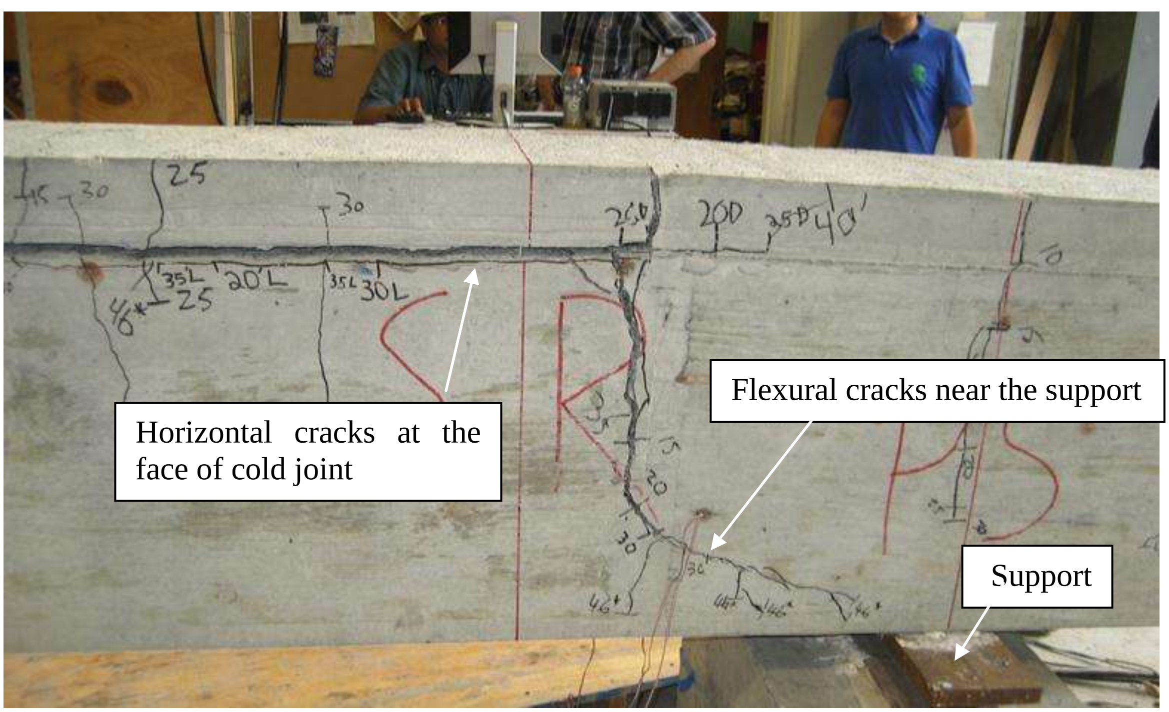


Figure 3.33 – Beam 5 (with a cold joint) at failure

3.3.3.2 Crack progression-Beam 5

Maximum measured crack width versus load for Beam 5 is shown in Figure 3.34; the crack map for Beam 5 is presented in Figure 3.35. The first flexural and horizontal cracks formed at the supports at an average end load of 10 kips (total load of 20 kips). At an average end load of 15 kips, flexural and horizontal cracks were present at both ends of the splice region and both supports (Figure 3.36). At an average end load of 40 kips, a 45-mil width flexural crack and 35-mil width horizontal crack were recorded. At this point, the beam was unloaded. The load was reapplied, and at the last load prior to failure at which cracks were marked (average end load of 40 kips), the maximum width of the cracks had not increased from first loading (Figure 3.34). Although the crack width was approximately the same, several cracks had increased in length.

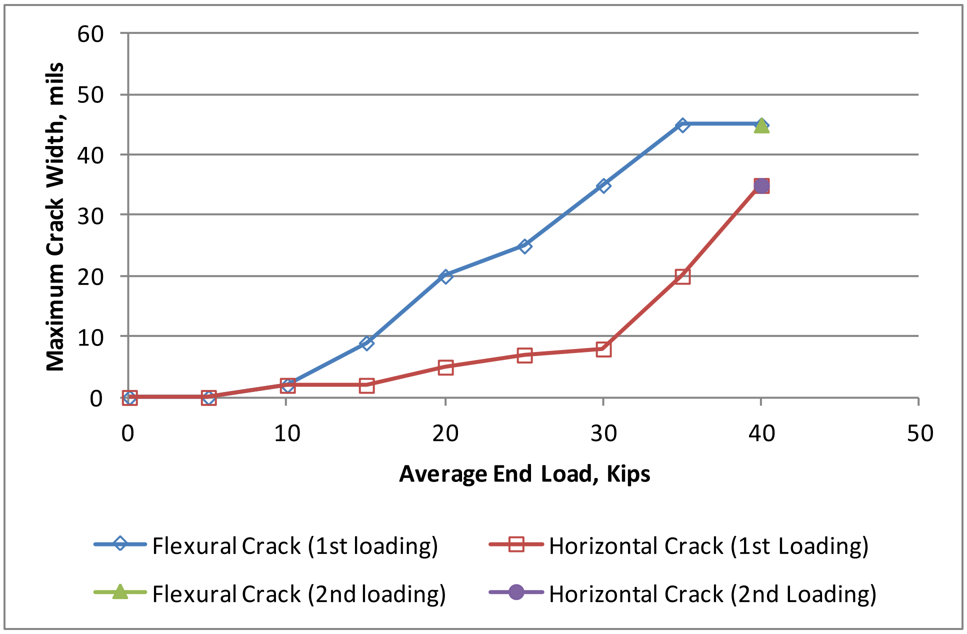


Figure 3.34 – Maximum crack width vs. average end load for Beam 5.

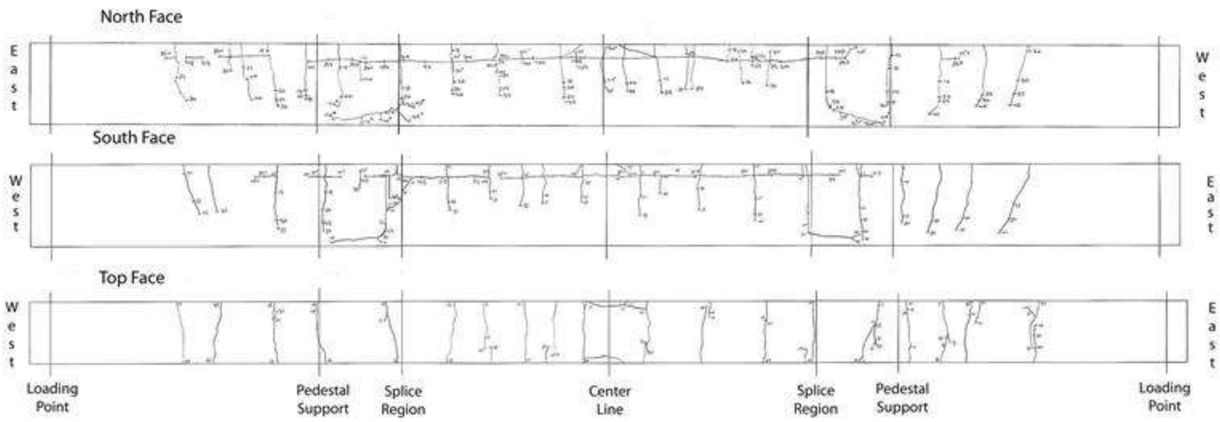


Figure 3.35 – Crack map for Beam 5. Numbers indicate maximum average end load when cracks marked. See Figure C.5 in Appendix C for greater detail.

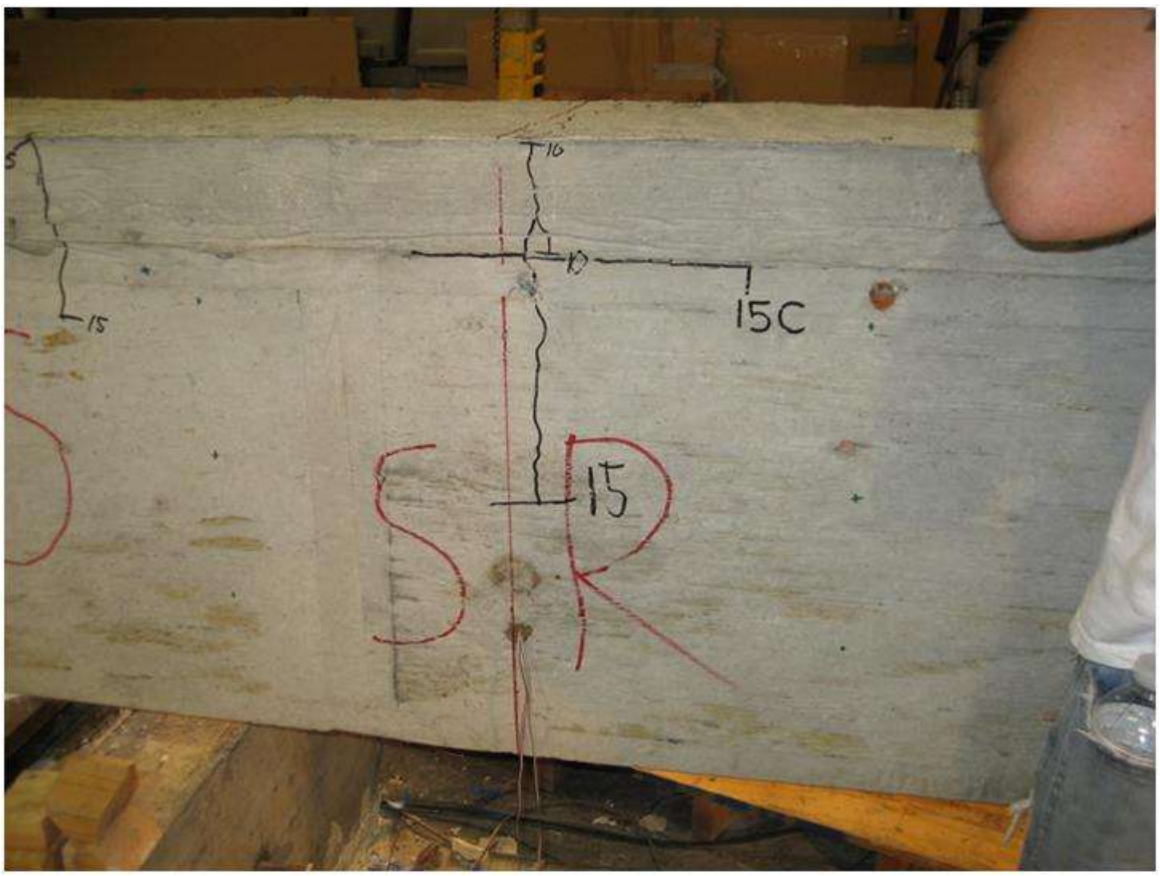


Figure 3.36 – Beam 5, northeast splice region with horizontal crack, 15 kip end load.

Failure occurred at an average end load of 48 kips (total load of 96 kips), slightly lower than the failure load for Beam 4 (average end load of 52 kips, total load of 105 kips), which was subjected to monotonically-increasing load up to failure. At failure of Beam 5, the concrete above the cold joint separated from the remainder of the beam, with the horizontal crack propagating along the cold joint throughout a region that was somewhat longer than the splice region. A small region near the centerline was restrained by the No. 3-bar hoop (Figure 3.37) and had a tighter horizontal crack and a failure surface that passed through the top of the beam in the vicinity of the hoop, as shown in Figure 3.35. As with the other beams, large flexural cracks were also present near both ends of the splice region (Figure 3.38).

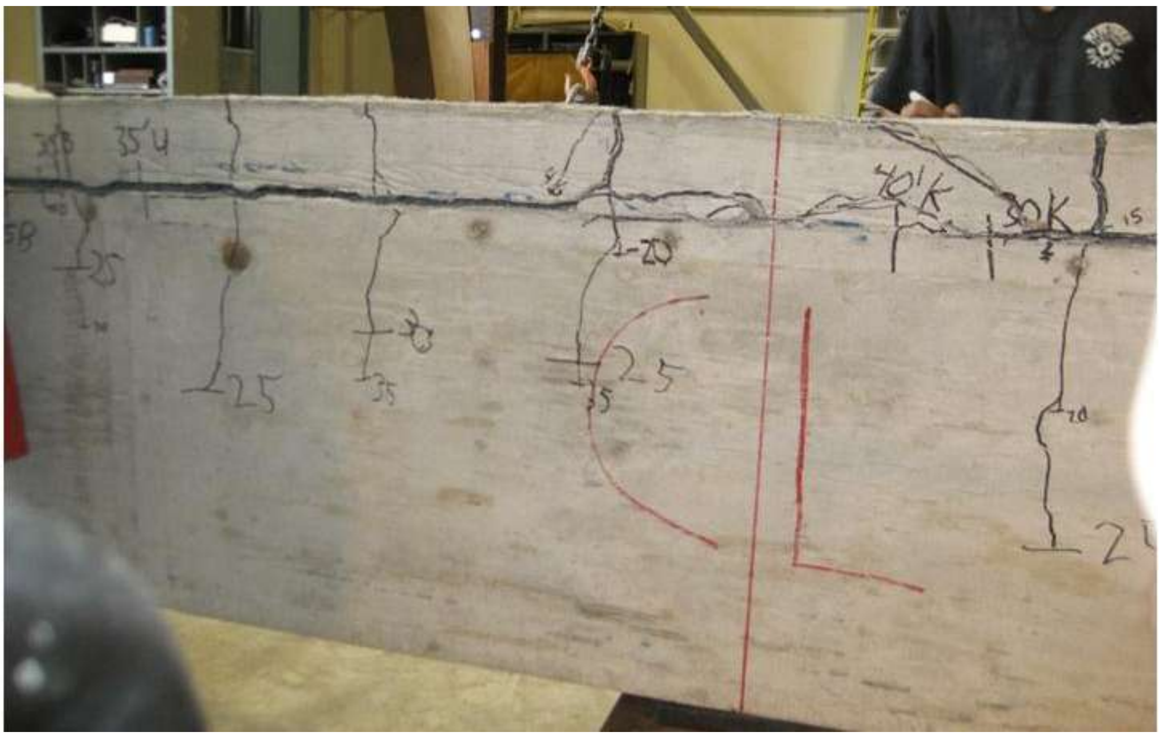


Figure 3.37 – Beam 5, centerline showing separation of concrete, 48-kip end load.

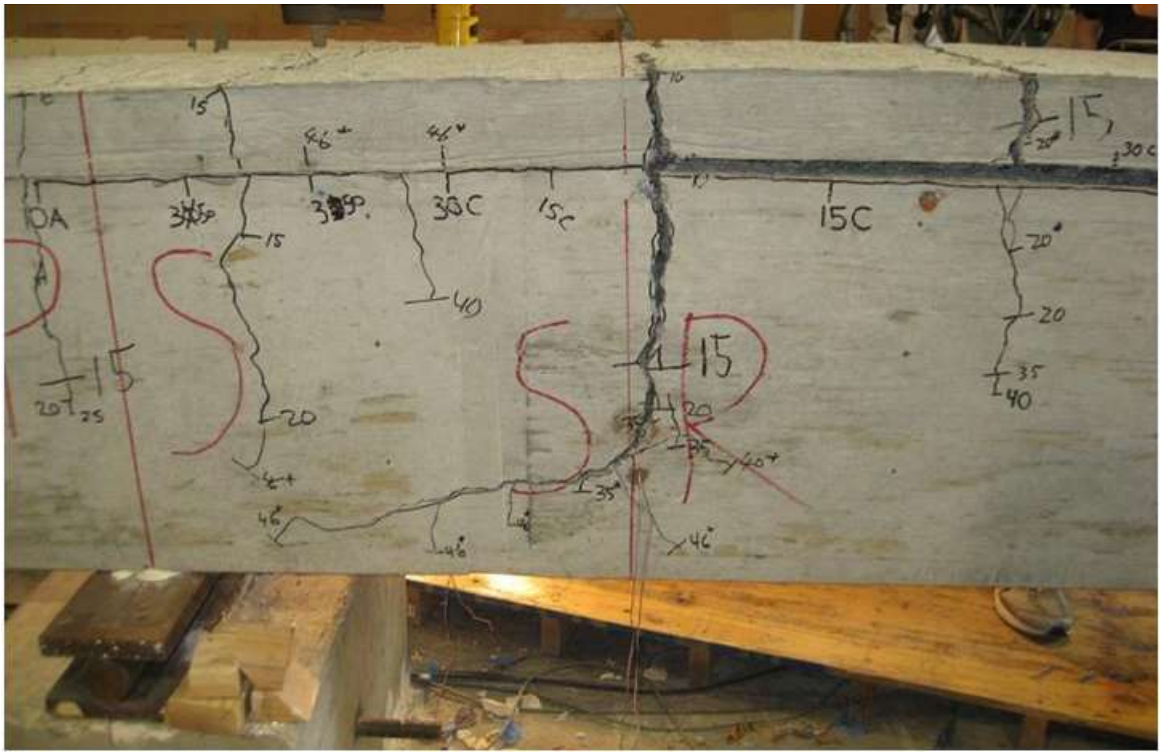


Figure 3.38 – Beam 5, splice region, 48-kip end load.

3.3.4 Beam 6 (cold joint, cycled)

3.3.4.1 Beam 6 load-deflection curve

The configuration and loading protocol of Beam 6 were similar to those of Beam 5. The beams were cast using the same procedures and at the same time and were tested in the same manner, except that unloading was much slower for Beam 6 and the beam was inspected for cracks more often during the second loading. The testing protocol for Beam 6 is presented in Table 2.5.

The individual load cell readings are plotted versus total deflection in Figure 3.39. As shown in Figure 3.39, the readings for the four load cells were identical, which verifies the assumption in Section 3.3.3 that the load was evenly distributed on the four load rods.

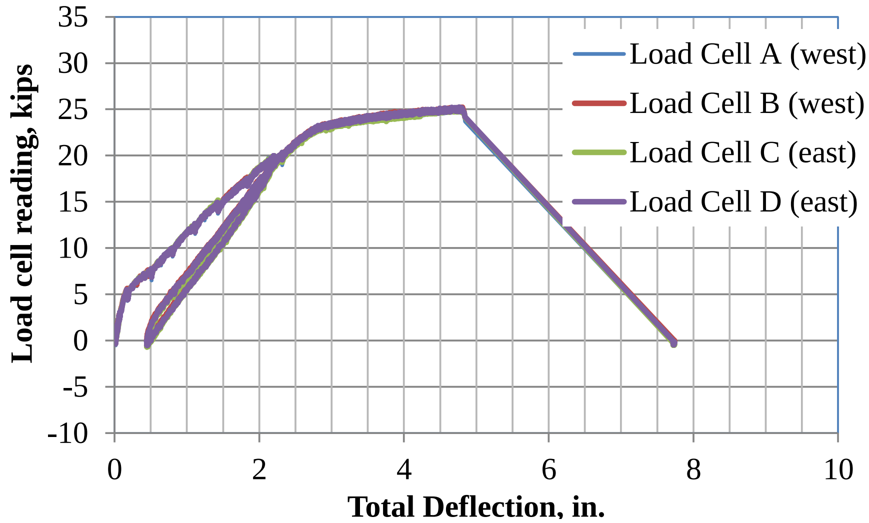


Figure 3.39 – Individual load cell readings (Beam 6)

The total load versus total deflection for Beam 6 is plotted in Figure 3.0. Overall, Beam 6 performed very similar to Beam 5. Yielding of the flexural reinforcement was observed at a total load of 92 kips and a total displacement of 2.7 in., compared with 91 kips and 2.7 in. for Beam 5. The maximum horizontal crack width at the unloading point was 30 mils (0.03 in.), compared with 35 mils (0.035 in.) for Beam 5. Beam 6 also failed due to splice failure (Figure 3.41) at a total load of 100 kips, corresponding to a bar stress of 69 ksi, and a total deflection of 4.7 in. (versus 96 kips and 3.6 in. for Beam 5).

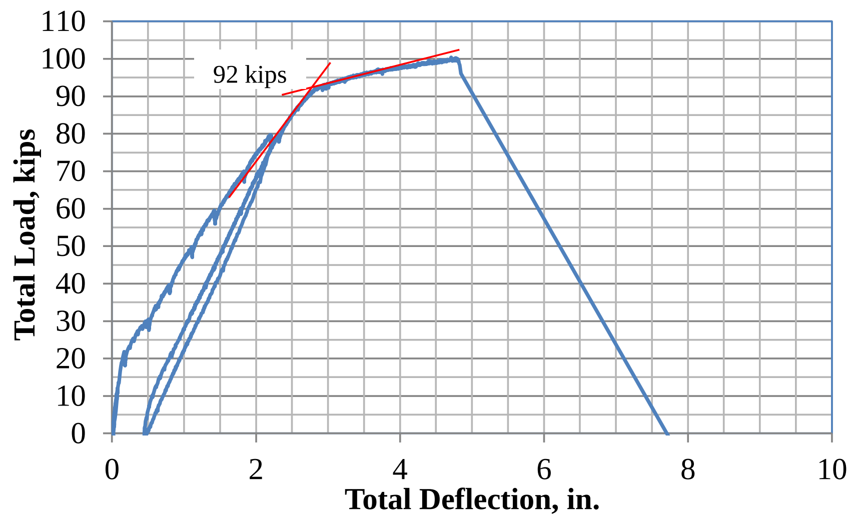


Figure 3.40 – Total load vs. total deflection for Beam 6 (with a cold joint)

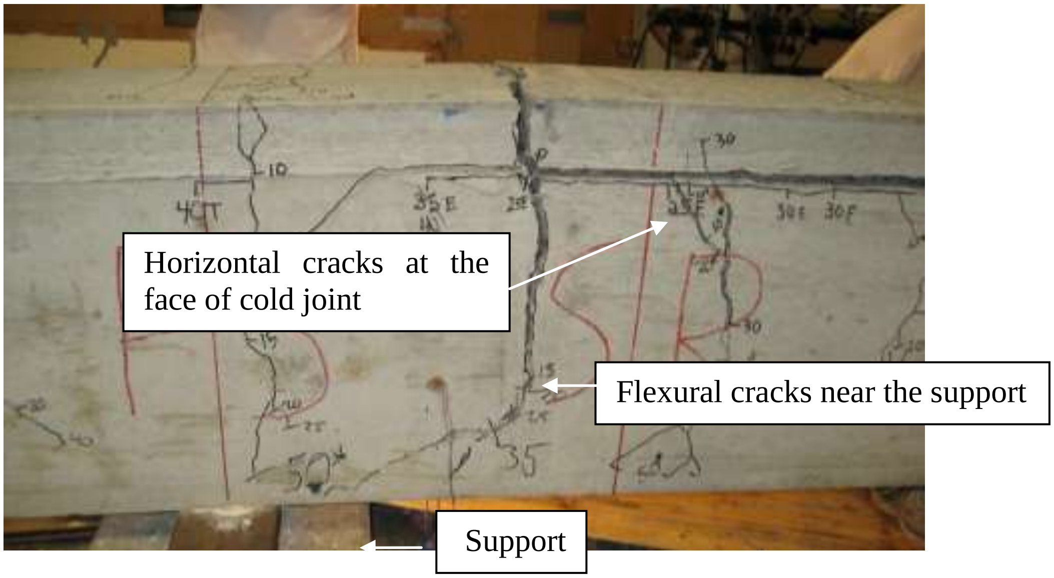


Figure 3.41 – Beam 6 (with a cold joint) at failure

3.3.4.2 Crack progression-Beam 6

Maximum measured crack width versus load for Beam 6 is shown in Figure 3.42; the crack map for Beam 6 is presented in Figure 3.43. The first flexural cracks formed at the east splice region and support at an average end load of 10 kips. At an average end load of 25 kips, flexural and horizontal cracks were present at both ends of the splice region and both supports (Figure 3.44). At an average end load of 40 kips, a 35-mil (0.035 in.) wide flexural crack and 30-mil (0.03 in.) wide horizontal crack were recorded. At this point, the beam was unloaded. The load was reapplied, and at the last load prior to failure at which cracks were marked (average end load of 40 kips), the crack width had not increased with respect to first loading (Figure 3.42). Although the maximum crack widths remained the same, several cracks had increased in length.

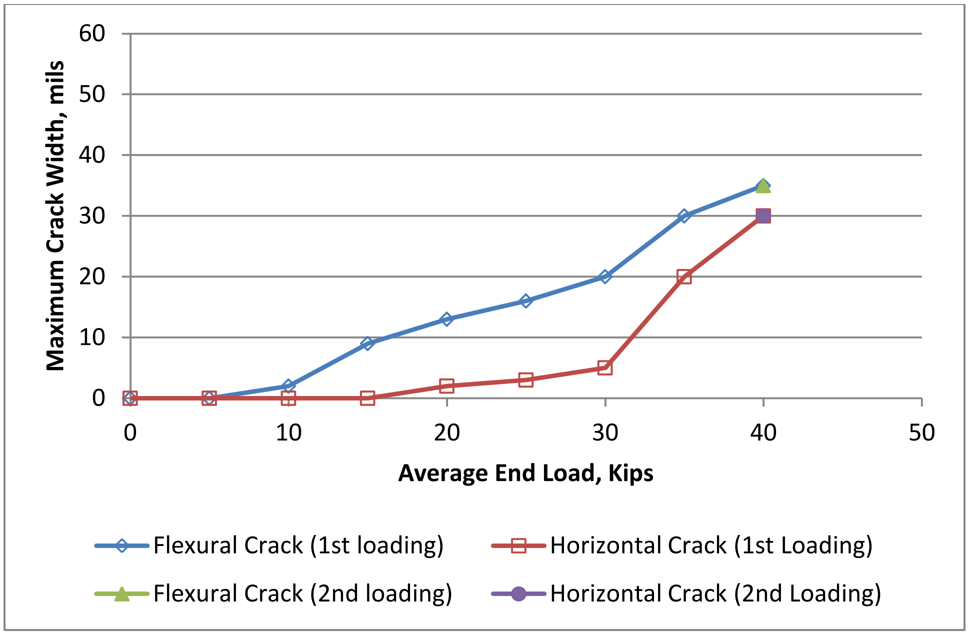


Figure 3.42 – Maximum crack width vs. average end load for Beam 6.

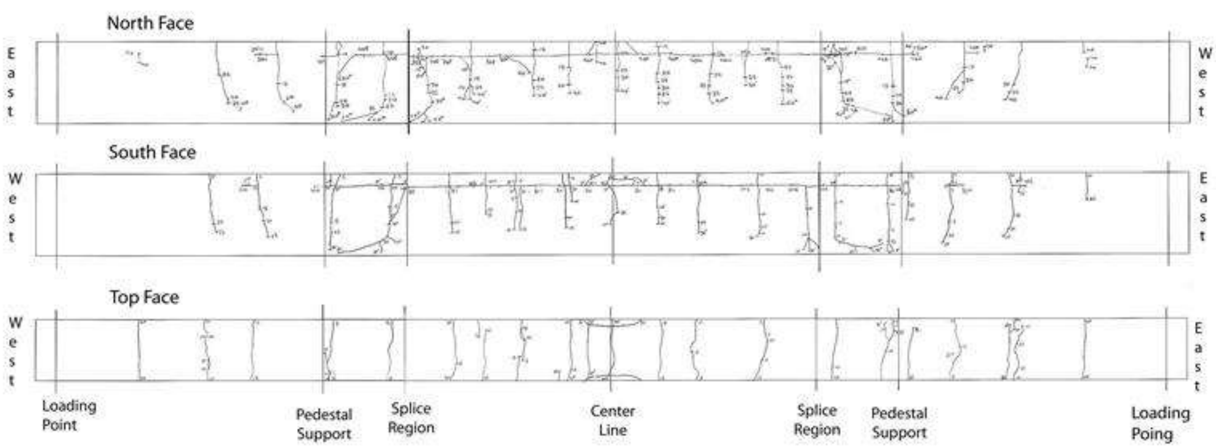


Figure 3.43 – Crack map for Beam 6. Numbers indicate maximum average end load when cracks marked. See Figure C.6 in Appendix C for greater detail.

Figure 3.44 – Beam 6, splice region with horizontal crack, 25-kip end load.

Failure occurred at an average end load of 50 kips, slightly lower than for Beam 4 (average end load of 52 kips, total load of 105 kips), and higher than Beam 5 (average end load of 48 kips, total load of 96 kips). As observed in Beams 2 through 5, at failure occurred at the cold joint with the upper concrete separating from the remainder of the beam, with the horizontal crack propagating along the cold joint between the pedestal supports. As for Beam 5, a small region near the centerline was restrained by the No. 3-bar hoop (Figure 3.45) and had a tighter horizontal crack and a failure surface that passed through the top of the beam in the vicinity of the hoop, as shown in Figure 3.49. As in the case of the other beams, large flexural cracks were also present near both ends of the splice region (Figure 3.46).

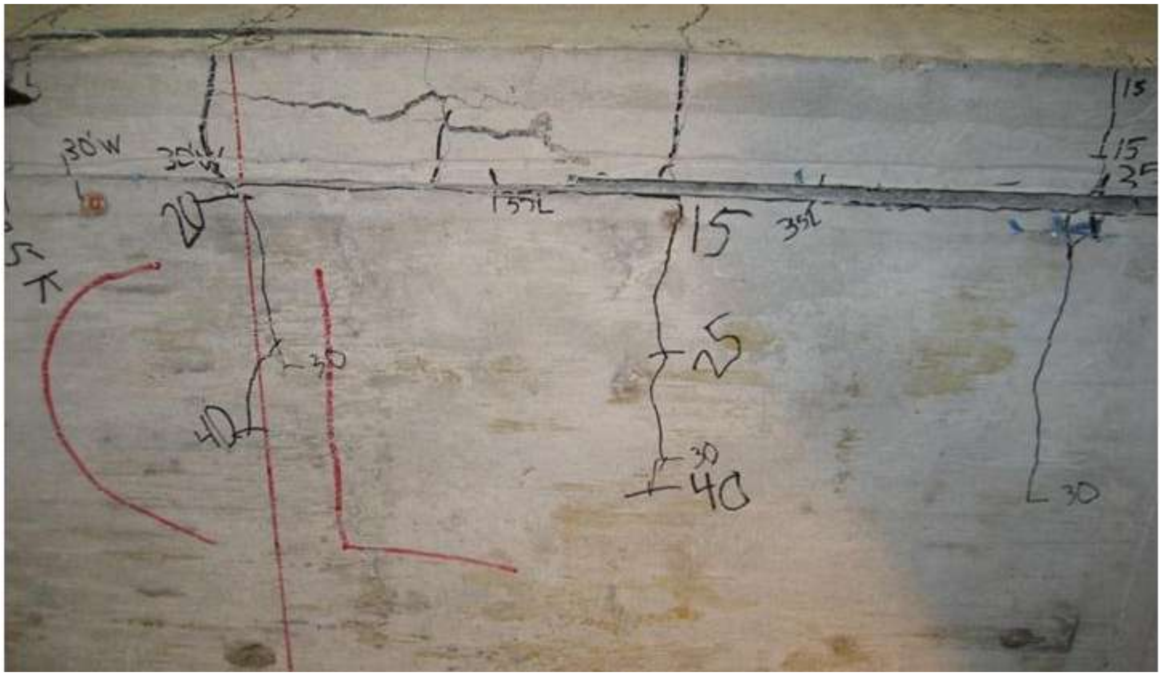


Figure 3.45 – Beam 6, centerline showing separation of concrete, 50 kip end load.

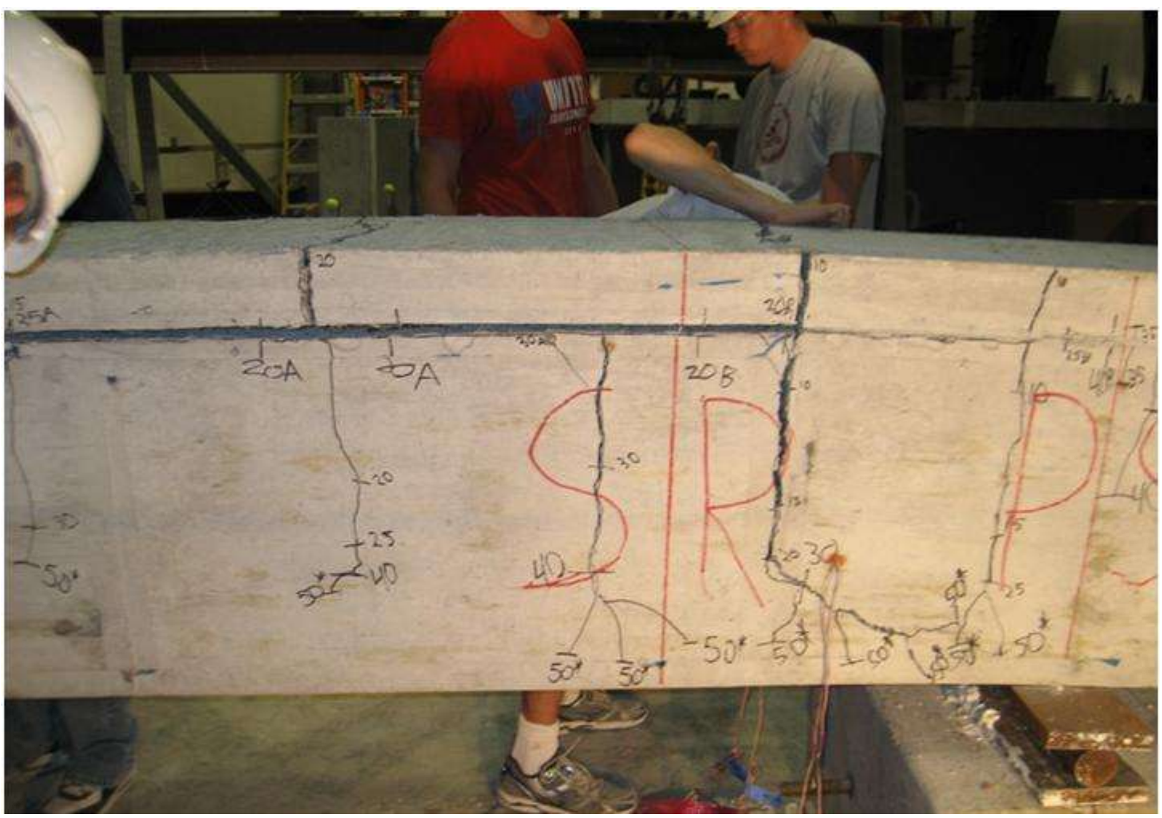


Figure 3.46 – Beam 6, splice region, 50-kip end load.

4 Summary and Conclusions

The effect of preexisting cracks, oriented in the plane of and parallel to the reinforcing steel, on the strength of No. 11-bar lap splices was investigated by testing six beams – three with a splice length of 79 in. and three with a splice length of 120 in. One of the beams with a 79-in. splice was cast monolithically and loaded monotonically to failure. To simulate the cracks, the other five specimens were cast with a cold joint at the mid-height of the reinforcing steel. Two beams (one with a 79-in. splice and one with a 120-in. splice) were loaded monotonically to failure. The other three beams were pre-loaded to develop horizontal cracks in the face of the cold joint, unloaded and then loaded to failure; those beams developed horizontal cracks with widths of 20, 30 and 35 mils (0.02, 0.03, 0.035 in.) just prior to unloading. The test results are summarized below:

1. For the beam with a splice length of 79 in. and cast with monolithic concrete, the reinforcing steel yielded and the beam failed in flexure.
2. For the beam with a splice length of 79 in., cast with a cold joint, and subjected to monotonically-increasing load to failure, splice failure took place at a bar stress of 62 ksi, about 8% below the bar yield strength of 67 ksi.
3. For the beam with a splice length of 79 in., cast with a cold joint and subjected to cyclic loading, horizontal cracks with a maximum width of 20 mils (0.02 in) developed prior to failure. Splice failure took place prior at a bar stress of 57 ksi, about 15% below the.bar yield strength.
4. For the beam with a splice length of 120 in., cast with a cold joint, and subjected to monotonically-increasing load, the reinforcing steel yielded prior to a splice failure, which occurred in the strain-hardening region of the stress-strain curve at a bar stress of 72 ksi.
5. For the two beams with a splice length of 120 in., cast with a cold joint, and subjected to cyclic loading, horizontal cracks with maximum widths of 30 and 35 mils (0.03 and 0.035 in.) developed prior to splice failure, which occurred at bar stresses of 67 and 69 ksi, respectively, values that equaled or exceeded the bar yield strength..

The following conclusions are based on the test results and analyses presented in this report.

1. The methods described in this report provide a viable means of simulating a crack in the plane of flexural reinforcement.
2. The cyclically load beams incorporating a cold joint to simulate crack in the plane of the reinforcement exhibited slightly reduced lap splice capacity compared to the monotonically loaded beams.
3. In the presence of a simulated crack in the plane of the reinforcing bars, the lap-spliced No. 11 bars with a 79-in. splice length achieved bar stresses of 62 and 57 ksi.
4. In the presence of a simulated crack in the plane of the reinforcing bars, the lap-spliced No. 11 bars with a 120-in. splice length achieved bar stresses greater than or equal to the yield strength, 67 ksi.

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Appendix A: Pilot Tests – Preliminary Study of the Effect of Simulated Cracks on Lap Splice Strength of Reinforcing Bars using Beams with Single Splices

1. Introduction

This appendix presents the findings of a pilot study consisting of two lap splice beam tests that were performed to investigate how a test specimen with a preexisting crack parallel to the plane of the reinforcement could be developed and tested. The test program described in the body of this report was developed using lessons obtained in this pilot study.

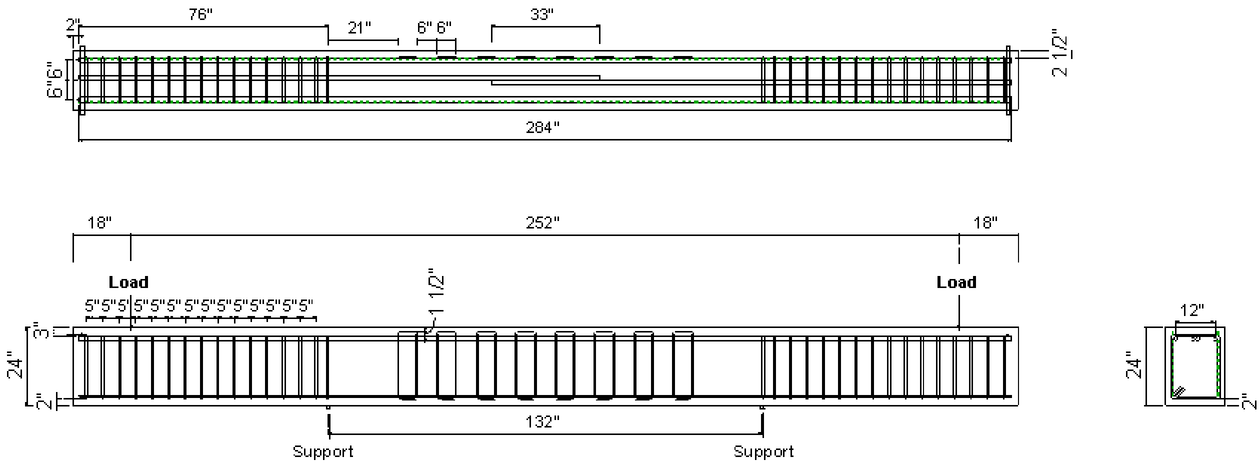
The two beams were cast simultaneously and tested monotonically to failure seven days after casting. Because this project involves a larger number of physical simulations, the testing of these two beams is referenced throughout the report as Stage 1 of the project. Both beams had main flexural reinforcement consisting of three No. 11 bars, two of them continuous and one of them spliced at the center of the beam (Figure A.1). The splice lengths in the two beams were 33 in. and 79 in., respectively. All other dimensions and material properties were identical. The beam with a splice length of 33 in. will be referenced throughout this appendix as Beam A1 and the beam with a splice length of 79 in. will be referenced as Beam A2.

The two beams were instrumented with strain gages placed on all bars at the edge of the splice region (Figure A.2). Beam displacements and applied loads were monitored during the tests using displacement transducers and load cells.

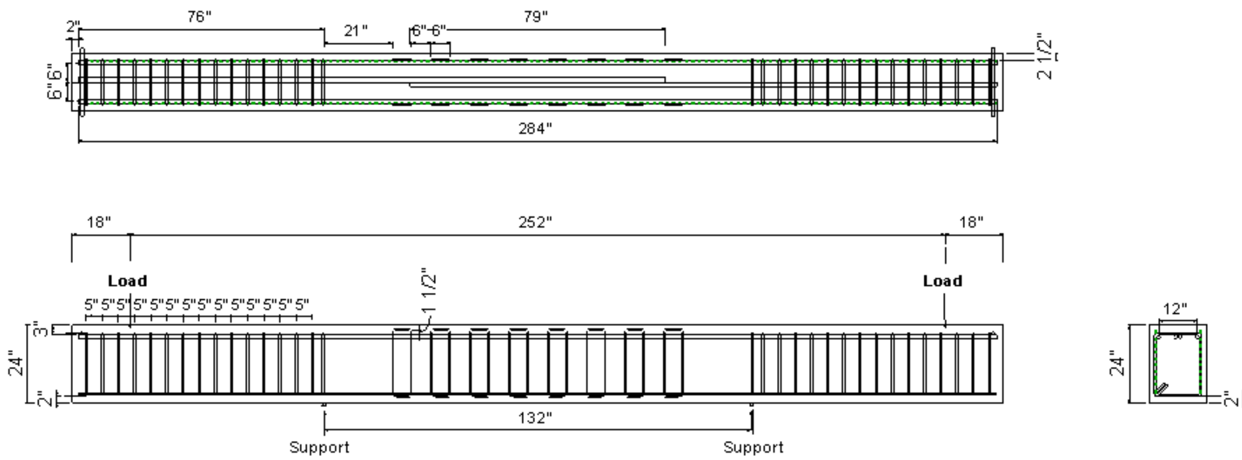
The following sections present brief descriptions of the beams, the test process, and outline the major findings from the tests.

2. Beam Casting

Casting was performed in two separate stages. The first stage of the casting process consisted of placing concrete over the full depth of the beam at the end sections and up to the mid-height of the flexural reinforcement in the splice region (Figs. A.1 and A.3). The first concrete was placed on May 3, 2012. The concrete surface at the location of the cold joint was roughened and the beam was wet-cured for 24 hours (Figure A.4). Two layers of painters tape were placed adjacent to the bars to simulate the effects of a preexisting crack parallel to the plane of the flexural reinforcement (Figure A.5). Concrete was placed above the cold joint on May 4, and the beams were subsequently moist cured for 7 days.



(a)



(b)

Figure A.1 – Reinforcing steel drawing. (a) Beam A1 – specimen with a 33-in. splice length. (b) Beam A2 – specimen with a 79-in. splice length

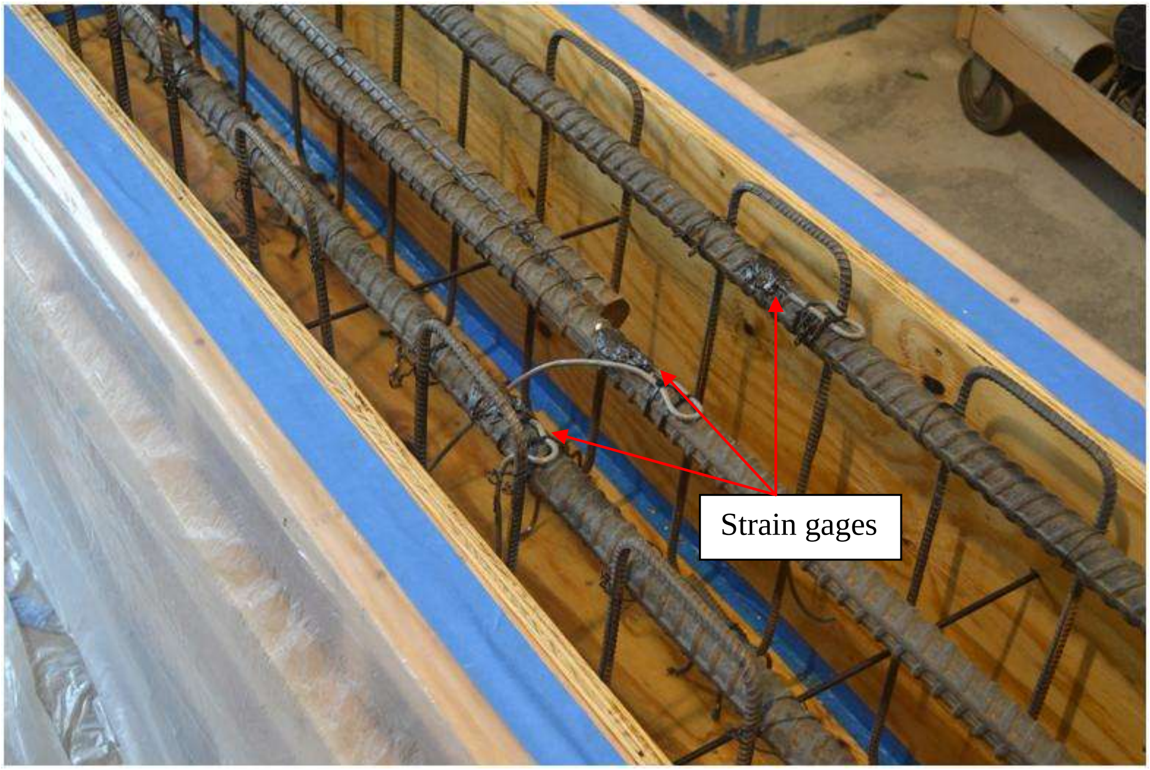


Figure A.2 – Strain gages placed on bars at the edge of the splice region of Beam A2.

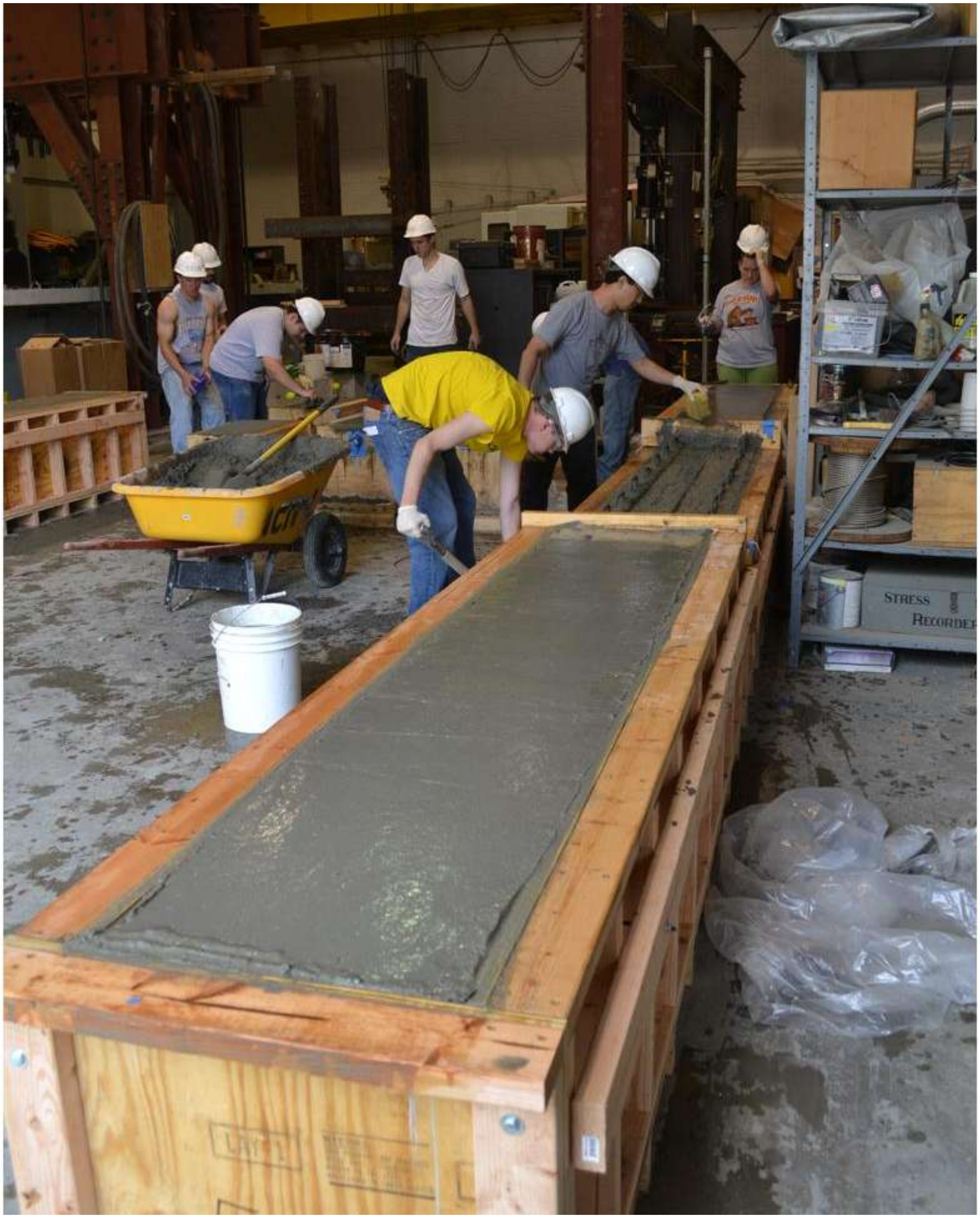


Figure A.3 – Beam A2 after first concrete placement was completed.

Figure A.4 – Beam A1 being moist cured after the first placement was completed.

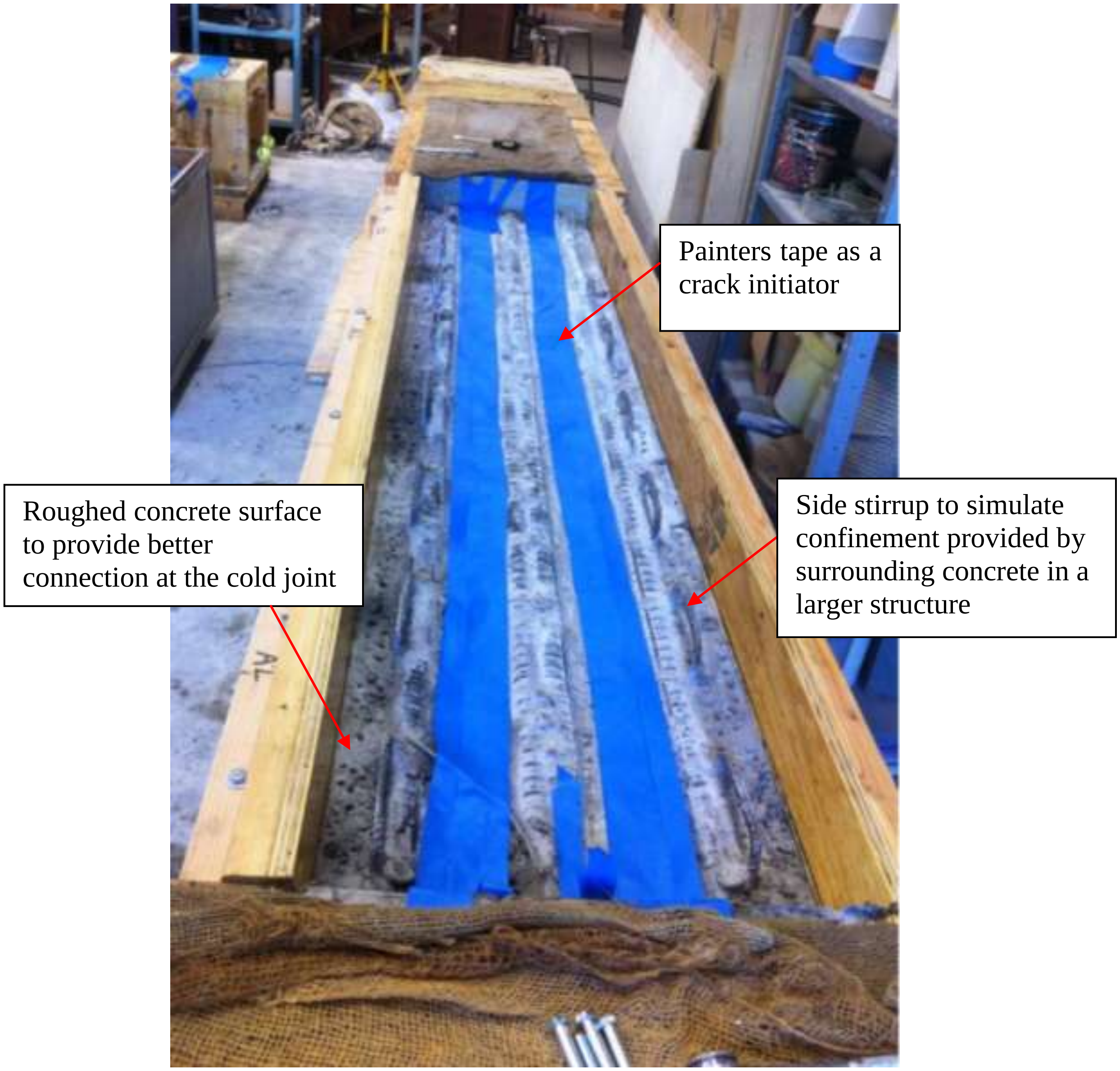


Figure A.5 – Painters tape placed to simulate a preexisting crack at the plane of the reinforcement in Beam A2.

3. Test apparatus and loading protocol

The two beams were tested using a four-point loading configuration. To facilitate inspection of the splice region during the test, the loads were applied in the downward direction (Figure A.6) so that the main flexural reinforcement would be located at the top of the beam. The splice region was located between the two supports (Figure A.7), in the central constant moment region of the beam.

In addition to strain gages, the beams were instrumented to measure displacement and load. The four load rods used in the test were instrumented to record load, and displacements were recorded using displacement transducers and dial gages for redundancy. Three displacement transducers were used to monitor the displacement at the center of the beam and at each of the two load points (Figure A.6). Dial gages were mounted at a distance of 3 in. from the load points.

Loads were applied with four hydraulic rams connected to a manual pump through a distribution system with two separate manifolds. The manifold system allowed adjustments in the pressure of each ram separately and adjustment of the pressure in each pair of rams allowing for loading in tandem. The force in each of the four load rods (Figure A.6) was monitored throughout the test and the pressure in the rams was constantly adjusted to maintain the force in each of the rods approximately equal.

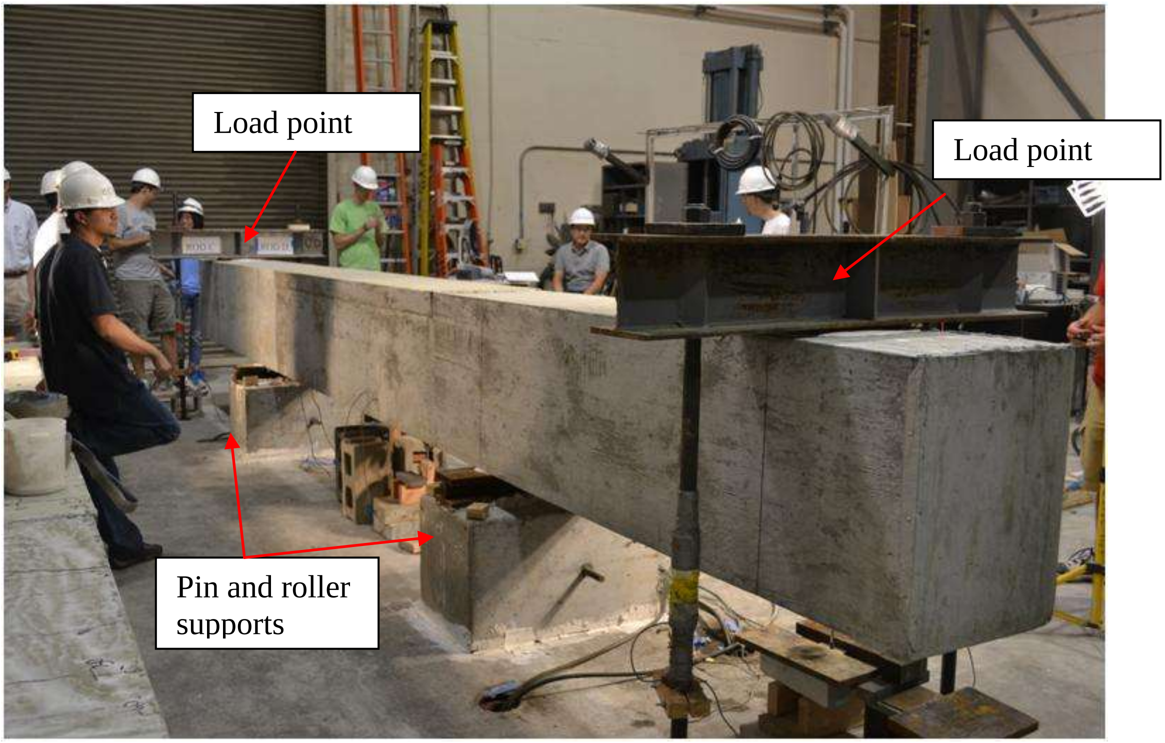


Figure A.6 – Test apparatus

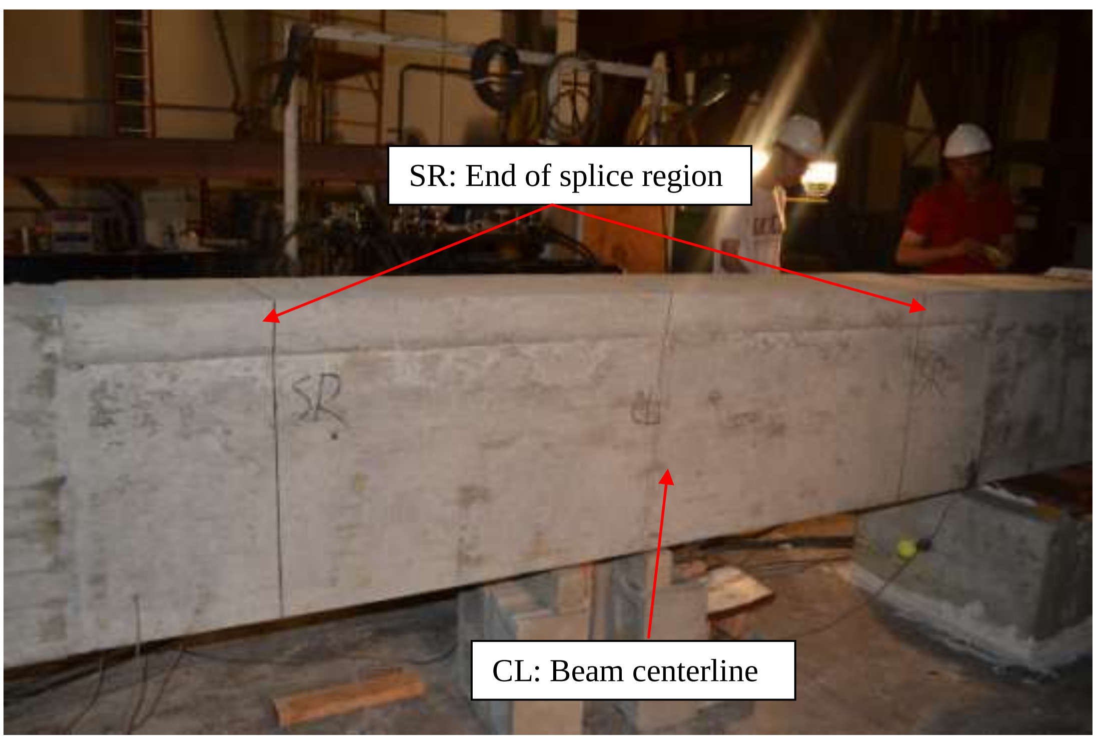


Figure A.7(a) – Splice region of Beam A2 prior to loading

Figure A.7(b) – East support of Beam A2 prior to loading

The loading protocol consisted of monotonically-increasing load applied at both the ends of the beams. Loading was paused at increments in the total force of 10 kips (5-kip increments applied at each end of the beam) to monitor crack widths, mark crack locations, and record dial gage readings (Figure A.8). After all these quantities were recorded, loading resumed until the next increment was completed. Given the potential for brittle failure and the large amount of energy stored in the beam, crack location, crack width, and dial gage readings were not recorded after the total load exceeded 140 kips (forces at beam ends exceeded 70 kips). After this point, the load was increased steadily until the end of the test. Measurements from load and displacement sensors were recorded without interruption during the test.

Figure A.8 – Marking cracks during test

4. Material Properties

The beams were tested on May 10, 2012, seven days after initial casting. On the day of the test the compressive strength of the concrete was 5090 psi in the body of the beam and 5150 psi above the cold joint.

A segment of the No. 11 bars used in the beams was tested in tension. The stress-strain curve for the No. 11 bar is shown in Figure A.9. To avoid damage, the extensometer was removed at approximately 3% elongation; force was recorded until failure. As shown in the figure, the No. 11 bar did not have a well-defined yield point. The yield stress calculated using the 0.2% offset method was 71 ksi, the proportional limit was approximately 67 ksi, and the measured elastic modulus was 27,666 ksi. The tensile strength of the steel was 108 ksi.

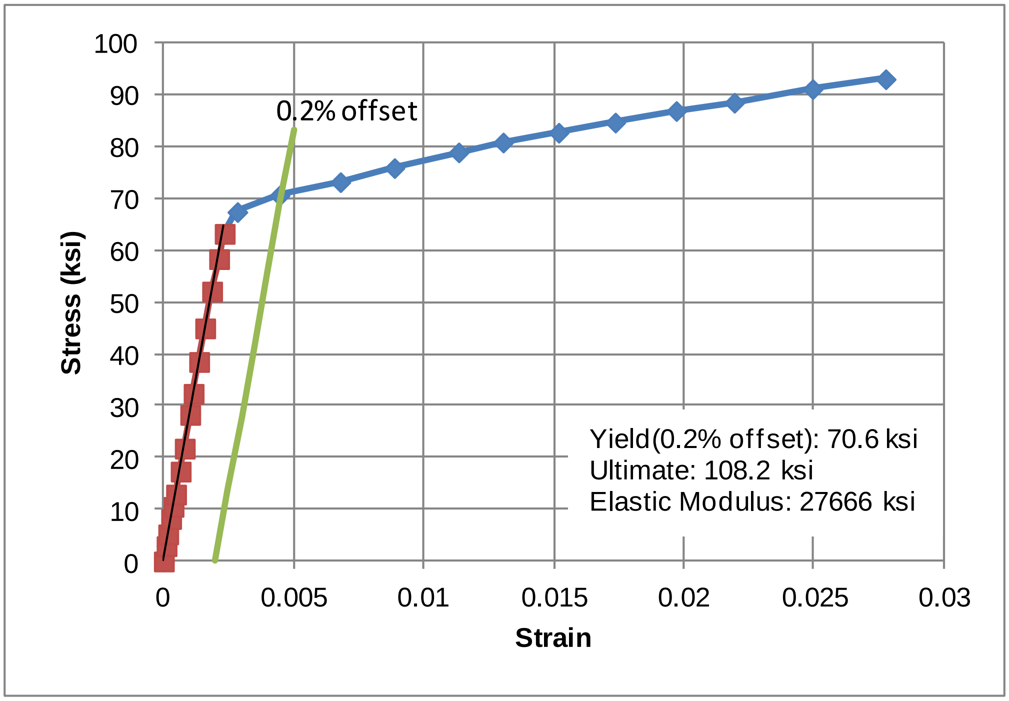


Figure A.9 – Measured stress-strain curve for the No. 11 bar used in the beams

5. <u>Test Results</u>

The load-deflection curves for Beams 1 (33-in. splice) and 2 (79-in. splice) are shown in Figures A.10 and A.11, respectively. The displacement shown in both figures was calculated by adding the average displacement at the two load points and the displacement at the center of the beam. The load shown in Figures A.10 and A.11 corresponds to the total load applied to the beam. Based on the shape of the load-deflection curves shown in Figures A.10 and A.11, it is concluded that a splice failure took place in Beam A1 and a flexural failure occurred in Beam A2.

For Beam A1 (33-in. splice length), the peak total load recorded was 140 kips, at a corresponding total displacement of 1.14 in. (Figure A.10). At a total load of 140 kips, the stress in the bars calculated using elastic cracked section theory was approximately 54 ksi. After the displacement exceeded 1.14 in., the total load dropped in a sudden manner to approximately 133 kips. If it is assumed that the tension force is carried in its entirety by the two continuous bars, a total force of 133 kips corresponds to a calculated bar stress equal to the yield stress of 71 ksi (based on linear elastic cracked section theory). These calculations indicate that splice failure occurred at a displacement of 1.14 in. and that the splice lost all its load carrying capacity in a sudden manner. The total load tended to increase again at displacements greater than 1.6 in., which is attributed to the effects of strain hardening in the two continuous bars.

The load-deflection curve for Beam A2, with a splice length of 79-in., is presented in Figure A.11. Loading was stopped when crushing of the concrete in the compression zone was observed in the constant moment region, in the areas adjacent to the two beam supports, at a total displacement of approximately 2.5 in. Unlike the curve for Beam A1, there was no sudden drop in load associated with failure of the splice. In the case of Beam A2, a sharp decrease in the slope of the load-deflection curve was observed at a total load of approximately 172 kips and total displacement of approximately 1.4 in. The stress in the three bars calculated based on moment-curvature analysis at this load is approximately 67 ksi (Table A.1), which corresponds to the observed proportional limit of the measured stress-strain relationship of the steel (Figure A.9). The calculated steel stress indicates that the sharp decrease in the slope of the load-deflection curve at 172 kips was caused by yielding of the reinforcing steel, not by failure of the splice. After yielding began, the total load continued to increase with increasing displacement, as the reinforcing steel strain hardened. The maximum load prior to flexural failure was approximately 186 kips, which corresponds to a bar stress of 72 ksi in all three bars (Table A.1). At a total load of 186 kips, horizontal splitting cracks on the beam top surface were observed (described in more detail below).

After the tests we completed, the beams were autopsied to determine the actual cover on the bars. For Beam A1, the top cover was 4 in., and side covers to the continuous bars were 3.5 (North) and 3.75 in. (South). For Beam A2, the top cover was 4 in., and side covers to the continuous bars were 3.5 in. (North and South). (These values are reflected in the bar stresses in the previous paragraph and summarized in Table A.1)

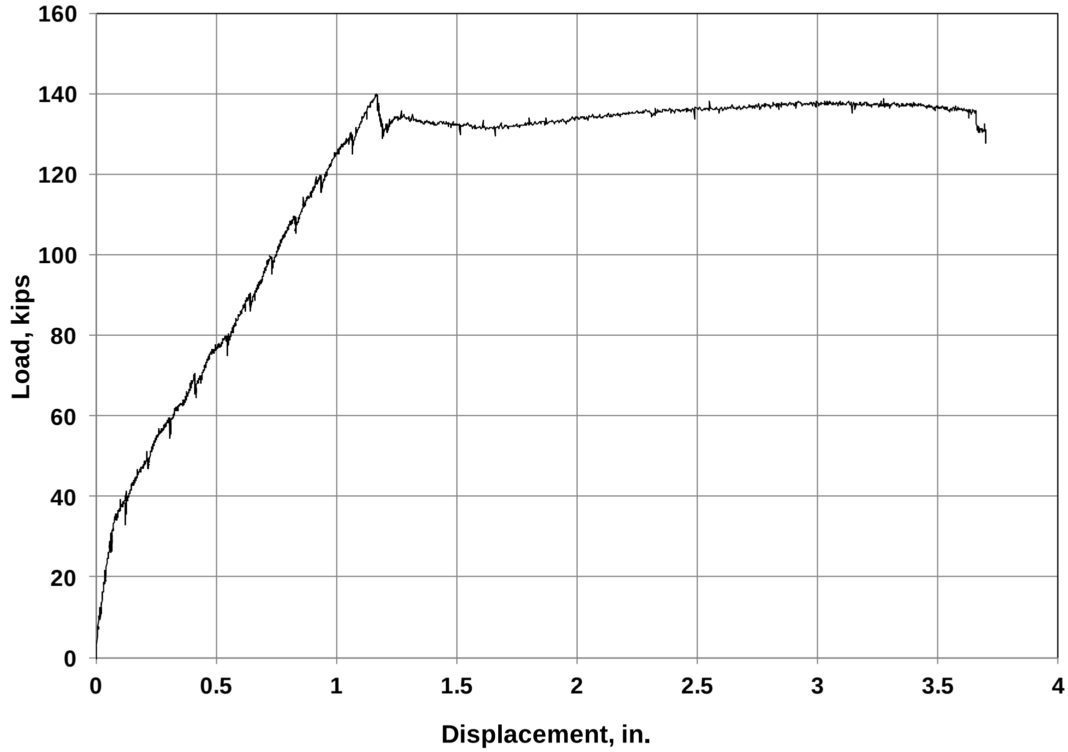


Figure A.10 – Total load vs. total deflection for Beam A1 (33-in. splice length)

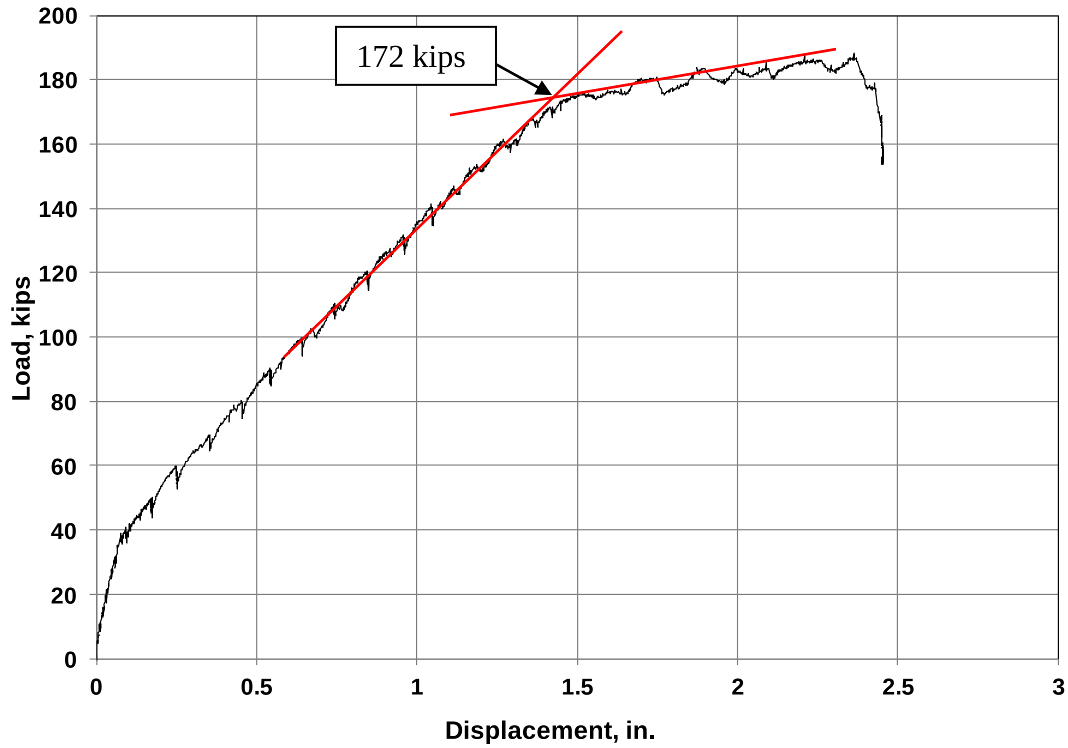


Figure A.11 – Total load vs. added deflection for Beam A2 (79-in. splice length)

Loads, moments, and bar stresses for the beams were calculated assuming that loads and reactions acted along the longitudinal centerline of the beam. Reactions and moments were calculated based on load cell readings and the weight of the loading assemblies. The self-weight of the beam was included in the calculations based on average beam dimensions and an assumed concrete density of 150 pcf.

The calculated moment, bar stress at splice failure, and calculated bar stress using the splice strength equation developed by ACI Committee 408 (2003) are shown in Table A.1. It is important to note that the splice strength expression developed by Committee 408 was calibrated on the basis of beams without preexisting cracks in the plane of the flexural reinforcement, and for this reason are presented only as a reference. For Beam A1 (with a 33-in. long splice), the bar stress at splice failure calculated based on a moment-curvature analysis was approximately 54 ksi. The calculated splice strength using the expression developed by ACI Committee 408 (ACI 408R) was 70 ksi. For Beam A2 (with a 79-in. long splice), the calculated bar stress at flexural failure was approximately 72 ksi, while the calculated splice strength using the ACI 408 expression was 140 ksi.

Table A.1 – Bar stresses at splice failure

Splice length	Failure mode	Total load at splice failure, kips	Calculated moment at splice failure, kip-ft	Inferred bar stress at failure based on moment-curvature relationship, ksi	Predicted bar stress (**uncracked** concrete-ACI 408R)
33-in.	splice failure	140	355	54	70
79 in.	flexural failure	186	472	72	140

The strain in the No. 11 bars was measured using strain gages located 2 in. outside the splice region (Figure A.2). The relationships between measured strain and total load are shown in Figures A.12 and A.13 for beams 1 and 2, respectively.

As shown in Figure A.12, the strain in the spliced bars (East-center and West-center gages) of Beam A1 increased to a maximum of 1750 and 1700 microstrain, respectively, and then dropped in a sudden manner. The maximum strain in the spliced bar was recorded at a total load of approximately 130 kips and corresponds to a bar stress of approximately 50 ksi, which is very close to the failure value of 54 ksi inferred on the basis of moment-curvature analysis (Table A.1). Strain readings from the east-center gage on the spliced bar show that the strain dropped from approximately 1700 to approximately 1300 microstrain at a total load of 130 kips, corresponding to a sudden reduction in capacity of approximately 25%. When the total load reached 140 kips, the strain in the east-center gage dropped suddenly to almost zero. Strain readings from the east continuous bar (East-Side 1) show a sudden increase from 2100 microstrain to more than 2500 microstrain at the failure total load of 140 kips. The strain gage readings indicate that failure of the splice led to a rapid decrease in the stress in the spliced bars, and that the tension force that was lost due to failure of the splice was transferred to the continuous bars, causing yielding of the continuous bars at a total force of 140 kips.

For Beam A2 (79-in. long splice), the recorded strains show a plateau (Figure A.13) due to exceeding the limiting strain allowed by the gain in the data acquisition system.

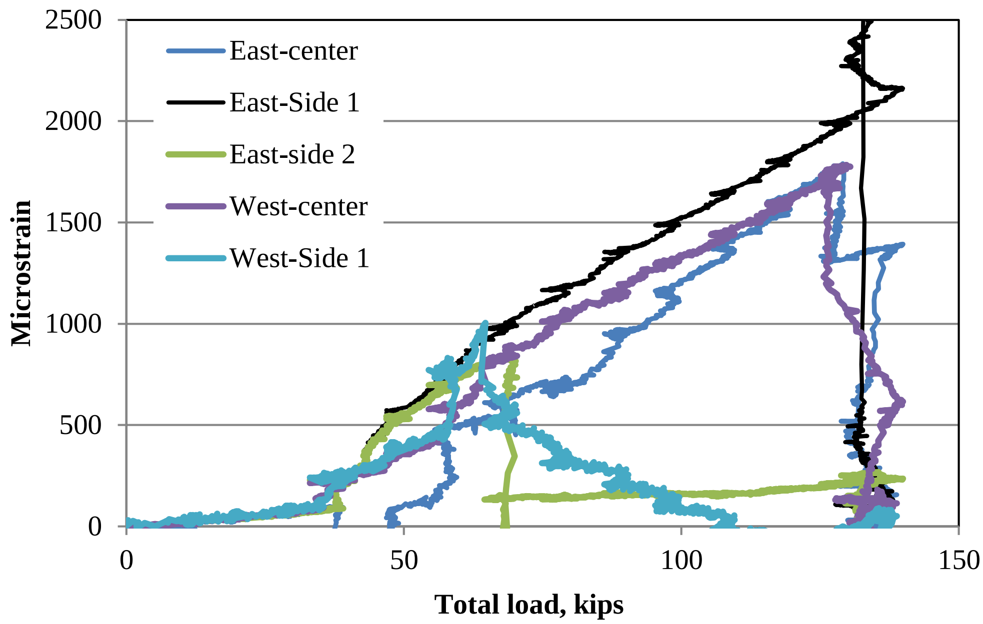


Figure A.12 – Measured strain in the reinforcing bars vs. total load for Beam A1 (33-in. splice length). (Note: The beam was oriented in an east-west direction; "center" identifies strain gages on the spliced bars and "side" means strain gauges on the continuous bars)

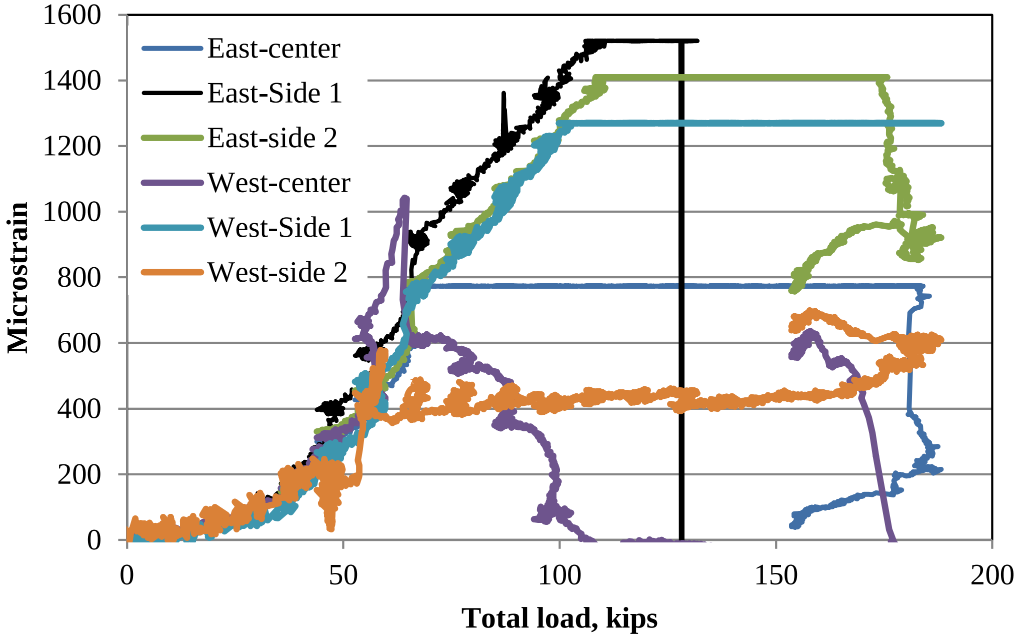


Figure A.13 – Measured strain in the reinforcing bars vs. total load for Beam A2 (79-in. splice length). (Note: The beam was oriented in an east-west direction; "center" identifies strain gages on the spliced bars and "side" means strain gauges on the continuous bars)

6. <u>Beam crack patterns</u>

Figures A.14 through A.18 are photographs taken after the conclusion of the two tests. For Beam A1 (33-in. splice length), splitting cracks were observed on the top surface between the vertical edges of the cold joint (Figures A.14 and A.15). The cracks were approximately ¼ in. wide, as shown in Figure A.16. Splitting cracks above the splice were also noted in Beam A2 (79-in. splice length) (Figures A.17 and A.18), although they were much narrower than those observed in the Beam A1.

The crack patterns for both beams show that the side stirrups were effective in keeping the cover in place, even after failure of the splice for Beam A1. In the case of Beam A1, the cracks were wider, which is consistent with the sudden drop in bar force that occurred at splice failure. For Beam A2, the cracks were much narrower, and it is apparent that the splice was able to sustain the same bar force as the continuous bars at displacements large enough to cause flexural failure of the beam.

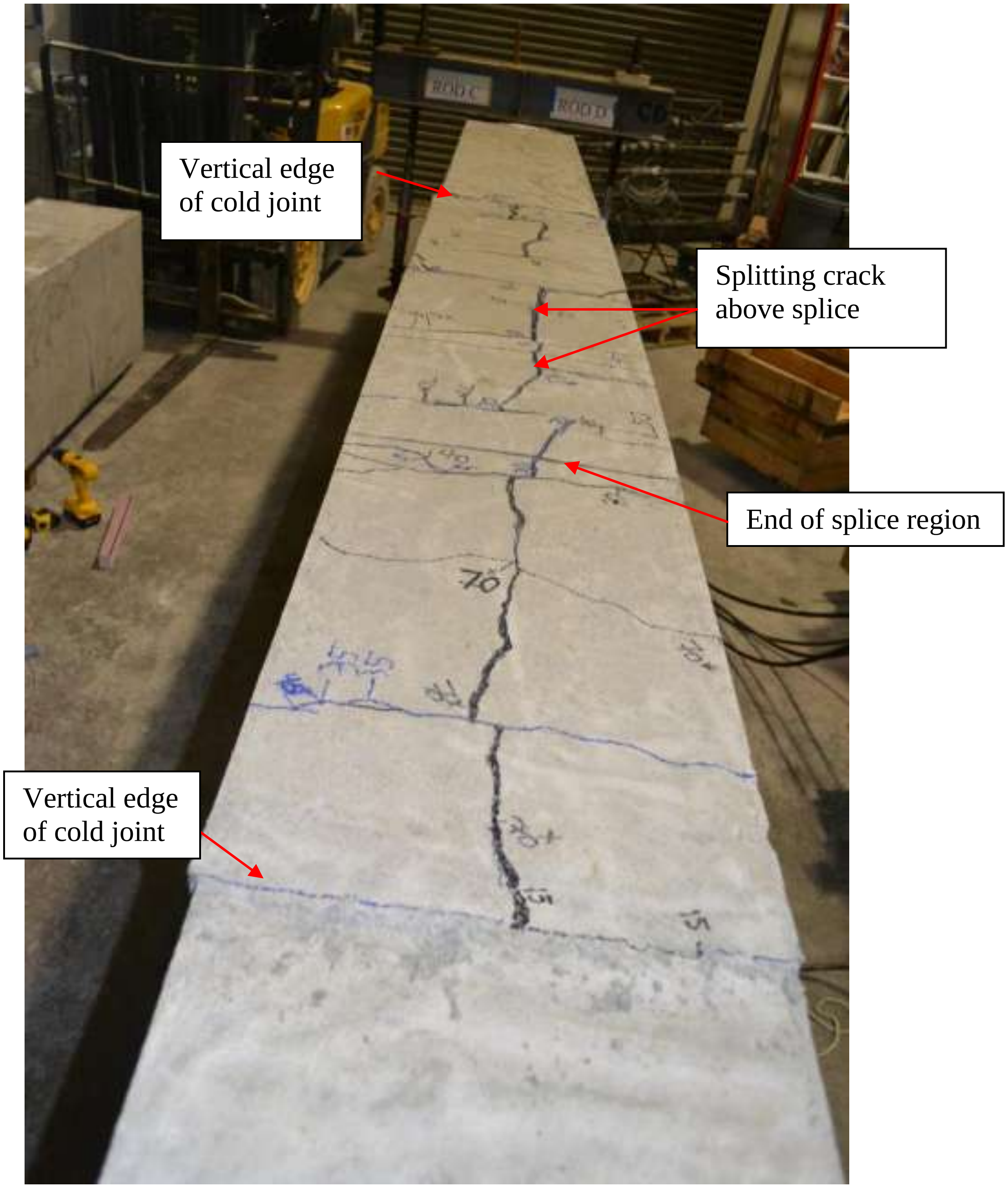


Figure A.14 – Splitting crack at the top of the splice region for Beam A1 (33-in. splice length).

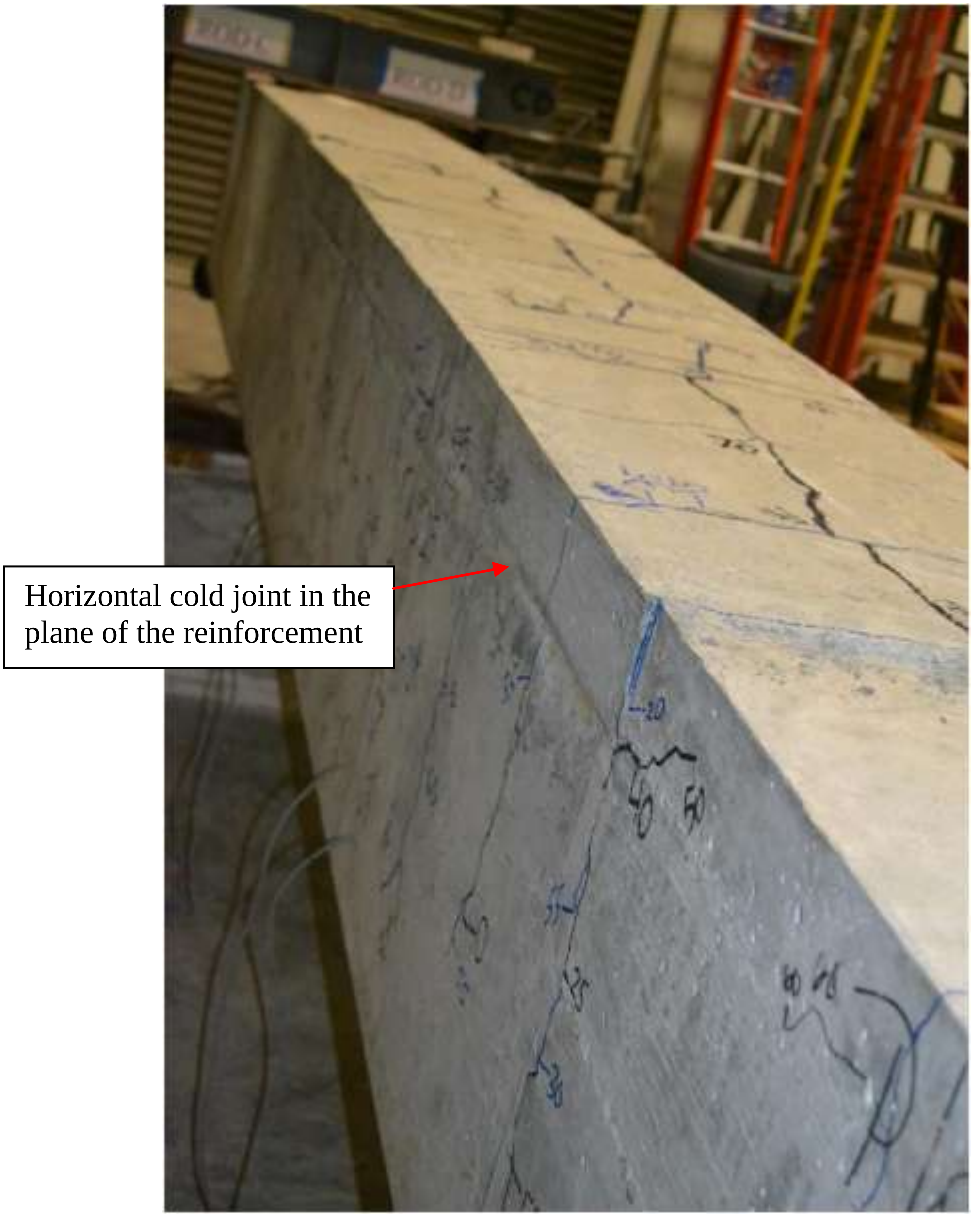


Figure A.15 – Crack pattern in the splice region for Beam A1 (33-in. splice length).

Figure A.16 – Splitting crack at the top of the splice region of Beam A1 (33 in. splice length).

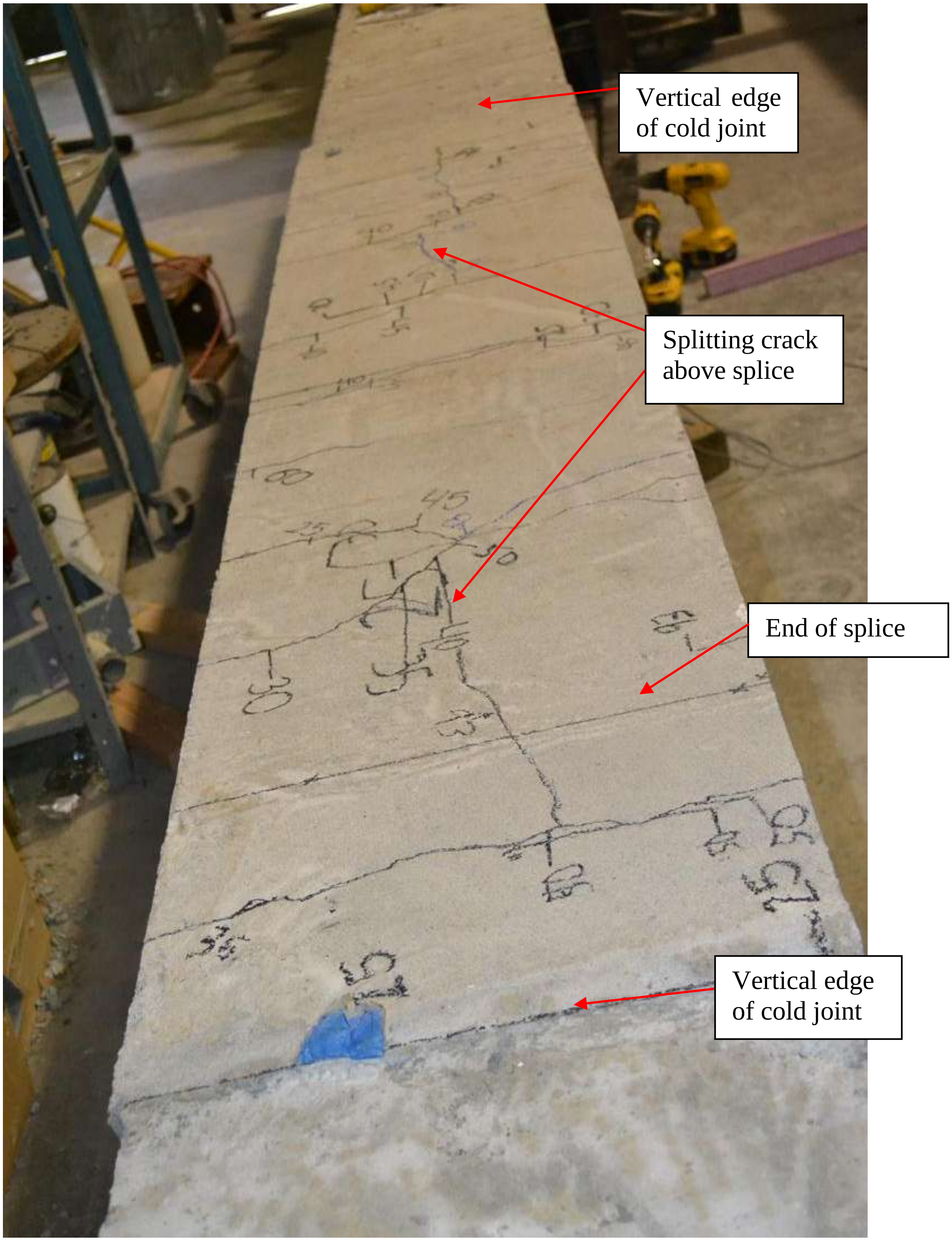


Figure A.17 – Splitting crack at the top of the splice region of Beam A2 (79 in. splice length).

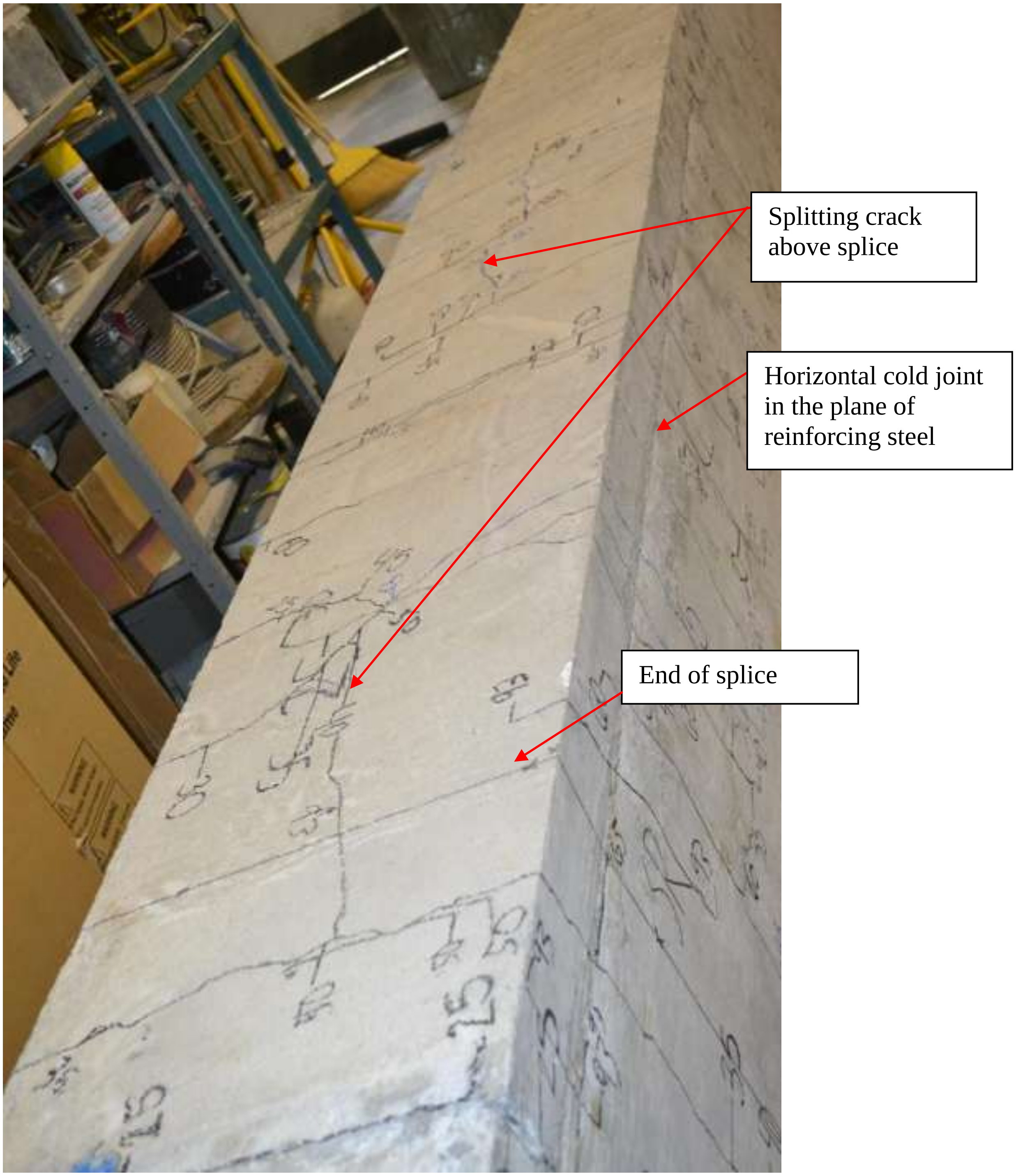


Figure A.18 – Crack pattern in the splice region of Beam A2 (79-in. splice length)

Appendix B: Reinforcing Steel Drawings

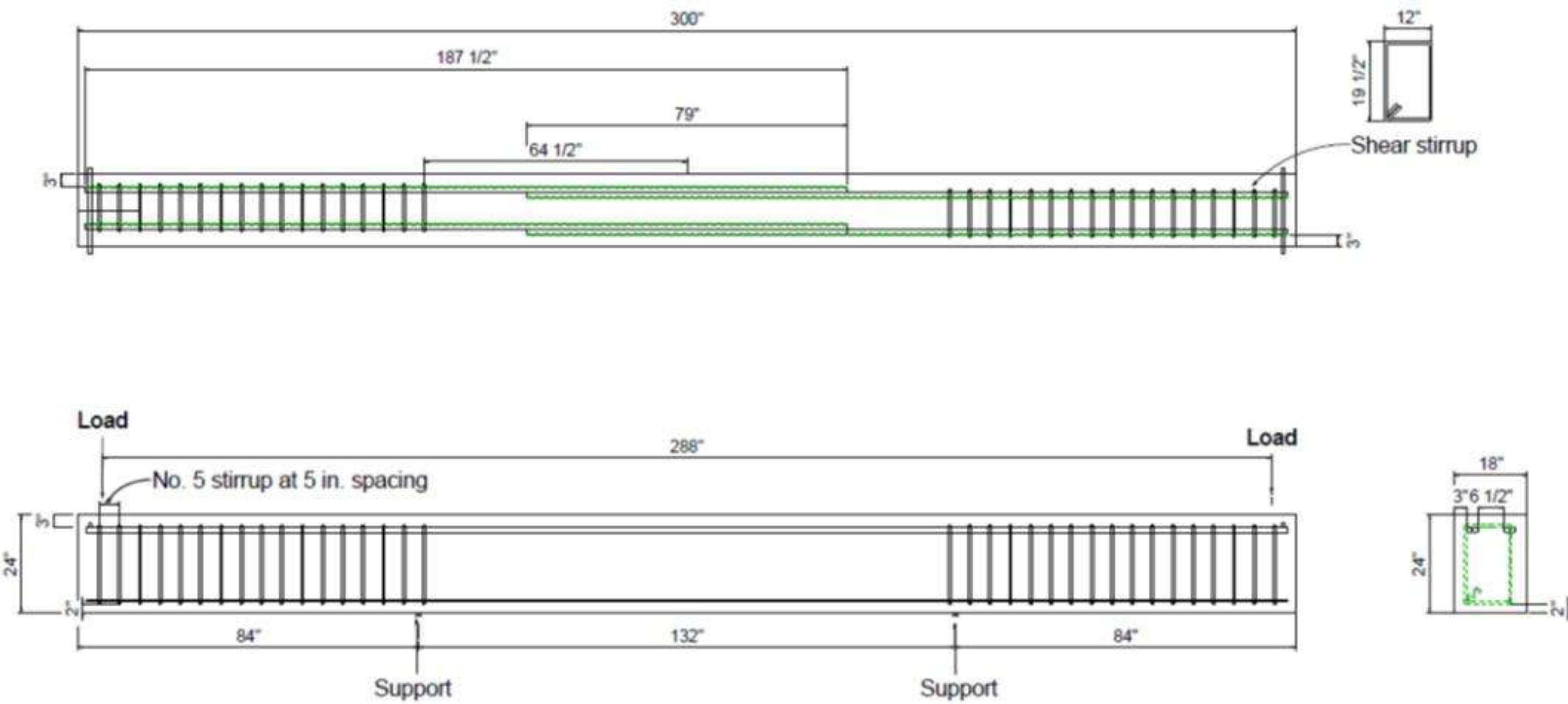


Figure B. 1 – Reinforcing steel drawing for beams with 79 in. splice length - monolithic

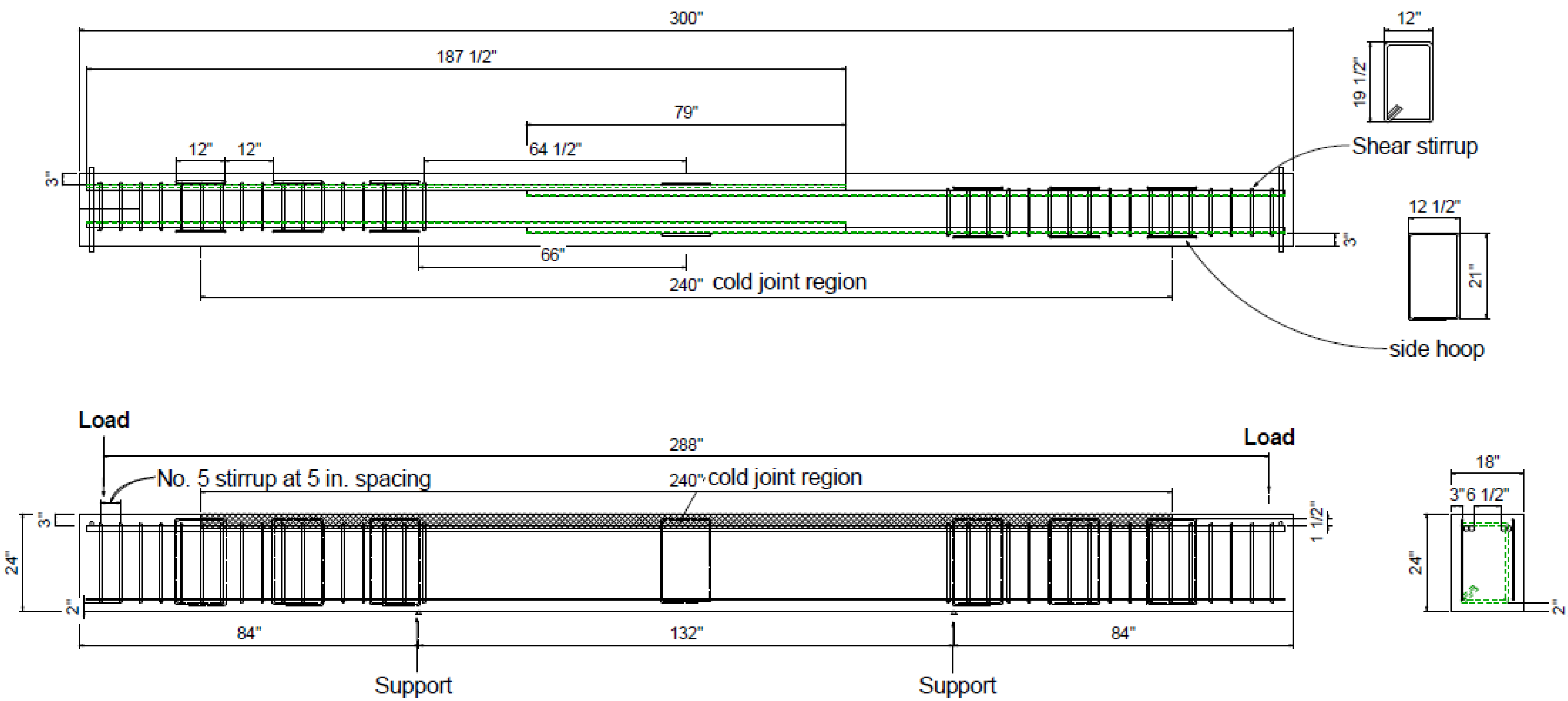


Figure B.2 – Reinforcing steel drawing for beams with 79 in. splice length – with crack

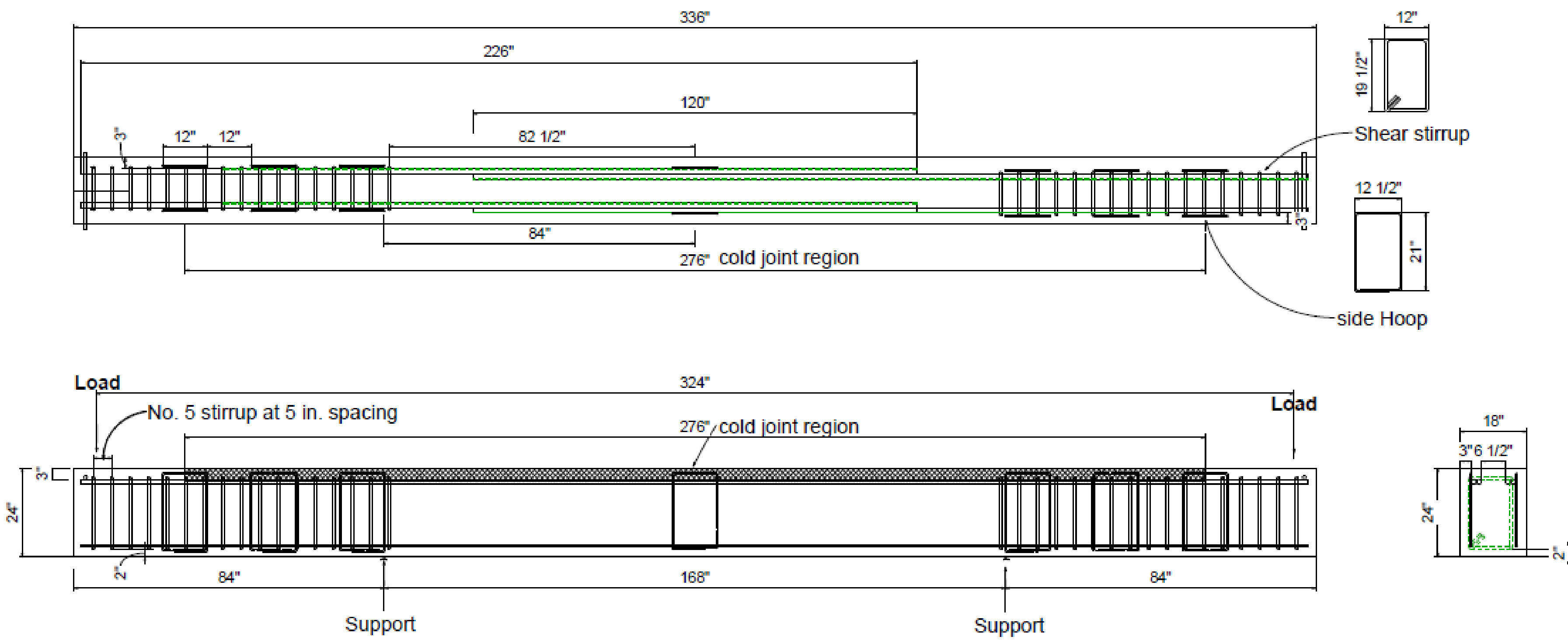
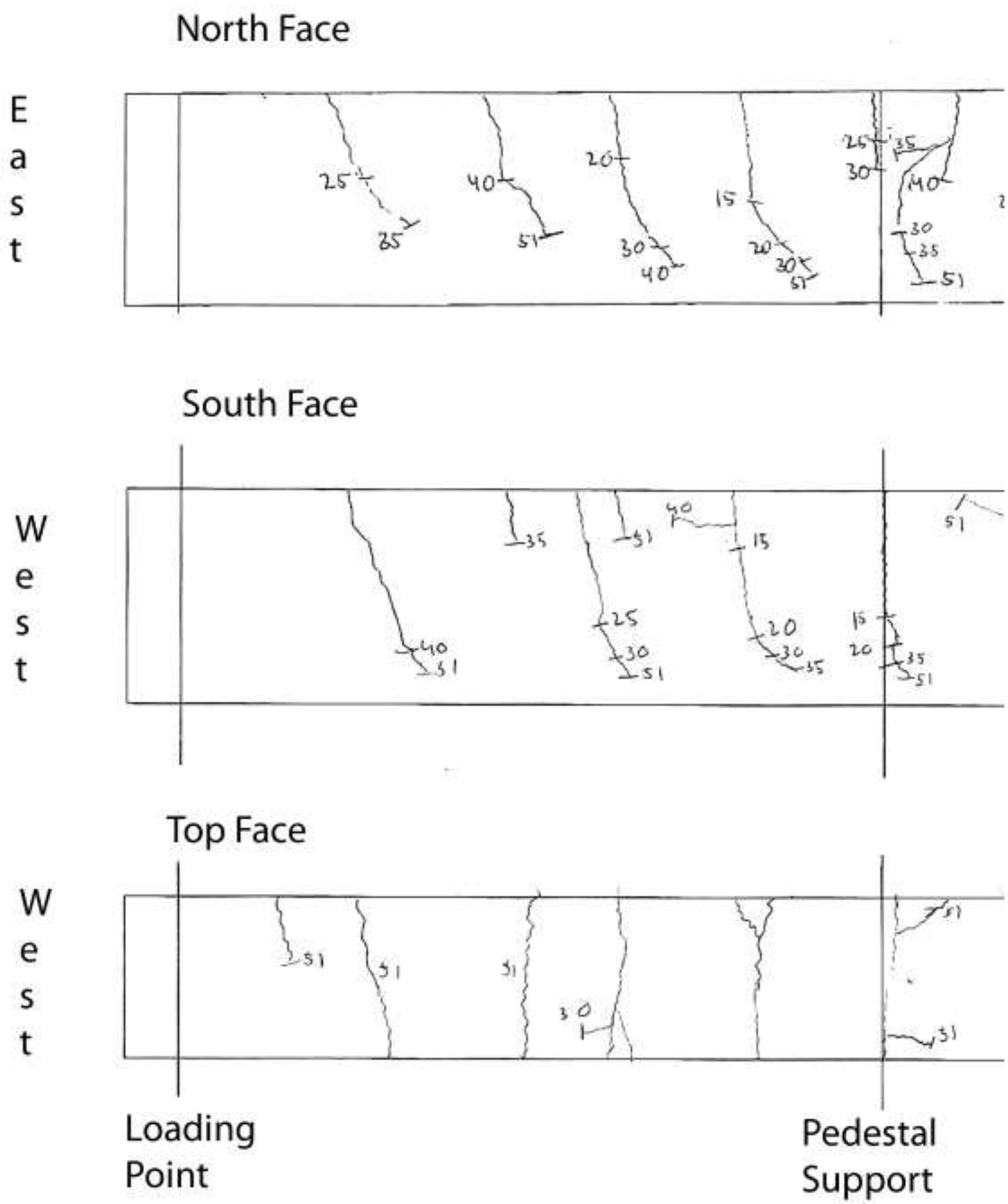
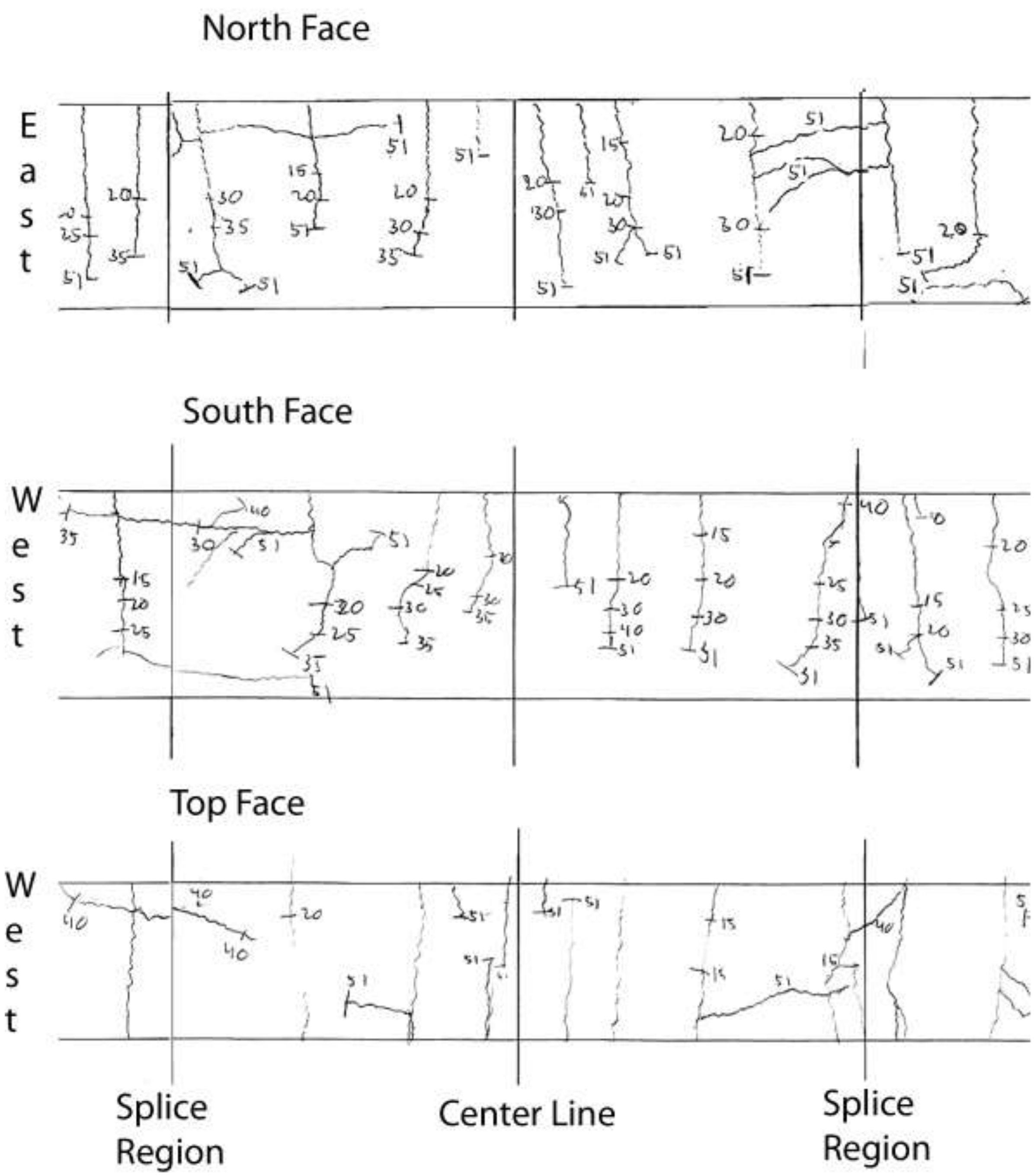


Figure B.3 – Reinforcing steel drawing for beams with 120 in. splice length – with crack

Appendix C: Detailed crack maps of Beams 1 – 6

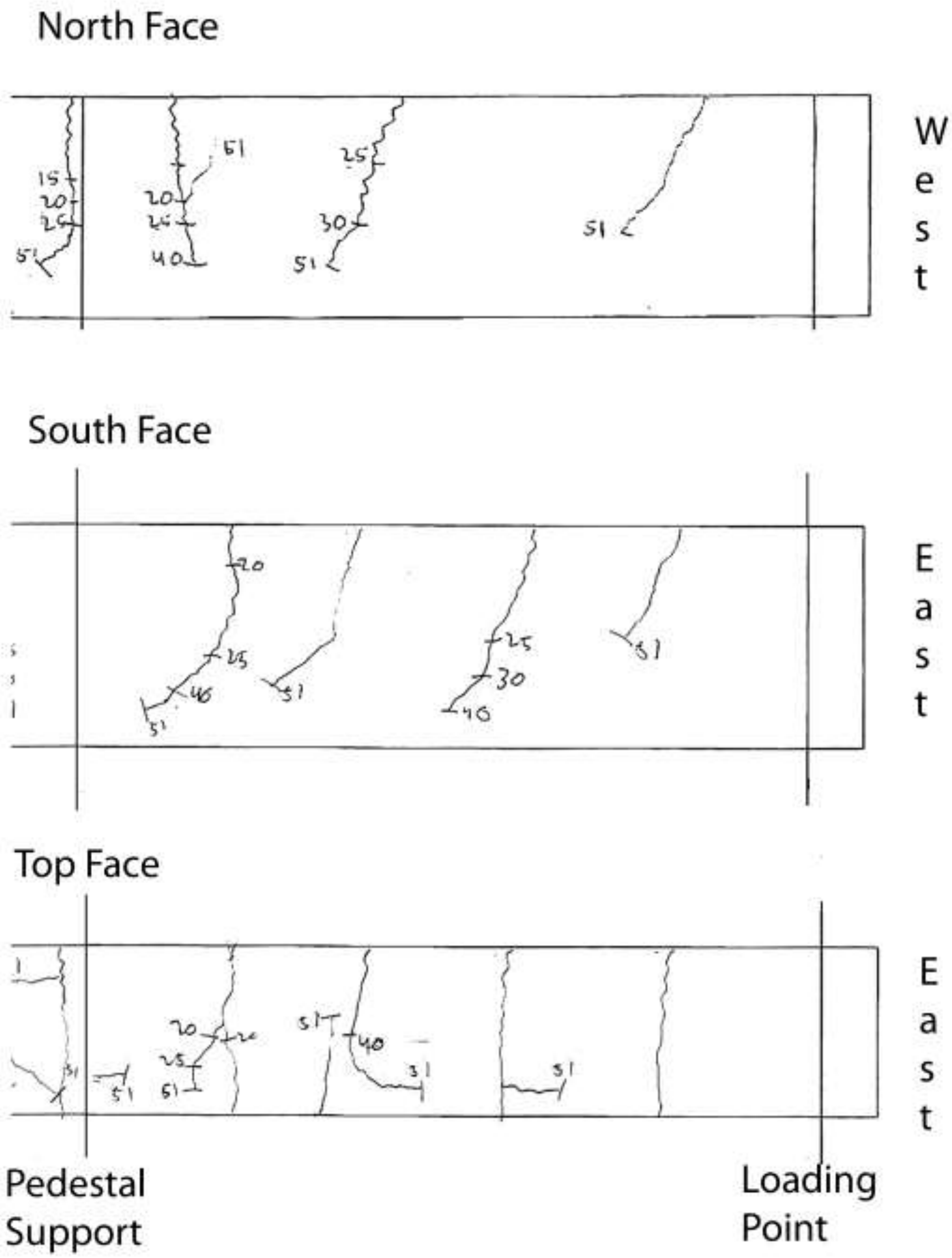

North Face
East
South Face
West
Top Face
West
Loading Point
Pedestal Support

(a)



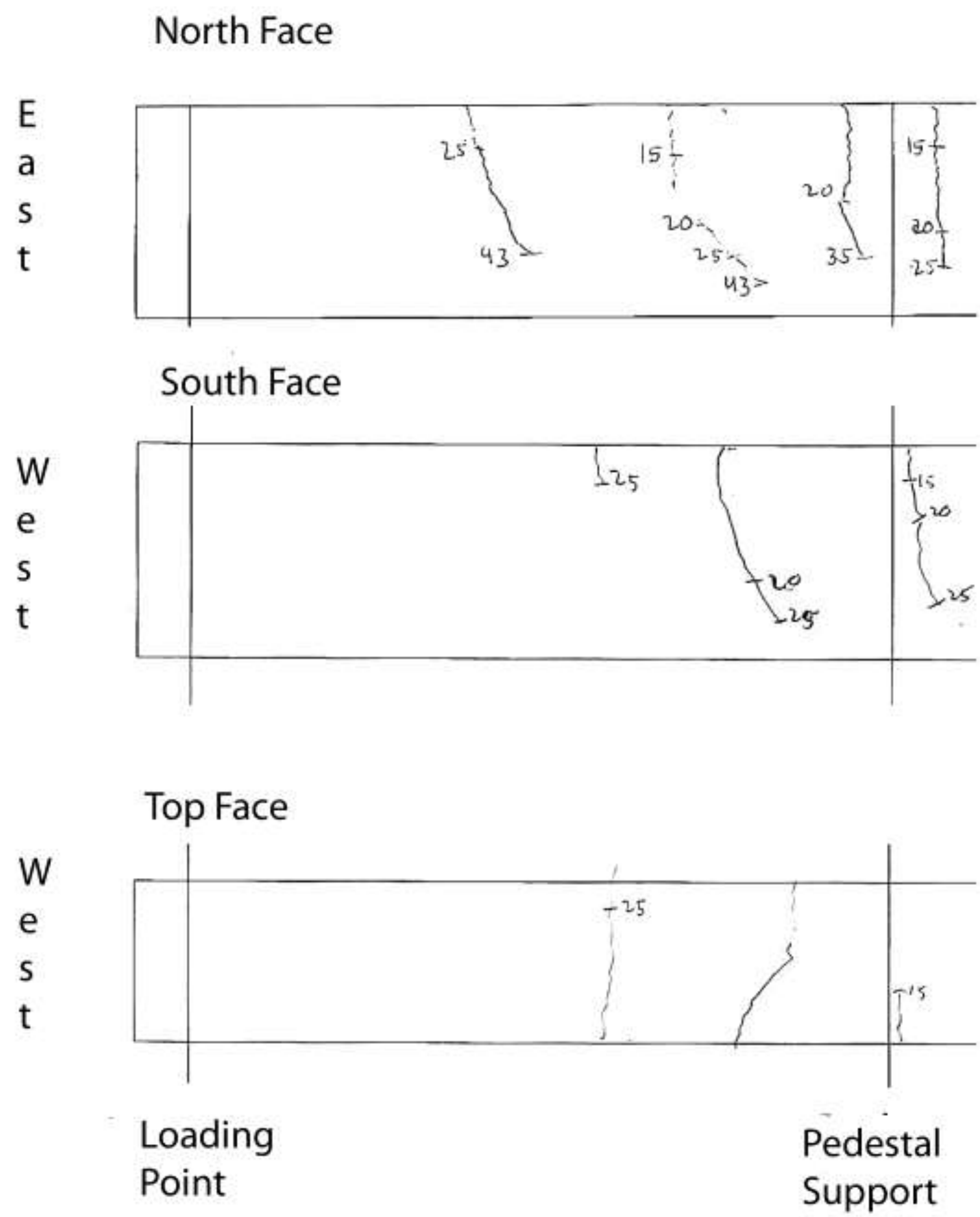
North Face
East
South Face
West
Top Face
West
Splice Region
Center Line
Splice Region

(b)

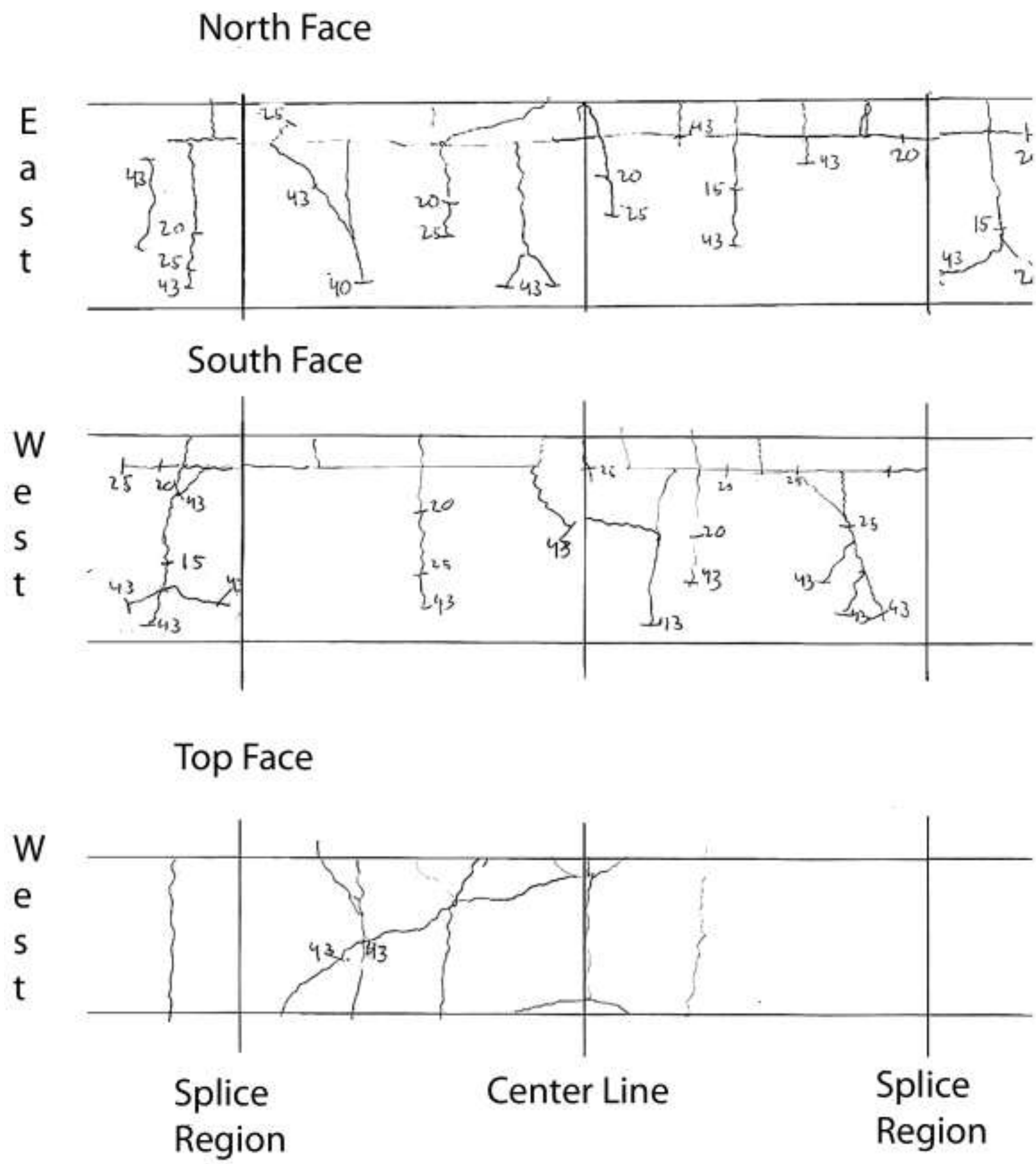


(c)

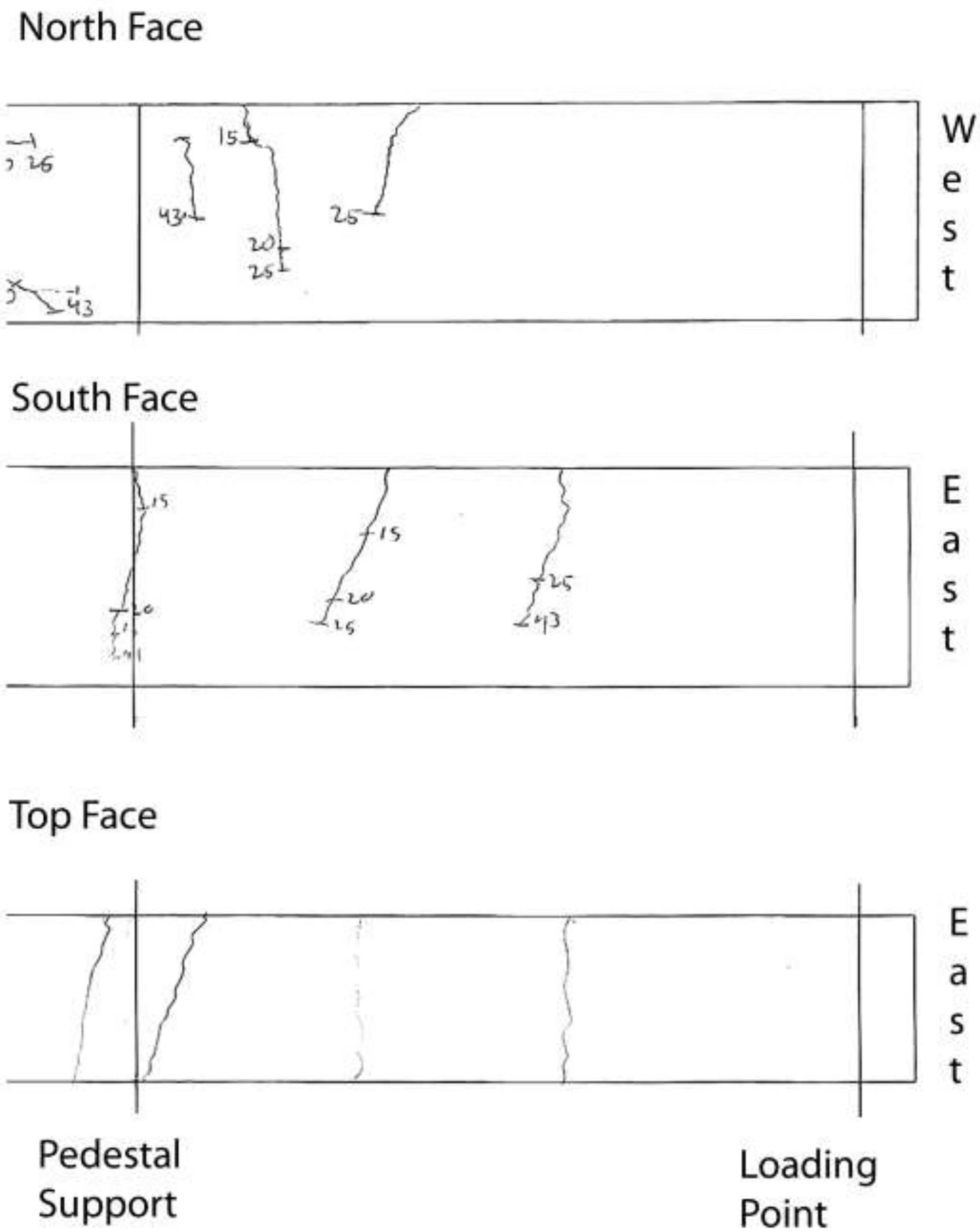
Figure C.1 – Crack map for Beam 1. Numbers indicate maximum average end load when cracks marked.


North Face
East
South Face
West
Top Face
West
Loading Point
Pedestal Support

(a)

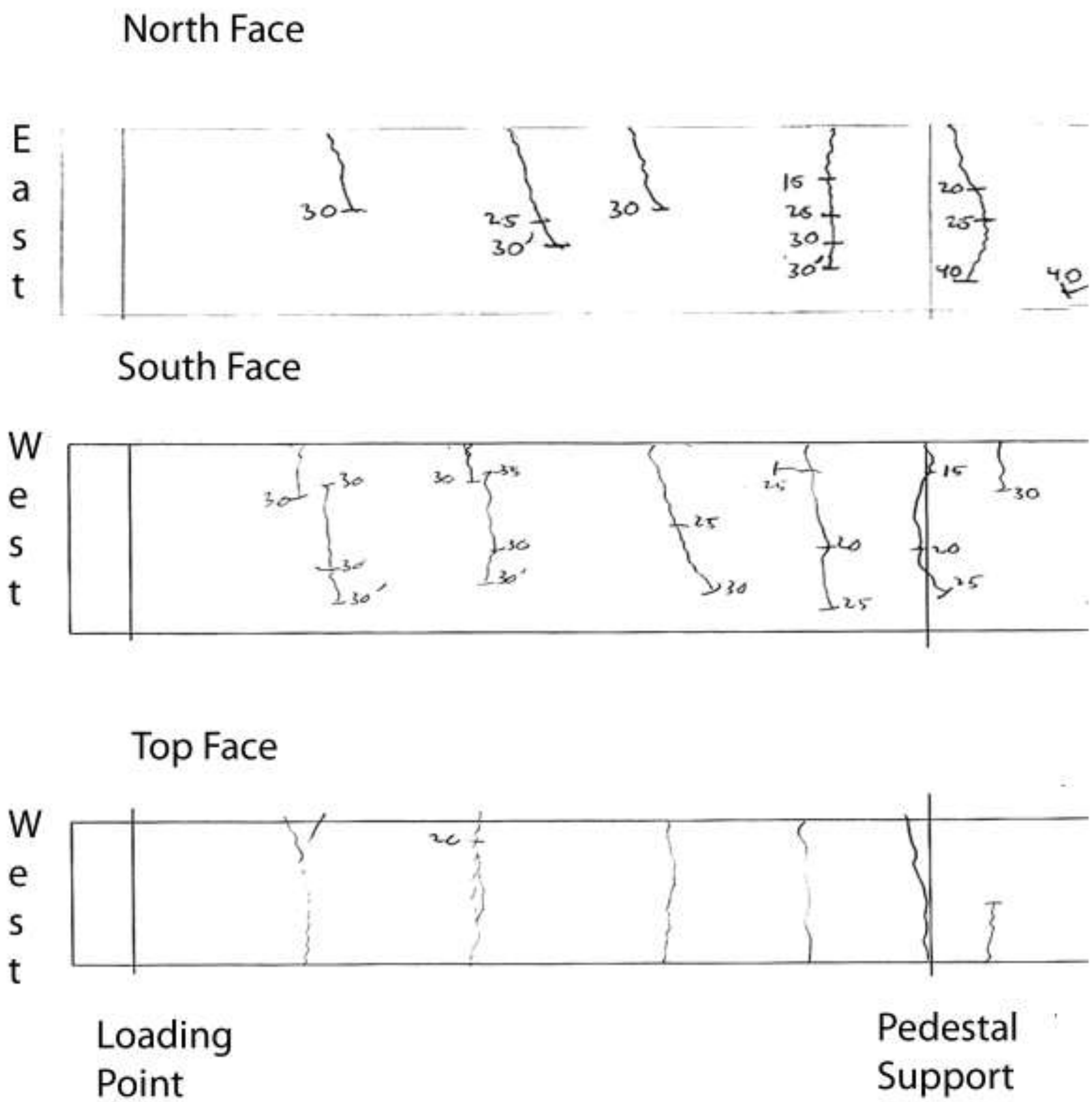

North Face
East
South Face
West
Top Face
West
Splice Region
Center Line
Splice Region

(b)

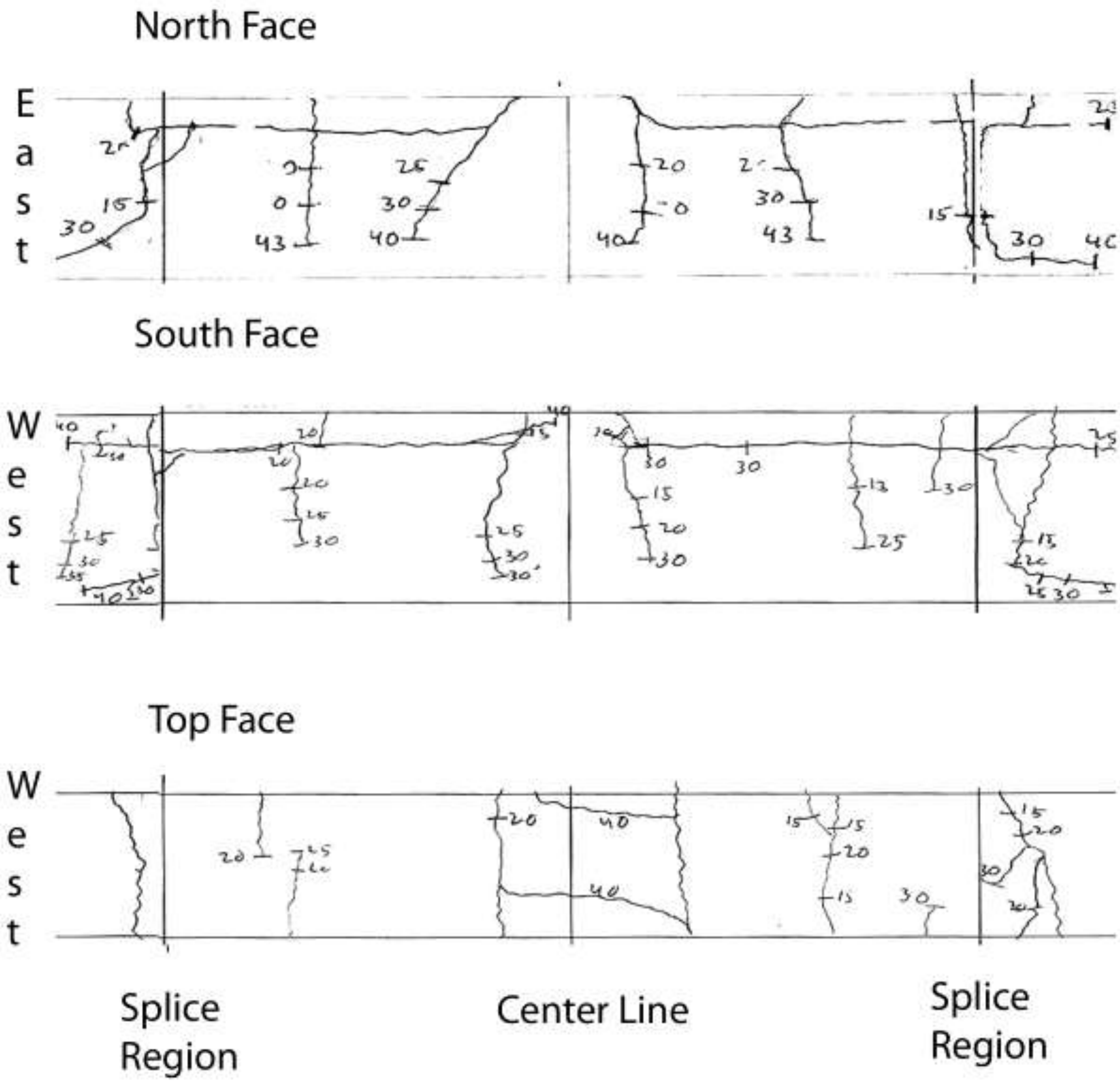


(c)

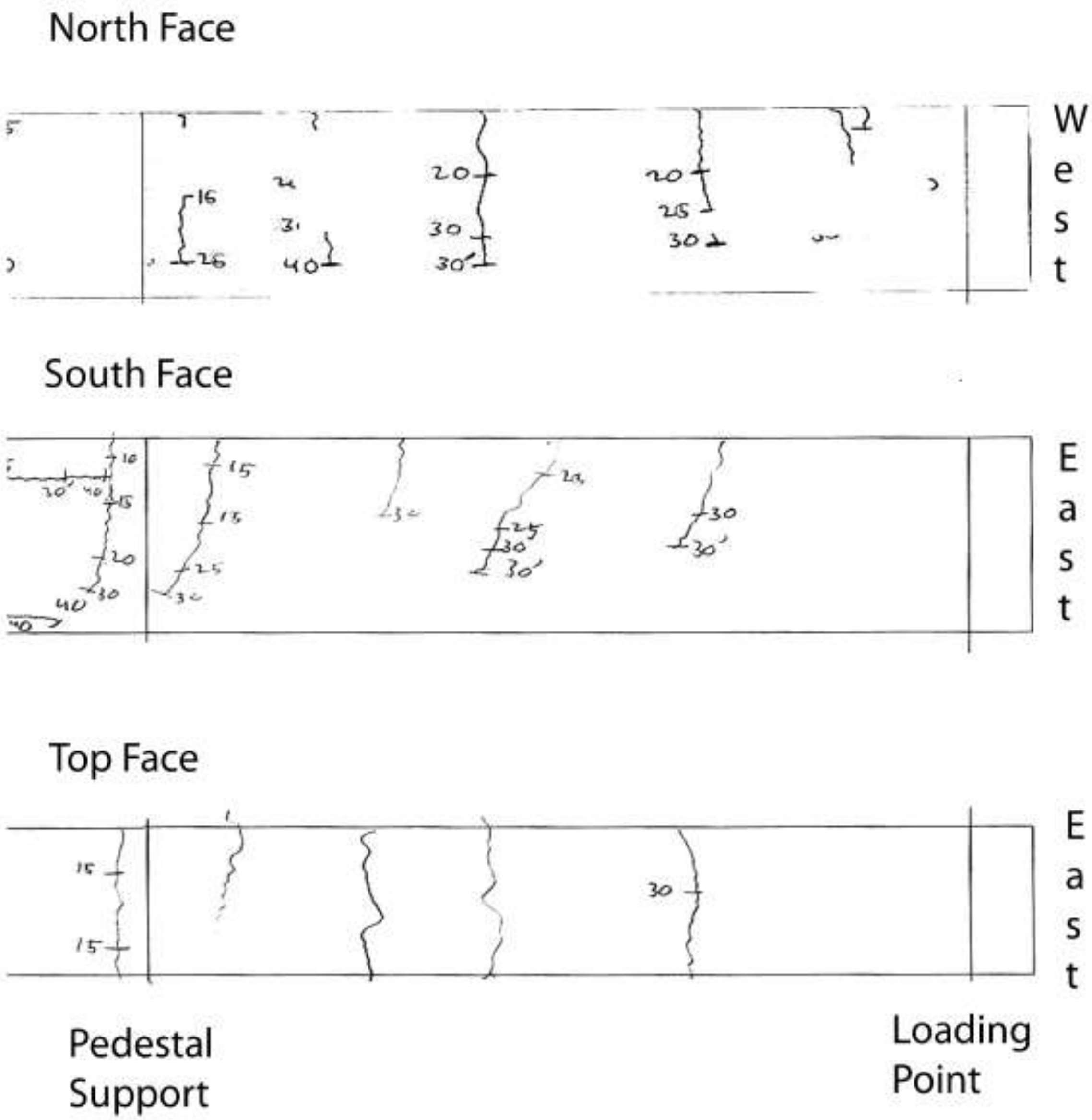
Figure C.2 – Crack map for Beam 2. Numbers indicate maximum average end load when cracks marked.


North Face
East
South Face
West
Top Face
West
Loading Point
Pedestal Support

(a)

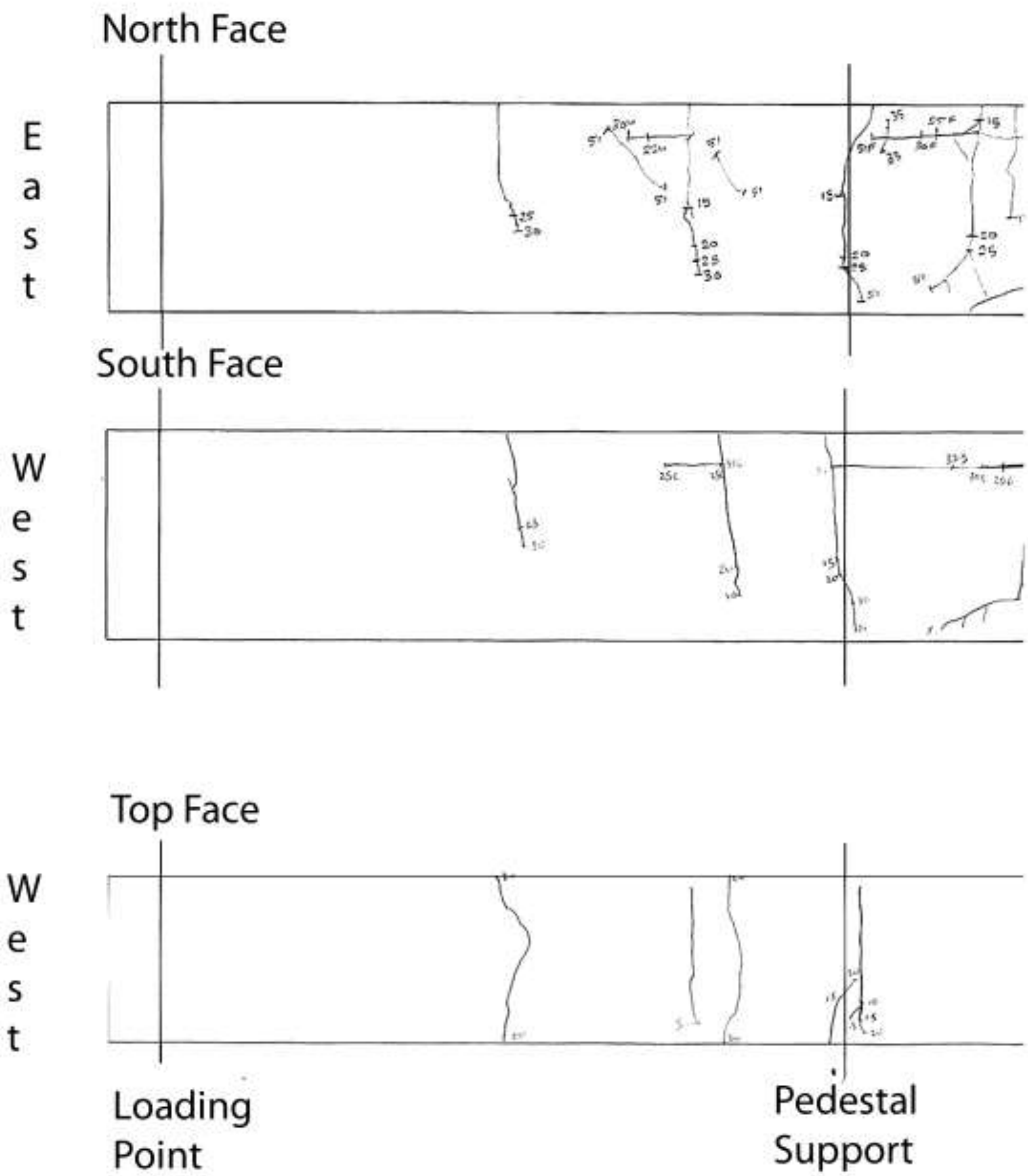

North Face
East
South Face
West
Top Face
West
Splice Region
Center Line
Splice Region

(b)

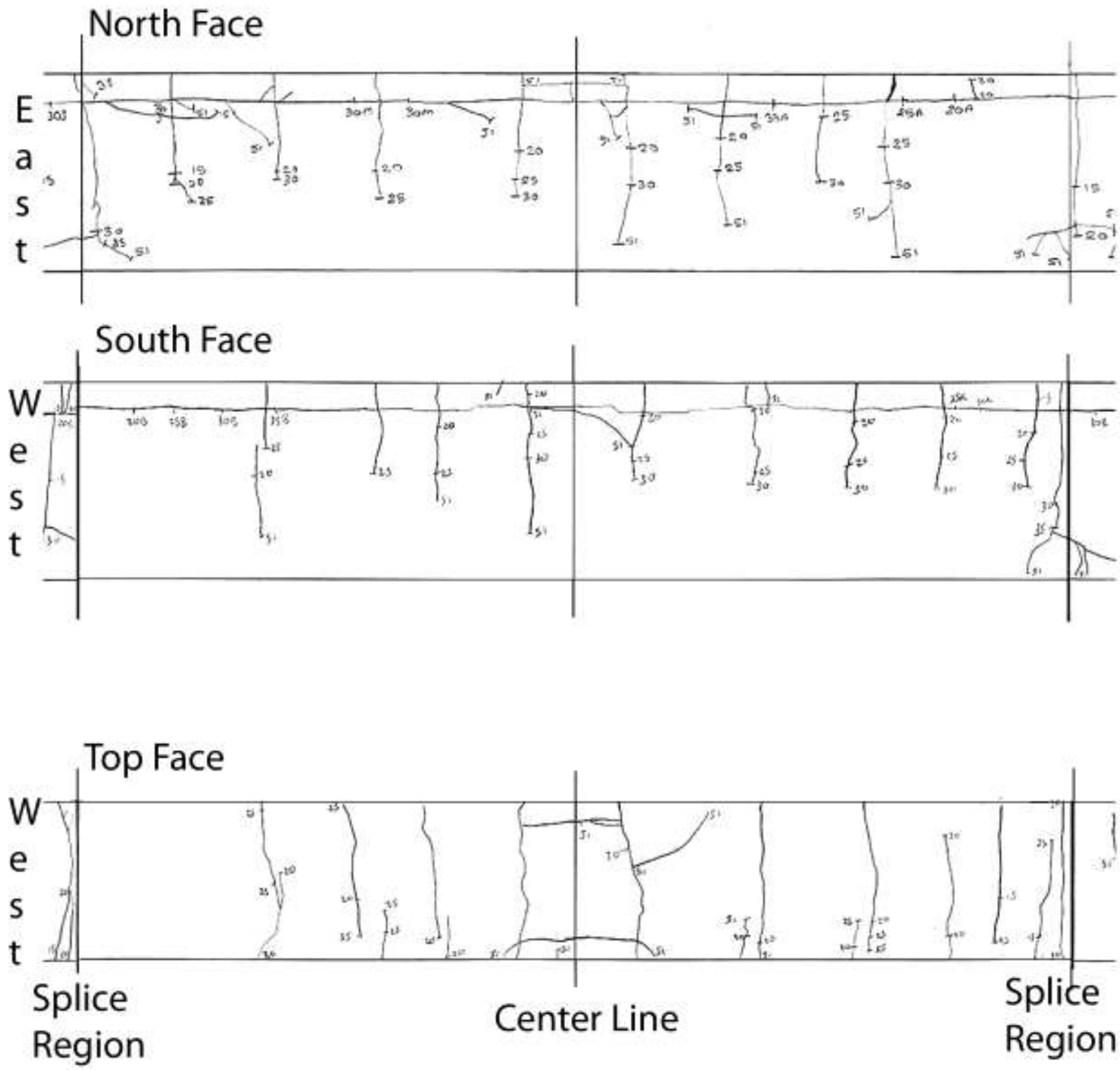


(c)

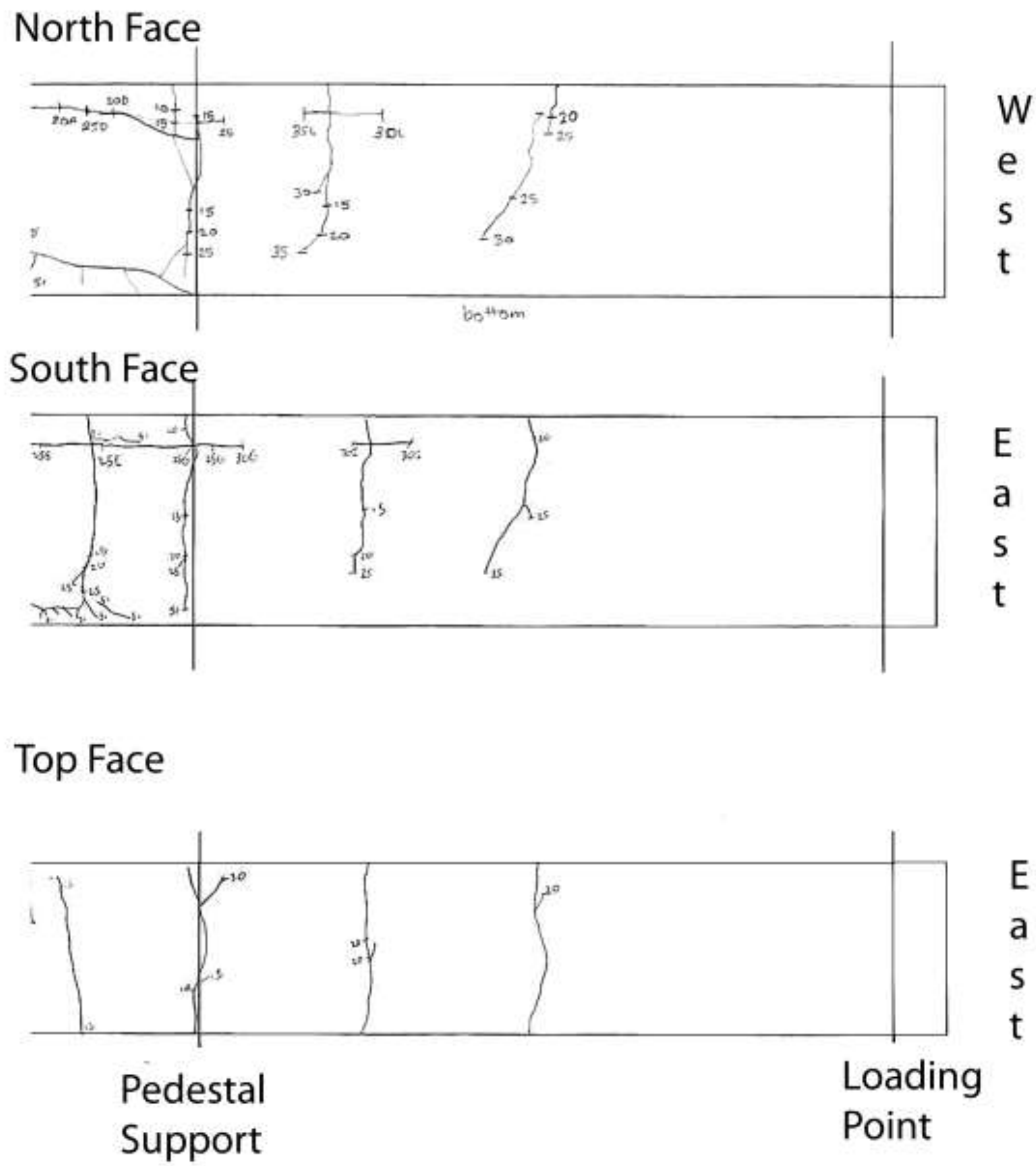
Figure C.3 – Crack map for Beam 3. Numbers indicate maximum average end load when cracks marked.


North Face
East
South Face
West
Top Face
West
Loading Point
Pedestal Support

(a)

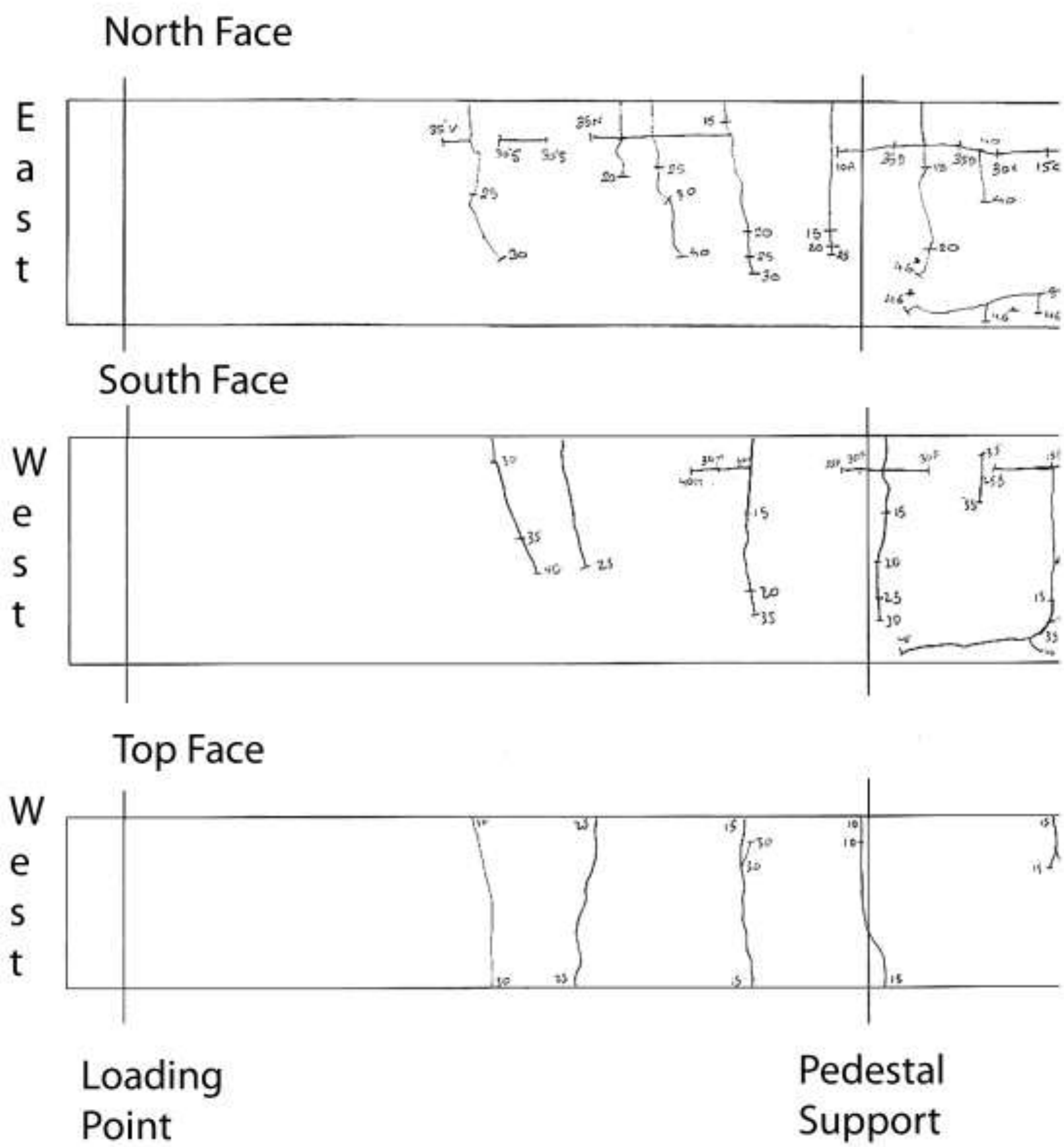

North Face
East
South Face
West
Top Face
West
Splice Region
Center Line
Splice Region

(b)

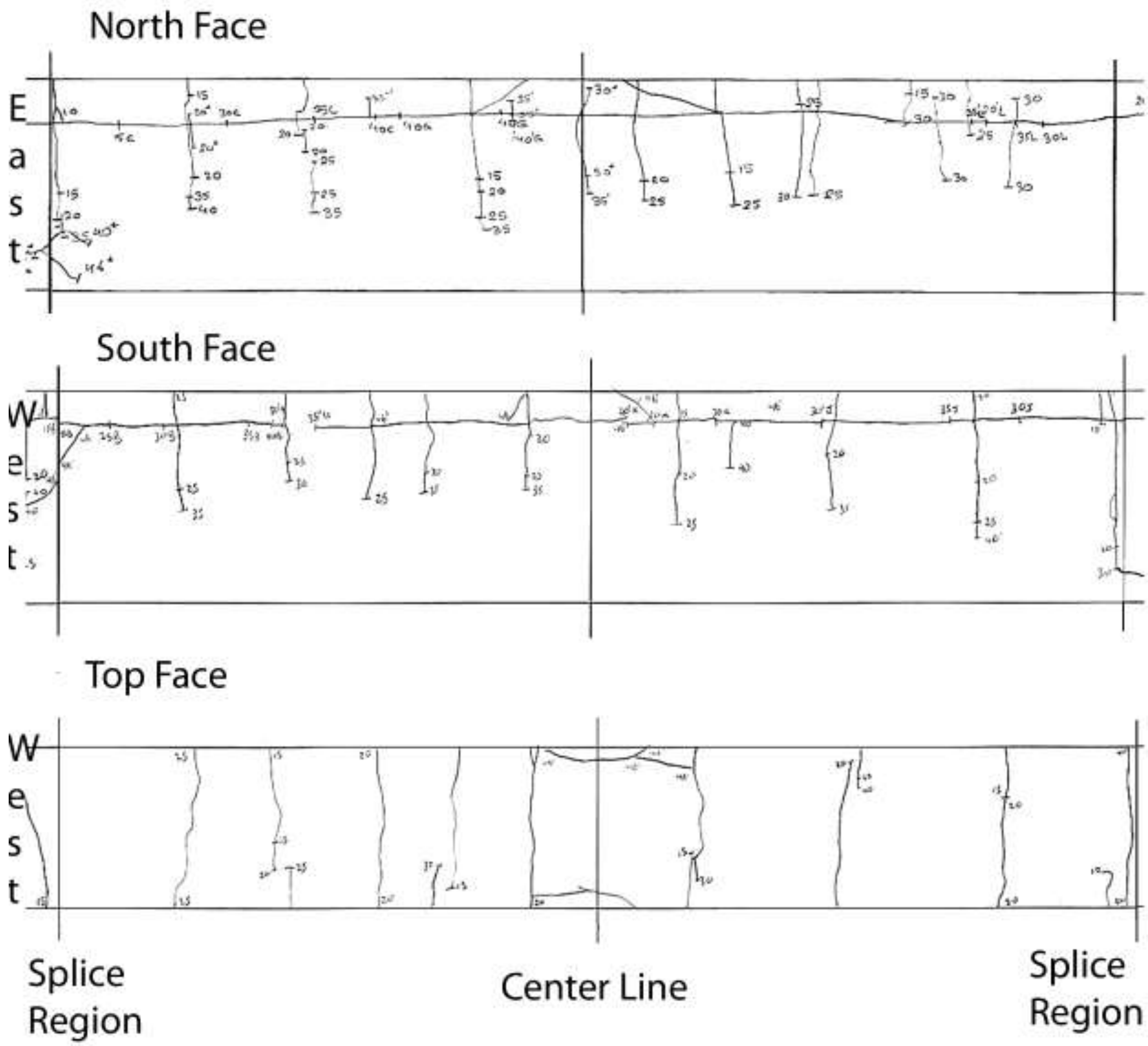


(c)

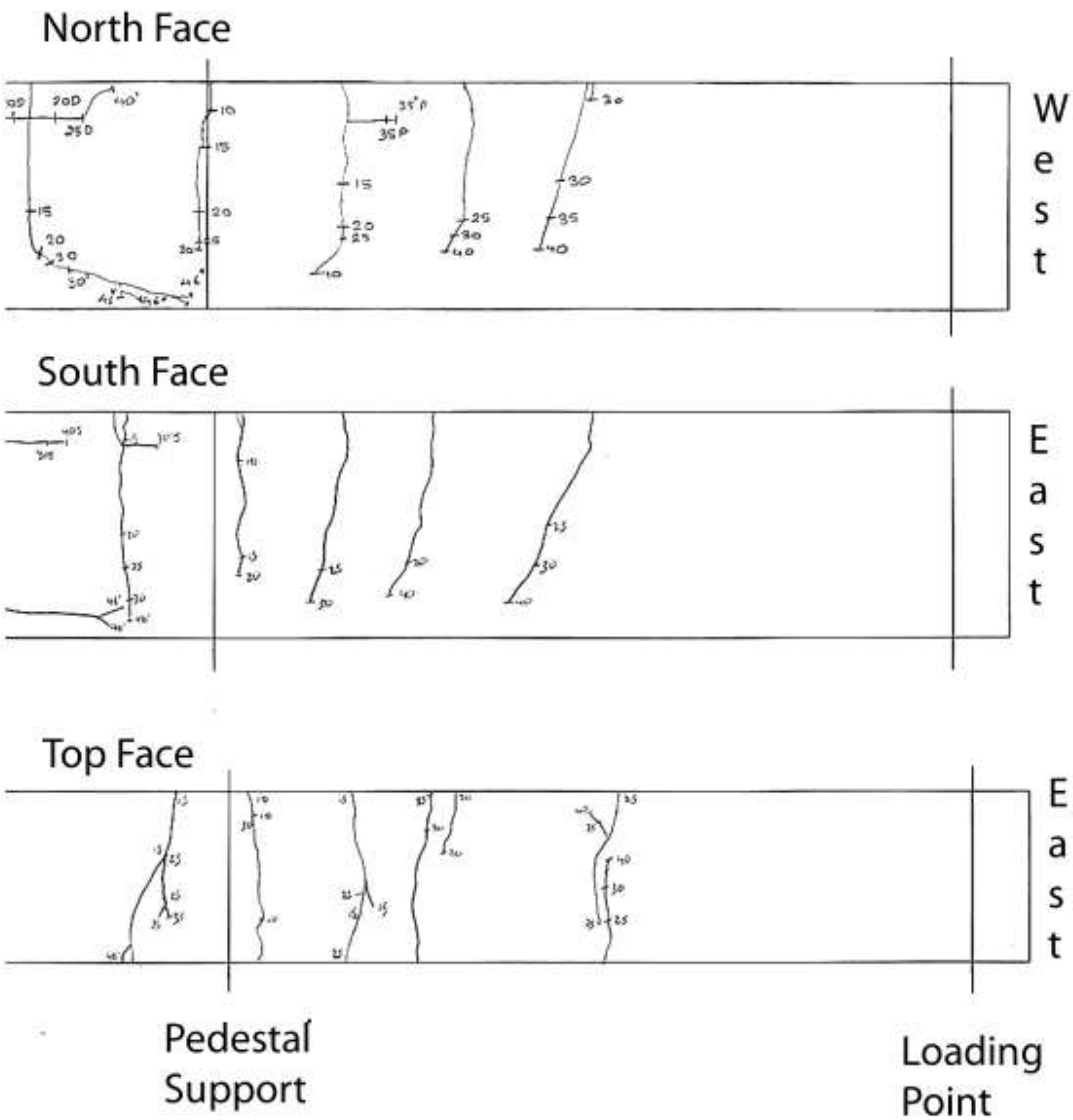
Figure C.4 – Crack map for Beam 4. Numbers indicate maximum average end load when cracks marked.


North Face
East
South Face
West
Top Face
West
Loading Point
Pedestal Support

(a)

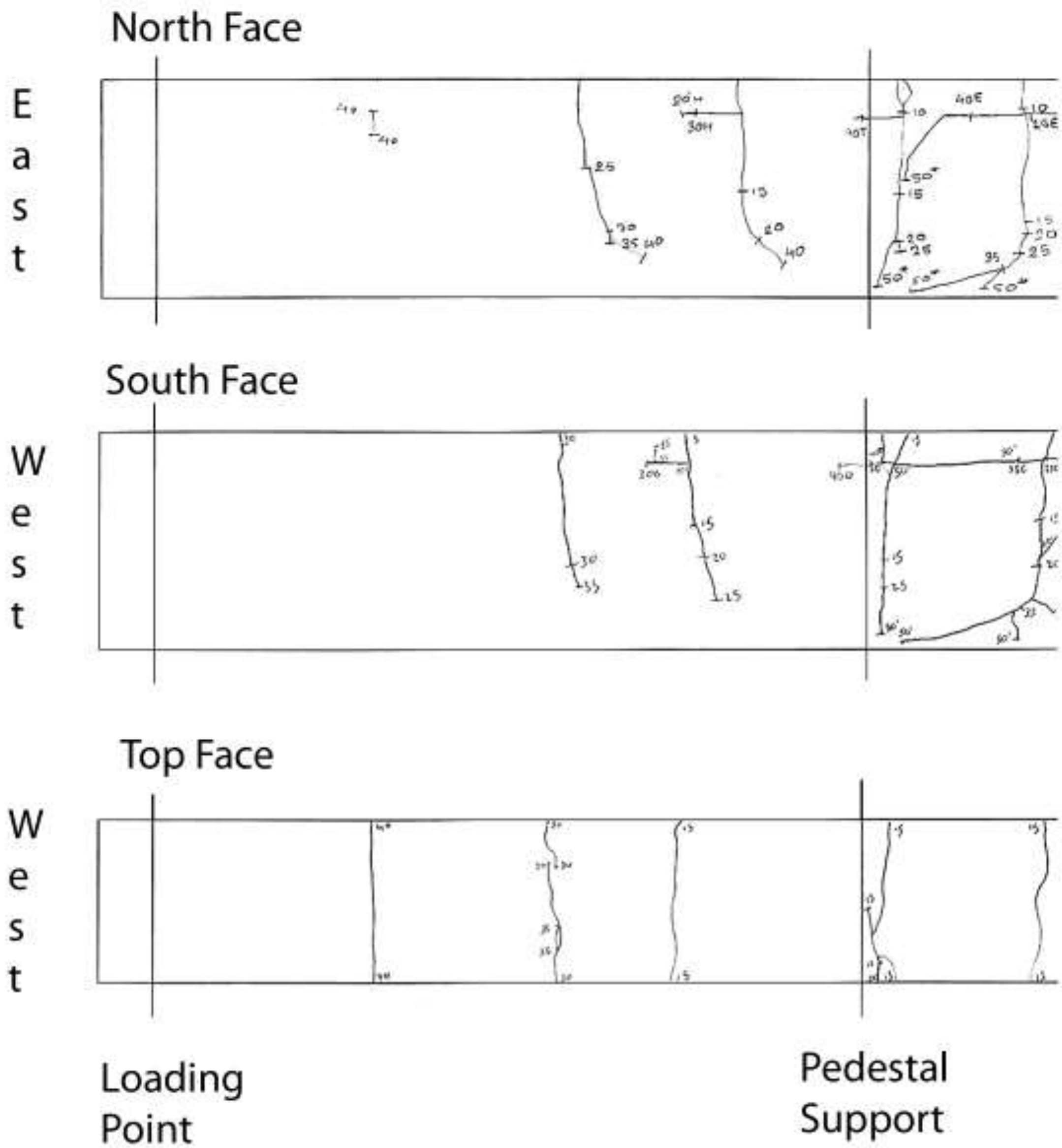

North Face
East
South Face
West
Top Face
West
Splice Region
Center Line
Splice Region

(b)

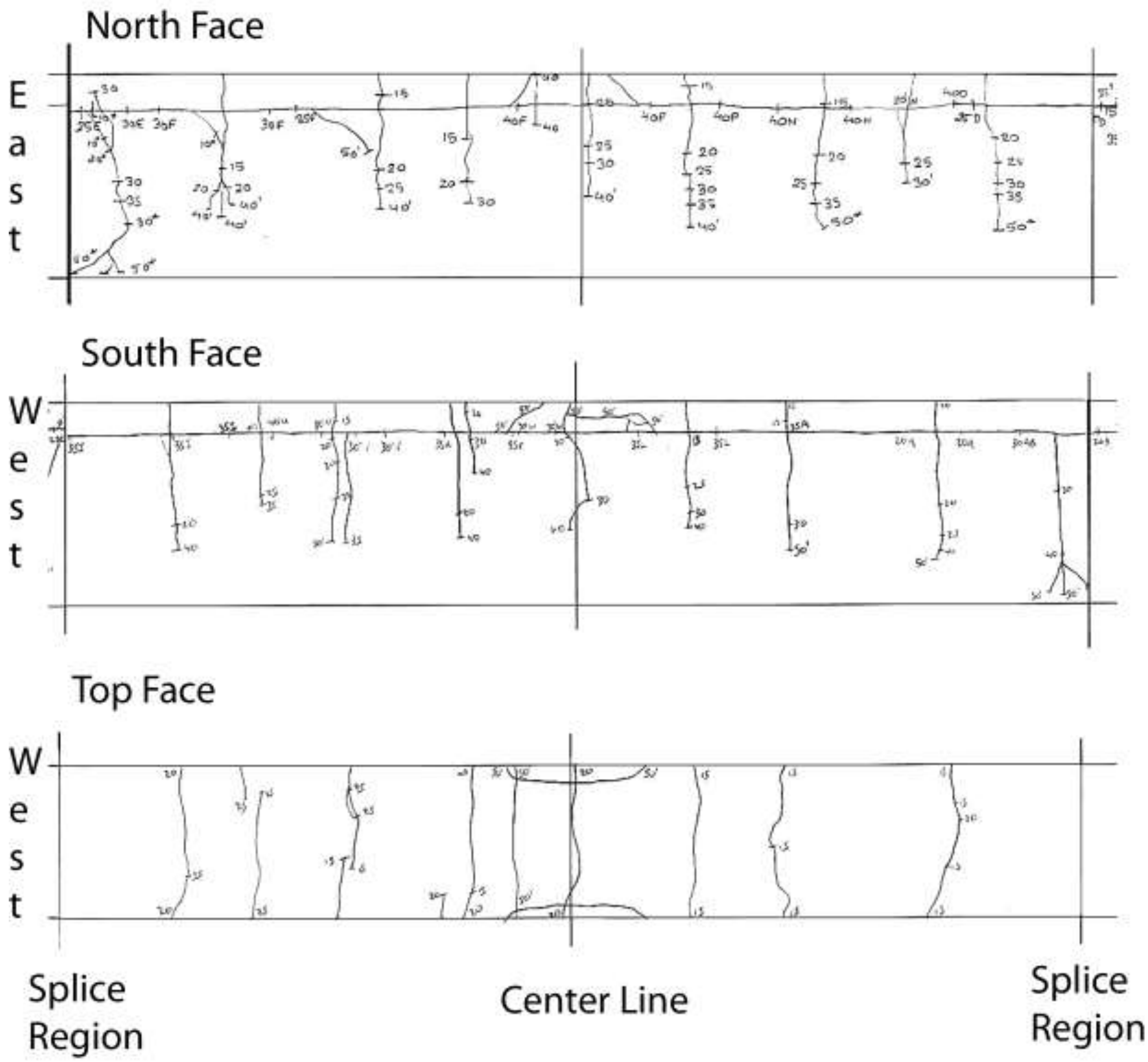


(c)

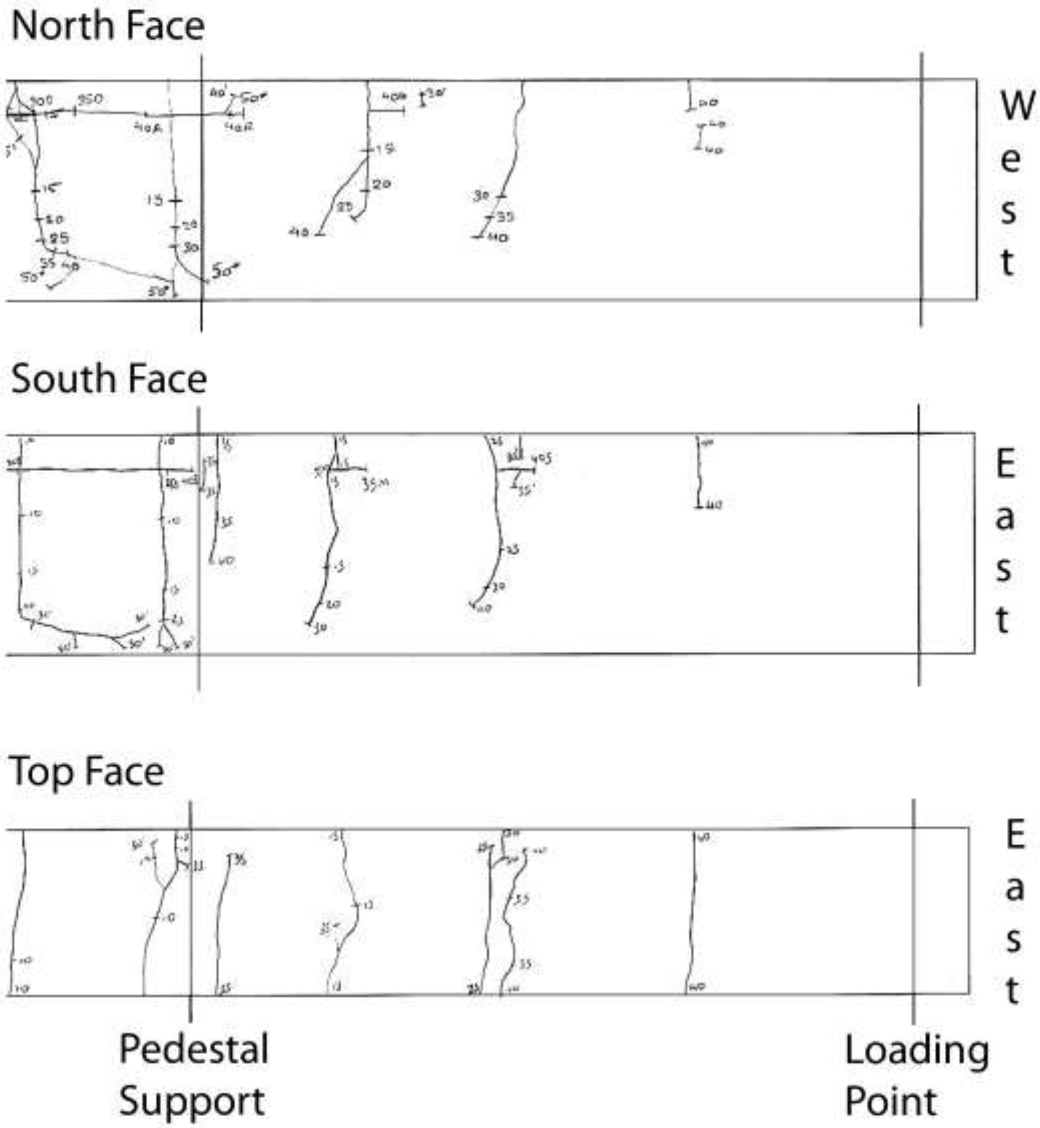
Figure C.5 – Crack map for Beam 5. Numbers indicate maximum average end load when cracks marked.


North Face
East
South Face
West
Top Face
West
Loading Point
Pedestal Support

(a)


North Face
East
South Face
West
Top Face
West
Splice Region
Center Line
Splice Region

(b)



(c)

Figure C.6 – Crack map for Beam 6. Numbers indicate maximum average end load when cracks marked.

Appendix D: Reinforcing steel mill certification and deformation measurements

NUCOR
NUCOR STEEL KANKAKEE, INC.

Mill Certification
1/3/2012

NUCOR STEEL KANKAKEE, INC.
One Nucor Way
Bourbonnais, IL 60914-4127
(815) 937-3131
Fax: (815) 939-5599

Sold To: AMBASSADOR STEEL CORP
PO BOX 2340
KOKOMO, IN 46904-2340
(765) 453-2100
Fax: (765) 455-4225

Ship To: AMBASSADOR STEEL CORP-EPOXY
FOR EPOXY COATING ONLY
KOKOMO, IN 00000
(765) 453-2100
Fax: (765) 453-7452

Customer P.O.	0000107878	Sales Order	280865.12
Product Group	Rebar	Part Number	900000367204200
Grade	ASTM A615/A615M-09b GR 60[420] AASHTO M31-07	Lot #	KN1110608801
Size	36/#11 Rebar	Heat #	KN11106088
Product	36/#11 Rebar 60' A615M Gr 420 (Gr60)	B.L. Number	K1-436165
Description	A615M GR 420 (Gr60)	Load Number	K1-5107878
Customer Spec		Customer Part #	S109

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.

Melt Date: 12/22/2011

C	Mn	P	S	Si	Cu	Ni	Cr	Mo	V	Cb	CEA706
0.38%	1.08%	0.015%	0.047%	0.24%	0.37%	0.30%	0.15%	0.082%	0.016%	0.002%	0.58%

CEA706: A706 CARBON EQUIVALENT

Roll Date: 12/26/2011

Yield 1: 66,488psi (458MPa)	Tensile 1: 94240psi (650MPa)	Elongation: 14% in 8"(% in 203.3mm)
Bend OK	Weight Variation -002.9% %	Avg Deformation Height: 0.079in

ALL MANUFACTURING PROCESSES OF THE STEEL MATERIALS IN THIS PRODUCT, INCLUDING MELTING, HAVE OCCURRED WITHIN THE UNITED STATES. ALL PRODUCTS PRODUCED ARE WELD FREE. MERCURY, IN ANY FORM, HAS NOT BEEN USED IN THE PRODUCTION OR TESTING OF THIS MATERIAL.

Curtis Glenn
Division Metallurgist

NSMG-10 January 1, 2012

Page 2 of 2

Figure D1 – Mill certification of No. 11 bar

Mill Certification Details

Customer: Ambassador Steel Corporation
Bill of Lading #: 435560-NUK
Chief Metallurgist: Curtis Glenn
Heat #: KN1110558901
Product: 16MM(#5) REBAR X 60-0 GR420(60)
Grade: 60
Comments:

Date: 11/18/2011
Tag #: KN1111128344
Mill: Nucor Kankakee
Size: 5
Division: Kansas City, MO

Chemical Properties - Wt.%

Mn	Cu	C	Ni	Si	Cr	Mo	S	P	V	Nb	Pb	Sn	Ti	N	Ca	Al	B	Ceq
1.000	.390	.360	.250	.200	.130	.063	.050	.014	.009	.002	.000	.000	.000	.000	.000	.000	.000	.560

Carbon Equivalent= 0.56

Physical Properties

Imperial = psi

Tensile: 101,855
Yield: 67,514
Elongation (in 8 inches): 14.62
Elongation (in 2 inches):

Bend Test: OK

The testing was conducted in accordance with the requirements of this specification. All melting and manufacturing processes were performed in the United States of America.

Curtis Glenn
Chief Metallurgist

Figure D2 – Mill certification of No. 5 bar

Mill Certification Details

Customer: Ambassador Steel Corporation

Bill of Lading #: 421874-NUK

Chief Metallurgist: Curtis Glenn

Heat #: KN1110073801

Product: Rebar ASTM A615/A615M-09b GR 60[420] AASHTO M31-07 10/#3 Rebar

Grade: A61560

Comments:

Date: 2/16/2011

Tag #: KN1111018783

Mill: Nucor Kankakee

Size: 10/#3 Rebar

Division: Kansas City, MO

Chemical Properties - Wt.%

Mn	C	Cu	Si	Ni	Cr	Mo	S	P	Nb	V	Sn	Ti	Pb	N	Ca	Al	B	Ceq
1.040	.360	.320	.210	.180	.110	.067	.047	.017	.002	.002	.000	.000	.000	.000	.000	.000	.000	.560

Carbon Equivalent= 0.56

Physical Properties

Imperial = psi

Tensile: 104,944

Yield: 68,917

Elongation (in 8 inches): 16.00

Elongation (in 2 inches):

Bend Test: OK

The testing was conducted in accordance with the requirements of this specification. All melting and manufacturing processes were performed in the United States of America.

Curtis Glenn
Chief Metallurgist

Figure D3 – Mill certification of No. 3 bar

Appendix E: Concrete Mixture Proportions

Table E.1 – Aggregate gradations

Sieve Size	Percent Retained on Each Sieve			
Sample	Granite 1 ½ in.	Granite ¾ in.	Pea Gravel	Sand
Specific Gravity	2.71	2.71	2.60	2.62
Absorption, %	0.65	0.98	0. 93	0.86
37.5-mm (1½-in.)	0%	0%	0%	0%
25-mm (1-in.)	19.0%	0%	0%	0%
19-mm (¾-in.)	28.7%	4.5%	0%	0%
12.5-mm (½-in.)	34.5%	38.7%	0%	0%
9.5-mm (⅜-in.)	14.2%	30.6%	0%	0%
4.75-mm (No. 4)	3.1%	24.5%	11.0%	1.7%
2.36-mm (No. 8)	0%	0.9%	44.8%	7.8%
1.18-mm (No. 16)	0%	0%	31.2%	16.9%
0.60-mm (No. 30)	0%	0%	6.0%	27.7%
0.30-mm (No. 50)	0%	0%	2.6%	36.4%
0.15-mm (No. 100)	0%	0%	1.1%	8.5%
0.075-mm (No. 200)	0%	0%	03%	0.9%
Pan	0.5%	0.7%	2.9%	0.1%

Table E.2 – Mixture proportions and concrete properties

	Trial Batch	Beam #1, 2, 3				Beam #1, 2, 3			
		Below cold joint		Above cold joint		Below cold joint		Above cold joint	
		Design	Actual	Design	Actual	Design	Actual	Design	Actual
w/c	0.42	0.42	0.43	0.42	0.43	0.42	0.42	0.42	0.41
Cement content, lb/yd^3	588	588	592	588	580	588	587	588	590
Water content, lb/yd^3	246	246	255	246	251	246	245	246	244
Granite 1 ½ in., lb/yd^3	687	687	687	687	675	687	688	687	690
Granite ¾ in., lb/yd^3	1050	1050	1050	1050	1060	1050	1055	1050	1055
Pea Gravel, lb/yd^3	836	836	838	836	837	836	844	836	840
Sand, lb/yd^3	720	720	718	720	724	720	739	720	730
Water Reducer, (ADVA 140M), oz/yd^3	24	40*	55	50	50	60		60	
Batch Size, yd^3	0.04	9		1		10		2	
Slump, in.	3.5	2.25		2.25		3		2.75	
Unit Weight, lb/ft^3	152	153		152		154		150	
Temperature, °F	81	82		76.4		82		86	
Compressive Strength, psi									
3 –Day strengths	--	--		3640+		--		4520+	
4-Day Strengths	3915	4010		--		4310+		--	
6-Day Strengths	4310	4670		4330++		4680		5490++	
7-Day Strengths	4490	5330++		--		5230++		--	
Modulus of Rupture									
ASTM C78	--	435++		--		370++		470++	
ASTM C496 (monolithic)	--	570++		--		600++		700++	
ASTM C96 (cold joint)	--	140++		--		274++		--	

* An extra 15 oz/yd^3 of water reducer was added on the job site.
\+ Tests were performed on the day when the forms were removed.
++ Tests were performed on the day of beam-splice specimen testing.

Appendix F: Data recording forms

Table F.1 – Dimensions of formwork

Specimen ID:		Date:	
Measured by:		Checked by:	

	Width	Height	Length
Design	18 in.	24 in.	
Tolerance	± ½ in.	± ½ in.	± 1 in.
Measurement 1			
Measurement 2			
Measurement 3			
Measurement 4			
Measurement 5			
Measurement 6			
Measurement 7			
Measurement 8			
Measurement 9			

Table F.2 – Dimensions of reinforcing steel within in the test region

Specimen ID:		Date:	
Measured by:		Checked by:	

		Side cover	Bottom to top of all-thread rod	Splice length
Design		3 in.		
Tolerance		± ½ in.	± ½ in.	± ½ in.
Splice 1	Measurement 1			
	Measurement 2			
	Measurement 3			
Splice 2	Measurement 1			
	Measurement 2			
	Measurement 3			

Measured bar diameter:

Splice 1:______________

Splice 2:______________

Table F.3 – Plastic concrete testing and concrete compressive strength

Specimen ID:		Date:	
Measured by:		Checked by:	

Plastic concrete testing

Slump, in.	Unit weight, lb/ft^3	Concrete temperature, °F

Concrete compressive strength

Cylinder ID	Cast date	Test date	Age, days	Dia., in.	Area, in.^{2}	Load, kips	Strength, psi	Notes

Table F.4 – Test setup – span spacing

Specimen ID:		Date:	
Measured by:		Checked by:	

	Measurement 1, in.	Measurement 2, in.	Measurement 3, in.	Average, in.
Pin centerline to roller centerline				
East end to east support				
East end to east splice end				
East end to beam centerline				
East end to west splice end				
East end to west support				
East end to west end				

Table F.5 – Dial gage readings

Specimen ID:		Date:	
Measured by:			

	Load, kips	Dial gage 1, in.	Dial gage 2, in.	Dial gage 3, in.
Reading 1				
Reading 2				
Reading 3				
Reading 4				
Reading 5				
Reading 6				
Reading 7				
Reading 8				
Reading 9				
Reading 10				
Reading 11				
Reading 12				
Reading 13				
Reading 14				
Reading 15				
Reading 16				
Reading 17				
Reading 18				
Reading 19				
Reading 20				

Appendix G: Load cell and displacement transducer calibration

The load cells and displacement transducers were calibrated before and after testing each three beams. A least-squares linear regression analysis was performed on force and displacement versus sensor output to determine the calibration constant. The calibration constant is presented in Tables G.1 and G.2 and the force and displacement versus sensor output are plotted in Figures G.2 to G.21.

Table G.1 – Load cells and displacement transducers calibration before and after testing Beams 1, 2, and 3

	Load Cell A	Load Cell B	Load Cell C	Load Cell D	LVDT	String Pot 1	String Pot 2	note
Calibration #1, slope	17930	17729	17705	17651	-0.2011	-1.986	-1.966	before testing Beams #1,2,and 3
Calibration #2, slope	17796	17758	17731	17801	-0.2011	-1.985	-1.973	After testing Beams #1,2,and 3
Deviation, %	0.74	0.16	0.15	0.84	0	0.02	0.35	

Table G.2 – Load cells and displacement transducers calibration before and after testing Beams 4, 5, and 6

	Load Cell A	Load Cell B	Load Cell C	Load Cell D	LVDT	String Pot 1	String Pot 2	note
Calibration #1, slope	17930	17729	17705	17651	-0.2011	-1.986	-1.966	before testing Beams #4,5,and 6
Calibration #3, slope	17938	17745	17809	17703	-0.2017	-1.988	-1.970	After testing Beams #4,5,and 6
Deviation, %	0.04	0.09	0.59	0.29	0.30	0.1	0.2	

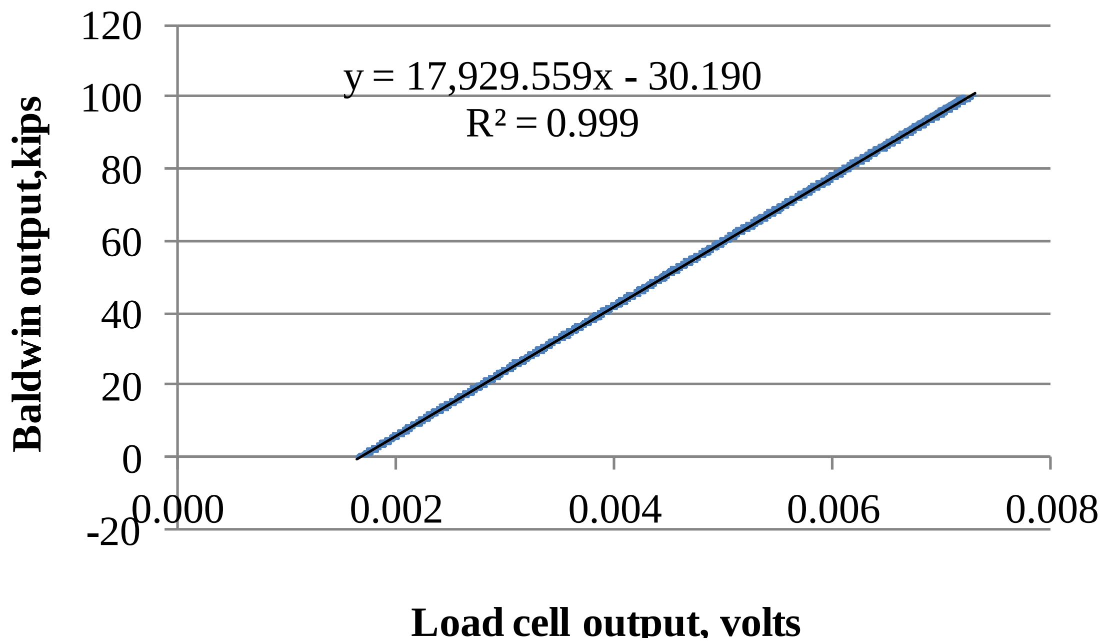


Figure G.1 – Load cell 2-0 calibration #1

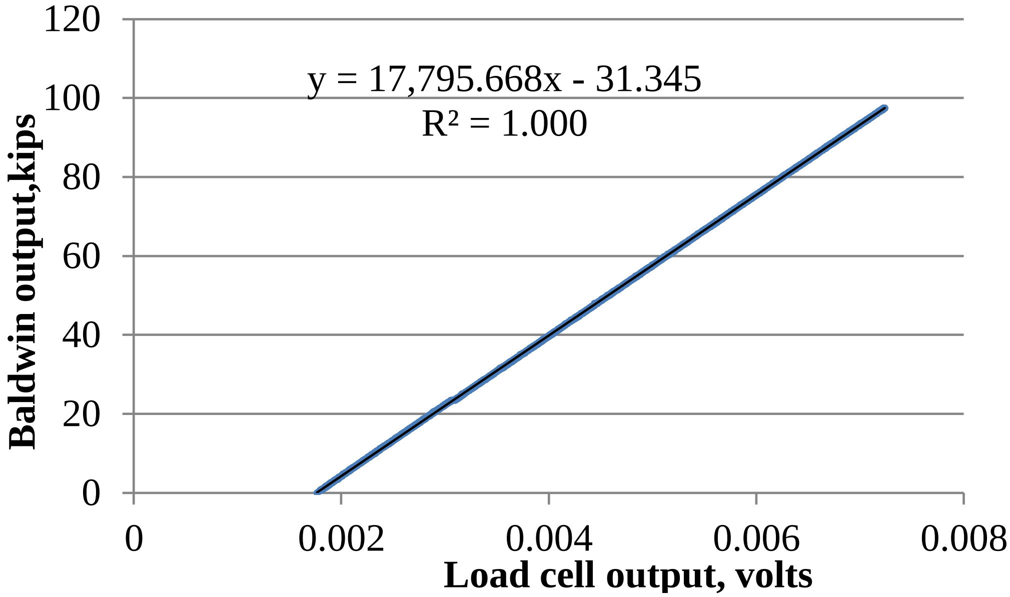


Figure G.2 – Load cell 2-0 calibration #2

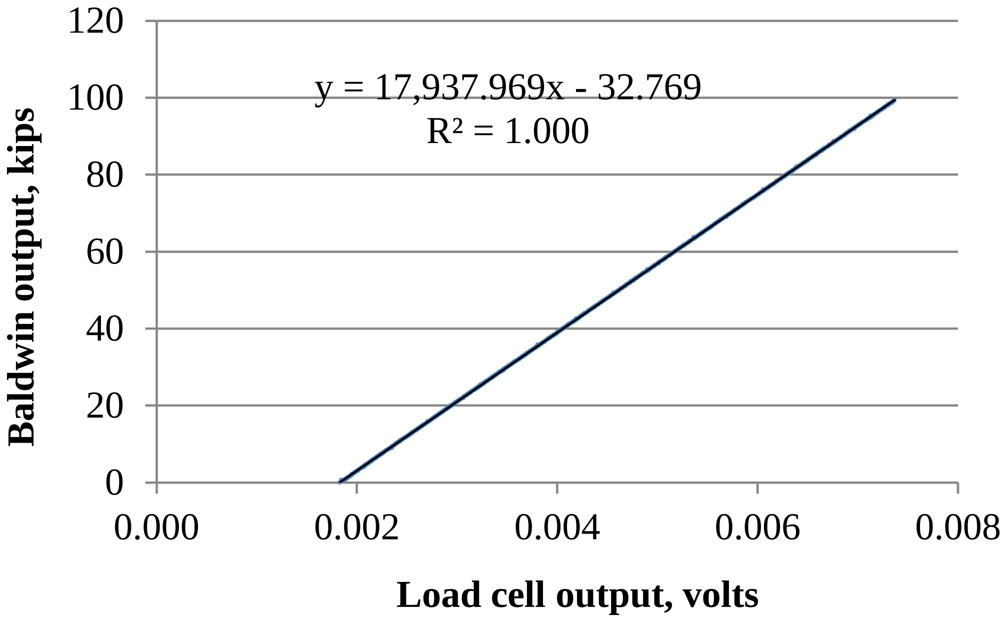


Figure G.3 – Load cell 2-0 calibration #3

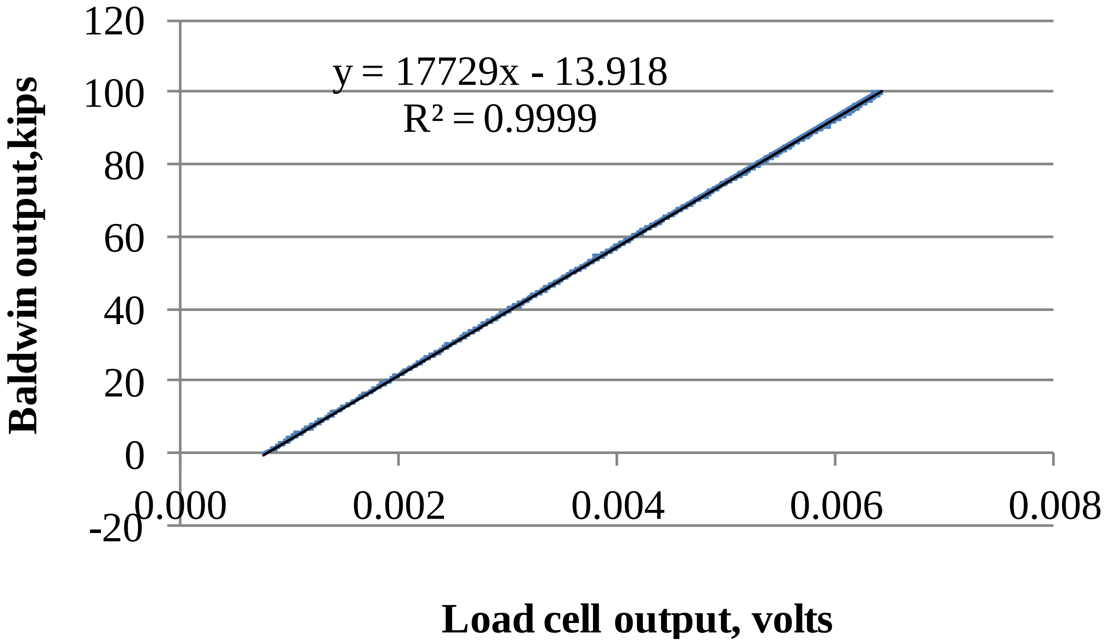


Figure G.4 – Load cell 2-1 calibration #1

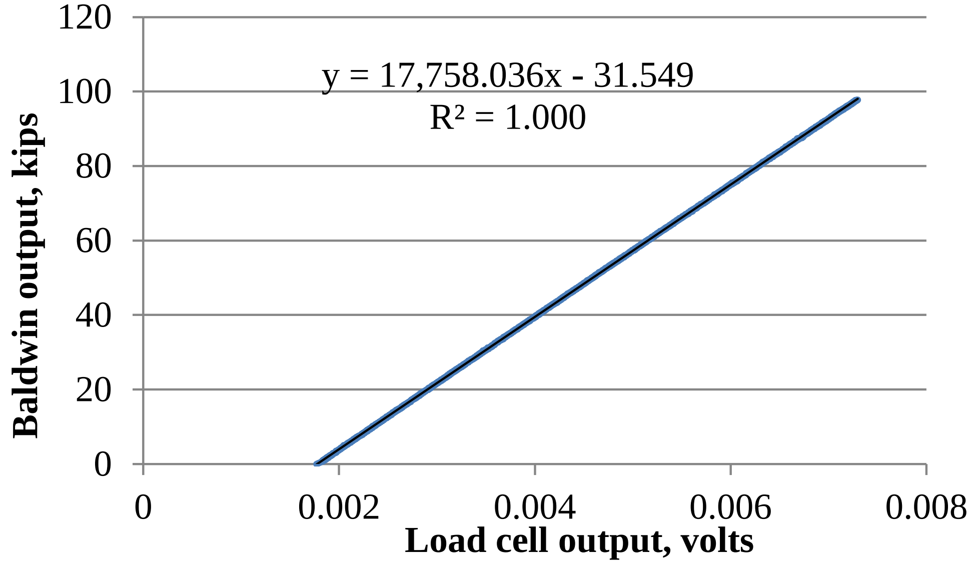


Figure G.5 – Load cell 2-1 calibration #2

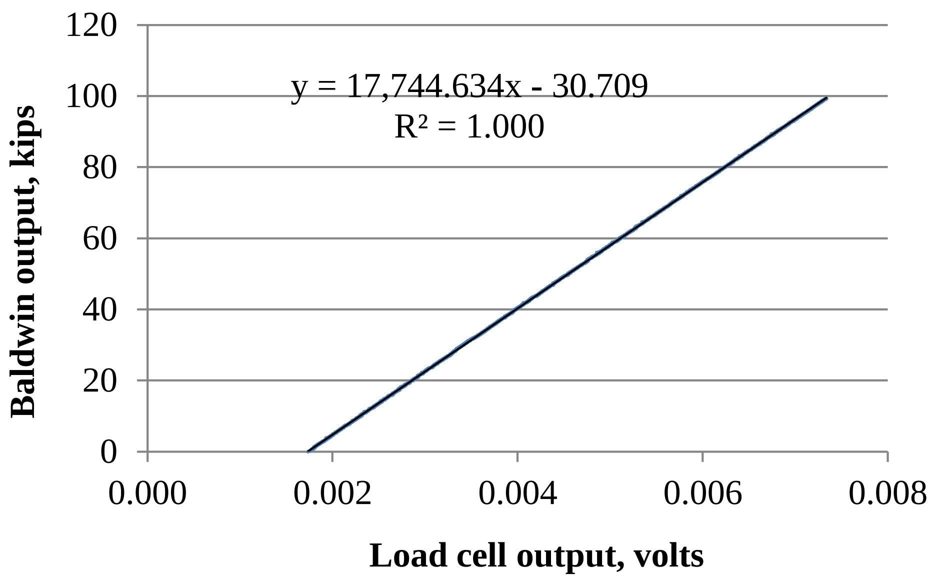


Figure G.6 – Load cell 2-1 calibration #3

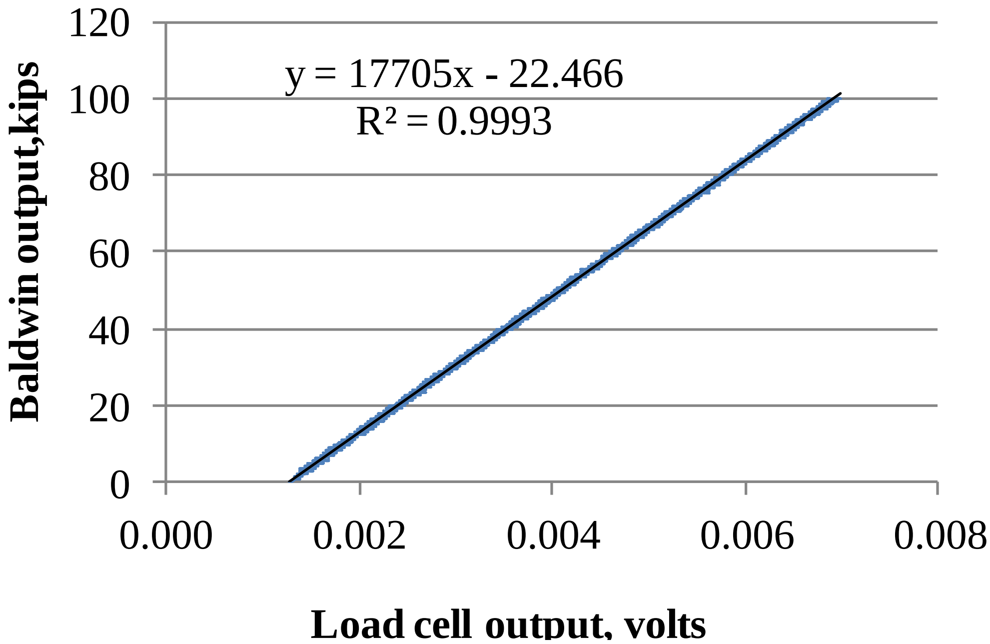


Figure G.7 – Load cell 2-2 calibration #1

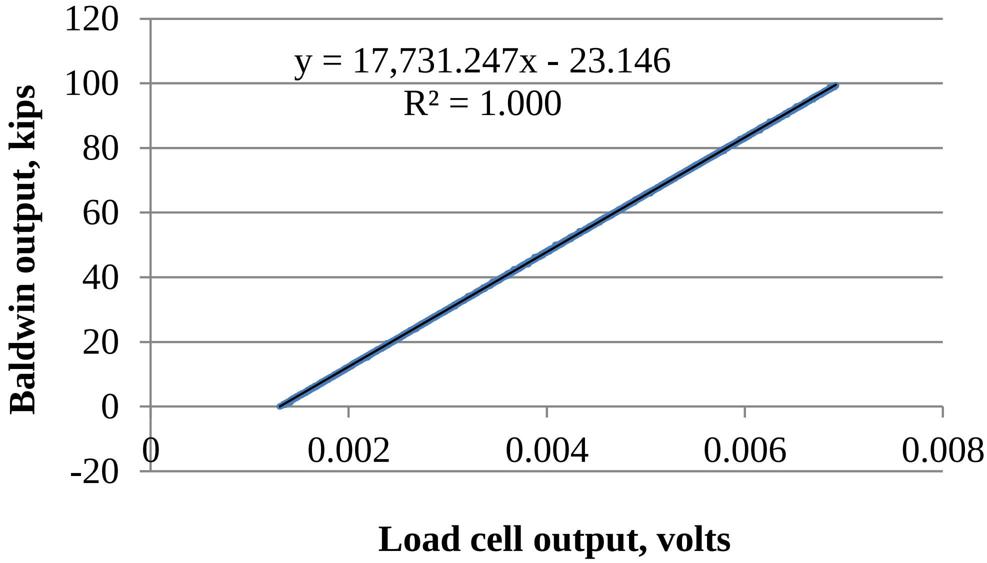


Figure G.8 – Load cell 2-2 calibration #2

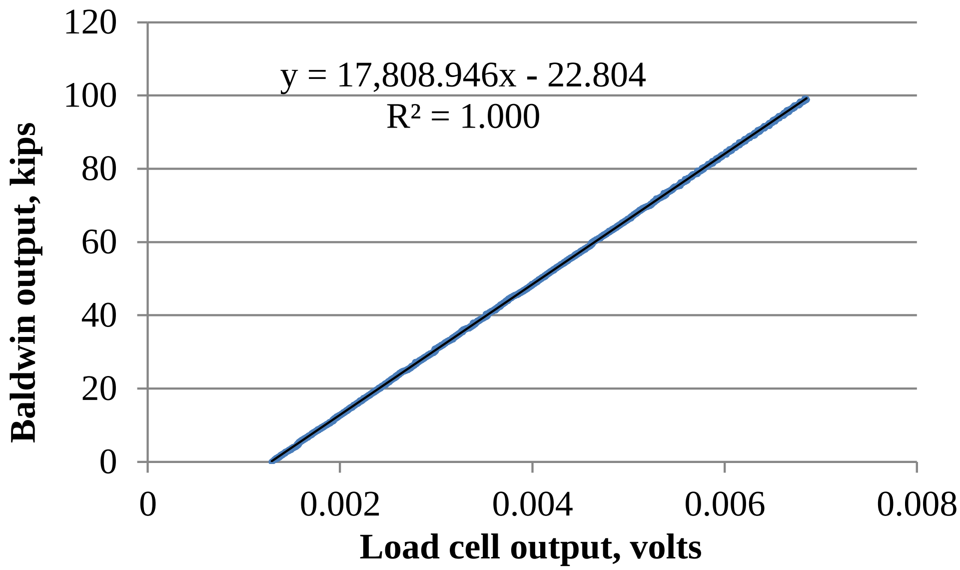


Figure G.9 – Load cell 2-2 calibration #3

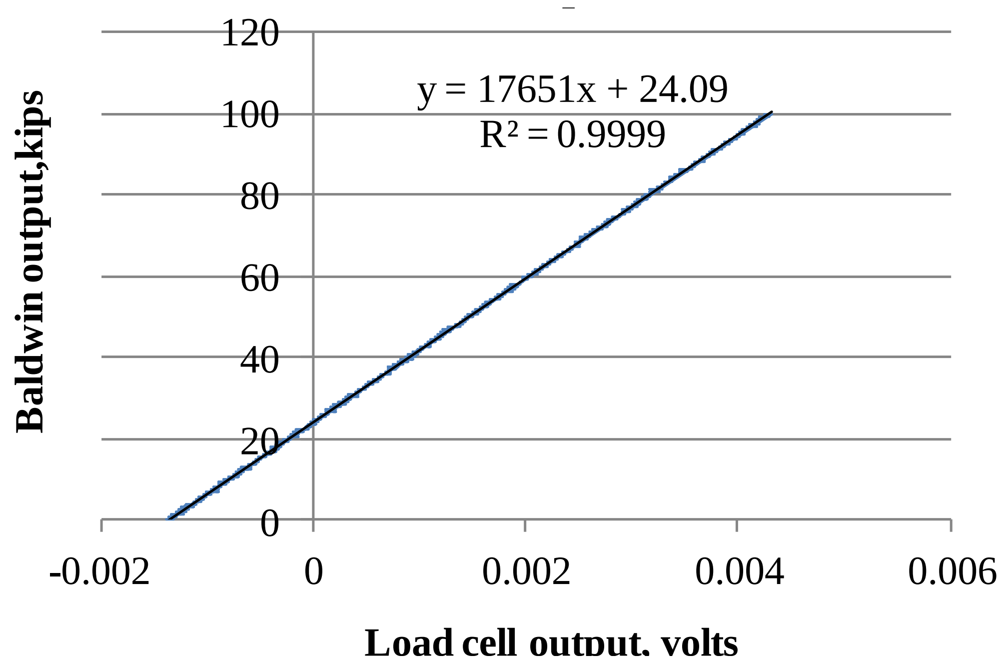


Figure G.10 – Load cell 2-3 calibration #1

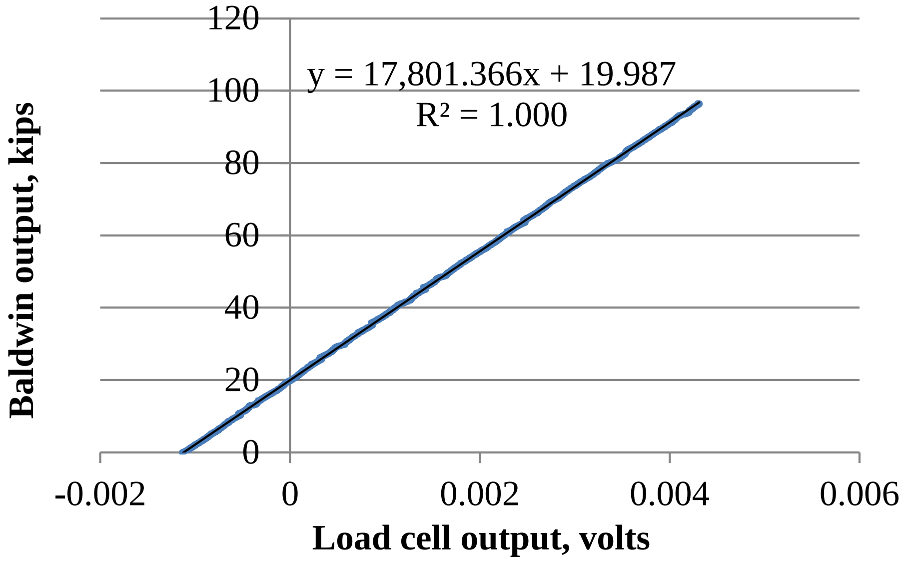


Figure G.11 – Load cell 2-3 calibration #2

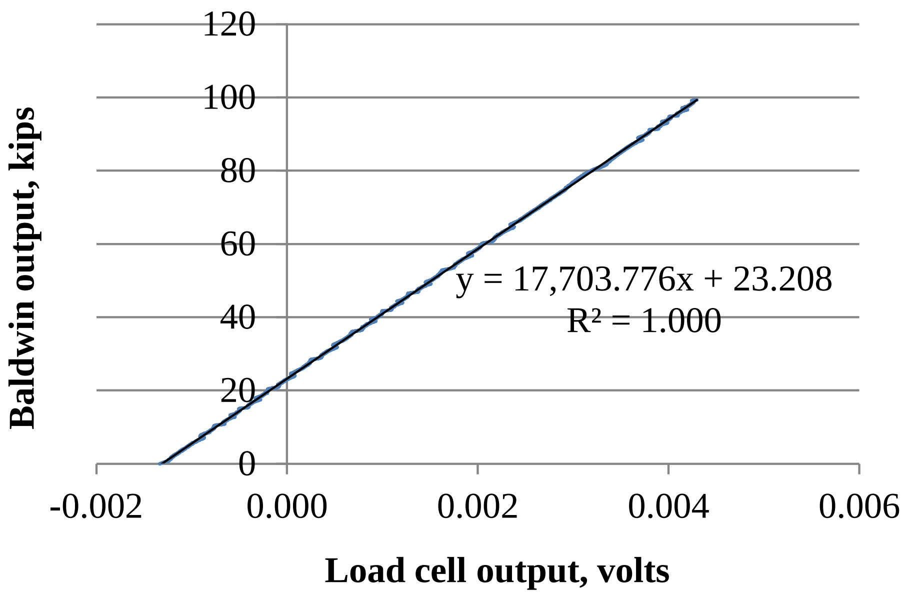


Figure G.12 – Load cell 2-3 calibration #3

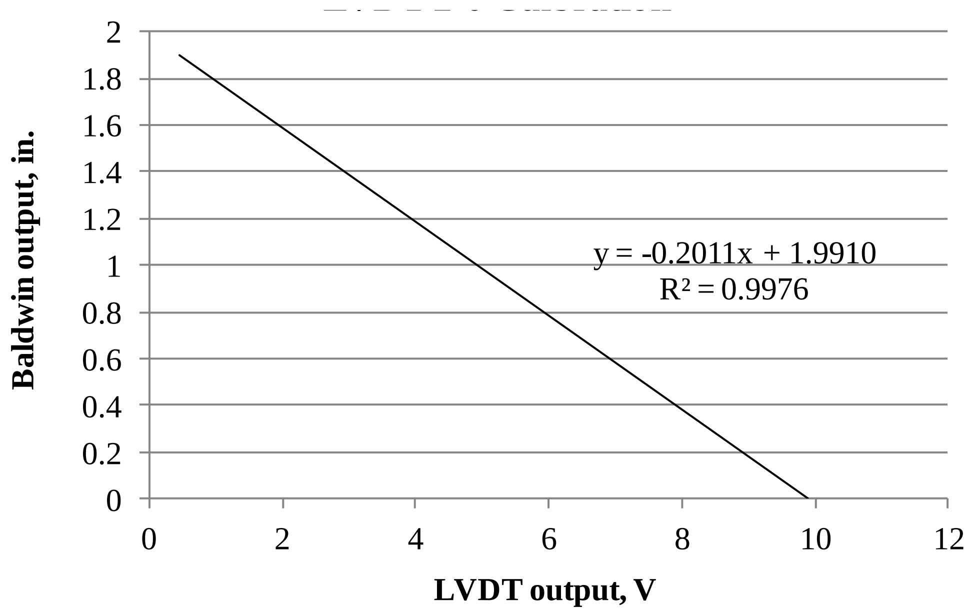


Figure G.13 – LVDT calibration #1

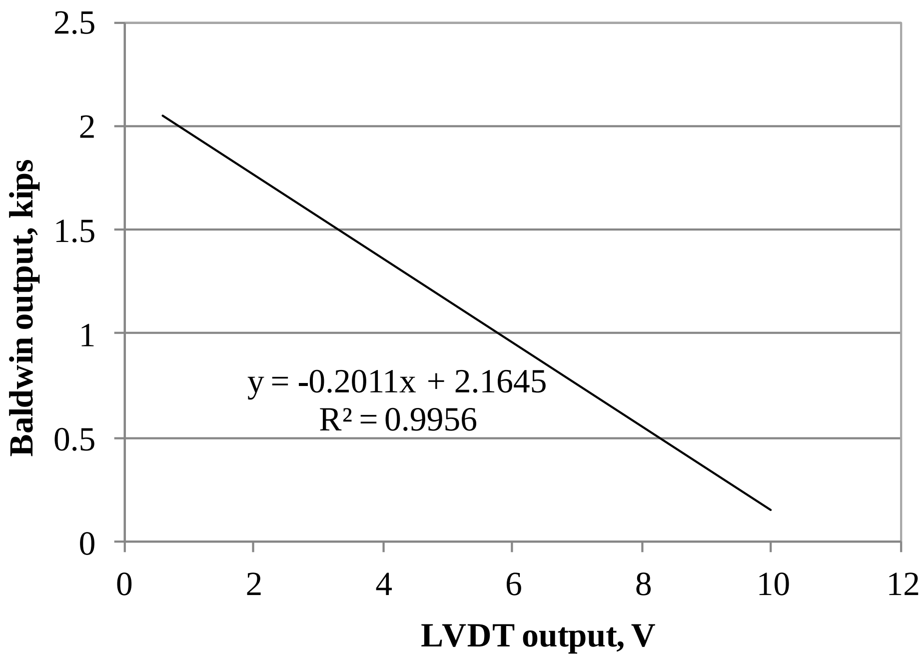


Figure G.14 – LVDT calibration #2

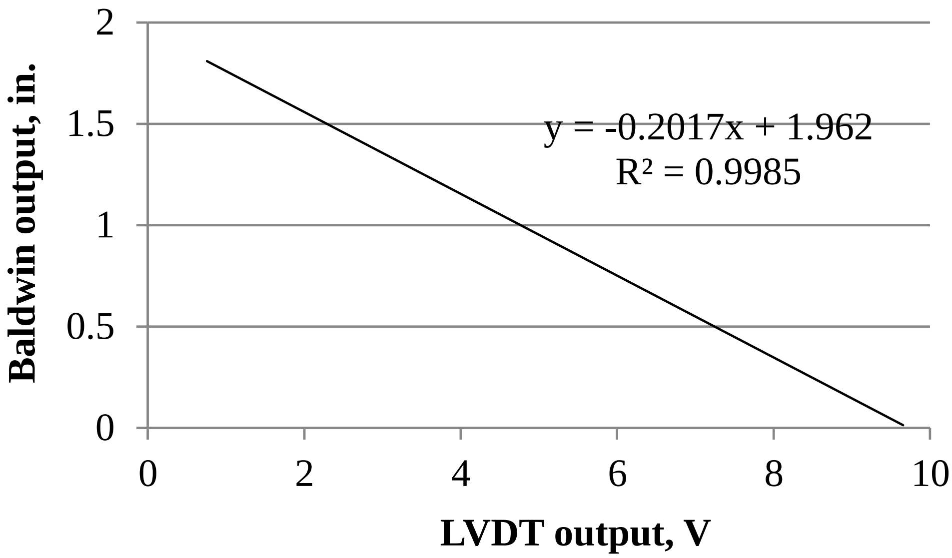


Figure G.15 – LVDT calibration #3

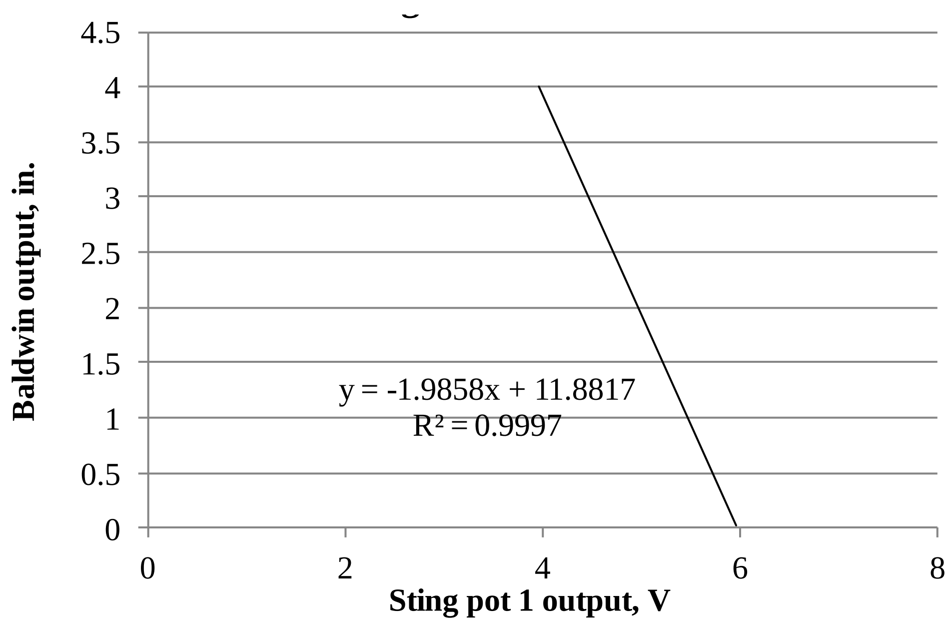


Figure G.16 – String pot 1 calibration #1

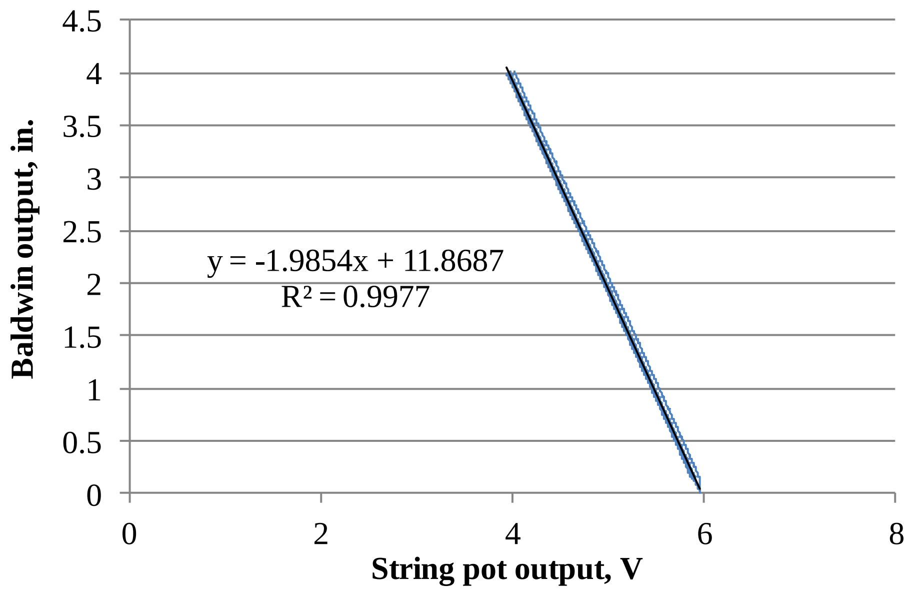


Figure G.17 – String pot 1 calibration #2

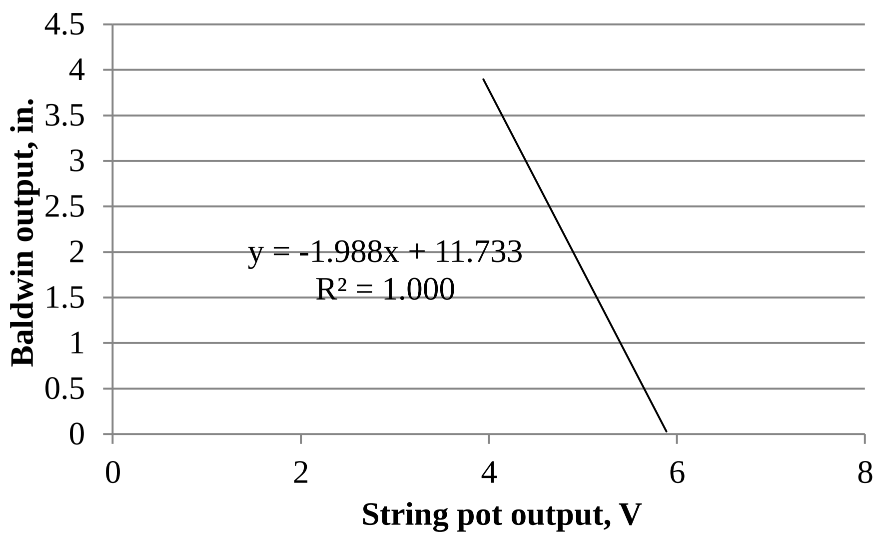


Figure G.18 – String pot 1 calibration #3

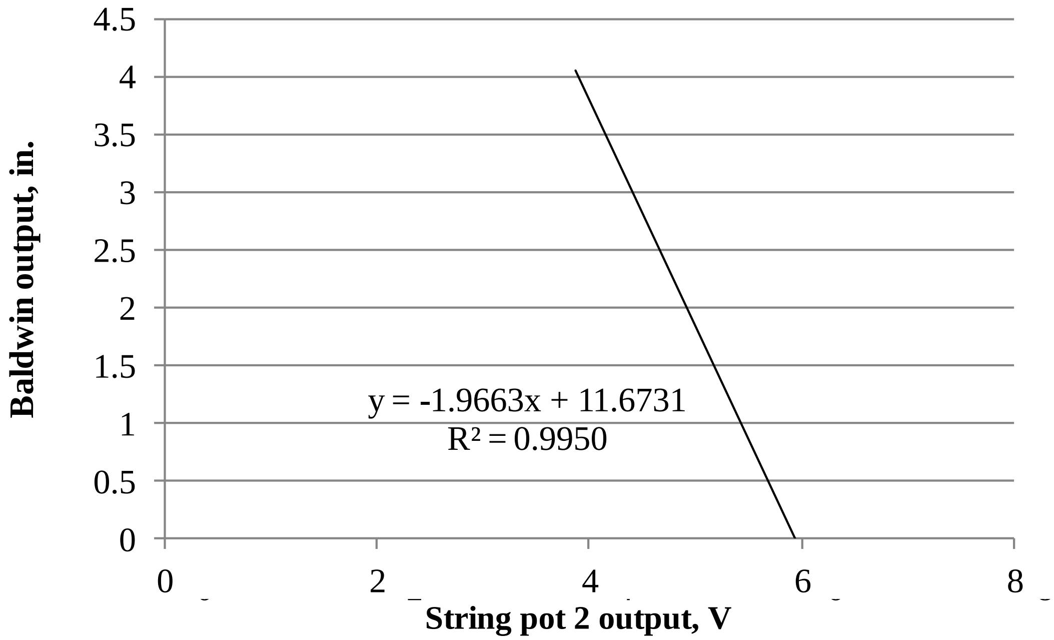


Figure G.19 – String pot 2 calibration #1

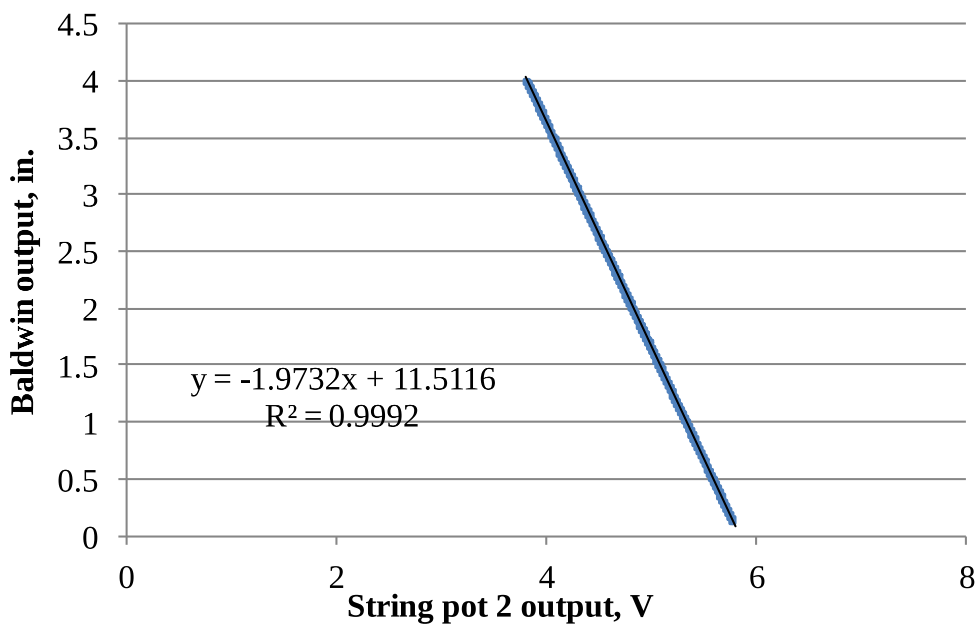


Figure G.20 – String pot 2 calibration #2

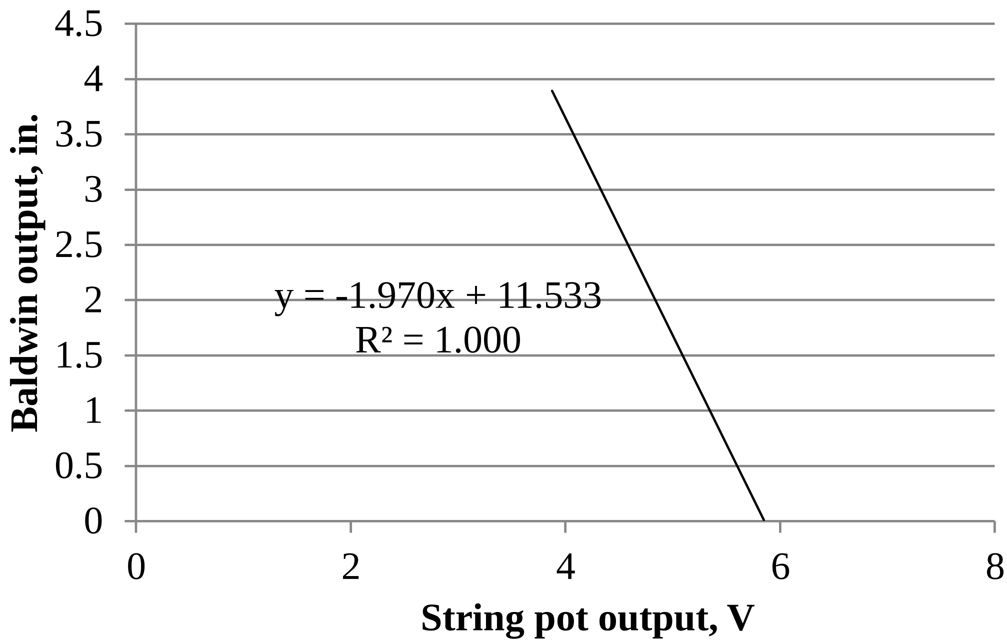


Figure G.21 – String pot 2 calibration #3

Appendix H: Training forms, Trip tickets, concrete properties, specimen dimensions, and crack recording during beam tests

Training Form (#1, 2, and 3 beams)

This form is prepared to indicate those who will work on casting the above mentioned beams have received proper training and they are qualified for what they will work on.

Last Name	First Name	ACI qualified? (if yes, certification #)	signature	date
Al-Khafaji	Ali	No	[illegible]	5/22/12
Eisenbarth	Brad			
Fink	Dalen	No	Dalen Fink	5/22/12
Guernsey	Edward	No	Ed Guernsey	5/22/12
~~Hawk~~	~~Kaleb~~			
Lyon	Adam	Yes	Adam Lyon	5/22/12
O'Reilly	Matt	01090586	Matt OReilly	5/24/12
Peckover	Jeff	Yes 01146212	[illegible]	5/22/12
Pendergrass	Ben	Yes	[illegible]	5/22/12
Routh	Jon	NO	Jon [illegible]	5/22/12
~~Schneider~~	~~Aaron~~			
Searle	Nate	Yes	Nate Searle	5/24/12
Shrestha	Pankaj	NO	[illegible]	5/24/12.
Sperry	Jayne	Yes	Jayne Sperry	5/22/12
~~Williams~~	~~Eric~~			
Yuan	Jiqiu	Yes 01082687	[illegible]	5/24/12
Zhen	Chen	Yes.	[illegible]	05/22/2012
Jeronimus	Robbie	nO	Robbie Jeronimus	5/22/12
~~Jocilyn (Joci)~~				
Hansen	Jocilyn (Joci)	no	J Hansen	5/22/12
Bohaty	Brent	No	[illegible]	05/24/12

University of Kansas
CEAE Department
2150 Learned Hall
1530 W. 15th St.
Lawrence, Kansas 66045-7609
Phone: 785.864.3885 Fax: 785.864.5631

CONCRETE MIX DESIGN

Contractor:	KU
Project:	Bond Test- Beam #1, 2, and 3
Source of Concrete:	Ready Mix Concrete
Date:	5/24/2012
Placement Type:	Conventional

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 100%	588 lb	3.20	2.94
Water	246 lb	1.00	3.94
MCM -1.5 in. / Granite 1.5" / 20.86%	687 lb	2.71	4.06
MCM -0.75 in. / Granite 0.75" / 31.89%	1050 lb	2.71	6.21
Pea Gravel / Pea Gravel / 25.39%	836 lb	2.60	5.15
VPsand / VP Sand / 21.86%	720 lb	2.62	4.40
Total Air, percent	0		0.27
Air Entraining Agent / Air R Us	0 fl oz (US)	1.01	0.00
Superplasticizer / Admixtures R Us	40 fl oz (US)	1.20	0.02
			27.00

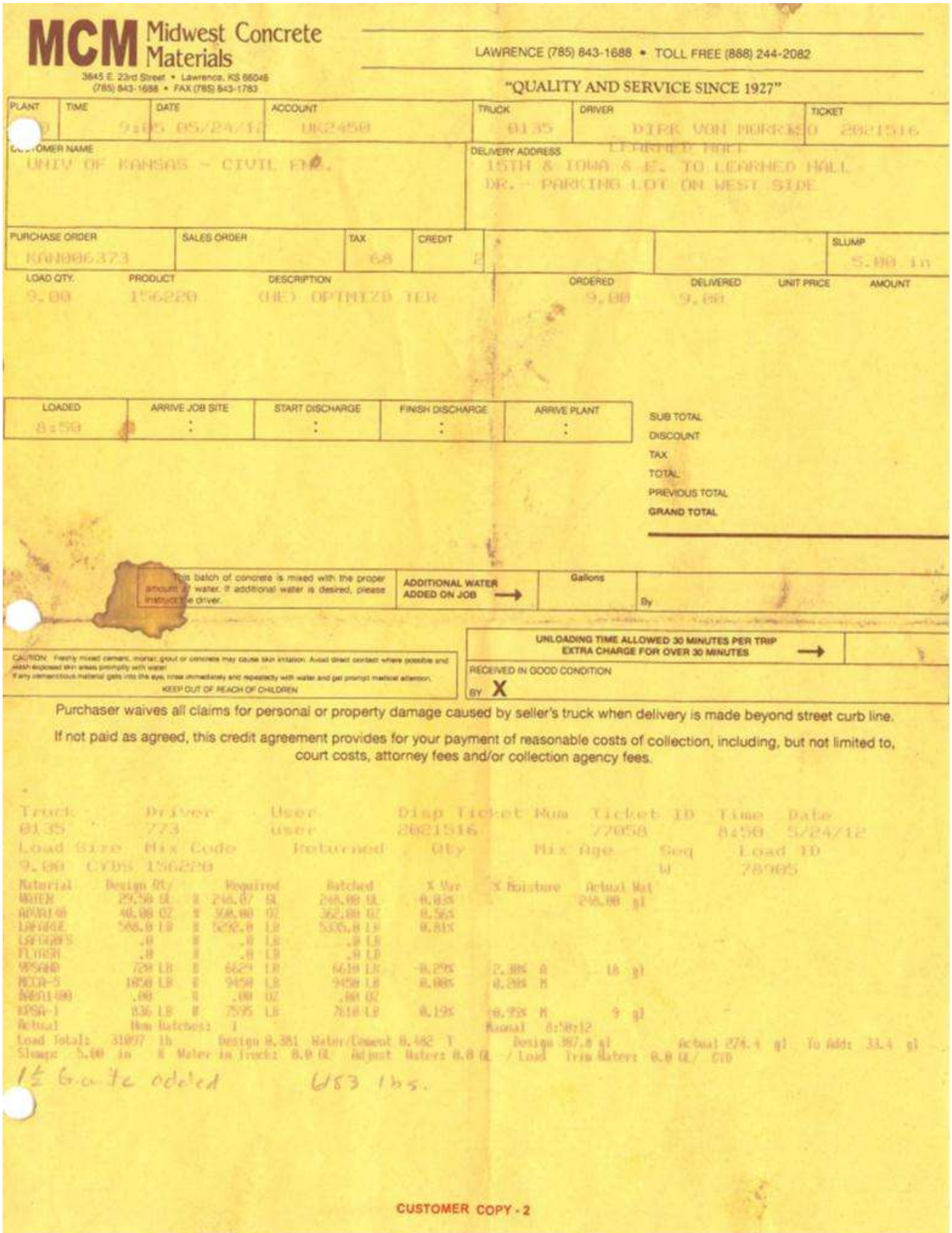
1 The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.

Total Water Content (including water in admixtures), lb		247		
Water / Cementitious Material Ratio:		0.42		
Concrete Unit Weight, pcf		152.9		
Target Slump, in.		5 in.		
Paste Content, percent		25.55%		
Workability Factor (WF)	Target:	35.0	Actual:	32.1
Coarseness Factor (CF)	Target:	57.9	Actual:	63.0
Prepared On:		5/16/12 2:20 PM		

Prepared By:

cast on 5/24/12

Jiqiu Yuan



MCM Midwest Concrete Materials
3645 E. 23rd Street • Lawrence, KS 66046
(785) 843-1688 • FAX (785) 843-1783

LAWRENCE (785) 843-1688 • TOLL FREE (888) 244-2082

"QUALITY AND SERVICE SINCE 1927"

PLANT	TIME	DATE	ACCOUNT	TRUCK	DRIVER	TICKET
	9:05	05/24/12	UK2450	0135	DIRK VON MORRISO	2021516

CUSTOMER NAME: UNIV OF KANSAS - CIVIL ENG.

DELIVERY ADDRESS: LEARNED HALL 15TH & IOWA & E. TO LEARNED HALL DR. - PARKING LOT ON WEST SIDE

PURCHASE ORDER	SALES ORDER	TAX	CREDIT	SLUMP
KAN006373		68		5.00 in

LOAD QTY.	PRODUCT	DESCRIPTION	ORDERED	DELIVERED	UNIT PRICE	AMOUNT
9.00	156220	(HE) OPTMIZD TER	9.00	9.00		

LOADED	ARRIVE JOB SITE	START DISCHARGE	FINISH DISCHARGE	ARRIVE PLANT
8:50	:	:	:	:

SUB TOTAL
DISCOUNT
TAX
TOTAL
PREVIOUS TOTAL
GRAND TOTAL

This batch of concrete is mixed with the proper amount of water. If additional water is desired, please instruct the driver.

ADDITIONAL WATER ADDED ON JOB → Gallons By

UNLOADING TIME ALLOWED 30 MINUTES PER TRIP EXTRA CHARGE FOR OVER 30 MINUTES →

CAUTION: Freshly mixed cement, mortar, grout or concrete may cause skin irritation. Avoid direct contact where possible and wash exposed skin areas promptly with water. If any cementitious material gets into the eye, rinse immediately and repeatedly with water and get prompt medical attention. KEEP OUT OF REACH OF CHILDREN

RECEIVED IN GOOD CONDITION
BY X

Purchaser waives all claims for personal or property damage caused by seller's truck when delivery is made beyond street curb line.

If not paid as agreed, this credit agreement provides for your payment of reasonable costs of collection, including, but not limited to, court costs, attorney fees and/or collection agency fees.

Truck	Driver	User	Disp Ticket Num	Ticket ID	Time	Date
0135	773	user	2021516	72058	8:50	5/24/12

Load Size	Mix Code	Returned	Qty	Mix Age	Seq	Load ID
9.00 CYDS	156220				W	78905

Material	Design Qty	Required	Batched	% Var	% Moisture	Actual Wat
WATER	29.50 GL	246.07 GL	246.00 GL	-0.03%		246.00 gl
ADVA140	40.00 OZ	360.00 OZ	362.00 OZ	0.56%		
LAFARGE	588.0 LB	5292.0 LB	5335.0 LB	0.81%		
LAFSGBFS	.0	.0 LB	.0 LB			
FLYASH	.0	.0 LB	.0 LB			
9#SAND	720 LB	6629 LB	6610 LB	-0.29%	2.30% A	18 gl
MCCA-5	1050 LB	9450 LB	9450 LB	0.00%	0.20% M	
DARA100	.00	.00 OZ	.00 OZ			
KPSA-1	836 LB	7595 LB	7610 LB	0.19%	0.95% M	9 gl

Actual Num Batches: 1 Manual 8:50:12
Load Total: 31097 lb Design 0.381 Water/Cement 0.482 T Design 307.8 gl Actual 274.4 gl To Add: 33.4 gl
Slump: 5.00 in # Water in Truck: 0.0 GL Adjust Water: 0.0 GL / Load Trim Water: 0.0 GL/ CYD

1½ Gal Fe added 683 lbs.

CUSTOMER COPY - 2

University of Kansas
CEAE Department
2150 Learned Hall
1530 W. 15th St.
Lawrence, Kansas 66045-7609
Phone: 785.864.3885 Fax: 785.864.5631

CONCRETE MIX DESIGN

Contractor:	KU
Project:	Bond Test- Beam #1, 2, and 3
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Date:	5/24/2012
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Total Air, percent	0		0.27
Air Entraining Agent / Air R Us	0 fl oz (US)	1.01	0.00
Superplasticizer / Admixtures R Us	(50) 50 fl oz (US)	1.20	0.02
			27.00

1 The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.

Total Water Content (including water in admixtures), lb			247	
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Paste Content, percent			25.55%	
Workability Factor (WF)	Target:	35.0	Actual:	32.1
Coarseness Factor (CF)	Target:	57.9	Actual:	63.0
Prepared On:			5/16/12 2:20 PM	

Prepared By:

Cast on 5/25/12

Jiqiu Yuan

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(785) 843-1688 • FAX (785) 843-1783

LAWRENCE (785) 843-1688 • TOLL FREE (888) 244-2082

"QUALITY AND SERVICE SINCE 1927"

PLANT	TIME	DATE	ACCOUNT	TRUCK	DRIVER	TICKET
	10:11	05/25/12	UK2450	0124	KOMA WISDOM	2021578

CUSTOMER NAME: UNIV OF KANSAS - CIVIL ENG.

DELIVERY ADDRESS: LEARNED HALL - 15TH & IOWA & E. TO LEARNED HALL DR.- PARKING LOT ON WEST SIDE

PURCHASE ORDER	SALES ORDER	TAX	CREDIT			SLUMP
KAN006373		6A				5.00 in

LOAD QTY.	PRODUCT	DESCRIPTION	ORDERED	DELIVERED	UNIT PRICE	AMOUNT
1.00	156220	(HE) OPTMIZD TER	1.00	1.00		
		SHORT LOAD CHARGE				

LOADED	ARRIVE JOB SITE	START DISCHARGE	FINISH DISCHARGE	ARRIVE PLANT
9:55	:	:	:	:

SUB TOTAL
DISCOUNT
TAX
TOTAL
PREVIOUS TOTAL
GRAND TOTAL

This batch of concrete is mixed with the proper amount of water. If additional water is desired, please instruct the driver.

ADDITIONAL WATER ADDED ON JOB → | Gallons | By

CAUTION: Freshly mixed cement, mortar, grout or concrete may cause skin irritation. Avoid direct contact where possible and wash exposed skin areas promptly with water.
If any cementitious material gets into the eye, rinse immediately and repeatedly with water and get prompt medical attention.
KEEP OUT OF REACH OF CHILDREN

UNLOADING TIME ALLOWED 30 MINUTES PER TRIP
EXTRA CHARGE FOR OVER 30 MINUTES →

RECEIVED IN GOOD CONDITION
BY X

Purchaser waives all claims for personal or property damage caused by seller's truck when delivery is made beyond street curb line.

If not paid as agreed, this credit agreement provides for your payment of reasonable costs of collection, including, but not limited to, court costs, attorney fees and/or collection agency fees.

Truck	Driver	User	Disp Ticket Num	Ticket ID	Time	Date
0124	905	user	2021578	77120	9:55	5/25/12

Load Size	Mix Code	Returned	Qty	Mix Age	Seq	Load ID
1.00 CYDS	156220				W	78967

Material	Design Qty		Required	Batched	% Var	% Moisture	Actual Wat
WATER	28.50 GL	#	26.16 GL	26.00 GL	-0.57%		26.00 gl
ADVA140	50.00 OZ	#	50.00 OZ	50.00 OZ	0.00%		
LAFARGE	508.0 LB	#	508.0 LB	500.0 LB	-1.36%		
LAFBGBFS	.0	#	.0 LB	.0 LB			
FLYASH	.0	#	.0 LB	.0 LB			
VPSAND	730 LB	#	735 LB	740 LB	0.63%	2.14% A	2 gl
MCCA-5	1050 LB	#	1050 LB	1070 LB	1.90%	0.95% M	
DARAI400	.00	#	.00 OZ	.00 OZ			
KPSA-1	836 LB	#	849 LB	850 LB	0.17%	1.50% M	2 gl

Actual Num Batches: 1 Manual 9:55:59
Load Total: 3460 lb Design 0.381 Water/Cement 0.492 ↑ Design 34.2 gl Actual 29.4 gl To Add: 4.8 gl
Slump: 5.00 in # Water in Truck: 0.0 GL Adjust Water: 0.0 GL / Load Trim Water: 0.0 GL/ CYD

1.5' CA 687 (did) 0.75%

CUSTOMER COPY - 2

Form 3: Plastic concrete testing and concrete compressive strength

Specimen ID:	Beam #1, 2, 3	Date:	5/24/2012
Measured by:	Nate Searle	Checked by:	Jayne

Plastic concrete testing

Slump, in.	Unit weight, lb/ft^3	Concrete temperature, °F
2 1/4"	153 lb/ft^3	82°F

Tare weight: 7.84 lb
Total weight: 46.04 lb
Volume: 0.2497 ft^2

Concrete compressive strength

Cylinder ID	Cast date	Test date	Age, days	Dia., in.	Area, in.^{2}	Load, kips	Strength, psi	Notes	
5/24 #1	5/24	5/28	4	6.015	28.416	113,000	3980	4010	Plasti
5/24 #2	5/24	5/28	4	6.025	28.51	115,000	4030		
5/25 #1	5/25	5/28	3	6.022	28.48	105,000	3690	3640	Plasti
5/25 #2	5/25	5/28	3	6.015	28.42	102,000	3590		
5/24 #3	5/24	5/30	6	6.025	28.51	131,000	4600	4670	Plasti
5/24 #4	5/24	5/30	6	6.02	28.46	134500	4730		
5/24 #1	5/24	5/31	7	6.015	28.42	146500	5150		steel
5/24 #2	5/24	5/31	7	6.009	28.36	159500	5620	5330	
5/24 #3	5/24	5/31	7	5.995	28.23	147000	5210		
5/25 #1	5/25	5/31	6	5.995	28.23	133000	4710		
5/25 #2	5/25	5/31	6	5.997	28.25	126000	4460	4330	steel
5/25 #3	5/25	5/31	6	5.994	28.22	108000	3830		

3

Form 3: Plastic concrete testing and concrete compressive strength

Specimen ID:	Beam #1, 2, 3 top layer	Date:	5/25/12
Measured by:	NATE SEARLE	Checked by:	Jayne

Plastic concrete testing

Slump, in.	Unit weight, lb/ft^3	Concrete temperature, °F
2 1/4"	152	76.4°F

wt. container = 7.85 lb
wt. container + conc. = 40.72 lb
Vol. container = 0.2497 ft^3

Concrete compressive strength

Cylinder ID	Cast date	Test date	Age, days	Dia., in.	Area, $in.^2$	Load, kips	Strength, psi	Notes

9

University of Kansas Flexure Beam Tests

DATE: 5/31/2012

BEAM TESTS: Beams 1, 2, 3

Flexure Beam ID*	Date Made	Height** in.	Width** in.	Date Tested	Load lb.	Age*** Days	Modulus of Rupture psi.
Mono-1	5/24/2012	6.04	6.27	5/31/2012	7750	7	610
Mono-2	5/24/2012	6.06	6.16	5/31/2012	6600	7	525
					Avg. Monolithic		570
CJ-1	5/24/2012	6.08	6.12	5/31/2012	1200	7	95
CJ-2	5/24/2012	6.06	6.08	5/31/2012	2250	7	180
					Avg. Cold Joint		140

*Mono = monolithic concrete; CJ = cold joint at midspan
**Measured at fracture plane
***The cold joint specimens contain concrete that is 7 and 6 days of age

Matt O'Reilly
Matt O'Reilly

University of Kansas Split Cylinder Tests

DATE: 5/31/2012

BEAM TESTS: Beams 1, 2, 3

Cylinder	Date Made	Length in.	Diameter in.	Date Tested	Load kips	Age Days	Modulus of Rupture psi.
1	5/24/2012	12.19	6.04	5/31/2012	50.5	7	435
2	5/24/2012	12.15	5.97	5/31/2012	49.0	7	430
						Avg.	435

Notes: Concrete from 5/24 cast

Matt O'Reilly
Matt O'Reilly

Form 1: Dimensions of formwork

Beam #1

Specimen ID:	Mono - 79-1	Date:	05/24/12
Measured by:	Matt ORreilly	Checked by:	Jeff Peckover

	Width	Height	Length
Design	18 in.	24 in.	25ft (300in
Tolerance	± ½ in.	± ½ in.	± 1 in.
Measurement 1	17 7/8"	24	300 3/8"
Measurement 2	17 7/8"	24	300 3/8"
Measurement 3	18"	24	300 3/8"
Measurement 4	17 15/16"	24	
Measurement 5	17 7/8"	24	
Measurement 6	17 3/4"	24 1/16	
Measurement 7	17 1/4"	24	
Measurement 8	18"	24	
Measurement 9	[illegible]	24	

Avg 17.75 24

Form 2: Dimensions of reinforcing steel within in the test region

Beam #1

Specimen ID:	Mono-79-1	Date:	05/24/12
Measured by:	Matt Opeitty	Checked by:	Jeff Peckover

		Side cover	Bottom to top of all-thread rod	Splice length
Design		3 in.		79
Tolerance		± ½ in.	± ½ in.	± ½ in.
Splice 1	Measurement 1	3 1/4	19 3/8	~~[illegible]~~ ~~19 3/8~~ 79
	Measurement 2	2 7/8	19 1/4	~~19 5/16~~
	Measurement 3	2 3/4	19 3/8	~~19 3/8~~
Splice 2	Measurement 1	2 3/4	19 3/8	79 1/4
	Measurement 2	2 15/16	19 5/16	
	Measurement 3	3 3/8	19 3/8	

Measured bar diameter:

Splice 1:____________

Splice 2:____________

2

Form 4: Test setup – span spacing

Beam #1

Specimen ID:	Beam 1	Date:	5/31/12
Measured by:	Matt O'Reilly	Checked by:	BH

	Measurement 1, in.	Measurement 2, in.	Measurement 3, in.	Average, in.
Pin centerline to roller centerline				
West end to West support	84 3/16	84 1/4	84 1/8	84.1875
West end to West splice end	110 5/8	110 3/4	110 5/8	110.667
West end to beam centerline	150 1/4	150 3/8	150 3/16	150.271
West end to East splice end	189 5/8	189 5/8	189 5/8	189.625
West end to East support	216 1/4	216 1/4	216 5/16	216.270
West end to East end	300 3/8	300 7/16	300 3/8	300.396

4

Form 5: Dial gage readings

Specimen ID:	Beam 1	Date:	05/31/12
Measured by:	Matt O'Reilly		

	Load, kips	Dial gage 1, in.	Dial gage 2, in.	Dial gage 3, in.
Reading 1	0	0.817	2.695	1.427
Reading 2	5	0.846	2.688	1.459
Reading 3	10	0.874	2.679	1.503
Reading 4	15	0.991	2.652	1.628
Reading 5	20	1.131	2.623	1.860
Reading 6	25	1.253	2.597	1.992
Reading 7	30	1.374	2.570	2.115
Reading 8	35	1.501	2.541	2.239
Reading 9	40	1.651	2.514	2.357
Reading 10				
Reading 11				
Reading 12				
Reading 13				
Reading 14				
Reading 15				
Reading 16				
Reading 17				
Reading 18				
Reading 19				
Reading 20				

Form 1: Dimensions of formwork

Beam #2

Specimen ID:	~~[illegible]~~ CJ-7-2-1	Date:	08/24/12
Measured by:	Matt O'Reilly	Checked by:	[illegible]

	Width	Height	Length
Design	18 in.	24 in.	25 ft (300 in)
Tolerance	± ½ in.	± ½ in.	± 1 in.
Measurement 1	17 15/16	24	25 1/4 ~
Measurement 2	18	20 1/8	25 1/4
Measurement 3	18	20 3/16	25 3/16
Measurement 4	17 15/16	20 3/16	
Measurement 5	18	20 1/8	
Measurement 6	18	20 3/16	
Measurement 7	17 15/16	20 1/4	
Measurement 8	18	20 1/8	
Measurement 9	18	24	
	17.98		

1

Form 2: Dimensions of reinforcing steel within in the test region

Beam #2

Specimen ID:	~~SP~~ CJ-79-1	Date:	05/24/12
Measured by:	Matt O'Reilly	Checked by:	[illegible]

		Side cover	Bottom to top of all-thread rod	Splice length
Design		3 in.		
Tolerance		± ½ in.	± ½ in.	± ½ in.
Splice 1	Measurement 1	2 6/8	19 3/8	79
	Measurement 2	3	19 3/8	
	Measurement 3	3 3/8	19 1/4	
Splice 2	Measurement 1	2 13/16	19 3/8	79 1/16
	Measurement 2	3 1/16	19 6/16	
	Measurement 3	3 3/16	19 3/8	

Measured bar diameter:

Splice 1:____________

Splice 2:____________

2

Form 4: Test setup – span spacing

Beam 2 Beam #2

Specimen ID:	~~[illegible]~~ J +	Date:	5/31
Measured by:	Matt O'Reilly	Checked by:	[illegible]

	Measurement 1, in.	Measurement 2, in.	Measurement 3, in.	Average, in.
Pin centerline to roller centerline				
West end to West support	84 3/16"	84 3/8"	84 1/8"	84.229
West end to West splice end	110 1/2"	110 3/4"	110 5/8"	110.625
West end to beam centerline	150 3/16"	150 3/8"	150 1/4"	150.271
West end to East splice end	189 9/16"	189 3/4"	189 5/8"	189.646
West end to East support	216 1/8"	216 1/4"	216"	216.125
West end to East end	300 1/4"	300 3/8"	300 1/4"	300.292

4

Form 5: Dial gage readings

Beam #2

Specimen ID:	Beam 2	Date:	05/31/12
Measured by:	Matt O'Reilly		

	Load, kips	Dial gage 1, in.	Dial gage 2, in.	Dial gage 3, in.
~~Reading 1~~	~~5~~	~~0.845"~~	~~0.602"~~	
Reading 2	0	0.819"	2.609"	0.952"
Reading 3	5	0.843"	2.602"	0.976"
Reading 4	10	0.882	2.592	1.013
Reading 5	15	~~0.897~~ 0.997	2.566	1.126
Reading 6	20	1.129	2.534	1.254
Reading 7	25	1.252	2.504	1.396
Reading 8	30	1.398	2.471	1.552
Reading 9				
Reading 10				
Reading 11				
Reading 12				
Reading 13				
Reading 14				
Reading 15				
Reading 16				
Reading 17				
Reading 18				
Reading 19				
Reading 20				

5

Form 1: Dimensions of formwork

Beam #3

Specimen ID:	~~Sp~~ GF79-2	Date:	06/29/12
Measured by:	Mark OReilly	Checked by:	Jeff Peckover

	Width	Height	Length
Design	18 in.	24 in.	25 ft (300 in.)
Tolerance	± ½ in.	± ½ in.	± 1 in.
Measurement 1	18	24	300 1/4
Measurement 2	18	20 1/16	300 1/4
Measurement 3	18 1/8	20 1/8	300 1/4
Measurement 4	17 3/4	20	
Measurement 5	17 3/4	20 1/8	
Measurement 6	17 11/16	20 3/16	
Measurement 7	17 13/16	20 1/8	
Measurement 8	18	20 1/8	
Measurement 9	18	24	

17.93

1

Form 2: Dimensions of reinforcing steel within in the test region

Beam #3

Specimen ID:	CJ-79-2	Date:	08/29/12
Measured by:	Matt O'Reilly	Checked by:	Jeff Peckover

		Side cover	Bottom to top of all-thread rod	Splice length
Design		3 in.		79"
Tolerance		± ½ in.	± ½ in.	± ½ in.
Splice 1	Measurement 1	3 7/16"	19 1/2"	79"
	Measurement 2	3 3/8"	19 3/8"	
	Measurement 3	3 1/4"	19 3/8"	
Splice 2	Measurement 1	2 5/8"	19 3/8"	78 15/16"
	Measurement 2	2 7/8"	19 3/8"	
	Measurement 3	3 1/4"	19 3/8"	

Measured bar diameter:

Splice 1:____________

Splice 2:____________

2

Form 4: Test setup – span spacing

Specimen ID:	Beam 3 Beam #3	Date:	5/31/12
Measured by:	Matt OReilly	Checked by:	[signature]

	Measurement 1, in.	Measurement 2, in.	Measurement 3, in.	Average, in.
Pin centerline to roller centerline				
West end to West support	84 1/16	84 3/16	84 3/16	84.146
West end to West splice end	111	111 1/16	110 13/16	110.292
West end to beam centerline	150 3/16	150 3/16	150 5/16	150.229
West end to East splice end	189 5/8	189 11/16	189 3/4	189.688
West end to East support	216 1/8	216 3/16	216 5/16	216.208
West end to East end	300 1/4	300 3/8	300 1/4	300.292

4

Form 5: Dial gage readings

Specimen ID:	Beam 3 Beam #3	Date:	05/31/12
Measured by:	Matt O Reilly		

	Load, kips	Dial gage 1, in.	Dial gage 2, in.	Dial gage 3, in.
Reading 1	0	0.916	2.681	0.813
Reading 2	5	0.943	2.674	0.839
Reading 3	10	0.977	2.665	0.876
Reading 4	15	1.090	2.639	0.996
Reading 5	20	1.236	2.605	1.124
Reading 6	25	1.390	2.573	1.260
Reading 7	30	1.534	2.543	1.402
~~Reading 8~~	Un Load			
Reading 9	0	1.045	2.665	0.935
Reading 10	5	1.089	2.644	0.980
Reading 11	10	1.180	2.623	1.068
Reading 12	15	1.268	2.603	1.151
Reading 13	20	1.360	2.581	1.238
Reading 14	25	1.451	2.561	1.324
Reading 15	30	1.554	2.538	1.421
Reading 16	35	1.674	2.509	1.549
Reading 17				
Reading 18				
Reading 19				
Reading 20				

Training Form (#1, 2, and 3 beams testing)

This form is prepared to indicate those who will work on testing the above mentioned beams have received proper training and they are qualified for what they will work on.

Last Name	First Name	ACI qualified? (if yes, certification #)	signature	date
Al-Khafaji	Ali	No	[illegible]	5/31/12
~~Eisenbarth~~	~~Brad~~			
Fink	Dalen	No	[illegible]	5/31/12
Guernsey	Edward	N.	[illegible]	5/31/12
~~Hawk~~	~~Kaleb~~			
Lyon	Adam	YES	[illegible]	5/31/2012
O'Reilly	Matt	Yes	[illegible]	5/31/12
Peckover	Jeff	Yes	[illegible]	5/31/12
Pendergrass	Ben	Yes	[illegible]	5/31/12
Schneider	Aaron	NO	[illegible] Schneider	5/31/12
Searle	Nate	Yes	[illegible]	5/31/12
Shrestha	Pankaj	NO	[illegible]	5/31/12
Sperry	Jayne			
~~Williams~~	~~Eric~~			
Yuan	Jiqiu	Yes	[illegible]	5/31/12
Zhen	Chen	Yes	[illegible]	5/31/12
Steele	Jim			
Nickolaus	Gary	~~Yes~~ NO [illegible] 5/31/12	Gary Nickolaus	5/31/12
Kummer	Lou			
BARNARD	JAY		[illegible]	5-31-12
BAIR	RICHARD		[illegible]	5/31/12
Bohnty	Brent		[illegible]	5/31/12

①/2

Beam Test Recording

Specimen :	Beam #1 (mono)	Test Date:	5/31/12
Recorded by:	Jiqiu Yuan		

Load, kip	Note
5	
10	1 mil SE SR 1 mil SE PS
15	2 mil out both PSs 5 mil out NW splice region
	7 mil NW PS. 5 mil SW PS
20	7 mil SE PS 3 mil SE SR
	7 mil out NW SR 3 mil CL
	9 mil SW PS 5 mil out SW PS
25	7 mil out SE PS 2 mil NW PS
	6 mil SE PS 3 mil CL
	13 mil out W SR 13 mil SW PS
	2 mil horizontal / NE PS
30	16 mil out W PS 13 mil on W PS
	18 mil out W SR 5 mil CL
	9 mil SE SR 13 E PS
	2 mil horizontal at W SR

②/2

Beam Test Recording

Specimen :	Beam #1 (mono)	Test Date:	5/31/12
Recorded by:	Jiqiu Yuan		

Load, kip	Note
35	13 mil out NW PS 18 mil out SW PS
	23 mil SW SR 15 mil out SE SR
	9 mil CL 5 mil horizontal W SR (previous 2 mil)
	10 mil E SR 13 mil SE PS
40	20 mil SE PS 16 mil NW PS
	16 mil SE SR 9 mil CL
	13 mil W SR
	2 mil horizontal on top E SR
	13 mil . . on top W SR
	expand to 9 mil ✓
51	
	1/8" SE PS 6 mil SW PS
	4 mil out W SR 3/8" W SR
	1/8" horizontal SW SR 12 mil CL
	15 mil NW SR

①/1

Beam Test Recording

Specimen :	Beam #2	Test Date:	5/31
Recorded by:	Jiqin		

Load, kip	Note
5	
unload	take initial dial gage reading
5, 10	nothing
15	3 mil crack both at support 3 mil at end of splice region
	delamination finding north side / 7 mil at west splice region
20	5 mil east PS / 5 mil out east PS / 16 mil out east SR
	5 mil horizontal N west SR 5 mil inside east SR
	7 mil horizontal S west SR 3 mil East PS
	6 mil W PS 3 mil out E SR
25	6 mil out SW PS 20 mil SW PS 3 mil out NWPS
	6~13 mil horizontal SW SR 16 mil out WSR
	6 mil inside W SR 20 mil E SR
30	last dial gage reading
42	horizontal crack extending
43	fail
	8 mil horizontal SW PS 1/4" horizontal SW SR
(58 ksi) bar stress	8 mil out SR 1/2 in horizontal NW SR

①/4

Beam Test Recording

cycle load)

Specimen :	Beam #3 (cold joint	Test Date:	5/31/12
Recorded by:	Jiqiu Yuan		

Load, kip	Note
5	
10	1 mil NE SR
15	3 mil NW PS 7 mil out NW SR
	5 mil NE SR 5 mil NE PS
	horizontal out SW SR
20	13 mil NW PS 9 mil out SW PS
	2 mil horizontal NW SR
	9 mil horizontal SW SR
	9 mil NE PS
25	8 mil horizontal NW SR over 2ft long
	13 mil SE PS
	16 mil horizontal SW SR
	5 mil horizontal NE SR
	7 mil horizontal NE PS
	3 mil horizontal out SW PS
	3 mil horizontal out SE SR

②/4

Beam Test Recording

Specimen :	Beam #3	Test Date:	5/31/12
Recorded by:	Jieqin		

Load, kip	Note
30	(only check horizontal crack width)
	2 mil horizontal at NW PS
	1 mil — at NW PS
	10 mil at NE SR
	20 mil at SW SR
	6 mil at NW SR
	5 mil at SE SR
	Center of splice no horizontal crack
	3 mil horizontal at SE SR
	6 mil on top of WPS (flexural)
	7 mil at centerline
	40 SE SR
unload 0 kip	3 mil 7 mil almost invisible
	13 mil flexural at WPS

③/4

Beam Test Recording

Specimen :	Beam #7	Test Date:	5/31/12
Recorded by:	Jiqin		

Load, kip	Note
up agg	dial gage reading only
5 kip	dial gage reading
10	〃
15	〃
20	〃
25	〃
30	recheck for crack
	35 mil horizontal at NW SR
2 mil	New crack at SW PS and between SW PS + SR
	55 mil at NW SR (old crack expand)
	5 in extension on the horizontal crack NE SR
	no horizontal crack at CL
35	dial gage only

④/4

Beam Test Recording

Specimen :	Beam #3	Test Date:	5/31/12
Recorded by:	Jiqiu		

Load, kip	Note
40 kip	failure
	3/8" SW SR horizontal
	0.5" SE SR
	1/8" on top of CL
	3/8" NW SR

Training Form (#4, 5, and 6 beams)

This form is prepared to indicate those who will work on casting the above mentioned beams have received proper training and they are qualified for what they will work on.

Last Name	First Name	ACI qualified? (if yes, certification #)	signature	date
Al-Khafaji	Ali			
Eisenbarth	Brad			
Fink	Dalen			
Guernsey	Edward	No	Ed [illegible]	
Hawk	Kaleb	No	Kaleb Hawk	
Lyon	Adam	YES	[illegible]	6/7/12
O'Reilly	Matt	Yes 01090386	Matt OReilly	06/07/12
Peckover	Jeff	Yes	[illegible]	6/7/12
Pendergrass	Ben	Yes	[illegible]	6/7/12
Routh	Jon			
Schneider	Aaron	NOPE	Aaron Schneider	6/7/12
Searle	Nate	Yes	Nate Searle	6/7/12
Shrestha	Pankaj	No.	[illegible]	6/7/12
Sperry	Jayne	Yes	Jayne Sperry	6/7/12
Williams	Eric			
Yuan	Jiqiu			
Zhen	Chen	Yes	[illegible]	
Somogie	Isaac	No	[illegible]	6/7/12
Hansen	Joci	No	[illegible]	6/7/12
Assenmacher	Laurakate	No	Laura Kate Assenmacher	6/7/12

University of Kansas
CEAE Department
2150 Learned Hall
1530 W. 15th St.
Lawrence, Kansas 66045-7609
Phone: 785.864.3885 Fax: 785.864.5631

CONCRETE MIX DESIGN

Contractor: KU
Project: Bond Test- Beam #4, 5, and 6
Source of Concrete: Ready Mix Concrete
Date: 6/10/2012
Placement Type: Conventional

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 100%	588 lb	3.20	2.94
Water	246 lb	1.00	3.94
MCM -1.5 in. / Granite 1.5" / 20.86%	687 lb	2.71	4.06
MCM -0.75 in. / Granite 0.75" / 31.89%	1050 lb	2.71	6.21
Pea Gravel / Pea Gravel / 25.39%	836 lb	2.60	5.15
VPsand / VP Sand / 21.86%	720 lb	2.62	4.40
Total Air, percent	0		0.27
Air Entraining Agent / Air R Us	0 fl oz (US)	1.01	0.00
Superplasticizer / Admixtures R Us	60 fl oz (US)	1.20	0.02
			27.00

1 The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.

Total Water Content (including water in admixtures), lb			247	
Water / Cementitious Material Ratio:			0.42	
Concrete Unit Weight, pcf			152.9	
Target Slump, in.			5 in.	
Paste Content, percent			25.55%	
Workability Factor (WF)	Target:	35.0	Actual:	32.1
Coarseness Factor (CF)	Target:	57.9	Actual:	63.0
Prepared On:			5/16/12 2:20 PM	

Prepared By:

Jiqiu Yuan

MCM Midwest Concrete Materials

3645 E. 23rd Street • Lawrence, KS 66046
(785) 843-1688 • FAX (785) 843-1783

LAWRENCE (785) 843-1688 • TOLL FREE (888) 244-2082

"QUALITY AND SERVICE SINCE 1927"

PLANT	TIME	DATE	ACCOUNT	TRUCK	DRIVER	TICKET
2[illegible]	7:2?	06/13/12	UK2450	0131	BEN JONES	2022120

CUSTOMER NAME: UNIV OF KANSAS - CIVIL ENG.

DELIVERY ADDRESS: LEARNED HALL
15TH & IOWA & E. TO LEARNED HALL
DR. - PARKING LOT ON WEST SIDE

PURCHASE ORDER	SALES ORDER	TAX	CREDIT			SLUMP
KANBBB6373		68				4.00 in

LOAD QTY.	PRODUCT	DESCRIPTION	ORDERED	DELIVERED	UNIT PRICE	AMOUNT
10.00	156720	KC DURABLE	10.00	10.00		

LOADED	ARRIVE JOB SITE	START DISCHARGE	FINISH DISCHARGE	ARRIVE PLANT
7:16	:	:	:	:

SUB TOTAL
DISCOUNT
TAX
TOTAL
PREVIOUS TOTAL
GRAND TOTAL

This batch of concrete is mixed with the proper amount of water. If additional water is desired, please instruct the driver.

ADDITIONAL WATER ADDED ON JOB → Gallons By

UNLOADING TIME ALLOWED 30 MINUTES PER TRIP
EXTRA CHARGE FOR OVER 30 MINUTES →

CAUTION: Freshly mixed cement, mortar, grout or concrete may cause skin irritation. Avoid direct contact where possible and wash exposed skin areas promptly with water. If any cementitious material gets into the eye, rinse immediately and repeatedly with water and get prompt medical attention.
KEEP OUT OF REACH OF CHILDREN

RECEIVED IN GOOD CONDITION
BY **X**

Purchaser waives all claims for personal or property damage caused by seller's truck when delivery is made beyond street curb line.

If not paid as agreed, this credit agreement provides for your payment of reasonable costs of collection, including, but not limited to, court costs, attorney fees and/or collection agency fees.

Truck	Driver	User	Disp Ticket Num	Ticket ID	Time	Date
0131	349	user	2022120	77657	7:16	6/13/12

Load Size	Mix Code	Returned	Qty	Mix Age	Seq	Load ID
10.00 CYDS	156720				0	79509

Material	Design Qty		Required	Batched	% Var	% Moisture	Actual Wat
WATER	25.00 GL	*	238.64 GL	238.00 GL	-0.27%		238.00 gl
ADVA140	60.00 OZ	*	600.00 OZ	600.00 OZ	0.00%		
LAFARGE	588.0 LB	*	5880.0 LB	5870.0 LB	-0.17%		
LAFBAAFS	.0	*	.0 LB	.0 LB			
VPSAND	720 LB	*	7320 LB	7390 LB	0.84%	1.78% A	15 gl
KPSA-1	836 LB	*	8393 LB	8440 LB	0.55%	0.48% M	4 gl
DARAI400	.00	*	.00 OZ	.00 OZ			
MCLA-5	1050 lb	*	10500 lb	10550 lb	0.48%	0.35% M	
MCCA-7	687 lb	*	6870 lb	6880 lb	0.15%	0.10% M	

Actual Num Batches: 1 Manual 7:16:09
Load Total: 41154 lb Design 0.441 Water/Cement 0.451 T Design 317.0 gl Actual 257.5 gl To Add: 59.5 gl
Slump: 4.00 in Water in Truck: 0.0 GL Adjust Water: 0.0 GL / Load Trim Water: 0.0 GL/ CYD

CUSTOMER COPY - 2

X

University of Kansas
CEAE Department
2150 Learned Hall
1530 W. 15th St.
Lawrence, Kansas 66045-7609
Phone: 785.864.3885 Fax: 785.864.5631

CONCRETE MIX DESIGN

Contractor:	KU
Project:	Bond Test- Beam #4, 5, and 6
Source of Concrete:	Ready Mix Concrete
Date:	6/14/2012
Placement Type:	Conventional

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 100%	588 lb	3.20	2.94
Water	246 lb	1.00	3.94
MCM -1.5 in. / Granite 1.5" / 20.86%	687 lb	2.71	4.06
MCM -0.75 in. / Granite 0.75" / 31.89%	1050 lb	2.71	6.21
Pea Gravel / Pea Gravel / 25.39%	836 lb	2.60	5.15
VPsand / VP Sand / 21.86%	720 lb	2.62	4.40
Total Air, percent	0		0.27
Air Entraining Agent / Air R Us	0 fl oz (US)	1.01	0.00
Superplasticizer / Admixtures R Us	60 fl oz (US)	1.20	0.02
			27.00

1 The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.

Total Water Content (including water in admixtures), lb			247		
Water / Cementitious Material Ratio:			0.42		
Concrete Unit Weight, pcf			152.9		
Target Slump, in.			5 in.		
Paste Content, percent			25.55%		
Workability Factor (WF)	Target:	35.0	Actual:	32.1	
Coarseness Factor (CF)	Target:	57.9	Actual:	63.0	
Prepared On:			5/16/12 2:20 PM		

Prepared By:

Jiqiu Yuan

6 lb/yd³ ice

MCM Midwest Concrete Materials

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"QUALITY AND SERVICE SINCE 1927"

PLANT	TIME	DATE	ACCOUNT	TRUCK	DRIVER	TICKET
	9:05	06/14/12	UK2450	0095	BRUCE WISE	2022187

CUSTOMER NAME: UNIV OF KANSAS - CIVIL ENG.

DELIVERY ADDRESS: LEARNED HALL
15TH & IOWA & E. TO LEARNED HALL
DR. - PARKING LOT ON WEST SIDE

PURCHASE ORDER	SALES ORDER	TAX	CREDIT			SLUMP
KAN006373		68	2			4.00 in

LOAD QTY.	PRODUCT	DESCRIPTION	ORDERED	DELIVERED	UNIT PRICE	AMOUNT
2.00	156720	KC DURABLE	2.00	2.00		
		SHORT LOAD CHARGE				

LOADED	ARRIVE JOB SITE	START DISCHARGE	FINISH DISCHARGE	ARRIVE PLANT
8:59	:	:	:	:

SUB TOTAL
DISCOUNT
TAX
TOTAL
PREVIOUS TOTAL
GRAND TOTAL

This batch of concrete is mixed with the proper amount of water. If additional water is desired, please instruct the driver.

ADDITIONAL WATER ADDED ON JOB → Gallons By

CAUTION: Freshly mixed cement, mortar, grout or concrete may cause skin irritation. Avoid direct contact where possible and wash exposed skin areas promptly with water. If any cementitious material gets into the eye, rinse immediately and repeatedly with water and get prompt medical attention.
KEEP OUT OF REACH OF CHILDREN

UNLOADING TIME ALLOWED 30 MINUTES PER TRIP EXTRA CHARGE FOR OVER 30 MINUTES →

RECEIVED IN GOOD CONDITION
BY **X**

Purchaser waives all claims for personal or property damage caused by seller's truck when delivery is made beyond street curb line.

If not paid as agreed, this credit agreement provides for your payment of reasonable costs of collection, including, but not limited to, court costs, attorney fees and/or collection agency fees.

Truck	Driver	User	Disp Ticket Num	Ticket ID	Time	Date
0095	989	user	2022187	77723	8:59	6/14/12

Load Size	Mix Code	Returned	Qty	Mix Age	Seq	Load ID
2.00 CYDS	156720				W	79576

Material	Design Qty		Required	Batched	% Var	% Moisture	Actual Wat
WATER	24.60 GL		45.29 GL	45.00 GL	-0.65%		45.00 gl
ADVA140	60.00 OZ		120.00 OZ	120.00 OZ	0.00%		
LAFARGE	588.0 LB		1176.0 LB	1180.0 LB	0.34%		
LAF000FS	.0		.0 LB	.0 LB			
YPSAND	720 LB		1466 LB	1460 LB	-0.40%	1.80% M	3 gl
KPSA-1	836 LB		1679 LB	1680 LB	0.06%	0.40% M	1 gl
DARA1400	.00		.00 OZ	.00 OZ			
MCCA-5	1050 lb		2100 lb	2110 lb	0.48%	0.35% M	
MCCA-7	687 lb		1374 lb	1380 lb	0.44%	0.10% M	

Actual Num Batches: 1 Manual 8:59:26
Load Total: 8193 lb Design 0.441 Water/Cement 0.448 T Design 63.4 gl Actual 48.9 gl To Add: 14.5 gl
Slump: 4.00 in Water in Truck: 0.0 GL Adjust Water: 0.0 GL / Load Trim Water: 0.0 GL/ CYD

CUSTOMER COPY - 2

X

Form 3: Plastic concrete testing and concrete compressive strength

Specimen ID:	Beams #4,5,6 Bottom layer	Date:	6/13/12
Measured by:	NATE SEARLE	Checked by:	Jaypie Spurj

Plastic concrete testing

Slump, in.	Unit weight, lb/ft^3	Concrete temperature, °F
3"	154.1	81.8 °F

wt. container: 9.74 lb
wt. conc. + container: 48.22 lb
Volume of container: 0.2497

Concrete compressive strength

Cylinder ID	Cast date	Test date	Age, days	Dia., in.	Area, in.^{2}	Load, kips	Strength, psi	Notes
Plastic #1	6/13	6/17	4	6.021	28.47	124,500	4370	4310
Plastic #2	6/13	6/17	4	6.030	28.56	121,000	4240	
Plastic #3	6/13	6/19	6	5.970	27.99	135,000	4820	4680
Plastic #4	6/13	6/19	6	6.09	29.13	132200	4530	
Steel #1	6/13	6/20	7	6.007	28.33	146500	5170	
Steel #2	6/13	6/20	7	5.999	28.25	145000	5130	5230
Steel #3	6/13	6/20	7	6.018	28.45	153000	5380	

9

X

Form 3: Plastic concrete testing and concrete compressive strength

Specimen ID:	Beam #4 S.6 top layer	Date:	6/14/2012
Measured by:	NATE SEARLE	Checked by:	Jeff Peckover

Plastic concrete testing

Slump, in.	Unit weight, lb/ft^3	Concrete temperature, °F
2 3/4"	149.9	86.2°

Wt. Container = 9.74
Vol. Container = 0.2497
Wt. Concrete + Container = 47.16 lb

Concrete compressive strength

Cylinder ID	Cast date	Test date	Age, days	Dia., in.	Area, in.^{2}	Load, kips	Strength, psi	Notes
plastic#1	6/14	6/17	3	6.026	28.52	121,500	4260	4520
plastic #2	6/14	6/17	3	6.027	28.53	136,500	4780	
steel#1	6/14	6/20	6	6.008	28.34	152500	5380	
steel#2	6/14	6/20	6	6.042	28.66	160000	5580	5490
steel#3	6/14	6/20	6	5.996	28.22	155500	5510	

University of Kansas Flexure Beam Tests

DATE: 6/20/2012

BEAM TESTS: Beams 4, 5, 6

Flexure Beam ID*	Date Made	Height** in.	Width** in.	Date Tested	Load lb.	Age*** Days	Modulus of Rupture psi.
Mono-W1	6/13/2012	6.03	6.35	6/20/2012	8650	7	674
Mono-W2	6/13/2012	5.89	6.27	6/20/2012	6350	7	525
						Avg. Monolithic (Wed.)	600
Mono-R1	6/14/2012	6.07	6.28	6/20/2012	8500	6	661
Mono-R2	6/14/2012	6.085	6.25	6/20/2012	9500	6	739
						Avg. Monolithic (Thur.)	700
CJ-1	6/13/2012	6.05	6.08	6/20/2012	3950	7	319
CJ-2	6/13/2012	6.05	6.14	6/20/2012	2850	7	228
						Avg. Cold Joint	274

*Mono = monolithic concrete; CJ = cold joint at midspan
**Measured at fracture plane
***The cold joint specimens contain concrete that is 7 and 6 days of age

Matt O'Reilly
Matt O'Reilly

University of Kansas Split Cylinder Tests

DATE: 6/20/2012

BEAM TESTS: Beams 4, 5, 6

Cylinder	Date Made	Length in.	Diameter in.	Date Tested	Load kips	Age Days	Modulus of Rupture psi.
W1	6/13/2012	12.25	5.98	6/20/2012	37.5	7	326
W2	6/13/2012	12.03	5.98	6/20/2012	47.0	7	416
						Avg. (Wed.)	**370**
R1	6/14/2012	12.15	5.99	6/20/2012	56.5	6	494
R2	6/14/2012	12.24	6.00	6/20/2012	51.0	6	442
						Avg. (Thur.)	**470**

Matt O'Reilly

Matt O'Reilly

Form 1: Dimensions of formwork

Specimen ID:	Beam 4	Date:	06/12/12
Measured by:	Matt O'Reilly	Checked by:	

	Width	Height	Length
Design	18 in.	24 in.	336"
Tolerance	± ½ in.	± ½ in.	± 1 in.
Measurement 1	18 1/16	24 1/8	335 1/2
Measurement 2	17 7/8	24 1/16	335 5/8
Measurement 3	18	20 3/16	335 5/8
Measurement 4	18 1/16	20 1/4	
Measurement 5	17 15/16	20 1/4	
Measurement 6	18	20 1/4	
Measurement 7	17 15/16	20 1/4	
Measurement 8	17 15/16	24	
Measurement 9	18	24	

7

Form 1: Dimensions of formwork

Specimen ID:	Beam 5	Date:	06/12/12
Measured by:	Matt O'Reilly	Checked by:	

	Width	Height	Length
Design	18 in.	24 in.	336"
Tolerance	± ½ in.	± ½ in.	± 1 in.
Measurement 1	18"	24 1/8"	~~335~~ 335 3/8
Measurement 2	18"	24 1/16"	335 1/2
Measurement 3	18 1/8"	20 1/8	333 3/8
Measurement 4	17 7/8	20 1/4	
Measurement 5	17 7/8	20 1/4	
Measurement 6	18 1/16	20 3/16	
Measurement 7	17 13/16	20 1/4	
Measurement 8	18	24	
Measurement 9	18 1/16"	24 1/8	

7

Form 1: Dimensions of formwork

Specimen ID:	Beam 6	Date:	06/12/12
Measured by:	Matt O'Reilly	Checked by:	

	Width	Height	Length
Design	18 in.	24 in.	336
Tolerance	± ½ in.	± ½ in.	± 1 in.
Measurement 1	18 1/4	24	335 5/8
Measurement 2	18 1/4	24	335 3/4
Measurement 3	17 15/16	20 3/16	335 3/4
Measurement 4	18	20 1/4	
Measurement 5	17 7/8	20 3/8	
Measurement 6	17 15/16	20 3/8	
Measurement 7	18 1/8	20 5/16	
Measurement 8	18 1/8	24 1/16	
Measurement 9	18	24	

7

Form 2: Dimensions of reinforcing steel within in the test region

Specimen ID:	Beam 4	Date:	06/12/12
Measured by:	Matt O'Reilly	Checked by:	

		Side cover	Bottom to top of all-thread rod	Splice length
Design		3 in.		120"
Tolerance		± ½ in.	± ½ in.	± ½ in.
Splice 1	Measurement 1	2 13/16	19 3/8	120 1/16
	Measurement 2	2 3/4	19 3/8	|
	Measurement 3	2 3/4	19 3/8	|
Splice 2	Measurement 1	3 7/16	19 3/16	120 [illegible]/16
	Measurement 2	3 7/16	19 7/16	|
	Measurement 3	3 3/8	19 3/8	|

Measured bar diameter:

Splice 1:____________

Splice 2:____________

8

Form 2: Dimensions of reinforcing steel within in the test region

Specimen ID:	Beam 5	Date:	06/12/12
Measured by:	Matt OReilly	Checked by:	

		Side cover	Bottom to top of all-thread rod	Splice length
Design		3 in.		120"
Tolerance		± ½ in.	± ½ in.	± ½ in.
Splice 1	Measurement 1	3 1/4	19 3/8	120
	Measurement 2	3 3/16	19 1/4	
	Measurement 3	3	19 3/8	
Splice 2	Measurement 1	3	19 3/8	120 1/18
	Measurement 2	3 1/8	19 5/16	
	Measurement 3	3 1/4	19 5/16	

Measured bar diameter:

Splice 1:____________

Splice 2:____________

8

Form 2: Dimensions of reinforcing steel within in the test region

Specimen ID:	Beam 8	Date:	06/12/12
Measured by:	Matt OReilly	Checked by:	

		Side cover	Bottom to top of all-thread rod	Splice length
Design		3 in.		120"
Tolerance		± ½ in.	± ½ in.	± ½ in.
Splice 1	Measurement 1	3 7/16	19 3/8	119 15/16
	Measurement 2	3 1/8	19 3/8	[illegible]
	Measurement 3	2 13/16	19 5/16	
Splice 2	Measurement 1	3 5/16	19 3/8	120 1/16
	Measurement 2	2 7/8	19 3/8	
	Measurement 3	2 5/8	19 3/8	

Measured bar diameter:

Splice 1:____________

Splice 2:____________

8

48
12
56
280
336

X

Form 4: Test setup – span spacing

Specimen ID:	Beam 4	Date:	06/20/12
Measured by:	Matt Opeilly	Checked by:	NATE SEARLE

	Measurement 1, in.	Measurement 2, in.	Measurement 3, in.	Average, in.
Pin centerline to roller centerline	~~84~~			
West end to West support	84 1/4"	84 1/4	84 5/16	
West end to West splice end	108 1/4"	108 1/4	108 5/16	
West end to beam centerline	168 1/4"	168 1/4"	168 5/16	
West end to East splice end	228 3/16"	228 5/16	228 1/4	
West end to East support	252 3/16"	252 1/4	252 1/8	
West end to East end	336 1/2	336 1/2	336 1/2	

4

X

Form 4: Test setup – span spacing

Specimen ID:	Beam 5	Date:	08/20/12
Measured by:	Matt OReilly	Checked by:	NATE SEARLE

	Measurement 1, in.	Measurement 2, in.	Measurement 3, in.	Average, in.
Pin centerline to roller centerline				
West end to West support	84 3/16	84 5/16	84 3/8	
West end to West splice end	108 3/16	108 1/4	108 1/4	
West end to beam centerline	168 1/8	168 1/4	168 5/16	
West end to East splice end	228 3/8	228 1/4	228 1/4	
West end to East support	262 1/8	252 1/4	252 1/4	
West end to East end	336 3/8	336 3/16	336 1/4	

4

X

Form 4: Test setup – span spacing

Specimen ID:	Beam 6	Date:	05/20/12
Measured by:	Matt O'Reilly	Checked by:	NATE SEARLE

	Measurement 1, in.	Measurement 2, in.	Measurement 3, in.	Average, in.
Pin centerline to roller centerline				
West end to West support	84 3/8	84 3/8	84 3/16	
West end to West splice end	108 1/4	108 5/16	108 1/8	
West end to beam centerline	168 1/4	168 5/16	168 1/8	
West end to East splice end	228 5/16	228 5/16	228 1/8	
West end to East support	252 5/16	252 1/4	252 1/8	
West end to East end	336 1/2	336 9/16	336 1/2	

4

Form 5: Dial gage readings

Specimen ID:	Beam 4	Date:	06/20/12
Measured by:	Matt O'Reilly		

	Load, kips	Dial gage 1, in.	Dial gage ~~2~~ 3, in.	Dial gage ~~3~~ 2, in.
Reading 1	0	0.776	2.630	1.696
Reading 2	~~10~~ 5	0.806	2.519	1.727
Reading 3	10	0.862	2.602	1.773
Reading 4	16	0.994	2.572	1.896
Reading 5	20	1.057	2.514	2.051
Reading 6	25	1.296	2.476	2.186
Reading 7	30	1.442	2.428	2.331
Reading 8	35	1.602	2.383	2.487
Reading 9				
Reading 10				
Reading 11				
Reading 12				
Reading 13				
Reading 14				
Reading 15				
Reading 16				
Reading 17				
Reading 18				
Reading 19				
Reading 20				

5

Form 5: Dial gage readings

Specimen ID:	Beam 5	Date:	06/20/12
Measured by:	Matt O'Reill,		

	Load, kips	Dial gage 1, in.	Dial gage ~~2~~ 3, in.	Dial gage ~~3~~ 2, in.
Reading 1	0	0.937	1.745	0.995
Reading 2	5	0.984	1.728	1.043
Reading 3	10	1.020	1.716	1.087
Reading 4	15	1.160	1.684	1.201
Reading 5	20	1.304	1.631	1.363
Reading 6	25	1.443	1.591	1.511
Reading 7	30	1.586	1.542	1.658
Reading 8	35	1.768	1.497	1.822
Reading 9	40	1.914	1.448	1.980
Reading 10	0	1.135	1.696	1.184
Reading 11	5	1.224	1.666	1.278
Reading 12	10	1.317	1.637	1.376
Reading 13	15	1.410	1.609	1.468
Reading 14	20	1.513	1.580	1.576
Reading 15	30	1.726	1.509	1.788
Reading 16	35	1.867	1.468	1.933
Reading 17	40	2.037	1.418	2.091
Reading 18	–			
Reading 19				
Reading 20				

5

Form 5: Dial gage readings

Specimen ID:	Beam 6	Date:	06/20/12
Measured by:	Matt OReilly		

	Load, kips	Dial gage 1, in.	Dial gage ~~2~~ 3, in.	Dial gage ~~3~~ 2, in.
Reading 1	0	0.818	2.828	1.334
Reading 2	5	0.850	2.816	1.368
Reading 3	10	0.890	2.801	1.423
Reading 4	15	1.039	2.769	1.569
Reading 5	20	1.167	2.717	1.694
Reading 6	25	1.300	2.681	1.836
Reading 7	30	1.438	2.636	1.982
Reading 8	35	1.615	2.586	2.168
Reading 9	40	1.779	2.530	2.340
Reading 10	0'	1.015	2.777	1.638
Reading 11	10'	1.178	2.723	1.715
Reading 12	20'	1.385	2.567	1.931
Reading 13	30'	1.598	2.594	2.151
Reading 14	35'	1.705	~~2.585~~ Stuck	2.265
Reading 15	40'	1.826	2.516	2.394
Reading 16				
Reading 17				
Reading 18				
Reading 19				
Reading 20				

5

Training Form (#4, 5, and 6 beams testing)

This form is prepared to indicate those who will work on testing the above mentioned beams have received proper training and they are qualified for what they will work on.

Last Name	First Name	ACI qualified? (if yes, certification #)	signature	date
Al-Khafaji	Ali			
Eisenbarth	Brad			
Fink	Dalen			
Guernsey	Edward	No	[illegible]	6/20/12
Hawk	Kaleb	No	[illegible]	6-20
Lyon	Adam			
O'Reilly	Matt	Yes 01090380	Matt O'Reilly	06/20/12
Peckover	Jeff			
Pendergrass	Ben	Yes	[illegible]	6/20/12
Schneider	Aaron	NO	Aaron Schneider	6/20/12
Searle	Nate	Yes	[illegible]	6/20/12
Shrestha	Pankaj	NO	[illegible]	6/20/12
Sperry	Jayne	Yes	Jayne Sperry	6/20/12
Williams	Eric			
Yuan	Jiqiu	Yes	[illegible]	6/20/12
Zhen	Chen	Yes	[illegible]	6/20/12
Steele	Jim			
Nickolaus	Gary			
Kummer	Lou			
Jeronimus	Robbie	no	[illegible]	6/20/12
Sanegie	Isaac	NO	[illegible]	6/20/2012
Long	Eric	No	[illegible]	6/20/2012
Schuler	Joe	NO	[illegible]	6/20/2012
BAIR	RICHARD	NA	[illegible]	6/20/12
Kummer	Louis	NO	[illegible]	6/20/12

1/4

Beam Test Recording

Specimen :	#4	Test Date:	6/20/12
Recorded by:	Jiqiu		

Load, kip	Note
5	
6	2 mil top W PS
15	2 mil outside E PS
	8 mil out E PS
	8 mil out W SR
	8 mil inside E SR
	7 mil out W PS
	3 mil out W PS
20	2 mil out NW PS
	13 mil out SW PS
	5 mil horizontal out NW SR 20A
	3 mil — — SW SR 20B
	5 mil between NW CL and PS
	2 mil SE SR
	6 mil between SE SR + PS

2/4

Beam Test Recording

Specimen :	#4	Test Date:	6/20/12
Recorded by:	Jiqiu		

Load, kip	Note
25	16 mil on both N+S E PS
	18 mil between SE PS + SR
	7 mil SE CL
	8 mil NE CL
	20B horizontal opposite 9 mil → 25B
	20A horizontal " " 6 mil → 25A
	16 mil out SW PS
	2 mil horizontal out SW PS 25C
	2 mil " " on NW PS 25D
	7 mil out SE SR 25E
	3 mil NE PS 25F
	2 mil on SE PS 25G
	2 mil out NE PS 25H

3/4

Beam Test Recording

Specimen :	#4	Test Date:	
Recorded by:	Jiqiu		

Load, kip	Note
30	20mil out NE PS
	7 mil out NE SR
	30 mil at SE SR
	5mil out NE SR. horizontal 30J
	2mil horizontal out SE PS 30K
	5 mil horizontal out NW PS 30L
35	25 mil on top of W PS
last dial gage reading	7 mil and 6 mil at CL
	25mil at E SR
	5mil horizontal out N CL 35M
	30J + 30E merge together → 16mil
	20 mil out NE PS
	A → 13mil B → 13mil

4/4

Beam Test Recording

Specimen :	#4	Test Date:	
Recorded by:	Jiqiu		

Load, kip	Note
(51)	fail flexural failure w/ secondary splice failure
	1 1/8" horizontal Sw sp
	1/2 on top CL
	1/8" horizontal on top CL

1/5 ×

Beam Test Recording

Specimen :	#5	Test Date:	6/20/12
Recorded by:	Jiqiu Yuan		

Load, kip	Note
7kip	
6kip	1 mil at top of W PS
	2 mil horizontal NE PS → 6A
	2 mil at SE SR
	1 mil on top of E PS
15kips	
	4 mil SE PS
	5 mil at NW PS
	9 mil NW SR
	3 mil east of CL
	5 mil NE SR
	2 mil horizontal at SW SR 15C
	2 mil horizontal at NE SR 15D

2/5

Beam Test Recording

Specimen :	#5	Test Date:	
Recorded by:	Jiqiu		

Load, kip	Note
20	9 mil at NW PS
	15B → 3 mil
	20 mil at SW SR
	16A → 5 mil
	20 mil SE SR
	3 mil* 20E at NW SR
25	16 mil at W PS
	15D → 5 mil
	15B → 3 mil
	15C → 7 mil
	5 mil at SE PS
	25 mil at NW SR

3/5

Beam Test Recording

Specimen :	#5	Test Date:	
Recorded by:	Jiqiu		

Load, kip	Note
30	2 mil horizontal at SW PS → 30F
	16 mil at NW PS
	35 mil at SW SR
	3 mil horizontal at SW PS 30H
	C → 7 mil
	2 mil at SE SR 30J
	13 mil at CL
	2 mil at SE CL 30K
	35 mil at SE SR
	5 mil horizontal at NWSR 30L
35	45 mil SE PS
	20 mil at NW PS
	J → 6 mil
	40 mil at both W+E SR

4/5

Beam Test Recording

Specimen :	#X	Test Date:	
Recorded by:	Jiajun		

Load, kip	Note
35 cont	(B) → 20 mil
	2 mil out NE PS (35N)
	2 mil horizontal inside NE PS (35O)
	2 mil ... out NW PS (35P)
	(D) → 13 mil
40	35 mil at WPS
	(B) → 35 mil
	(D) → 3 mil
	16 mil East of CL
	(J) → 18 mil
	15 mil at E SR
	5 mil horizontal at CL (40R)
unload	

Load every 5 kip dial gage reading

5/5

Beam Test Recording

Specimen :	#5	Test Date:	
Recorded by:	Jiqiu		

Load, kip	Note
reload 20 kip	(L) extended about 2 in
reload 30 kip	(J) extended 1 ft
	5mil horizontal out NE PS (30' S)
	2mil h. at SE PS (30' T)
reload 35'	2mil horizontal out NE PS (35' V)
	2mil horizontal ind. SW SP (35' U)
reload 40'	(D) widen to 25 mil
46*	fail (flexural failure)

Beam Test Recording

Specimen :	#6	Test Date:	6/20/12
Recorded by:	Jiqiu		

Load, kip	Note
5	
10	2 mil SE SR
	2 mil NE PS
15	5 mil out NW PS
	7 mil out SE PS
	9 mil inside NW PS
	3 mil out of NW SR
	2 mil CL
	2 mil West of N CL
	4 mil out SW PS
20	10 mil out NW PS
	13 mil out SW SR
	10 mil out NW PS
	3 mil out NW SR
	2 mil horizontal inside SE SR 20A
	2 mil " " outside SE SR 20B

2

Beam Test Recording

Specimen :	#6	Test Date:	
Recorded by:	Jiqiu		

Load, kip	Note	
25	2 mil horizontal out SW SR	25Q
	3 mil out NE PS	
	2 mil horizontal Out NW SR	25P
	3 mil horizontal out NE SR	25E
	16 mil out SE PS	
30	22N + 20B connected	
	20 mil out SW SR	
	D widen 5mil	
	B → 5mil	
		30
	2 mil inside NE SR	
		30
	2 mil out SW PS	
		30
	2 mil out NE PS	

Beam Test Recording

Specimen :	#6	Test Date:	
Recorded by:	Jiqiu		

Load, kip	Note
35	C expand to 5 mil
	25 mil and NW PS
	30 mil at SW SR
	2mil horizontal inside SW SR 35 I
	D → 13 mil
	I + C connected
	E + H connected 20 mil
	2mil horizontal NE PS 35 J
	2mil at SW CL 35 K
	2mil east of S CL 35 L
	13mil on top of CL
	2mil hor at SE PS 35M

4

Beam Test Recording

Specimen :	#6	Test Date:	
Recorded by:	Jiqiu		

Load, kip	Note
40	
	A → 2mil
	2mil out NW SR 40N
	3mil SW PS 40O
	E → 3mil
	2 mil at the cold joint (top face)
	A+D merged
	2mil hori N of CL 40P
	3mil out of NW PS 40Q
	2mil out NW PS 40R
	1mil out SE PS 40S
	2mil out NE PS 40T

Beam Test Recording

5

Specimen :	#6	Test Date:	
Recorded by:	Jiqiu		

Load, kip	Note
unload	
reload	
10	
reload	
20	A + D merged
	H extended line
reload	
30'	2 mil horizontal inside SW SP 30'U
	1 mil - SW SR 30'V
	1 mil - at S CL 30'W
reload	
35'	
q.	

6

Beam Test Recording

Specimen :	#6	Test Date:	
Recorded by:	Jiqiu		

Load, kip	Note
reload	
40'	(U) extended 2 in
	(I) 〃 〃 4 in
50'	fail

Appendix I: Certificates of calibration for laboratory apparatus

Dial Gauge Verification:

Date Performed: 06/11/12

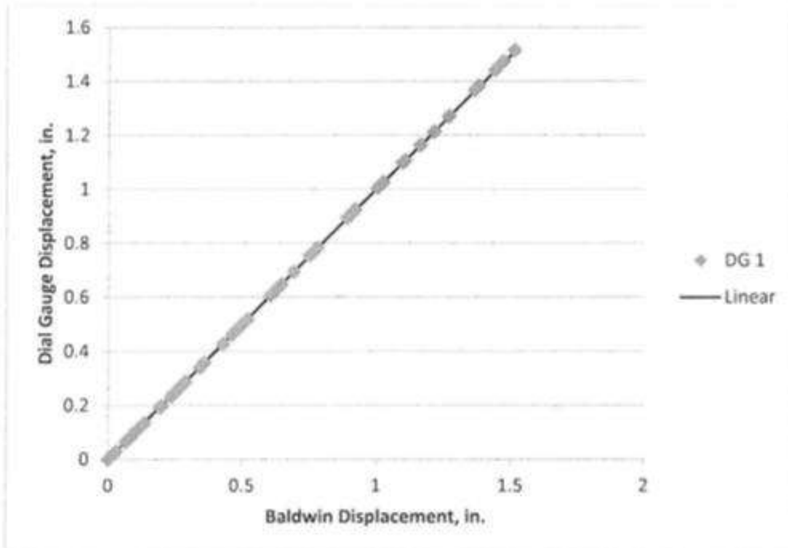
Operator: Matt O'Reilly

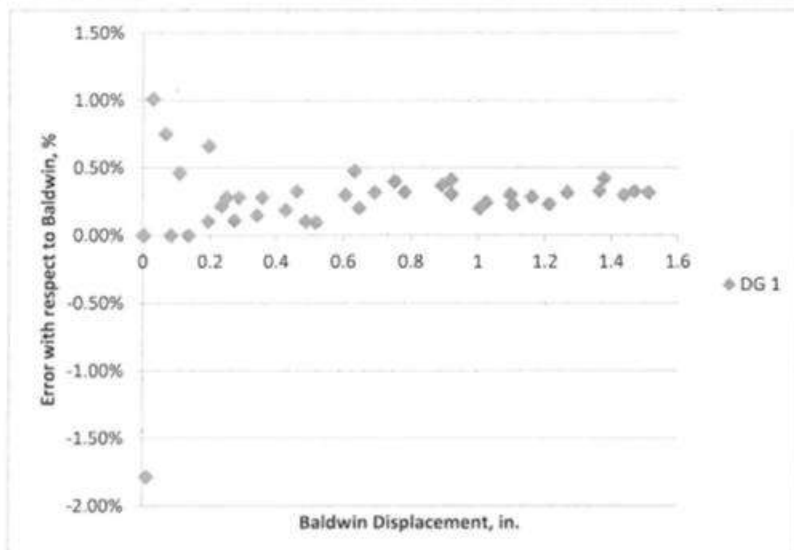
Test Frame ID: 472961 (120k Baldwin)

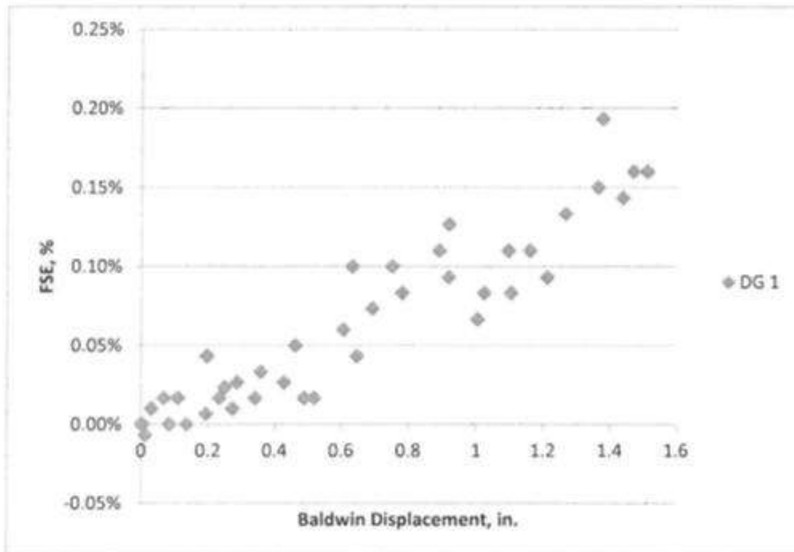
Test Frame Displacement
Calibration Certificate #: 1060726 11122608

Verification Data Attached.

ID:	DG1		
Baldwin	Gauge	% variation	FSE, %
0	0	0.00%	0.00%
0.0112	0.011	-1.79%	-0.01%
0.0297	0.03	1.01%	0.01%
0.083	0.083	0.00%	0.00%
0.135	0.135	0.00%	0.00%
0.1967	0.198	0.66%	0.04%
0.2493	0.25	0.28%	0.02%
0.2862	0.287	0.28%	0.03%
0.357	0.358	0.28%	0.03%
0.4605	0.462	0.33%	0.05%
0.5175	0.518	0.10%	0.02%
0.6457	0.647	0.20%	0.04%
0.6918	0.694	0.32%	0.07%
0.7805	0.783	0.32%	0.08%
0.9192	0.923	0.41%	0.13%
1.004	1.006	0.20%	0.07%
1.0957	1.099	0.30%	0.11%
1.1597	1.163	0.28%	0.11%
1.266	1.27	0.32%	0.13%
1.3772	1.383	0.42%	0.19%
1.4377	1.442	0.30%	0.14%
1.5112	1.516	0.32%	0.16%
1.4692	1.474	0.33%	0.16%
1.3635	1.368	0.33%	0.15%
1.2112	1.214	0.23%	0.09%
1.1035	1.106	0.23%	0.08%
1.0235	1.026	0.24%	0.08%
0.9192	0.922	0.30%	0.09%
0.8907	0.894	0.37%	0.11%
0.751	0.754	0.40%	0.10%
0.632	0.635	0.47%	0.10%
0.6052	0.607	0.30%	0.06%
0.4875	0.488	0.10%	0.02%
0.4272	0.428	0.19%	0.03%
0.3405	0.341	0.15%	0.02%
0.2727	0.273	0.11%	0.01%
0.2335	0.234	0.21%	0.02%
0.1938	0.194	0.10%	0.01%
0.1085	0.109	0.46%	0.02%
0.0665	0.067	0.75%	0.02%
0.003	0.003	0.00%	0.00%
0	0		0.00%

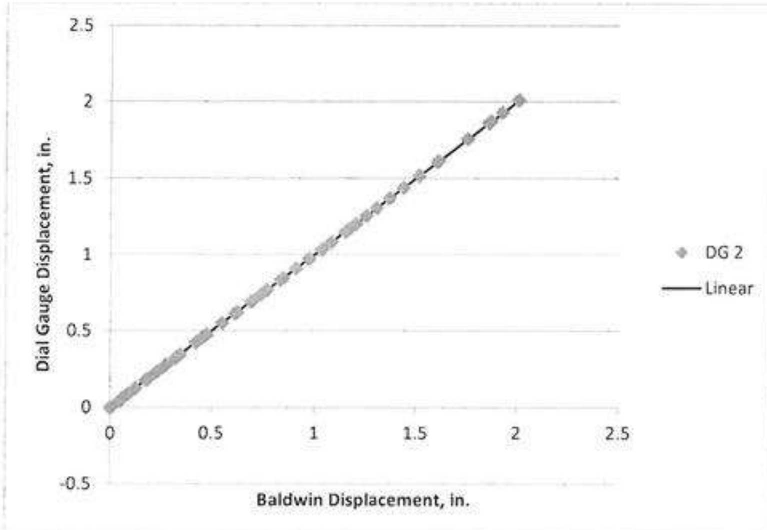

1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
Dial Gauge Displacement, in.
Baldwin Displacement, in.
DG 1
Linear


1.50%
1.00%
0.50%
0.00%
-0.50%
-1.00%
-1.50%
-2.00%
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
Error with respect to Baldwin, %
Baldwin Displacement, in.
DG 1

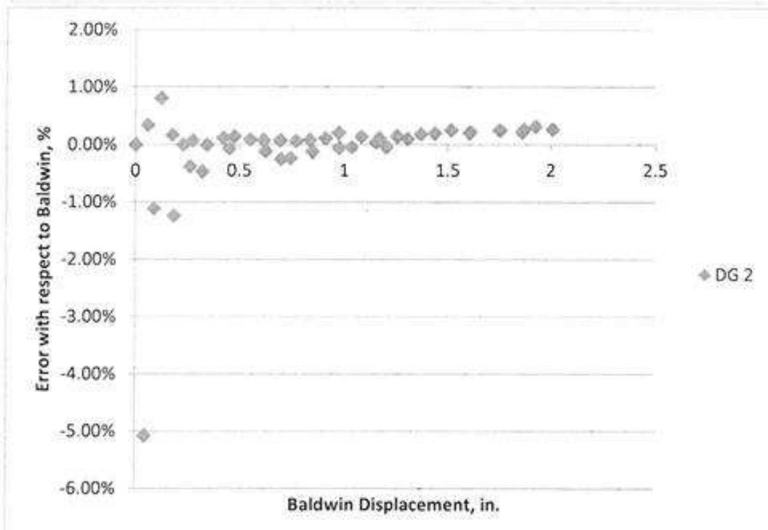


0.25%
0.20%
0.15%
0.10%
0.05%
0.00%
-0.05%
FSE, %
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
Baldwin Displacement, in.
DG 1

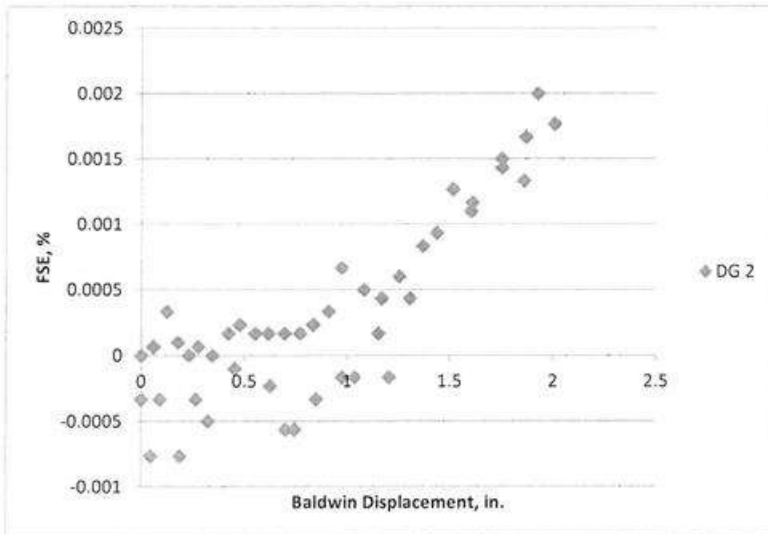
ID:	DG2		
Baldwin	Gauge	% variation	FSE, %
0	0	0.00%	0
0.0588	0.059	0.34%	6.67E-05
0.124	0.125	0.81%	0.000333
0.1777	0.178	0.17%	1E-04
0.23	0.23	0.00%	0
0.2758	0.276	0.07%	6.67E-05
0.345	0.345	0.00%	0
0.4225	0.423	0.12%	0.000167
0.4763	0.477	0.15%	0.000233
0.5515	0.552	0.09%	0.000167
0.6145	0.615	0.08%	0.000167
0.6935	0.694	0.07%	0.000167
0.7685	0.769	0.07%	0.000167
0.8333	0.834	0.08%	0.000233
0.909	0.91	0.11%	0.000333
0.972	0.974	0.21%	0.000667
1.0815	1.083	0.14%	0.0005
1.1677	1.169	0.11%	0.000433
1.2542	1.256	0.14%	0.0006
1.3695	1.372	0.18%	0.000833
1.5152	1.519	0.25%	0.001267
1.6037	1.607	0.21%	0.0011
1.7527	1.757	0.25%	0.001433
1.869	1.874	0.27%	0.001667
2.0067	2.012	0.26%	0.001767
1.925	1.931	0.31%	0.002
1.86	1.864	0.22%	0.001333
1.7515	1.756	0.26%	0.0015
1.6095	1.613	0.22%	0.001167
1.4372	1.44	0.19%	0.000933
1.3067	1.308	0.10%	0.000433
1.2035	1.203	-0.04%	-0.000167
1.1515	1.152	0.04%	0.000167
1.0355	1.035	-0.05%	-0.000167
0.9735	0.973	-0.05%	-0.000167
0.847	0.846	-0.12%	-0.000333
0.7417	0.74	-0.23%	-0.000567
0.6967	0.695	-0.24%	-0.000567
0.6237	0.623	-0.11%	-0.000233
0.4523	0.452	-0.07%	-1E-04
0.3235	0.322	-0.46%	-0.0005
0.264	0.263	-0.38%	-0.000333
0.1863	0.184	-1.23%	-0.000767
0.09	0.089	-1.11%	-0.000333
0.0453	0.043	-5.08%	-0.000767
0	-0.001		-0.000333



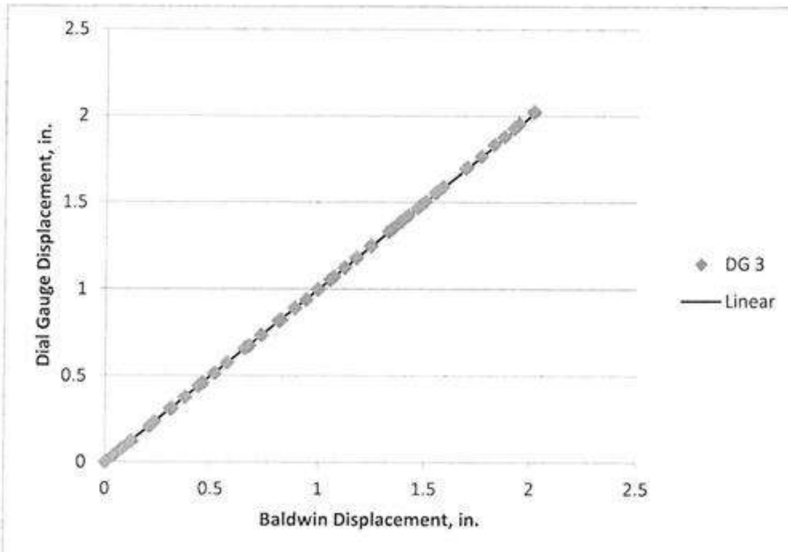
2.5
2
1.5
1
0.5
0
-0.5
0
0.5
1
1.5
2
2.5
Dial Gauge Displacement, in.
Baldwin Displacement, in.
DG 2
Linear

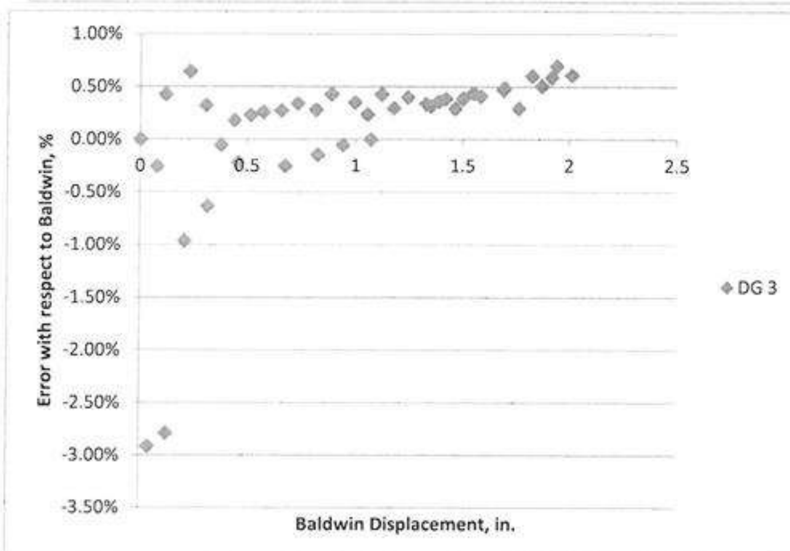


2.00%
1.00%
0.00%
-1.00%
-2.00%
-3.00%
-4.00%
-5.00%
-6.00%
0
0.5
1
1.5
2
2.5
Error with respect to Baldwin, %
Baldwin Displacement, in.
DG 2


0.0025
0.002
0.0015
0.001
0.0005
0
-0.0005
-0.001
FSE, %
0
0.5
1
1.5
2
2.5
Baldwin Displacement, in.
DG 2

Baldwin	Gauge	% variation	FSE, %
ID:	DG3		
0	0	0.00%	0.00%
0.0782	0.078	-0.26%	-0.01%
0.1175	0.118	0.43%	0.02%
0.2325	0.234	0.65%	0.05%
0.308	0.309	0.32%	0.03%
0.3782	0.378	-0.05%	-0.01%
0.4402	0.441	0.18%	0.03%
0.5158	0.517	0.23%	0.04%
0.5745	0.576	0.26%	0.05%
0.6572	0.659	0.27%	0.06%
0.7325	0.735	0.34%	0.08%
0.8157	0.818	0.28%	0.08%
0.8872	0.891	0.43%	0.13%
0.9955	0.999	0.35%	0.12%
1.0535	1.056	0.24%	0.08%
1.1202	1.125	0.43%	0.16%
1.243	1.248	0.40%	0.17%
1.3285	1.333	0.34%	0.15%
1.387	1.392	0.36%	0.17%
1.4647	1.469	0.29%	0.14%
1.5482	1.555	0.44%	0.23%
1.69	1.698	0.47%	0.27%
1.7618	1.767	0.30%	0.17%
1.8705	1.88	0.51%	0.32%
1.9405	1.954	0.70%	0.45%
2.0117	2.024	0.61%	0.41%
1.9177	1.929	0.59%	0.38%
1.824	1.835	0.60%	0.37%
1.6947	1.703	0.49%	0.28%
1.5825	1.589	0.41%	0.22%
1.5002	1.506	0.39%	0.19%
1.4185	1.424	0.39%	0.18%
1.3507	1.355	0.32%	0.14%
1.1765	1.18	0.30%	0.12%
1.07	1.07	0.00%	0.00%
0.9405	0.94	-0.05%	-0.02%
0.8242	0.823	-0.15%	-0.04%
0.6757	0.674	-0.25%	-0.06%
0.458	0.457	-0.22%	-0.03%
0.316	0.314	-0.63%	-0.07%
0.208	0.206	-0.96%	-0.07%
0.1255	0.122	-2.79%	-0.12%
0.0412	0.04	-2.91%	-0.04%
0	0		


2.5
2
1.5
1
0.5
0
0
0.5
1
1.5
2
2.5
Dial Gauge Displacement, in.
Baldwin Displacement, in.
DG 3
Linear


1.00%
0.50%
0.00%
-0.50%
-1.00%
-1.50%
-2.00%
-2.50%
-3.00%
-3.50%
0
0.5
1
1.5
2
2.5
Error with respect to Baldwin, %
Baldwin Displacement, in.
DG 3

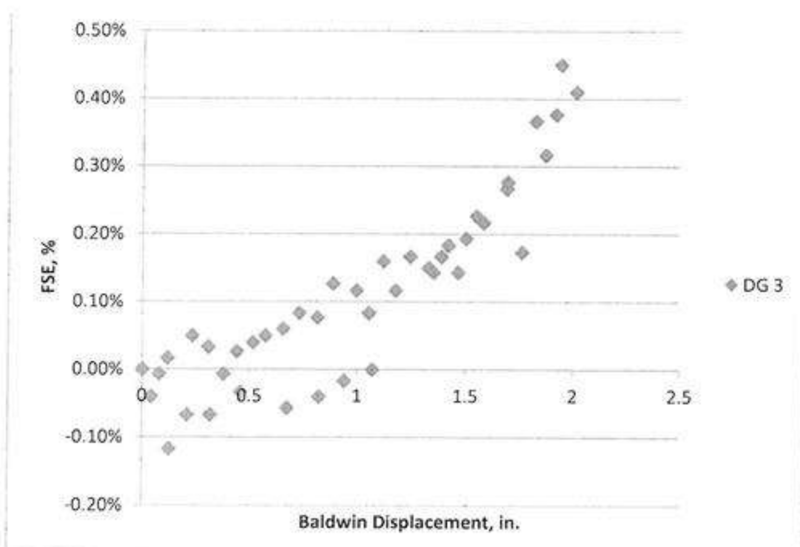
Slump Cone Verification

Date: 6/13/2012

Measured by: Matt O'Reilly

Measurements (in.)

	1	2	3	Avg	ASTM Spec.
Diameter at:					
Base	7.96	7.94	8.04	7.98	8 ± 0.125
Top	3.91	4.01	3.96	3.96	4 ± 0.125
Height	12.04	12.02	12.03	12.03	12 ± 0.125



Unit Weight Container Verification

Date: 6/13/2012

Measured by: Matt O'Reilly

Measurements (in.)

	1	2	3	Avg
Diameter	8.00	7.98	8.00	7.99
Height	8.50	8.56	8.53	8.53

Volume ($in.^3$) 428.1

Volume (ft^3) 0.2477 ASTM: Volume of container must exceed 95% of needed capacity

CERTIFICATE OF CALIBRATION

ISSUED BY : INSTRON CALIBRATION LABORATORY

DATE OF ISSUE : 26-Jul-2011 CERTIFICATE NUMBER: 106072611122608

NVLAP Lab code: 200301-0

INSTRON

Instron
825 University Avenue
Norwood, MA 02062-2643
Telephone: (800) 473 - 7838
Fax: (781) 575 - 5755
Email: service_requests@instron.com

APPROVED SIGNATORY

John Weiss

Type of Calibration: Displacement
Relevant Standard: astm e2309
Date of Calibration: 26-Jul-2011

Customer Requested Due Date: 26-Jul-2012

Customer		**Machine**	
	UNIVERSITY OF KANSAS 1032 LEARNED HALL 15TH ST. LAWRENCE, KS 66045	Serial No :	472961
		Make :	INSTRON
		Model :	55R120BTE

P.O. Number : SQ00003644
Contact : JIM WEAVER

Readout Verified

1. Digital Readout (in)

Certification Statement

This certifies that the displacements verified with machine indicator 1 (listed above) were verified by Instron in accordance with ASTM E2309-05 (Follow-the-Displacement Method) and Instron work instruction ICA-8-07.

The testing machine was verified on-site at customer location. Adjustments are noted in the comments section of this report with a reference to the "As Found" data.

The verification and equipment used conform to a controlled Quality Assurance program which meets the specifications outlined in ANSI/NCSL Z540-1, ISO 10012, ISO 9001:2000, and ISO/IEC 17025:2005. The Instron measurement equipment used for verification is traceable to NIST.

Summary of Results

Indicator 1 - Digital Readout (in)

Verified Range (in)	Max Error (in)	Max Error (%)	Max Repeat Error (in)	Max Repeat Error (%)	System Class*	Resolution (in)	Resolution Class	ASTM Lower Limit (in)
1 - 5	-0.0130	-0.368	0.0027	0.147	A	.0001	A	1

**System Class is derived from assessment of the following: error, repeatability, resolution, and standard device classification.*

The results indicated on this certificate and report relate only to the items verified. If there are methods or data included that are not covered by the NVLAP accreditation it will be identified in the comments. Any limitations of use as a result of this verification will be indicated in the comments. This report must not be used to claim product endorsement by NVLAP or the United States government. This report shall not be reproduced, except in full, without the approval of Instron.

CalproSDS version 3.3

CERTIFICATE OF CALIBRATION

ISSUED BY : INSTRON CALIBRATION LABORATORY

DATE OF ISSUE : **26-Jul-2011** CERTIFICATE NUMBER: **106072611122608**

Direction of Displacement : Ascending

Datapoint Summary - Indicator 1 - Digital Readout (in)

Suggested Value (in)	Run 1 Error (in)	Run 1 Error (%)	Run 2 Error (in)	Run 2 Error (%)	Run 3 Error (in)	Run 3 Error (%)	Repeat Error (in)	Uncertainty (in)*	Coverage Factor = k
1	-0.0036	-0.348	-0.0021	-0.201	-0.0031	-0.301	0.0015	0.0020	2.36
2	-0.0075	-0.368	-0.0062	-0.308	-0.0064	-0.313	0.0013	0.0019	2.26
3	-0.0094	-0.313	-0.0095	-0.311	-0.0085	-0.280	0.0010	0.0018	2.26
4	-0.0130	-0.321	-0.0113	-0.279	-0.0103	-0.255	0.0027	0.0036	2.78
5	-0.0101	-0.200	-0.0078	-0.154	-0.0089	-0.177	0.0023	0.0031	2.57

**The reported expanded uncertainty of measurement is based on a combined uncertainty multiplied by a coverage factor k to provide a level of confidence of approximately 95 %.*

Runs 1 and 2 are performed to comply with the requirements of ASTM E2309, run 3 is performed to calculate the uncertainty of measurement.

Data - Indicator 1 - Digital Readout (in)

Temperature at start of verification : 81.8 °F

Suggested Value	Run 1 Applied	Run 1 Indicated	Error Class	Run 2 Applied	Run 2 Indicated	Error Class	Run 3 Applied	Run 3 Indicated
1	1.0331	1.0295	A	1.0456	1.0435	A	1.0306	1.0275
2	2.0395	2.0320	A	2.0162	2.0100	A	2.0451	2.0387
3	3.0031	2.9937	A	3.0587	3.0492	A	3.0335	3.0250
4	4.0445	4.0315	A	4.0455	4.0342	A	4.0405	4.0302
5	5.0471	5.0370	A	5.0583	5.0505	A	5.0399	5.0310

For runs 1 and 2: the worst Resolution Class is A and the worst Repeatability Class is A.

Temperature at end of verification : 82.0 °F

Starting Point of crosshead : 10 in

Verification Equipment

Make/Model	Serial No.	Description	Cal Agency	Uncertainty of Calibration	Resolution	Cal Date	Due Date
Boeckeler DLG	6894	Linear Gage	A.A. JANSON	.000147 in	.0001 in	7-Dec-09	7-Dec-11
EXTECH 445580	956905	Thermometer	SYPRIS	.5 °F	.1 °F	11-Sep-09	11-Sep-11

The standards used for this verification are traceable to NIST.

CalproSDS version 3.3

CERTIFICATE OF CALIBRATION

ISSUED BY : INSTRON CALIBRATION LABORATORY

DATE OF ISSUE : **26-Jul-2011** CERTIFICATE NUMBER: **106072611122608**

Comments

Verified By: John Weiss

Senior Field Service Engineer

CERTIFICATE OF CALIBRATION

ISSUED BY: INSTRON CALIBRATION LABORATORY

DATE OF ISSUE: 26-Jul-11 CERTIFICATE NUMBER: **10607261110339**

NVLAP

Lab code 200301-0

INSTRON

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Fax: (781) 575-5750
Email: service_requests@instron.com

APPROVED SIGNATORY

John Weiss

Type of Calibration: Force
Relevant Standard: ASTM E4-10
Date of Calibration: 26-Jul-11
Customer Requested Due Date: 26-Jul-12

Customer

Name:	UNIVERSITY OF KANSAS
Address:	1032 Learned Hall, 15th St. Lawrence, KS 66045 WEAVERHJ@KU.EDU
P.O./Contract No.:	SQ00003644
Contact:	JIM WEAVER

Machine

Manufacturer:	BALDWIN TATE EMERY
Serial Number:	472961
System ID:	55R120BTE472961
Range Type:	Single

Transducer

Manufacturer:	BALDWIN TATE EMERY
Transducer ID:	472961
Capacity:	120000 lbf
Type:	Compression

Classification

1. Digital Readout - PASSED

Certification Statement

This certifies that the forces verified with machine indicator(s) (listed above) that passed are WITHIN ± 1 % accuracy, 1 % repeatability, and zero return tolerance.
All machine indicators were verified on-site at customer location by Instron in accordance with ASTM E4.
The certification is based on runs 1 and 2 only. A third run is taken to satisfy uncertainty requirements according to ISO 17025 specifications.
The verification and equipment used conform to a controlled Quality Assurance program which meets the specifications outlined in ANSI/NCSL Z540-1, ISO 10012, ISO 9001:2008 and ISO/IEC 17025:2005.

Method

The testing machine was verified in the 'as found' condition with no adjustments carried out.

Instron CalproCR Version 3.21

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
106072611103339

Summary of Results

Temperature at start of verification: 81.60 °F.

Indicator 1. - Digital Readout (lbf)

Range Full Scale (%)	Tested Force Range (lbf)	Mode	ASTM E4 Max Error (%)	ASTM E4 Max Repeat Error (%)	Zero Return	Resolution (lbf)	ASTM E4 Lower Limit (lbf)
100	-1192.4 to -119488.2	C	0.65	0.06	Pass	1	200

Temperature at end of verification: 81.80 °F.

Data Point Summary - Indicator 1. - Digital Readout (lbf)

COMPRESSION

% of Range	Run 1 Error (%)	Run 2 Error (%)	Run 3 Error (%)	ASTM E4 Repeat Error (%)	Relative Uncertainty* (%)	Uncertainty of Measurement* (± lbf)
100% Range (Full Scale: -119488.2 lbf)						
1	0.64	0.62	0.61	0.02	0.14	1.670
2	0.64	0.65	0.65	0.01	0.13	3.172
4	0.59	0.59	0.64	0.00	0.14	6.441
7	0.57	0.51	0.42	0.06	0.17	14.460
10	0.55	0.50	0.42	0.05	0.17	19.943
20	0.52	0.50	0.41	0.02	0.16	39.173
40	0.53	0.49	0.47	0.04	0.15	73.077
70	0.46	0.44	0.45	0.02	0.15	124.601
100	0.38	0.34	0.36	0.04	0.15	179.947

** The reported expanded uncertainty is based on a standard uncertainty multiplied by a coverage factor k = 2, providing a level of confidence of approximately 95%.*

Data - Indicator 1. - Digital Readout (lbf)

COMPRESSION

% of Range	Run 1 Indicated (lbf)	Run 1 Applied (lbf)	Run 2 Indicated (lbf)	Run 2 Applied (lbf)	Run 3 Indicated (lbf)	Run 3 Applied (lbf)
100% Range (Full Scale: -119488.2 lbf)						
0 Return	1		3		2	
1	-1200	-1192.4	-1200	-1192.6	-1200	-1192.75
2	-2400	-2384.8	-2400	-2384.5	-2400	-2384.6
4	-4800	-4772.05	-4800	-4771.95	-4800	-4769.4
7	-8400	-8352.6	-8400	-8357.4	-8400	-8364.6
10	-12000	-11934.6	-12000	-11940.6	-12000	-11950.2
20	-24000	-23875.8	-24000	-23881.2	-24000	-23901
40	-48000	-47748	-48000	-47763.6	-48000	-47777.4
70	-84000	-83613.6	-84000	-83635.2	-84000	-83625
100	-119900	-119450.4	-119900	-119488.2	-119000	-118572.6

Instron CalproCR Version 3.21

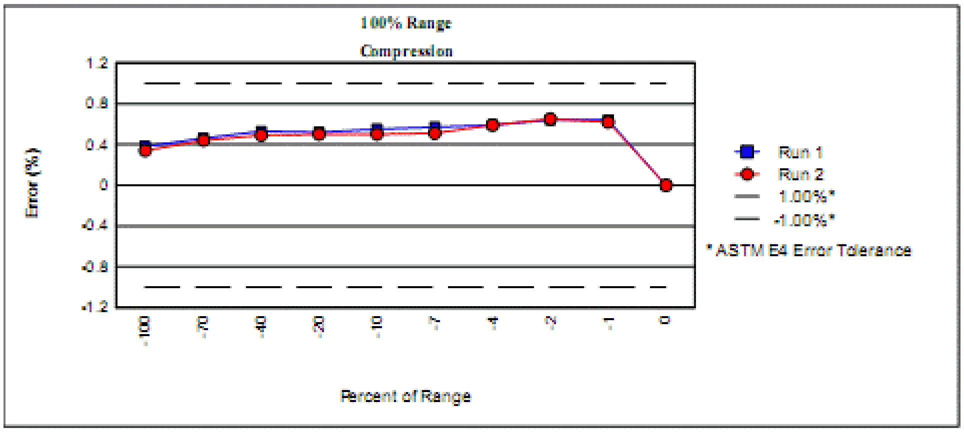
CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
106072611103339

The Return to Zero tolerance is ± the indicator resolution, 0.1 % of the maximum force verified in the range, or 1% of the lowest force verified in the range, whichever is greater.

Graphical Data - Indicator 1. - Digital Readout (lbf)



Verification Equipment

Make/Model	Serial Number	Description	Calibration Agency	Capacity	Cal Date	Cal Due
Extech 445580	956905	temp. indicator	Sypris	NA	11-Sep-09	11-Sep-11
Interface 9840	67002	force indicator	Interface	NA	29-Nov-10	29-Nov-12
Strainsense 930709D	930709D	load cell	Instron	120000 lbf	11-Nov-10	11-Nov-11
Flintec 198862	198862	load cell	Instron	12000 lbf	21-May-10	21-May-12

Verification Equipment Usage

Range Full Scale (%)	Standard Serial Number	Mode	Percent(s) of Range	Lower Limit for Standard Class A / A1 (lbf)
100	930709D	C	7/10/20/40/70/100	5000 / 5000
100	198862	C	1/2/4	200 / 200

Instron standards are traceable to NIST.

The standard Class A lower limit is used for systems with an accuracy of +/- 10% and the standard Class A1 lower limit is used for systems with an accuracy of +/- 0.5%.

Standard forces have been temperature compensated as necessary.

Comments

Instron CalproCR Version 3.21

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
106072611103339

Verified by: John Weiss
Field Service Engineer

NOTE: Clause 20 of ASTM E4 states; It is recommended that testing machines be verified annually or more frequently if required. In no case shall the time interval between verifications exceed 18 months (except for machines in which long term test runs beyond the 18 month period). Testing machines shall be verified immediately after repairs that may in any way affect the operation of the weighing system or values displayed. Verification is required immediately after a testing machine is relocated and where there is a reason to doubt the accuracy of the force indicating system, regardless of the time interval since the last verification.

CERTIFICATE OF CALIBRATION

ISSUED BY: INSTRON CALIBRATION LABORATORY

DATE OF ISSUE: 26-Jul-11   CERTIFICATE NUMBER: 106072611132239

Lab code 200301-0

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Email: service_requests@instron.com

APPROVED SIGNATORY

John Weiss

Type of Calibration: Force
Relevant Standard: ASTM E4-10
Date of Calibration: 26-Jul-11

Customer Requested Due Date: 26-Jul-12

Customer

Name:	UNIVERSITY OF KANSAS
Address:	1032 Learned Hall, 15th St. Lawrence, KS 66045 WEAVERHJ@KU.EDU
P.O./Contract No.:	SQ00003644
Contact:	JIM WEAVER

Machine

Manufacturer:	BALDWIN TATE EMERY
Serial Number:	64865
System ID:	60BTE64865
Range Type:	Single

Transducer

Manufacturer:	BALDWIN TATE EMERY
Transducer ID:	64865
Capacity:	60000 lbf
Type:	Compression

Classification

1. Dial Indicator - PASSED**

Certification Statement

This certifies that the forces verified with machine indicator(s) (listed above) that passed are WITHIN ± 1 % accuracy, 1 % repeatability, and zero return tolerance.
All machine indicators were verified on-site at customer location by Instron in accordance with ASTM E4.
The certification is based on runs 1 and 2 only. A third run is taken to satisfy uncertainty requirements according to ISO 17025 specifications.
The verification and equipment used conform to a controlled Quality Assurance program which meets the specifications outlined in ANSI/NCSL Z540-1, ISO 10012, ISO 9001:2008 and ISO/IEC 17025:2005.

**** within ± .5% accuracy and .5% repeatability.**

Method

The testing machine was verified in the 'as found' condition with no adjustments carried out.

Instron CalproCR Version 3.21

CERTIFICATE OF CALIBRATION	CERTIFICATE NUMBER: 10807 2811132239
NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0	

Summary of Results

Temperature at start of verification: 82.50 °F.

Indicator 1. - Dial Indicator (lbf)

Range Full Scale (%)	Tested Force Range (lbf)	Mode	ASTM E4 Max Error (%)	ASTM E4 Max Repeat Error (%)	Zero Return	Resolution (lbf)	ASTM E4 Lower Limit (lbf)
100	-597.1 to -60010.8	C	0.49	0.44	Pass	25	5000

Temperature at end of verification: 82.50 °F.

Data Point Summary - Indicator 1. - Dial Indicator (lbf)

COMPRESSION

% of Range	Run 1 Error (%)	Run 2 Error (%)	Run 3 Error (%)	ASTM E4 Repeat Error (%)	Relative Uncertainty* (%)	Uncertainty of Measurement* (± lbf)
100% Range (Full Scale: -60010.8 lbf)						
1	0.49	0.42	0.55	0.07	2.41	14.390
2	-0.30	0.14	-0.13	0.44	1.24	14.858
4	0.01	-0.21	-0.16	0.22	0.63	15.138
7	-0.11	-0.11	-0.13	0.00	0.37	15.474
10	0.02	0.10	0.04	0.08	0.29	17.211
20	-0.05	-0.20	-0.04	0.15	0.22	26.186
40	0.06	0.06	-0.02	0.00	0.17	40.546
70	0.07	0.04	-0.18	0.03	0.22	92.420
100	-0.02	-0.01	-0.01	0.01	0.15	90.613

** The reported expanded uncertainty is based on a standard uncertainty multiplied by a coverage factor k = 2, providing a level of confidence of approximately 95%.*

Data - Indicator 1. - Dial Indicator (lbf)

COMPRESSION

% of Range	Run 1 Indicated (lbf)	Run 1 Applied (lbf)	Run 2 Indicated (lbf)	Run 2 Applied (lbf)	Run 3 Indicated (lbf)	Run 3 Applied (lbf)
100% Range (Full Scale: -60010.8 lbf)						
0 Return	0		0		0	
1	-600	-597.1	-600	-597.5	-600	-596.7
2	-1200	-1203.6	-1200	-1198.35	-1200	-1201.55
4	-2400	-2399.65	-2400	-2404.95	-2400	-2403.95
7	-4200	-4204.55	-4200	-4204.7	-4200	-4205.5
10	-6000	-5998.8	-6000	-5994	-6000	-5997.6
20	-12000	-12006	-12000	-12024	-12000	-12005.4
40	-24000	-23985	-24000	-23986.2	-24000	-24004.8
70	-42000	-41969.4	-42000	-41984.4	-42000	-42073.8
100	-60000	-60010.8	-60000	-60008.4	-60000	-60006.6

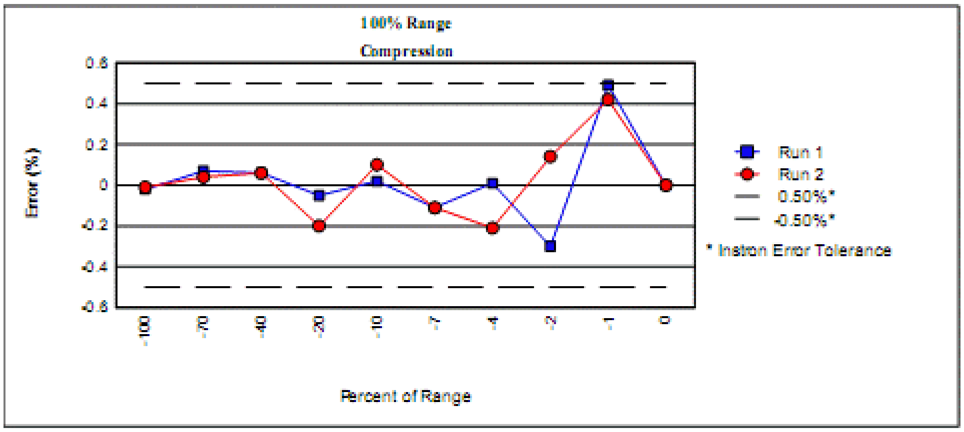
CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
106072611132239

The Return to Zero tolerance is ± the indicator resolution, 0.1 % of the maximum force verified in the range, or 1% of the lowest force verified in the range, whichever is greater.

Graphical Data - Indicator 1. - Dial Indicator (lbf)



Verification Equipment

Make/Model	Serial Number	Description	Calibration Agency	Capacity	Cal Date	Cal Due
Extech 445580	956905	temp. indicator	Sypris	NA	11-Sep-09	11-Sep-11
Interface 9840	67002	force indicator	Interface	NA	29-Nov-10	29-Nov-12
Strainsense 930709D	930709D	load cell	Instron	120000 lbf	11-Nov-10	11-Nov-11
Flintec 198862	198862	load cell	Instron	12000 lbf	21-May-10	21-May-12

Verification Equipment Usage

Range Full Scale (%)	Standard Serial Number	Mode	Percent(s) of Range	Lower Limit for Standard Class A / A1 (lbf)
100	930709D	C	10/20/40/70/100	5000 / 5000
100	198862	C	1/2/4/7	200 / 200

Instron standards are traceable to NIST.

The standard Class A lower limit is used for systems with an accuracy of +/- 10% and the standard Class A1 lower limit is used for systems with an accuracy of +/- 0.5%.

Standard forces have been temperature compensated as necessary.

Comments

Instron CalproCR Version 3.21

Verified by: John Weiss
Field Service Engineer

NOTE: Clause 20 of ASTM E4 states; It is recommended that testing machines be verified annually or more frequently if required. In no case shall the time interval between verifications exceed 18 months (except for machines in which long term test runs beyond the 18 month period). Testing machines shall be verified immediately after repairs that may in any way affect the operation of the weighing system or values displayed. Verification is required immediately after a testing machine is relocated and where there is a reason to doubt the accuracy of the force indicating system, regardless of the time interval since the last verification.

CERTIFICATE OF CALIBRATION

ISSUED BY: INSTRON CALIBRATION LABORATORY

DATE OF ISSUE: 27-Jul-11 CERTIFICATE NUMBER: **106072611142228B**

Lab code: 200301-0

Instron
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Email: service_requests@instron.com

APPROVED SIGNATORY

Justin Fry

Digitally signed by Justin Fry
DN: cn=Justin Fry, c=US, o=Instron
Reason: I am approving this document
Date: 2011.07.27 13:28:57 -04'00'

Type of Calibration: Force
Relevant Standard: ASTM E4-10
Date of Calibration: 26-Jul-11

Customer Requested Due Date: 26-Jul-12

Customer

Name:	UNIVERSITY OF KANSAS
Address:	1032 Learned Hall, 15th St. Lawrence, KS 66045 WEAVERHJ@KU.EDU
P.O./Contract No.:	SQ00003644
Contact:	JIM WEAVER

Machine

Manufacturer:	FORNEY / SATEC
Serial Number:	69084
System ID:	FORNEY / SATEC-69084
Range Type:	Single

Transducer

Manufacturer:	FORNEY / SATEC
Transducer ID:	69084
Capacity:	600000 lbf
Type:	Compression

Classification

1. Digital Readout - PASSED

Certification Statement

This certifies that the forces verified with machine indicator(s) (listed above) that passed are WITHIN ± 1 % accuracy, 1 % repeatability, and zero return tolerance.
All machine indicators were verified on-site at customer location by Instron in accordance with ASTM E4.
The certification is based on runs 1 and 2 only. A third run is taken to satisfy uncertainty requirements according to ISO 17025 specifications.
The verification and equipment used conform to a controlled Quality Assurance program which meets the specifications outlined in ANSI/NCSL Z540-1, ISO 10012, ISO 9001:2008 and ISO/IEC 17025:2005.

Method

The testing machine was verified in the 'as found' condition with no adjustments carried out.

Instron CalproCR Version 3.21

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
106072611142228B

Summary of Results

Temperature at start of verification: 90.10 °F.

Indicator 1. - Digital Readout (lbf)

Range Full Scale (%)	Tested Force Range (lbf)	Mode	ASTM E4 Max Error (%)	ASTM E4 Max Repeat Error (%)	Zero Return	Resolution (lbf)	ASTM E4 Lower Limit (lbf)
100	-5790.6 to -600312	C	-0.98	0.28	Pass	1	200

Temperature at end of verification: 90.80 °F.

Data Point Summary - Indicator 1. - Digital Readout (lbf)

COMPRESSION

% of Range	Run 1 Error (%)	Run 2 Error (%)	Run 3 Error (%)	ASTM E4 Repeat Error (%)	Relative Uncertainty* (%)	Uncertainty of Measurement* (± lbf)
100 % Range (Full Scale: -600312 lbf)						
1	-0.98	-0.70	-0.73	0.28	0.23	13.838
2	0.58	0.73	0.63	0.15	0.17	20.454
4	0.95	0.93	0.96	0.02	0.15	35.652
7	0.65	0.64	0.67	0.01	0.13	53.830
10	0.31	0.30	0.30	0.01	0.13	76.558
20	-0.24	-0.25	-0.23	0.01	0.13	153.924
40	-0.11	-0.12	-0.12	0.01	0.13	307.562
70	-0.23	-0.23	-0.23	0.00	0.13	534.640
100	-0.66	-0.68	-0.68	0.02	0.13	768.317

** The reported expanded uncertainty is based on a standard uncertainty multiplied by a coverage factor k = 2, providing a level of confidence of approximately 95%.*

Data - Indicator 1. - Digital Readout (lbf)

COMPRESSION

% of Range	Run 1 Indicated (lbf)	Run 1 Applied (lbf)	Run 2 Indicated (lbf)	Run 2 Applied (lbf)	Run 3 Indicated (lbf)	Run 3 Applied (lbf)
100 % Range (Full Scale: -600312 lbf)						
0 Return	-3		-91		47	
1	-6000	-6059.4	-5750	-5790.6	-6000	-6044.4
2	-11900	-11831.4	-11900	-11813.4	-11900	-11825.4
4	-23990	-23763.6	-23990	-23767.8	-24000	-23772
7	-42000	-41730	-42000	-41734	-42000	-41722
10	-60000	-59812	-59990	-59812	-59990	-59808
20	-120000	-120286	-119900	-120198	-120000	-120276
40	-240000	-240272	-240000	-240284	-240000	-240292
70	-420000	-420974	-420000	-420980	-420000	-420974
100	-596000	-599934	-596200	-600312	-596400	-600496

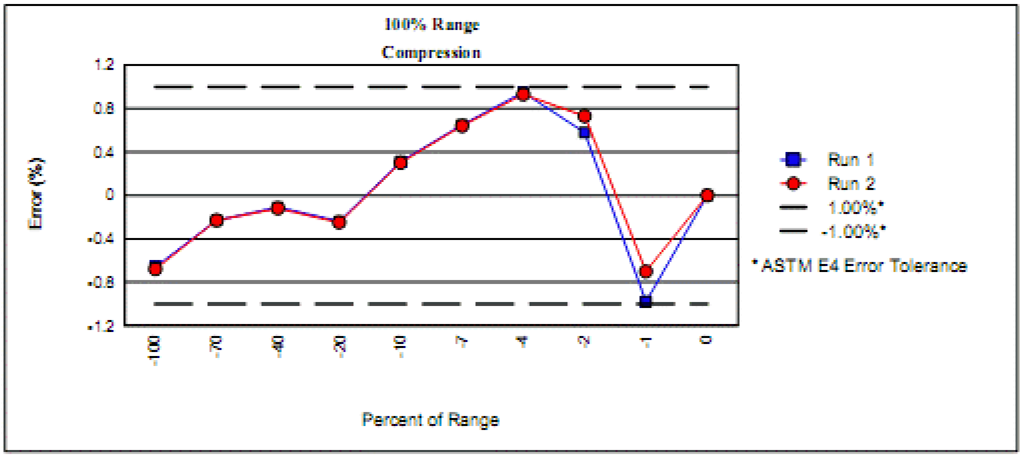
CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
106072611142228B

The Return to Zero tolerance is ± the indicator resolution, 0.1 % of the maximum force verified in the range, or 1% of the lowest force verified in the range, whichever is greater.

Graphical Data - Indicator 1. - Digital Readout (lbf)



Verification Equipment

Make/Model	Serial Number	Description	Calibration Agency	Capacity	Cal Date	Cal Due
Strainsense 3080402	3080402	load cell	Instron	600000 lbf	30-Sep-10	30-Sep-11
Strainsense 930709D	930709D	load cell	Instron	120000 lbf	11-Nov-10	11-Nov-11
Extech 445580	956905	temp. indicator	Sypris	NA	11-Sep-09	11-Sep-11
Interface 9840	67002	force indicator	Interface	NA	29-Nov-10	29-Nov-12

Verification Equipment Usage

Range Full Scale (%)	Standard Serial Number	Mode	Percent(s) of Range	Lower Limit for Standard Class A / A1 (lbf)
100	3080402	C	7/10/20/40/70/100	20000 / 20500
100	930709D	C	1/2/4	5000 / 5000

Instron standards are traceable to NIST.

The standard Class A lower limit is used for systems with an accuracy of +/- 1.0% and the standard Class A1 lower limit is used for systems with an accuracy of +/- 0.5%.

Standard forces have been temperature compensated as necessary.

Comments

This certificate replaces 106072611142228 to reflect the proper Machine Make/Serial Number.

Instron CalproCR Version 3.21

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
106072611142228B

Verified by: John Weiss
Field Service Engineer

NOTE: Clause 20 of ASTM E4 states; It is recommended that testing machines be verified annually or more frequently if required. In no case shall the time interval between verifications exceed 18 months (except for machines in which long term test runs beyond the 18 month period). Testing machines shall be verified immediately after repairs that may in any way affect the operation of the weighing system or values displayed. Verification is required immediately after a testing machine is relocated and where there is a reason to doubt the accuracy of the force indicating system, regardless of the time interval since the last verification.

CERTIFICATE OF CALIBRATION

ISSUED BY: INSTRON CALIBRATION LABORATORY

DATE OF ISSUE: 27-Jul-11 CERTIFICATE NUMBER: 106072711095017

NVLAP

Lab code 200301-0

INSTRON

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Fax: (781) 575-5750
Email: service_requests@instron.com

APPROVED SIGNATORY

John Weiss

Type of Calibration: Force
Relevant Standard: ASTM E4-10
Date of Calibration: 27-Jul-11

Customer Requested Due Date: 27-Jul-12

Customer

Name:	UNIVERSITY OF KANSAS
Address:	1032 Learned Hall, 15th St. Lawrence, KS 66045 WEAVERHJ@KU.EDU
P.O./Contract No.:	SQ00003644
Contact:	JIM WEAVER

Machine

Manufacturer:	FORNEY
Serial Number:	76125
System ID:	QC-50-106C76125
Range Type:	Multi

Transducer

Manufacturer:	FORNEY
Transducer ID:	76125
Capacity:	400000 lbf
Type:	Compression

Classification

1. Dial Indicator - PASSED

Certification Statement

This certifies that the forces verified with machine indicator(s) (listed above) that passed are WITHIN ± 1 % accuracy, 1 % repeatability, and zero return tolerance.
All machine indicators were verified on-site at customer location by Instron in accordance with ASTM E4.
The certification is based on runs 1 and 2 only. A third run is taken to satisfy uncertainty requirements according to ISO 17025 specifications.
The verification and equipment used conform to a controlled Quality Assurance program which meets the specifications outlined in ANSI/NCSL Z540-1, ISO 10012, ISO 9001:2008 and ISO/IEC 17025:2005.

Method

The testing machine was verified in the 'as found' condition with no adjustments carried out.

Instron CalproCR Version 3.21

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
10607271109501 7

Summary of Results

Temperature at start of verification: 89.00 °F.

Indicator 1. - Dial Indicator (lbf)

Range Full Scale (%)	Tested Force Range (lbf)	Mode	ASTM E4 Max Error (%)	ASTM E4 Max Repeat Error (%)	Zero Return	Resolution (lbf)	ASTM E4 Lower Limit (lbf)
100	-80030 to -400030	C	-0.40	0.36	Pass	500	100000
7.5	-6004.8 to -30018	C	-0.32	0.30	Pass	37.5	7500

Temperature at end of verification: 90.10 °F.

Data Point Summary - Indicator 1. - Dial Indicator (lbf)

COMPRESSION

% of Range	Run 1 Error (%)	Run 2 Error (%)	Run 3 Error (%)	ASTM E4 Repeat Error (%)	Relative Uncertainty* (%)	Uncertainty of Measurement* (± lbf)
100% Range (Full Scale: -400030 lbf)						
20	-0.04	-0.40	-0.32	0.36	0.44	353.694
25	0.00	-0.34	0.25	0.34	0.47	465.158
40	-0.08	-0.35	-0.45	0.27	0.31	500.675
60	0.08	-0.08	-0.22	0.16	0.25	590.853
80	-0.34	-0.27	0.05	0.07	0.29	916.889
100	-0.01	-0.02	-0.02	0.01	0.15	583.669
7.5% Range (Full Scale: -30018 lbf)						
20	-0.08	-0.11	-0.06	0.03	0.39	23.480
30	-0.01	-0.09	-0.28	0.08	0.33	29.288
40	-0.02	-0.32	-0.30	0.30	0.30	36.558
60	-0.18	-0.15	-0.70	0.03	0.41	73.152
80	0.07	0.04	-0.41	0.03	0.36	85.527
100	0.02	-0.06	0.16	0.08	0.21	62.974

** The reported expanded uncertainty is based on a standard uncertainty multiplied by a coverage factor k = 2, providing a level of confidence of approximately 95%.*

Data - Indicator 1. - Dial Indicator (lbf)

COMPRESSION

% of Range	Run 1 Indicated (lbf)	Run 1 Applied (lbf)	Run 2 Indicated (lbf)	Run 2 Applied (lbf)	Run 3 Indicated (lbf)	Run 3 Applied (lbf)
100% Range (Full Scale: -400030 lbf)						
0 Return	0		0		0	
20	-80000	-80030	-80000	-80322	-80000	-80256
25	-100000	-100002	-100000	-100346	-100000	-99754
40	-160000	-160132	-160000	-160556	-160000	-160730
60	-240000	-239820	-240000	-240192	-240000	-240540
80	-320000	-321086	-320000	-320856	-320000	-319830
100	-400000	-400030	-395000	-395064	-396000	-396068

Instron CalproCR Version 3.21

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
1060727110950 17

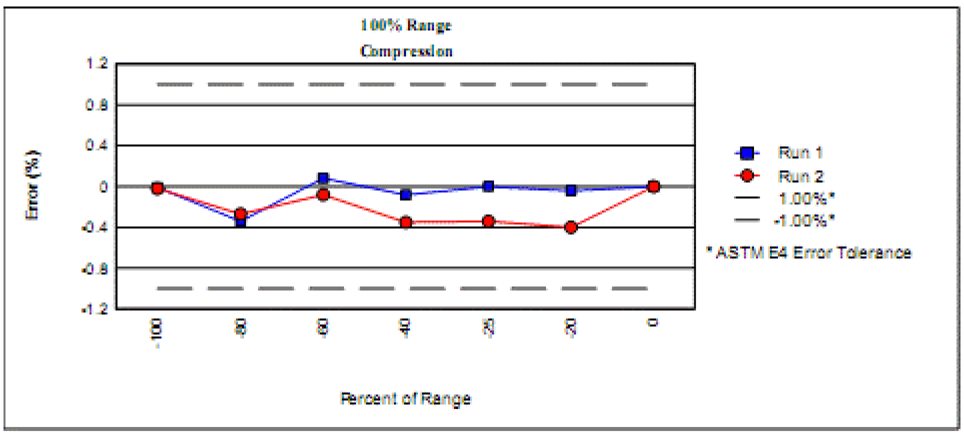
Data - Indicator 1. - Dial Indicator (lbf)

COMPRESSION

	Run 1		Run 2		Run 3	
% of Range	Indicated (lbf)	Applied (lbf)	Indicated (lbf)	Applied (lbf)	Indicated (lbf)	Applied (lbf)
7.5% Range (Full Scale: -30018 lbf)						
0 Return	0		0		0	
20	-6000	-6004.8	-6000	-6006.6	-6000	-6003.6
30	-9000	-9001.2	-9000	-9008.4	-9000	-9025.2
40	-12000	-12002.4	-12000	-12039	-12000	-12036
60	-18000	-18032.4	-18000	-18027	-18000	-18127.2
80	-24000	-23983.8	-24000	-23989.8	-24000	-24099.6
100	-30000	-29992.8	-30000	-30018	-30000	-29952.6

The Return to Zero tolerance is ± the indicator resolution, 0.1 % of the maximum force verified in the range, or 1% of the lowest force verified in the range, whichever is greater.

Graphical Data - Indicator 1. - Dial Indicator (lbf)



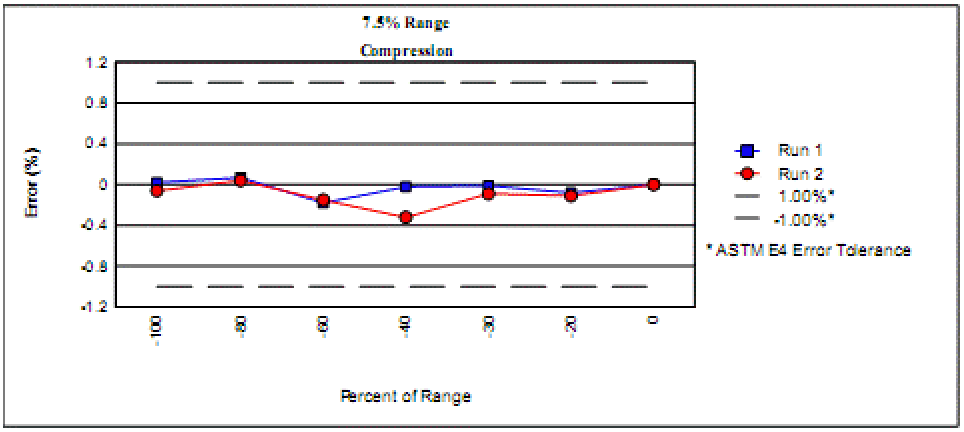
Instron CalproCR Version 3.21

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
10607271109 5017

Graphical Data - Indicator 1. - Dial Indicator (lbf)



Verification Equipment

Make/Model	Serial Number	Description	Calibration Agency	Capacity	Cal Date	Cal Due
Extech 445580	956905	temp. indicator	Sypris	NA	11-Sep-09	11-Sep-11
Interface 9840	67002	force indicator	Interface	NA	29-Nov-10	29-Nov-12
Strainsense 3080402	3080402	load cell	Instron	600000 lbf	30-Sep-10	30-Sep-11
Strainsense 930709D	930709D	load cell	Instron	120000 lbf	11-Nov-10	11-Nov-11

Verification Equipment Usage

Range Full Scale (%)	Standard Serial Number	Mode	Percent(s) of Range	Lower Limit for Standard Class A / A1 (lbf)
100	3080402	C	20/25/40/60/80/100	20000 / 20500
7.5	930709D	C	20/30/40/60/80/100	5000 / 5000

Instron standards are traceable to NIST.

The standard Class A lower limit is used for systems with an accuracy of +/- 10% and the standard Class A1 lower limit is used for systems with an accuracy of +/- 0.5%.

Standard forces have been temperature compensated as necessary.

Comments

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
10607271109501 7

Verified by: John Weiss
Field Service Engineer

NOTE: Clause 20 of ASTM E4 states; It is recommended that testing machines be verified annually or more frequently if required. In no case shall the time interval between verifications exceed 18 months (except for machines in which long term test runs beyond the 18 month period). Testing machines shall be verified immediately after repairs that may in any way affect the operation of the weighing system or values displayed. Verification is required immediately after a testing machine is relocated and where there is a reason to doubt the accuracy of the force indicating system, regardless of the time interval since the last verification.

CERTIFICATE OF CALIBRATION

ISSUED BY: INSTRON CALIBRATION LABORATORY

DATE OF ISSUE: 26-Jul-11 CERTIFICATE NUMBER: 106072611161214

NVLAP

Lab code 200301-0

INSTRON

Instron
825 University Avenue
Norwood, MA 02062-2643
Telephone: (800) 473-7838
Fax: (781) 575-5750
Email: service_requests@instron.com

APPROVED SIGNATORY

John Weiss

Type of Calibration: Force
Relevant Standard: ASTM E4-10
Date of Calibration: 26-Jul-11

Customer Requested Due Date: 26-Jul-12

Customer

Name:	UNIVERSITY OF KANSAS
Address:	1032 Learned Hall, 15th St. Lawrence, KS 66045 WEAVERHJ@KU.EDU
P.O./Contract No.:	SQ00003644
Contact:	JIM WEAVER

Machine

Manufacturer:	FORNEY
Serial Number:	82118
System ID:	QC-400C82118
Range Type:	Single

Transducer

Manufacturer:	FORNEY
Transducer ID:	82118
Capacity:	400000 lbf
Type:	Compression

Classification

1. Dial Indicator - PASSED

Certification Statement

This certifies that the forces verified with machine indicator(s) (listed above) that passed are WITHIN ± 1 % accuracy, 1 % repeatability, and zero return tolerance.
All machine indicators were verified on-site at customer location by Instron in accordance with ASTM E4.
The certification is based on runs 1 and 2 only. A third run is taken to satisfy uncertainty requirements according to ISO 17025 specifications.
The verification and equipment used conform to a controlled Quality Assurance program which meets the specifications outlined in ANSI/NCSL Z540-1, ISO 10012, ISO 9001:2008 and ISO/IEC 17025:2005.

Method

The testing machine was verified in the 'as found' condition with no adjustments carried out.

Instron CalproCR Version 3.21

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
10607281116 1214

Summary of Results

Temperature at start of verification: 93.20 °F.

Indicator 1. - Dial Indicator (lbf)

Range Full Scale (%)	Tested Force Range (lbf)	Mode	ASTM E4 Max Error (%)	ASTM E4 Max Repeat Error (%)	Zero Return	Resolution (lbf)	ASTM E4 Lower Limit (lbf)
100	-22454 to -377702	C	-0.72	0.40	Pass	250	50000

Temperature at end of verification: 93.30 °F.

Data Point Summary - Indicator 1. - Dial Indicator (lbf)

COMPRESSION

% of Range	Run 1 Error (%)	Run 2 Error (%)	Run 3 Error (%)	ASTM E4 Repeat Error (%)	Relative Uncertainty* (%)	Uncertainty of Measurement* (± lbf)
100% Range (Full Scale: -377702 lbf)						
5	-0.20	0.20	0.50	0.40	0.77	172.963
10	0.17	-0.15	0.31	0.32	0.47	187.799
20	0.61	0.56	-0.17	0.05	0.55	438.541
35	0.26	0.40	0.47	0.14	0.21	285.929
50	0.08	-0.25	-0.50	0.33	0.37	733.637
75	-0.27	-0.06	0.06	0.21	0.24	708.625
100	-0.72	-0.53	-0.33	0.19	0.26	987.667

** The reported expanded uncertainty is based on a standard uncertainty multiplied by a coverage factor k = 2, providing a level of confidence of approximately 95%.*

Data - Indicator 1. - Dial Indicator (lbf)

COMPRESSION

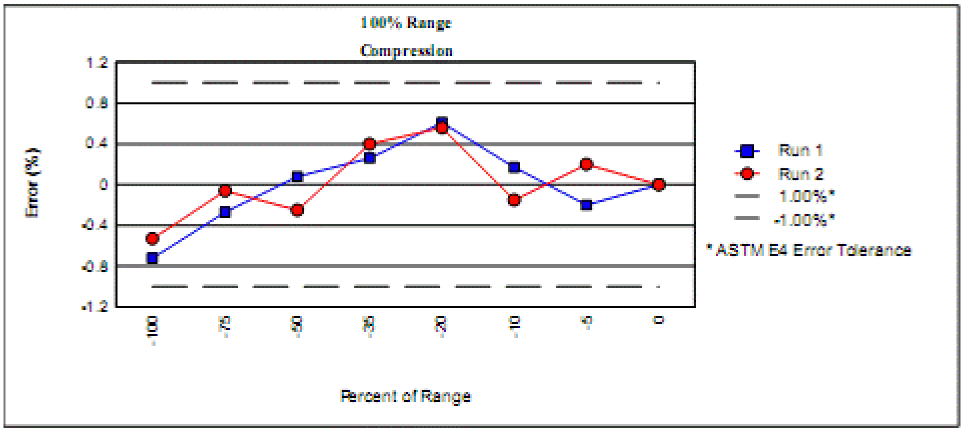
% of Range	Run 1 Indicated (lbf)	Run 1 Applied (lbf)	Run 2 Indicated (lbf)	Run 2 Applied (lbf)	Run 3 Indicated (lbf)	Run 3 Applied (lbf)
100% Range (Full Scale: -377702 lbf)						
0 Return	0		0		0	
5	-22500	-22546	-22500	-22454	-22500	-22388
10	-40000	-39934	-40000	-40060	-40000	-39878
20	-80000	-79516	-80000	-79554	-80000	-80134
35	-140000	-139638	-140000	-139448	-140000	-139346
50	-200000	-199834	-200000	-200510	-200000	-200998
75	-300000	-300798	-300000	-300174	-300000	-299822
100	-375000	-377702	-375000	-376984	-375000	-376230

The Return to Zero tolerance is ± the indicator resolution, 0.1 % of the maximum force verified in the range, or 1% of the lowest force verified in the range, whichever is greater.

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

Graphical Data - Indicator 1. - Dial Indicator (lbf)



Verification Equipment

Make/Model	Serial Number	Description	Calibration Agency	Capacity	Cal Date	Cal Due
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Interface 9840	67002	force indicator	Interface	NA	29-Nov-10	29-Nov-12
Strainsense 3080402	3080402	load cell	Instron	600000 lbf	30-Sep-10	30-Sep-11

Verification Equipment Usage

Range Full Scale (%)	Standard Serial Number	Mode	Percent(s) of Range	Lower Limit for Standard Class A / A1 (lbf)
100	3080402	C	5/10/20/35/50/75/100	20000 / 20500

Instron standards are traceable to NIST.

The standard Class A lower limit is used for systems with an accuracy of +/- 10% and the standard Class A1 lower limit is used for systems with an accuracy of +/- 0.5%.

Standard forces have been temperature compensated as necessary.

Comments

Verified by: John Weiss
Field Service Engineer

Instron CalproCR Version 3.21

NOTE: Clause 20 of ASTM E4 states; It is recommended that testing machines be verified annually or more frequently if required. In no case shall the time interval between verifications exceed 18 months (except for machines in which long term test runs beyond the 18 month period). Testing machines shall be verified immediately after repairs that may in any way affect the operation of the weighing system or values displayed. Verification is required immediately after a testing machine is relocated and where there is a reason to doubt the accuracy of the force indicating system, regardless of the time interval since the last verification.