

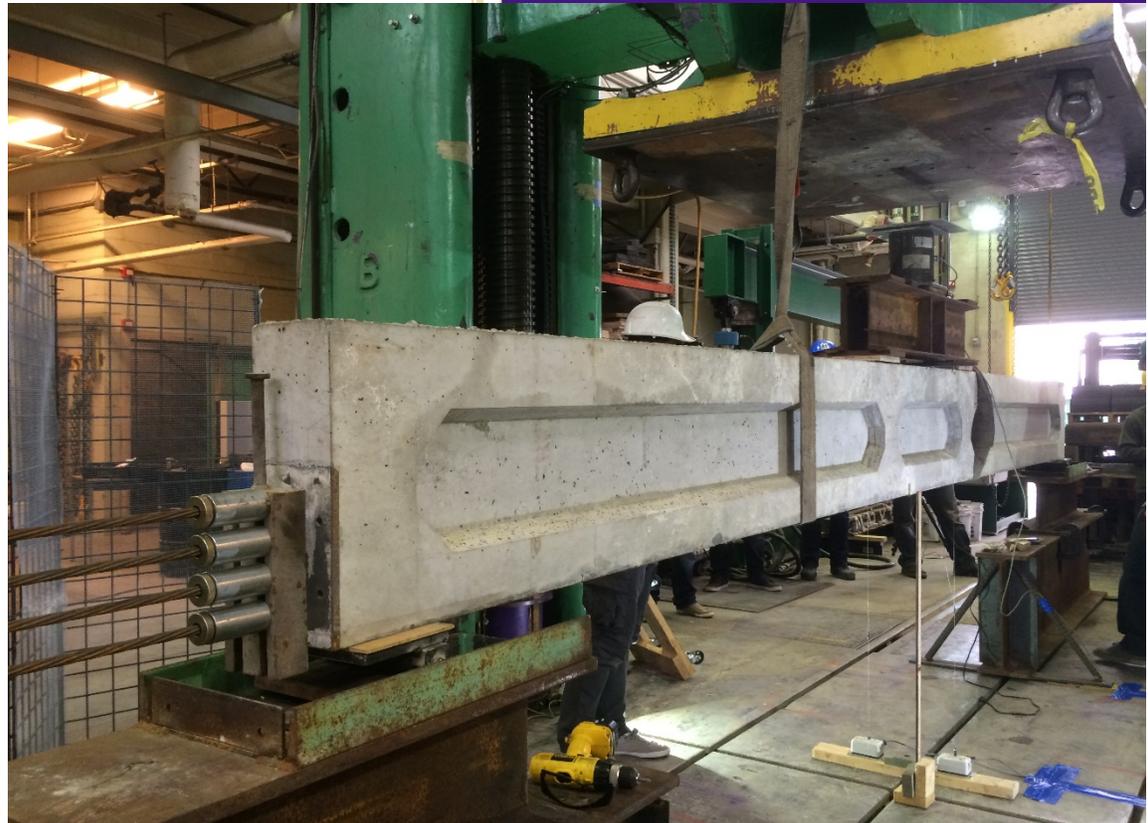


UNIVERSITY of WASHINGTON

Department of Civil Engineering
College of Engineering

2015

PCI Big Beam Report



**University of Washington's
Mr. Beam (Team #2)**

Faculty Advisor

Dr. John Stanton

PCI Producer Member

Concrete Technology Corporation

Student Members

Wai Lok Chung, Kok Wong, Si Kei Ngan, Yousif
Alshaba, Givens Lam



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Executive Summary

In 2015, the University of Washington's Mr. Beam Team entered the Big Beam Competition under the supervision of UW faculty advisor Professor John Stanton. The team was tasked with designing, constructing, and testing a prestressed concrete beam according to PCI Big Beam Competition criteria and while meeting ASTM and ACI standards.

The team comprised of three seniors and two juniors: Wai Lok Chung, Kok Wong, Si Kei Ngan, Givens Lam, and Yousif Alshaba, respectively. In the initial design stages, the seniors were enrolled in a reinforced concrete course, but no team members had any prior prestressed concrete experience. Throughout the project, the team learned from their faculty advisor, prestressed concrete lecture, and ACI 318-11. Some seniors would go on to take the prestressed concrete course the following quarter.

The team designed an I-Beam shaped beam of constant depth and cross-section with the exception of web stiffeners at the loading points and supports. To reinforce the beam, the team used WWF, two #3 reinforcing bars, and four prestressed strands with debonding of the middle 11'. The strands were partially debonded with the goal of reaching a higher deflection at peak load. A detailed description of the beam is given in the report's **Design Process** section.

After the design and construction stages were complete, the team worked on predicting and assessing the effect of the loads on the beam. Initial predictions were made during the design stage using hand calculations with the assistance of a spreadsheet. However, the greatest challenges lay in determining the strand stress and beam deflection at peak load because no suitable theory exists for performing those calculations.

The cracking load was established using conventional methods. The beam was analyzed as an uncracked member, and the external load needed to achieve the cracking stress was calculated. The two largest unknowns were the prestress loss and the true cracking strength of the concrete. However, the predicted cracking load was very close to the measured one.

For the ultimate load and corresponding deflection, the beam was treated as three rigid bodies (left end, middle and right end) connected by the tendon. For any given strand stress, the strand elongation—and, thus, the beam deflection—could be calculated. The challenge was in establishing the strand stress at ultimate conditions.

Initially, Equation (18-2) from ACI 318-11 was used. However, the result appeared too small—the strand stress was only slightly higher than the effective stress and the predicted deflection was only a fraction of an inch. The problem appeared to lie with the span/depth ratio, which for the beam was 11.3, but the ACI equation is based on test results for slabs, which are typically much more slender. This suggested that Eq (18-2) might give a poor estimate of f_{ps} for such a stocky member. Consequently, the same calculation was repeated by assuming the strand would yield and reach a total strain of 1%. This number was chosen arbitrarily, but some assumption was needed. The predictions and results are shown in the following table

Table 1. Predictions vs. Results

	Prediction	Results
Ultimate Load (kips)	40.7	40.8
Deflection at Ultimate Load (in)	6.69	2.23
Cracking Load (kips)	26.5	26.0

While load predictions were relatively accurate, deflection predictions were not. This may have been due to incorrect assumptions made during prediction calculations. A detailed discussion can be found in the **Results** section.



Acknowledgments

The team would like to express the deepest appreciation to those who have helped them prepare for competition.

First, the team would like to the support they received from the University of Washington. They would like to thank Dr. John Stanton, who guided them through the design process and subsequent analysis. He patiently and clearly explained concepts to those who had not yet learned all required for designing and constructing the beam. His passion for and experience in structural engineering are deeply appreciated.

The team would also like to thank graduate students, Mr. Matthew Sisley and Mr. Travis Thonstad. Sisley's willingness to help the team at every step was invaluable. Having participated in the 2014 Big Beam Competition, he shared what his team had done and used his experience to lead this year's team on aspects of beam construction and design. He sacrificed his own time to help them bend the WWF and prepare the prestressing strands. Thonstad's advice and guidance have also been invaluable. Having advised the 2014 team, he shared his experiences on the beam design process and what previous teams have encountered.

Additionally, individuals from Concrete Technology Corporation (CTC) have helped the team significantly. The team would like to thank Mr. Austin Maue, a design engineer at CTC. He reviewed shop drawings to ensure the construction process went according to the team's design. Moreover, he guided the team on assembling the shear reinforcement and helped solve a last minute problem they had before casting. For his experience and responsibility, the team would also like to thank Mr. Dave Chapman, the Chief Engineer at CTC. He helped check the team's work at the site to make sure that everything went smoothly. Moreover, the team would like to thank the crew at CTC for casting and prestressing the beam in an organized manner without delay.

The team would like to thank Davis Wire and Sumiden Wire Products Corporation for donating the WWF and strands, respectively.

Finally, the team would like to thank PCI and Sika for sponsoring the competition. The competition has been a challenging and unique experience from which team members will benefit in their future careers.

Design Process

This section will cover Mr. Beam’s design concept, flexural design, and shear design.

Design Concept

In initial meetings, the team went over prestressing concepts and competition rules, as well as brainstormed beam shapes. The team considered a “T,” “I,” and bulb-shaped cross-sections, as well as whether the beam should be prismatic or non-prismatic. After a few weeks, the team settled on an I-Beam of constant depth and cross-section with stiffeners at the loading points and supports. An I-Beam would resist torsional buckling better than a T-Beam while optimizing cost and weight. Due to construction issues faced by last year’s team, especially in terms of formwork construction, this year’s team chose a prismatic shape with straight prestressed strands.

The strands were designed to be unbonded to increase the deflection at peak load. The concept was that a significant strain increase would occur over a greater length of strand, leading to more strand elongation—and so more deflection—than would be the case for a bonded system

Additionally, to reduce weight and costs, the team designed the beam to be 18’ long instead of the allowed 19’. Because of the shortened beam and unbonded strands, steel plates and strand vices were added to the beam ends for additional strand anchorage.

Flexural Design

Several iterations were analyzed to select the appropriate number of strands, the location of the strands, and the size of the beam. The final beam cross-section dimension and reinforcement locations are shown below in **Figure 1**.

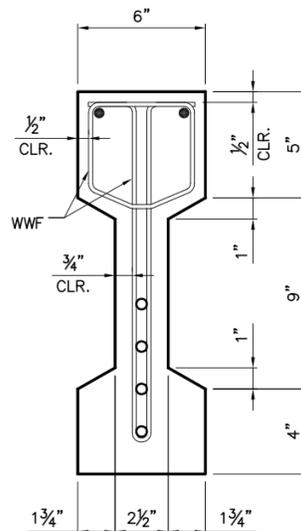


Figure 1. Typical Beam Cross-Section

Strand sizes readily available were 0.5 in. and 0.6 in. dia. The larger strands offered too few design choices, so the team concentrated on using 0.5 in. dia. strands. Using an assumed strand stress at failure of 260 ksi, an ultimate load of 40 kips could be achieved with the combinations shown in Table 2.

Table 2. Heights for Number of Strands

Number of Strands	h (in)
1	45.5
2	24.7
3	18.0
4	14.8
5	13.0
6	11.9

After evaluating the combinations for cost and constructability, the four-strand solution was selected. The strands were designed to be unbonded for 11' at the center of the beam with sheaths to increase ductility. The unbonded length was made as long as possible to maximize deflection at ultimate load. Meanwhile, the ends were bonded about 3' at each end to provide anchorage. Although this was expected to be enough for transfer, it risked being too short for the development length. For this reason, strand vices were attached at each end just before testing.

Moreover, two #3 rebars were added to the top flange to provide tensile strength at initial conditions and some additional compression strength at ultimate. Four stiffeners were added to help to transfer the vertical concentrated loads and the support reaction forces to the web.

Shear Design

To ensure flexural failure, a large amount of W2.9 3X3 welded wire fabric (WWF) was used to provide shear reinforcement. More reinforcement was used than last year due to availability and last year's experience with the strands bursting through the web. Due to this year's thin web, the W2.9 WWF was bent into a narrow U shape and used along the whole length of the beam. The WWF was also bent into a wider U shape to provide more shear reinforcement and concrete confinement in the stiffeners. This would later be tied to the confinement cage in the top flange. Instead of cutting and placing sheets of WWF, the bending was to provide better anchorage.

Materials

This section will cover the materials used in the project. This includes concrete, prestressing strand, welded-wire fabric, rebar, steel plate, and strand vices.

Concrete

CTC produced the concrete used in the beam. The same concrete mix was used for the whole beam. Lightweight coarse and fine aggregates were used for the beam because the team wanted to minimize beam weight. The concrete had a w/c ratio of 0.4, a design unit weight of 105.4 pcf, and an actual unit weight of 109.8 pcf. No slump was measured. Below is a summary of the mix.

Table 3. Concrete Mix for One cy of Concrete

Cementitious Materials	Aggregates	Admixtures	Strength @ 28 Days (psi)
610 lb Type II Cement	800 lb Fine	3.1 lb Adva 575	Design f'c: 9,000
135 lb Flyash	1,000 lb Coarse		Actual f'c: 9,170
			Tensile Strength: 986

Additional mix details can be found in the **Appendix**. The concrete was almost perfect for the application because it provided the lowest possible weight within the category of lightweight concrete, but at the same time gave almost the largest possible strength (10,00 psi) without the cost penalty of using high strength concrete. Below is the strength verses time curve of the mixture.

PLOT OF STRENGTH VS AGE

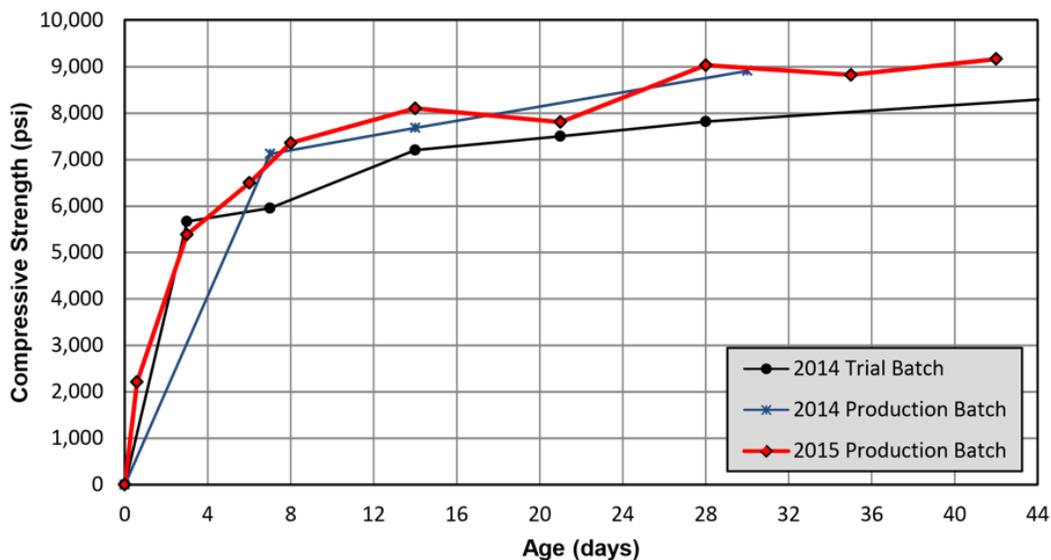


Figure 2: Concrete Strength vs. Time Curve

Prestressing Strands

The prestressing strands used in the beam were low relaxation ½" dia. ASTM A416 grade 270 strands. Sumiden Wire Products Corporation provided the strands with a yield point of 39.6 kips and a modulus of elasticity of 28,400 ksi. The strands were straight throughout the whole beam and covered in plastic sheathing to debond the strands. A steel plate and strand vices were applied to the ends of the beam to keep the strands from slipping. Detailed strand properties can be found in the **Appendix**.



Figure 3. Sheathed Strands

Welded Wire Fabric

Davis Wire provided the welded wire fabric (WWF) used in the beam. Three 5' x 10' sheets of WWF were used to construct the shear reinforcement. Two sheets of WWF were cut and bent into "U" shapes to provide web reinforcement along the whole beam length. Meanwhile, one sheet of WWF was cut and bent into a square cage to provide confinement for the top flange in the middle part of the beam. Last, about half a sheet of the remaining WWF was cut and bent into U shapes to provide reinforcement for the stiffeners.



Figure 4. WWF Reinforcement



Rebar

Two #3 ASTM A615 grade 60 rebar were used in the top flange throughout the whole beam to provide anchorage for the shear reinforcement and to act as compression steel. The yield strength and tensile strength for the rebar were 69 ksi and 108 ksi respectively, and elongation in 8" was 15.6%. At jacking and transfer states, the rebar could act as tension steel as well.

Beam Fabrication

Beam fabrication started with the shear reinforcement on April 9, 2015. By April 13, bending the shear reinforcement was finished in time for reinforcement assembly, which involved setting the reinforcement in the formwork, tying the reinforcement together, and pre-tensioning the strands. On April 14, the beam was cast, and on April 21, the strands were released and the beam was released from the form.

Reinforcement Construction

Reinforcement construction began with cutting and bending 3" x 3" - W2.9 Welded Wire Fabric (WWF) sheets. Three different shapes and sizes of shear reinforcement were to be assembled inside the beam: a narrow "U" shape throughout the beam, a wider "U" shape for the stiffeners at the load points and supports, and a confinement cage at the beam midspan. To create these shapes, three 5' x 10' WWF sheets were cut and bent to appropriate sizes and shapes. Due to the limited size of WWF sheet, the small "U" was separated into two 10' x 3' sections with a 2' of overlap at midspan. There were a total of four big "U" reinforcements needed—two for the supports and two for the stiffeners. Each required a 3/4' x 3' of WWF. Finally, the confinement cage required a 10' x 2' sheet of WWF to protect the top flange's critical region.



Figure 5. Cutting the WWF

After all the WWF pieces were cut into appropriate sizes, each of the shear reinforcement pieces was bent into the appropriate shapes using an oxy-acetylene torch. The wire was too brittle to cold-bend the tight radius needed. The small "U" was bent by marking the bending area and placing a 1/2" diameter steel bar at the middle to form the bent shape. Each took about 4 hours to bend; a total of 8 hours were used to bend the small "U"-shaped reinforcement.



Figure 6. Hot-Bending the Top Flange Confinement

Additionally, the shape of the cage was changed to rectangular due to construction difficulties—the WWF was difficult to bend into the non-rectangular configuration originally intended. Before bending, each area to be bent was marked. A ½” diameter steel bar was placed at the bending location to form the 90 degree angle. From the moment the torch first heated the steel, the process took about 5 hours to finish. A similar method was used to bend the big “U” reinforcements, which took only took 1 hour to finish.

The prepared reinforcement was then assembled at Concrete Technology Corporation (CTC). The two small “U” sections were made to overlap by cutting several vertical wires and forcing the horizontal wires to fit into the other’s grid. The big “U”s were placed at the four aforementioned positions—support and load point stiffeners—and the small “U” was placed between them. The sheathed strands were then slipped through the small “U.” Prior to casting, sheath lengths were checked, and the team discovered that the strands weren’t 11’ long. The lengths of each strand’s sheaths are tabulated below.

Table 4. Lengths of Sheaths for Each Strand

Location from Bottom (in)	Length of Sheaths (ft)
8	10.583
6	10.917
4	10.000
2	10.917

Afterward, the top flange confinement cage was fitted over the “U”-shaped web reinforcement at midspan. This involved cutting several of the small “U”s’ horizontal wires to ensure the cage would fit into the proper position. After all the WWF was in place, the compression steel bars were slipped through the cage. Finally, all the reinforcement was tied together and kept in place.



Figure 7. Assembling the WWF Reinforcement

Strand Prestressing and Beam Casting

The strands were then pre-tensioned—a two-step process. First, all the strands were pre-tensioning to half the prestress load. After checking that the reinforcement was still in the appropriate position, the pre-tensioning process continued to the appropriate stress. The next day, the beam was cast and the strands were released one week later on April 21.



Figure 8. Strand Jacking

Testing

Prior to test day, cylinders were tested at CTC, and the beam was placed on roller supports spaced 17' apart under the “Baldwin” testing machine. The team reused the previous year’s testing rig and supports.

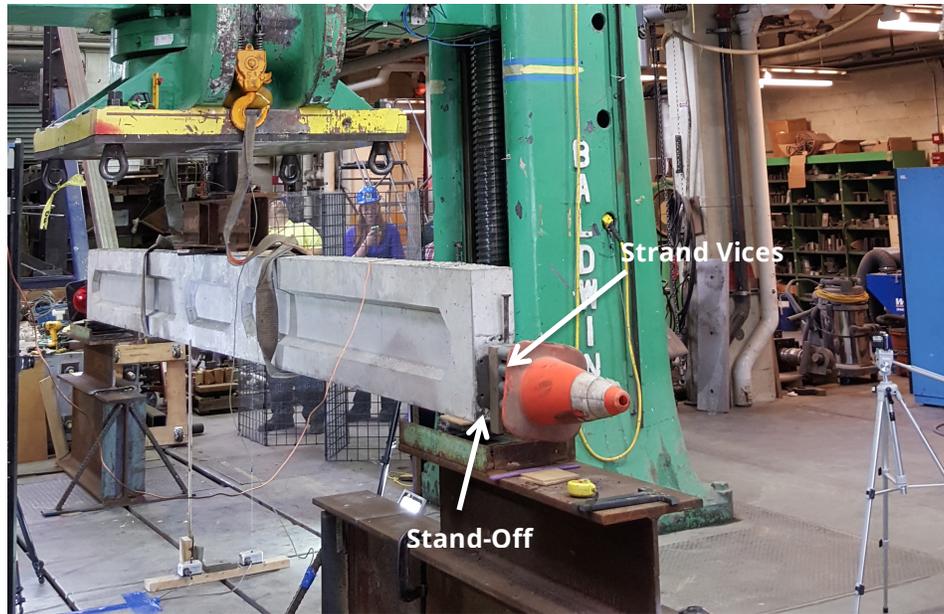


Figure 8. Testing Rig

On June 2, 2015, at the University of Washington’s More Hall Structures Lab, testing began. Before testing, strand vices were added to the ends of the strands to provide additional anchorage. A stand-off was placed between the end of the beam and the strand vices to facilitate subsequent removal, and the vices were tightened as closely as possible against the stand-off.

The location of the supports and loading points were then marked. A steel bar was hot-glued into place between the loading points. Strings tied to the ends of the bar were attached to potentiometers underneath the beam to measure deflection. Additionally, a meter stick was placed vertically at mid-span to provide manual deflection readings in case the electronic instrumentation malfunctioned. Two video cameras, one fixed still camera, and one mobile still camera recorded the test. The video cameras were placed on either side of the beam during testing. Because cracks are more difficult to visually detect in a prestressed beam than in a conventional reinforced beam, the video footage was used to relate displacements with when cracks were visibly—as well as audibly—noticeable. Individual tasks were assigned to each individual: time-keeping, data acquisition, video camera start and stop, still camera, watching for cracks and recording the time and load at which they occurred, etc.

The beam was initially loaded to 5 kips to test instruments. Afterward, the load was removed, the instruments were re-zeroed, and testing began. During loading, observers stood behind safety screens but watched for cracks. None were seen in this first loading stage. Once the load reached 22 kips, loading stopped, and the team checked the beam for cracks at close range. Then the beam was loaded to failure. A series of well-distributed cracks formed over the central 5' or so of the

beam. Eventually one crack, almost exactly at mid-span, dominated and became much wider. A load noise was heard, the large crack opened much wider, and the load dropped. The beam failed from crushing of the cover concrete above the confinement cage. The concrete in the cage appeared to be largely intact. The peak load was 40.8 kips, and the corresponding deflection was 2.23”.



Figure 9. Beam at Failure

Results

After the testing, collected data was used to plot the load-deflection curve shown in **Figure 6**. According to **Figure 6**, the load-deflection curve appears to be nonlinear, but the beam was still elastic with unbonded strands stretching inside. The reason for this is that the beam's cross-sectional area was changing after the cracks formed.

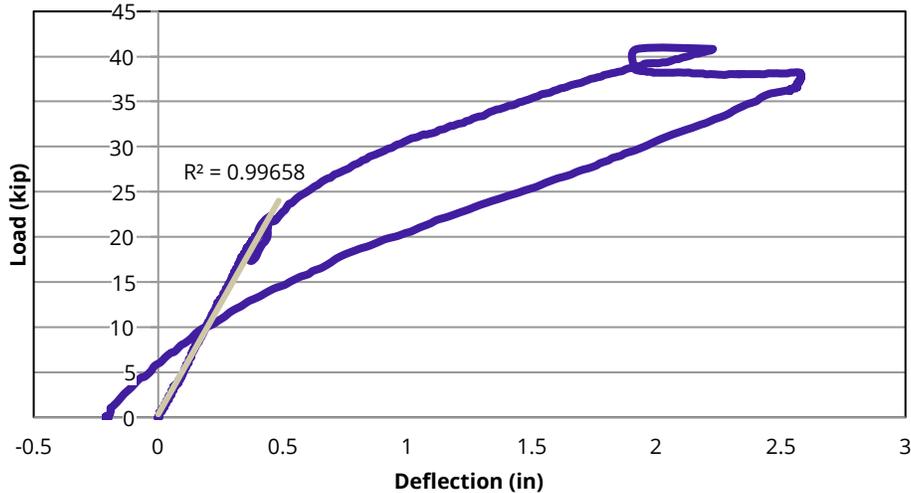


Figure 10. Load-Beam Deflection Curve

During testing, the team checked for any cracks formed at certain loads. At 22 kips, loading stopped for the PCI representative to check for cracks, and he found none. However, the curve seems to have lost its linearity at around 22 kips. **Figure 11.** shows a zoomed in version of the plot for loads from 15 to 25 kips.

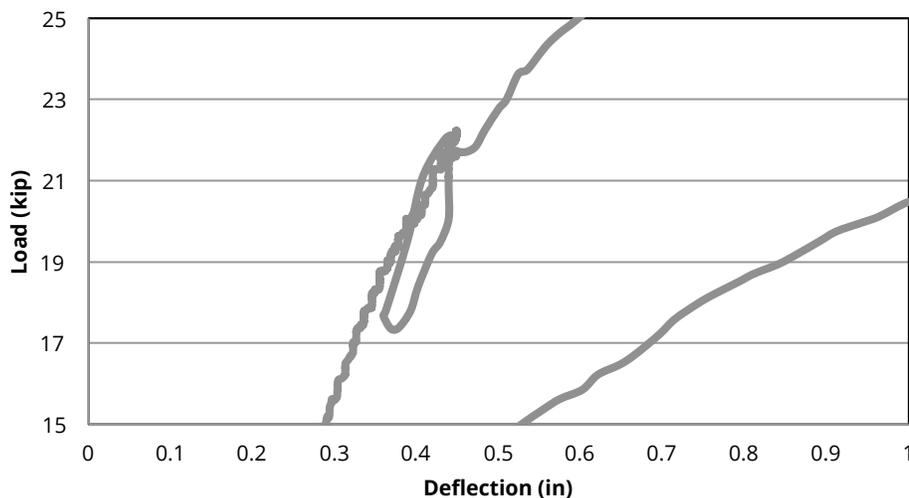


Figure 11. Load-Beam Deflection Curve: 15 - 25 kip Loads

The curve appears to be only slightly non-linear which may have been caused by micro-cracks.

Upon beam failure, the team determined that the ultimate load was 40.8 kips, which was 0.2% larger than the predicted 40.7 kips. From **Figure 10.** and audibly from the video footage, the first crack was determined to be around 26.0 kips, which was 2% smaller than the predicted 26.5 kips. Meanwhile, the deflection at peak load was found to be 2.23", which was 67% smaller than 6.69".

After failure, one major crack was found at the middle of the beam. This suggests that the force was concentrated at the middle of the two loading points. Beam failure occurred when the top flange concrete crushed. The deflected shape of the beam was V-shaped rather than U-shaped because as the beam had unbounded strands which distributed the moment throughout the strands and could deflect larger and quicker within the beam relative to the concrete.

Because the predicted deflection and the actual deflection were so different, the team set out to determine what went wrong during prediction calculations. With the failure load, actual deflection, and Dr. Beam software, the peak moment could be found, which in addition to a self-weight moment yielded a total moment of 1740 k-in. The force in the strands was then found to be 152 kips. From there, the deflection at the ultimate load can be back calculated to be 1.89", which is 17.9% smaller than the actual deflection but much closer to the actual than in the previous prediction.

This may have been caused by two events:

- Strands yielding and behaving nonlinearly
- Unbounded length being shorter or longer than used due to construction

All calculations can be found in the **Appendix.**

Lessons Learned

Throughout the competition, the team has learned many lessons design, construction, and post-failure analysis.

The first lessons came from prestressed concrete topics. Prior to and during competition, most of the team member had not taken any Prestressed Concrete course. Consequently, the team did not know much about prestressed concrete, but throughout the competition, the team learned about the difference between reinforced concrete and prestressed concrete. They learned about the different modes of failure, different tensile reinforcement, and different concerns for deflection. After understanding the basics materials, the team was able to design their own prestressed concrete beam.

The team also learned the importance of communicating through clear drawings—how to draft the beam using AutoCAD and how to use elevation, plan, and cross-section views to communicate their ideas to CTC. They found that appropriate selections of drawing views and dimensioning were key in good communication and that it could save time for repeating revisions.

The largest lesson learned in construction occurred in shear reinforcement construction. The team was responsible for constructing the beam shear reinforcements. They learned how to use a torch to hot bend long spans of reinforcement—that is, the cage and web reinforcement—and to use hand tools to cold bend short span reinforcement—that is, stiffener reinforcement. They not only learned about tool operation but also about the close correlation between design and construction. In designing the confinement cage, the dimension of the reinforcements was pushed to the design code's limit to maximize concrete confinement. While constructing the cage, they discovered how difficult it was to follow design dimensions. As a result, they decided to simplify the cage geometry. Moreover, they learned the importance of construction planning because brainstorm types of tools to use and sequencing the construction saved the team a lot of time.

Last year's beam had stiffeners that catastrophically failed during testing. As a result, this year's stiffeners were designed to be beefier by increasing their width and reinforcement. The team was glad to find that the stiffeners remained intact after failure.

The team chose to use lightweight concrete to decrease beam self-weight. The results showed that the beam's performance was comparable to high strength concrete. The compressive strength of the lightweight concrete was 9,220 psi with a modulus of rupture around 986 psi. Although costs were higher with lightweight concrete, the benefits offset the cost increase.

Using and analyzing unbonded strands introduced many new concepts to the team. The team decided to use unbonded strands to increase the deflection during loading, but results showed that unbonded strands did not help much with deflection. Although the unbonded strands would have allowed for wider cracks, the top flange concrete limited beam deflection, and the beam failed from crushing due to the high compressive stresses. Apart from concrete crushing between the loading points, most of the concrete remained intact. If unbonded strands were to be used to increase deflection, a different type of concrete may be considered for concrete between and around the loading points.



Because of a designed bonded length of about 3' on either end, the team prepared for strand slippage by adding strand vices for increased anchorage. However, to the team's surprise, no slippage occurred. Had slippage occurred, the strand vices would have exerted such a large force on the stand-offs at the beam ends that they wouldn't have been as easy to remove. The concrete strength, and possibly the web reinforcement, may have contributed to the excellent bond.

Rarely do engineers get to design, construct, and test to failure. The team is thankful for the opportunity to do so, and the experience and the lessons learned will no doubt serve the team well as they move on to their future careers.

Appendix

1. Certification from CTC
2. Test summary form
3. Calculations
 - a. Prediction of cracking load, max moment, and deflection at max moment
 - b. Post-Failure Analysis
 - c. Cost and weight
4. Total load/midspan deflection graph
5. Concrete Specifications
6. Strand Specifications
7. Drawings of beam cross section(s) and elevation

PCI BIG BEAM COMPETITION – 2015



CERTIFICATION

CONCRETE TECHNOLOGY CORPORATION
As a representative of (name of Producer Member or sponsoring organization)

UNIVERSITY OF WASHINGTON, TEAM #2
Sponsoring (name of school and team number)

I certify that:

- The big beam submitted by this team was fabricated and tested within the contest period.
- The calculations of predicted cracking load, maximum load, and deflection were done prior to testing of the beam.
- The students were chiefly responsible for the design.
- The students participated in the fabrication to the extent that was prudent and safe.
- The submitted test results are, to the best of my knowledge, correct, and the video submitted is of the actual test.

Certified by:

Austin D. Maue
Signature

AUSTIN D. MAUE
Name (please print)

06-11-15
Date

THIS CERTIFICATION MUST BE PART OF THE FINAL REPORT

Sponsored by  and 

BIG BEAM CONTEST 2015

06/11/2015

Date

University of Washington

Student Team (school name)

2

Team Number

04/14/2015

Date of Casting

Basic information

1. Age of beam at testing (days) 49

2. Compressive cylinder tests*

Number tested: 2
Size of cylinders: 4" x 8"
Average: 9220 psi

3. Unit weight of concrete (pcf) 109.8

Slump (in.): n/a

Air content (%): 0.98

Tensile strength (psi): 986

Circle one:

Split cylinder

MOR beam

4. Pretest Calculations

a. Applied point load at midspan to cause cracking (kip) 26.8

b. Maximum applied point load at midspan (kip) 40.7

c. Maximum anticipated deflection due to applied load only (in.)
6.69

Pretest calculations MUST be completed before testing.

*International entries may substitute the appropriate compressive strength test for their country.

Test summary forms must be included with the final report, due June 16, 2015

Judging Criteria

Teams MUST fill in these values.

a. Actual maximum applied load (kip) 40.8
b. Measured cracking load (kip)* 26.0
c. Cost (dollars) 93.03
d. Weight (lb) 1086
e. Largest measured deflection (in.) 2.23
f. Most accurate calculations 69.27.

(a) Absolute value of (maximum applied load – calculated applied load) / calculated applied load

0.27.

(b) Absolute value of (maximum measured deflection – calculated deflection) / calculated deflection

67%.

(c) Absolute value of (measured cracking load – calculated cracking load) / calculated cracking load

2%.

Total of three absolute values (a + b + c) = 69.27.

*Measured cracking load is found from the "bend-over" point in the load/deflection curve. Provide load/deflection curve in report.

Predictions: Deflection

Failure Deflection

According to ACI, the following equation is used to calculate the f_{ps}

$$f_{ps} = f_{pi} + f_c' + \frac{f_c'}{100 \times \rho_p} = 170.9 \text{ksi} + 9.2 \text{ksi} + \frac{9.2 \text{ksi}}{100 \times \frac{4 \times 0.1519 \text{in}^2}{6 \text{in} \times 13 \text{in}}} = 191.9 \text{ksi}$$

Then, $\Delta \epsilon_p$ can be calculated using f_{ps} .

$$\Delta \epsilon_p = \frac{f_{ps} - f_{pi}}{E_p} = \frac{191.9 \text{ksi} - 170.9 \text{ksi}}{28400 \text{ksi}} = 7.398 \times 10^{-4}$$

After that, assuming $c = 1$ in, ΔL_{ps} can be calculated by calculating L_{ps} .

$$L_{ps} = \text{Average(unbonded length)} + \frac{f_{ps} - f_{se}}{1000} \times d_s$$

$$L_{ps} = \frac{(10'7 + 10'11 + 10' + 10'11)}{4} + \frac{0.1 \times 191.9 \text{ksi}}{1000} \times 1/2"$$

$$L_{ps} = 127.26 \text{in}$$

$$\Delta L_{ps} = \Delta \epsilon_p \times L_{ps}$$

$$\Delta L_{ps} = 7.398 \times 10^{-4} \times 127.26 \text{in}$$

$$\Delta L_{ps} = 0.0941 \text{in}$$

The following can then be used to calculate $\Delta \theta$ (assume $c = 1$ in).

$$\Delta \theta = \frac{\Delta L_{ps}}{d_p - c} = \frac{0.0941 \text{in}}{13 \text{in} - 1 \text{in}} = 0.00784 \text{rad}$$

Finally, the expected deflection can be calculated

$$\Delta z \downarrow = \frac{L_{span}}{2} \times \Delta \theta = \frac{17 \times 12 \text{in}}{2} \times 0.00784 \text{rad} = 0.800 \text{in}$$

The deflection calculated was too low by using the ACI method. This may be due to the value of L/h . The length-to-height ratio for the beam is $(17 \times 12 \text{in}) / 18 \text{in} = 11.33$, but this value is far larger than the value from ACI when they are dividing the equation. Therefore, the ACI method may not be suitable for this beam.

As a result, the calculation was repeated by assuming the stress-strain curve is non-linear at failure and the total ϵ_p is equal to 1.00%. Using the equation,

$$\epsilon_{pe} = \frac{f_{pe}}{E_p} = \frac{170.9ksi}{28400ksi} = 0.00602$$

$$\epsilon_{ps} = \epsilon_{pe} + \Delta\epsilon_p \Rightarrow \Delta\epsilon_p = 0.00398$$

By using the PCI code, the following equation can be used to calculate f_{ps} .

$$f_{ps} = 268 - \frac{0.075}{\epsilon_{ps} - 0.0065} < 0.98f_{pu}(ksi) \text{ with } 270ksi \text{ strand}$$

$$f_{ps} = 268 - \frac{0.075}{0.01 - 0.0065}$$

$$f_{ps} = 246.57ksi$$

Then, c can be calculated.

$$C_c = F_{ps} = f_{ps} \times A_s = 246.57ksi \times 4 \times 0.1519in^2 = 149.82kips$$

$$a = \frac{C_c}{0.85 \times f'_c \times b} = \frac{149.82kips}{0.85 \times 9.2ksi \times 6in} = 3.193in$$

$$c = \frac{a}{\beta_1} = \frac{3.193in}{0.65}$$

By repeating the steps above. The failure deflection is calculated to be 6.39in.

Total Deflection

Deflection due to camber is calculated to be 0.148 in based on our pre-stress spreadsheet.

Deflection due to cracking load is 0.154 in based on our pre-stress spreadsheet.

The total deflection can be calculated using the following formula,

$$\Delta_{total} = \Delta_{camber} + \Delta_{cracking} + \Delta_{failure}$$

$$\Delta_{total} = 0.148in + 0.154in + 6.39in$$

$$\Delta_{total} = 6.692in$$

Predictions: Cracking Load

We are using the following equation for calculating the cracking moment, M_{cr} ,

$$-\frac{F_p}{A} - \frac{e_p F_p}{S} + \frac{M_{cr}}{S} = f_{cr}$$

$$-\frac{170.9\text{kips}\times 4\times 0.1519\text{in}^2}{80\text{in}^2} - \frac{(1.175 + 3.175 + 5.175 + 7.175)\text{in}\times 170.9\text{kips}\times 0.1519\text{in}^2}{300.259} + \frac{M_{cr}}{300.259}$$

$$= 0.965$$

$$M_{cr} = 1113 \text{ kip} - \text{in}$$

By that, we can calculate the cracking load, P_{cr} ,

$$P_{cr} = \frac{M_{cr}}{d_{\text{loading to support}}} = \frac{1113\text{kip} - \text{in}}{(7\times 12)\text{in}} = 13.25 \text{ kips for each loading point}$$

Predictions: Ultimate Load

Ultimate moment can be calculated using the following equation,

$$M_{ult} = F_{ps} \times \left(d - \frac{a}{2} \right)$$

$$M_{ult} = 149.82\text{kips} \times \left(13\text{in} - \frac{3.193\text{in}}{2} \right)$$

$$M_{ult} = 1708.5\text{kip} - \text{in}$$

The ultimate load can be calculated using the following equation,

$$P_{ult} = \frac{M_{ult}}{d_{\text{loading to support}}} = \frac{1708.5\text{kip} - \text{in}}{7\times 12\text{in}} = 20.34\text{kips for each loading point}$$

Comparing Predictions with Results

Equations

$$(1) \text{ Percent Difference} = \frac{|\text{Actual Value} - \text{Theoretical Value}|}{\text{Theoretical Value}} \times 100$$

Calculations

For ultimate load,

$$\text{Percent Difference for Ultimate Load} = \frac{40.8 - 40.7}{40.7} \times 100\% = 0.2\%$$

For the first crack,

$$\text{Percent Difference for the first crack} = \frac{26.5 - 26}{26} \times 100\% = 1.9\%$$

For maximum deflection,

$$\text{Percent Different for Max. Deflection} = \frac{2.23 - 6.69}{6.69} \times 100\% = 66.7\%$$



Post-Failure Analysis

Equations

$$(2) \theta = \frac{\frac{\Delta_e}{2}}{d_p - \frac{a}{2}}$$

$$(3) \Delta_{vertical} = \frac{\theta L}{2}$$

$$(4) \Delta_{total} = \Delta_{vertical} + \Delta_{camber} + \Delta_{decompression}$$

$$(5) M_n = F_p \left(d_p - \frac{1}{2} \left(\frac{F_b}{0.85 f'_c b} \right) \right)$$

$$(6) \text{Percent difference} = \frac{\text{Actual Value} - \text{Theoretical Value}}{\text{Theoretical Value}} \times 100\%$$

Calculations

$$\begin{aligned} \Delta_e &= 1'' \\ d_p &= 13'' \\ a &= 2'' \\ L &= 17' \\ M_n &= 1740k - in \end{aligned}$$

From our predictions,

$$\Delta_{camber} = 0.148'', \Delta_{decompression} = 0.154''$$

From Equation (2),

$$\theta = \frac{\frac{1}{2}}{13 - 1} = \frac{1}{24}$$

From Equation (3),

$$\Delta_{vertical} = \frac{17 \times 12 \times \frac{1}{24}}{2} = 1.59''$$

From Equation (4),

$$\Delta_{total} = 1.59 + 0.148 + 0.154'' = 1.89''$$

By using the "goal seek" tool in Excel with Equation (5),

$$F_p = 152.2 \text{ kips}$$

From Equation (6),

$$\text{Percent Difference} = \frac{2.23 - 1.89}{2.23} \times 100 = 17.9\%$$

Mr. Beam Cost and Weight Calculations

Length (ft) : 18

Typical Cross-Sectional Area Calculation

Flange Width (in) = 6
 Top Flange Height (in) = 5
 Bottom Flange Height (in) = 4
 Web Height (in) = 7
 Web Width (in) = 2.5
 Sloped Run (in) = 1.75
 Sloped Height (in) = 1

Web Area (in²) = 17.5
 Sloped Area (in²) = 4.25
 Top Flange Area (in²) = 30
 Bottom Flange Area (in²) = 24

Typical Cross-Section Area (in²) = 75.75

Detail Volumes

Triangular Volumes (in³) = 25.375
 Ding Point Stiffener Volumes (in³) = 113.75
 End Triangle Volumes (in³) = 6.125
 End Stiffener Volumes (in³) = 576

Beam Weight

Beam Volume (in³) = 17085
 (ft³) = 9.89
 (yd³) = 0.3662

Unit Weight (pcf) = 109.8

Beam Weight (lb) = 1086

Amounts of Steel

	(ft ²)	lb/sf	Total
Steel Plate	0.94	20.4	19.27
WWF	59	0.61	35.99
	(ft)	(lb/ft)	
Regular #3 Rebar	36	0.376	13.536

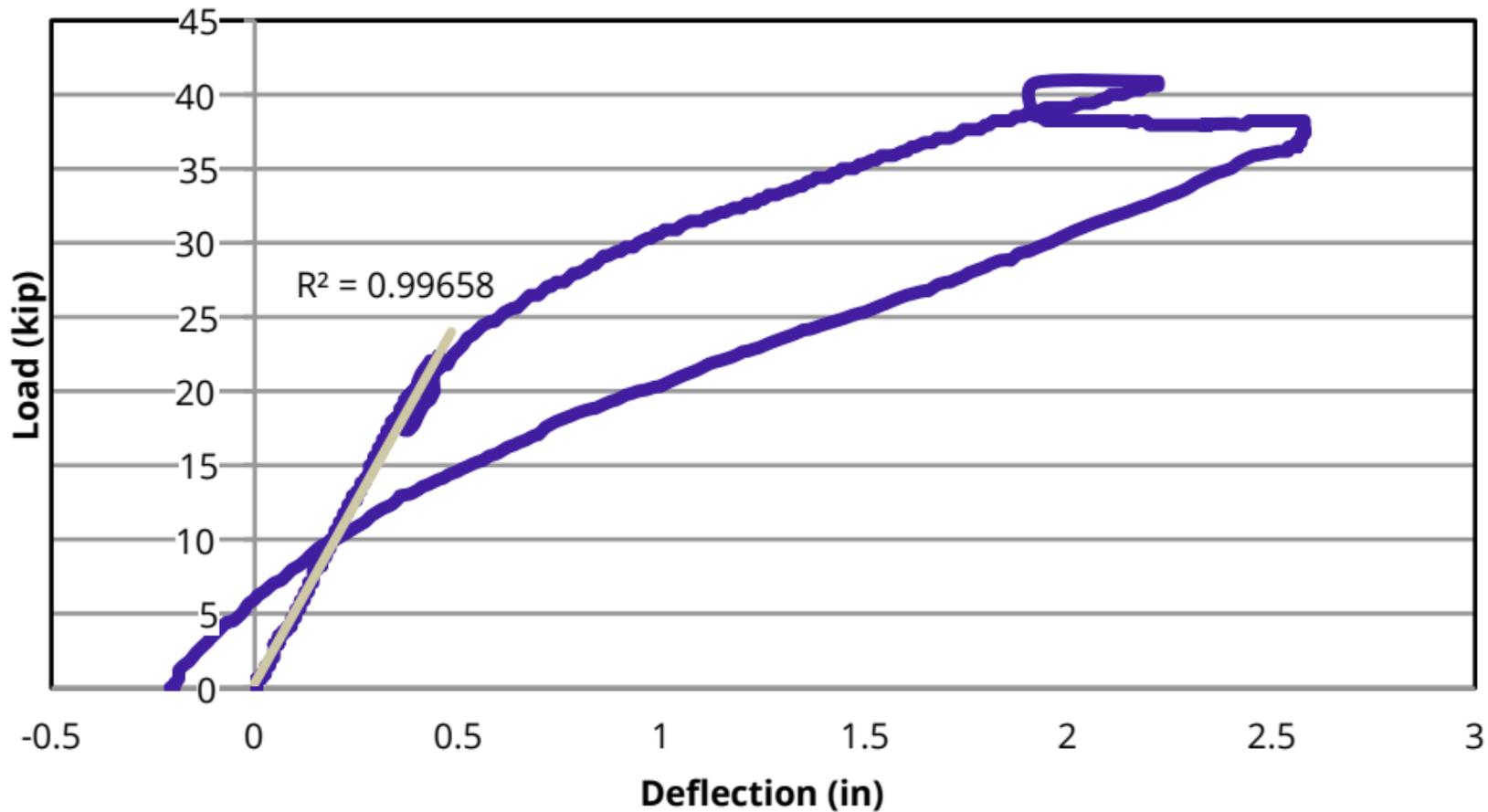
Cost

	Unit Cost	Cost
Concrete (\$/cu yd)	110	40.28
WWF (\$/lb)	0.50	18.00
Plate Steel (\$/lb)	0.06	1.06
Strands (\$/ft)	0.3	21.60
A615 (\$/lb)	0.45	6.09
Forming (\$/sf)	1.25	6.01

Total Cost (\$) = 93.03

Surface Area Calculations

Surface Area of Stiffeners (in²) = 73
 End Stiffener SA (in²) = 63.5
 Beam SA without Detailing (in²) = 7758
 Total Beam SA (in²) = 8304
 (ft²) = 4.81



Load-Beam Deflection Curve



JOB NAME: U.W. BIG BEAM 2014
JOB NO.: 15X05A

BY: ADM
DATE: 4/14/15

Target Values per ONE CUBIC YARD yield

MIX #: **LOB2**

Cementitious Material

Cement: **610** lbs.
 Flyash: **135** lbs.
 Silica: **0** lbs.

<u>S.G.</u>	<u>Vol. (ft³)</u>
3.15	3.10
2.00	1.08
2.20	0.00

Aggregate (SSD)

Fine: **800** lbs. = 44 % of total aggregate
 Coarse: **1,000** lbs. = 56 % of total aggregate

<u>Absrp.</u>		
1.82	10.0	7.04
1.50	8.0	10.68

Water

Water: **298** lbs. = 35.7 gal.

1.00	4.78
-------------	------

Admixtures (oz./100 wt)

WRDA 64: **0.0** = 0.0 oz. = 0.00 gal. = 0.0 lbs.
 Adva 575: **6.0** = 44.7 oz. = 0.35 gal. = 3.1 lbs.
 DCI: **0.00** = 0 oz. = 0.00 gal. = 0.0 lbs.
 Daravair 1000: **0.0** = 0.0 oz. = 0.00 gal. = 0.0 lbs.
 Entrained Air: %

1.21	0.00
1.07	0.05
0.84	0.00
1.00	0.00
	0.00

Σ **2,846** lbs.

Σ **26.74** ft³

γ Fresh Conc.: 105.4 pcf
 w/c : 0.400
 mortar: 16.0 cf/cy

Target Volume:	27.00	ft ³
Total Volume Air:	0.26	ft ³
% Air:	0.98	



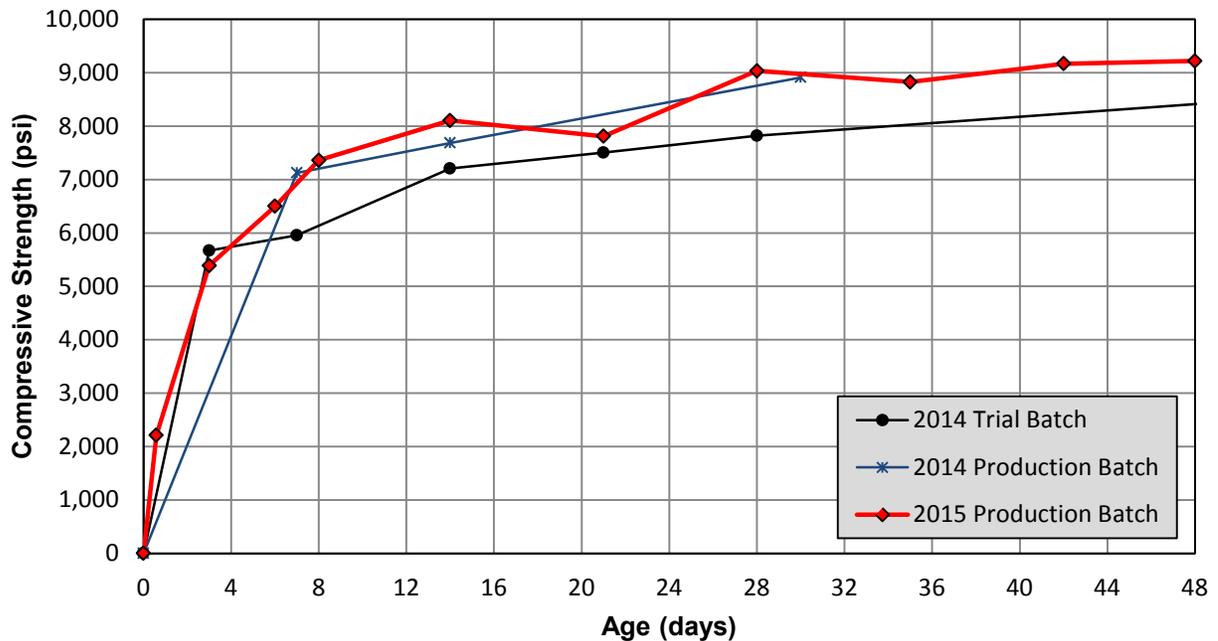
CYLINDER TEST SCHEDULE

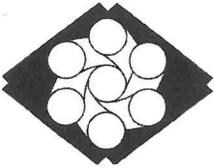
Date	Cylinder 1 f'c (psi)	Cylinder 2 f'c (psi)	Average f'c (psi)	MOE (ksi)
4/15/2015	1,940	2,480	2,210	
4/17/2015	5,480	5,300	5,390	
4/20/2015	6,710	6,290	6,500	
4/22/2015	7,690	7,030	7,360	2,919
4/28/2015	8,020	8,190	8,105	
5/5/2015	7,280	8,340	7,810	3,296
5/12/2015	9,220	8,850	9,035	
5/19/2015	8,850	8,800	8,825	3,481
5/26/2015	9,420	8,920	9,170	
6/1/2015	8,860	9,580	9,220	3,694

FLEXURE BEAM TEST SCHEDULE

Date	Beam 1 MOR (psi)	Beam 2 MOR (psi)	Beam 3 MOR (psi)	Average MOR (psi)
5/5/2015	993	997		995
6/1/2015	1,009	972	978	986

PLOT OF STRENGTH VS AGE





SUMIDEN WIRE PRODUCTS CORPORATION

Customer Service - PC Strand
East: Toll Free 866-491-5020 • 710 Marshall Stuart Dr., Dickson, TN 37055
West: Toll Free 866-246-3758 • 1412 El Pinal Dr., Stockton, CA 95205

SWPC

Import (NO Samples Necessary)

CERTIFICATE OF INSPECTION

Order Number: SLPC141188-1 Page No : 1 OF 1
B/L No: SIPC142530 Issue Date : 12/29/2014

Commodity: Steel Strand, Uncoated Seven Wire for Prestressed Concrete
Size & Grade: 1/2" x 270 KSI
Specification: ASTM A416 - Latest 1/2" - Low Relaxation
Customer Name: CONCRETE TECHNOLOGY CORPORATION
Customer P.O.: 6-02965
Destination: CONTEC-WA
State Job No:

No	Pack #	Heat #	B.S.	Elong.	Y.P.	Area	E-Modulus	CURVE#
			Min:41,300 (LB)	3.5 (%)	37,170 (LB)	(IN ²)	(MPSI)	
1	S128053-2	S0510714	43,001	5.0	40,043	0.1522	28.9	S128053
2	S128053-3	S0510714	43,001	5.0	40,043	0.1522	28.9	S128053
3	S128053-4	S0510714	43,001	5.0	40,043	0.1522	28.9	S128053
4	S128054-1	S0510714	42,816	5.1	39,747	0.1523	28.6	S128054
→ 5	S528228-5	S0588414	42,349	5.1	39,697	0.1519	28.4	S528228
6	S528229-6	S0588414	42,320	5.1	39,646	0.1516	28.7	S528229
7	S528229-7	S0588414	42,320	5.1	39,646	0.1516	28.7	S528229

We hereby certify that:

- * We have accurately carried out the inspection of COMMODITY and met the requirements in accordance with the applicable SPECIFICATION, both listed above.
- * The material described above will bond to concrete of a normal strength and consistency in conformance with the prediction equations for transfer and development length given in the ACI/AASHTO specifications.
- * The individual below has the authority to make this certificate legally binding for SWPC.

*Received by,
Det. W. K. Collins
12/31/14*

Date: 12/31/14 CMO: NO
PO: 6-02965
Job: Inventory
Item: Strand 1/2" Commercial

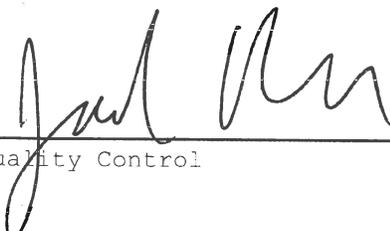
[Signature]

Quality Assurance Section

**CHEMICAL COMPOSITION**

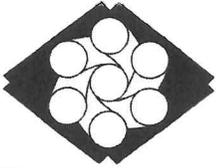
Date: 12/29/2014
Customer Name: Concrete Technology Corporation
B/L#: SIPC142530

Heat No.	C %	Mn %	P %	S %	Si %	Cr %	V %
S0510714	0.760	0.690	0.012	0.014	0.910	0.100	0.028
S0588414	0.750	0.680	0.014	0.008	0.810	0.100	0.038


Quality Control

12/29/14
Date

SUMIDEN WIRE PRODUCTS CORPORATION
PC STRAND DIVISION
1412 EL PINAL DRIVE, STOCKTON, CA 95205 209-466-8924 FAX 209-941-2990
710 MARSHALL STUART DRIVE, DICKSON, TN 37055 615-446-3199 FAX 615-740-1518



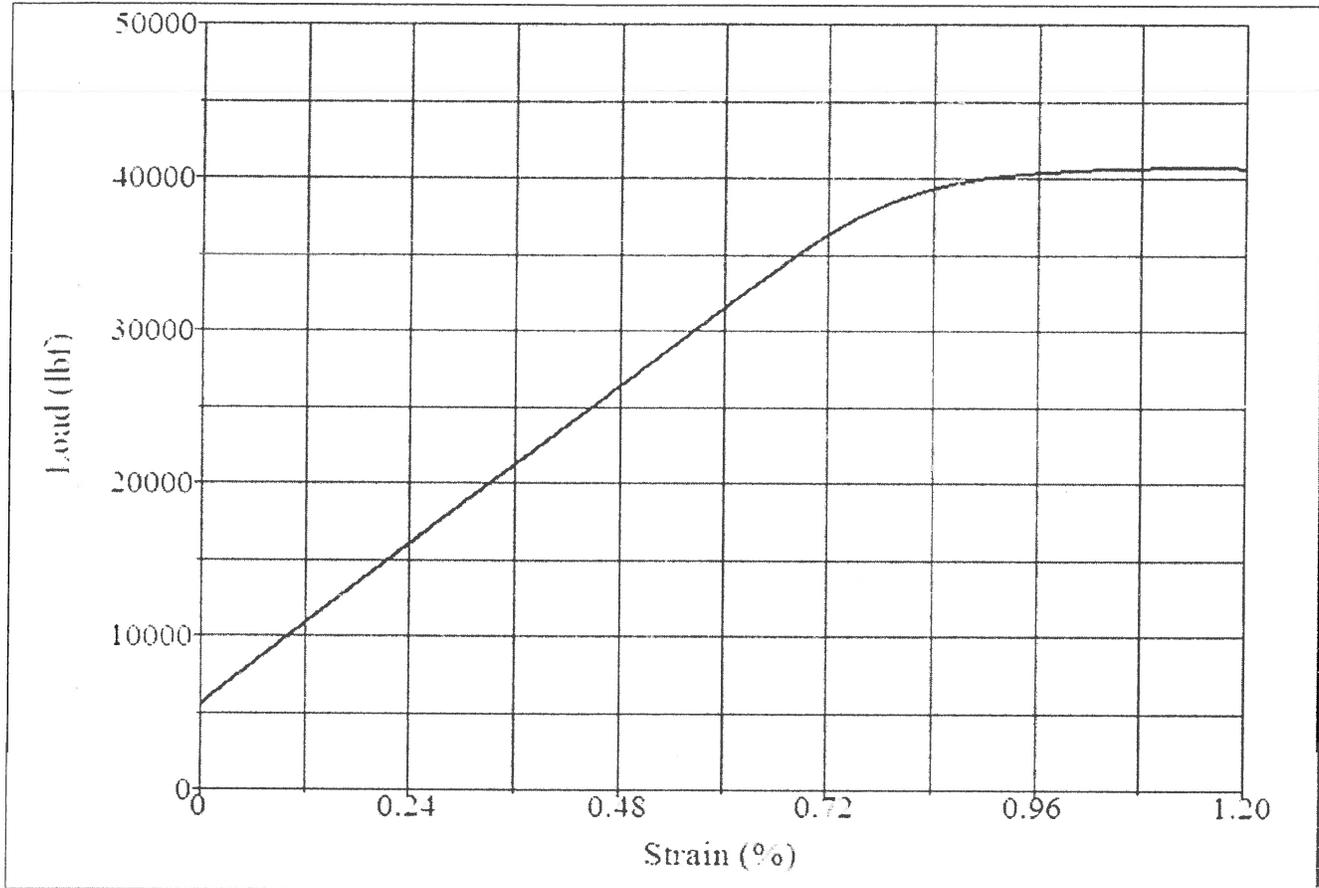
SUMIDEN WIRE PRODUCTS CORPORATION

Customer Service - PC Strand

East: Toll Free 866-491-5020 • 710 Marshall Stuart Dr., Dickson, TN 37055

West: Toll Free 866-246-3758 • 1412 El Pinal Dr., Stockton, CA 95205

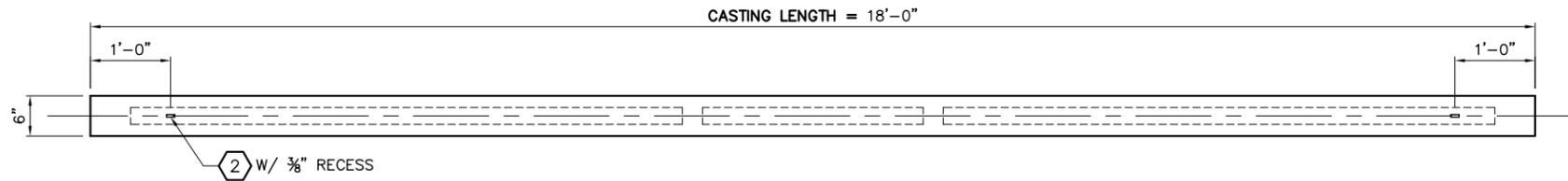
SWPC



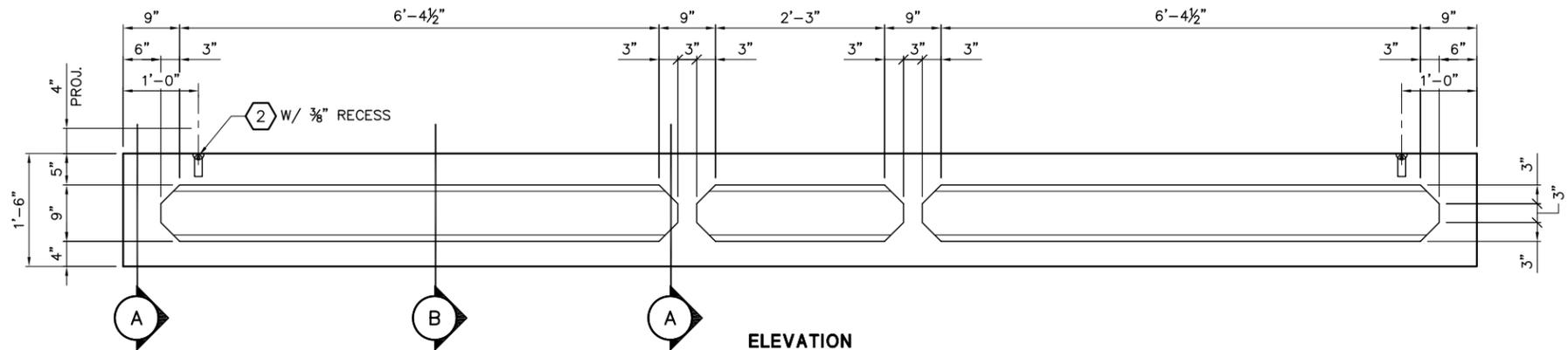
*Vertical Line is drawn at 1% Extension Under Load

Curve # S528228
Pack Tested S528228-1
Yield Point 39,697 lbf
Area 0.1519 in²
Modulus 28.4 Msi

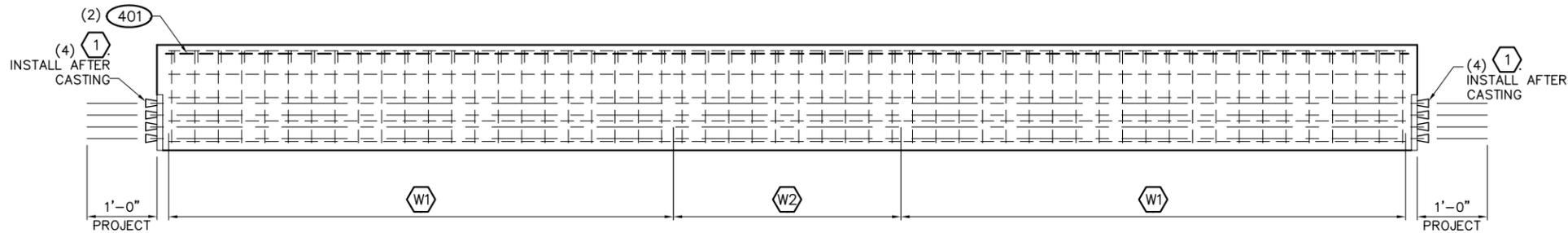
MARKED END



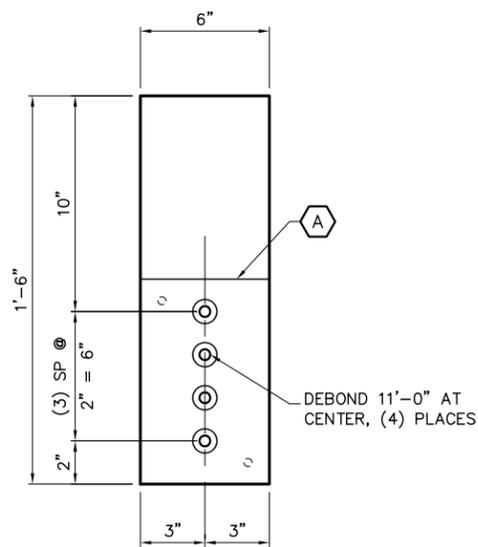
PLAN VIEW



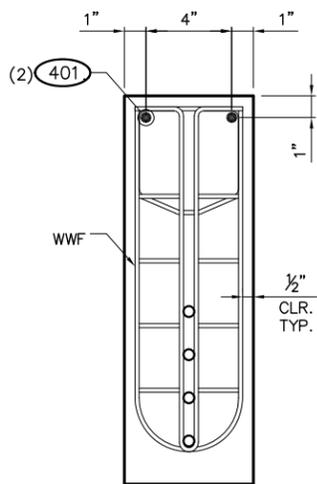
ELEVATION



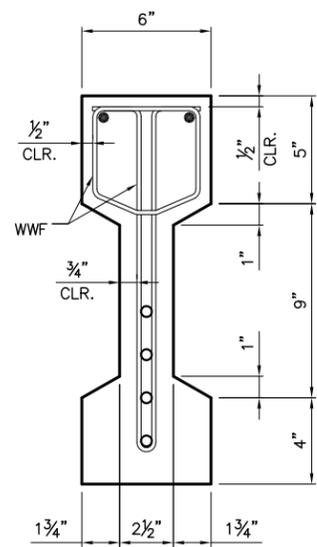
REINFORCEMENT ELEVATION



TYPICAL END VIEW



SECTION A



SECTION B

REINFORCEMENT SCHEDULE

GRADE	BAR MARK NUMBER	QUANTITY	STRAIGHT NO. BEBDS	CUTTING LENGTH	REBAR TYPE	BAR SIZE MARK NUMBER
60	401	2	X	17'-9"		X402

WWR SCHEDULE

MARK	DESCRIPTION	QTY	LENGTH
W1	4X4-W2.9xW2.9	2	7'-3"
W2	4X4-W2.9xW2.9	1	3'-3"

GENERAL NOTES

CONCRETE: STRENGTH @ RELEASE...5,000 P.S.I.
STRENGTH @ 28 DAYS...9,000 P.S.I.

PRESTRESS: 1/2"Ø 270KSI, 7 WIRE UNCOATED LOW RELAXATION STRAND **IMPORTED**

STRAIGHT: (4) STRANDS JACKED TO 123.6 KIPS (30.9 K/STR.)

FINISHES: TOP..... GOOD WOOD FLOAT
SIDES.... FORM FINISH
SOFFIT... FORM FINISH
ENDS..... FORM FINISH

YARD HAULING: 1'-0" FROM EACH END
STORAGE BUNKING: 1'-0" FROM EACH END
SHIPPING BUNKING: 1'-0" FROM EACH END

WSDOT STAMP REQ'D

INSERT & ASSEMBLY SCHEDULE

MARK	DESCRIPTION	PRIME	GALV.	PER PIECE	TOTAL
1	1/2"Ø STRAND CHUCK	X		8	
2	2.5T MEADOW BURKE RL-25 TECH ANCHOR			2	
A	R 6"x10"x1/2" W/(2) 3/8"Øx4" STUDS			2	

MARK	DESCRIPTION	PRIME	GALV.	PER PIECE	TOTAL
------	-------------	-------	-------	-----------	-------

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No.	Status	Revision	By	App'd	Date
1	FOR CONST.		DEOB	AM	3/17/15
0	SUBMITTAL		DEOB	AM	3/10/15

CONTRACTOR :

Design :
Drawn : DEOB
Checked : AM
Approved :

DO NOT SCALE

CONCRETE TECHNOLOGY CORP.
MANUFACTURERS OF PRESTRESSED CONCRETE
1123 PORT OF TACOMA ROAD
TACOMA, WASHINGTON 98421

UNIVERSITY OF WASHINGTON
"MR. BEAM"
UNIVERSITY OF WASHINGTON
SEATTLE, WASHINGTON

PRODUCTION DRAWING
PRODUCT
MARK: BM-2 QTY. 1 WT. 1.23 KIPS

Date: MARCH, 2015
Sh. No. of
Dwg. No. BM-2
Job. No. 15X05-A

CONCRETE TECHNOLOGY CORP.
HARDWARE ORDER

DATE MARCH, 2015
RE-ISSUE _____
SHEET _____ OF _____

JOB: UW BIG BEAM COMPETITION JOB NO.: 15X05 A

Assembly A **Make** 2

STEEL 6"x10"x1/2" THICK REBAR _____

ANCHORS (2) 3/8" x 4" STUD

WT. 8 # SPECIAL INSTRUCTIONS _____

FINISH PRIME GALV. TAPE EDGES YES NO

CONCRETE TECHNOLOGY CORP.
HARDWARE ORDER

DATE _____
RE-ISSUE _____
SHEET _____ OF _____

JOB: _____ JOB NO.: _____

Assembly _____ **Make** _____

STEEL _____ REBAR _____

ANCHORS _____

WT. _____ SPECIAL INSTRUCTIONS _____

FINISH PRIME GALV. TAPE EDGES YES NO

CONCRETE TECHNOLOGY CORP.
HARDWARE ORDER

DATE _____
RE-ISSUE _____
SHEET _____ OF _____

JOB: _____ JOB NO.: _____

Assembly _____ **Make** _____

STEEL _____ REBAR _____

ANCHORS _____

WT. _____ SPECIAL INSTRUCTIONS _____

FINISH PRIME GALV. TAPE EDGES YES NO

CONCRETE TECHNOLOGY CORP.
HARDWARE ORDER

DATE _____
RE-ISSUE _____
SHEET _____ OF _____

JOB: _____ JOB NO.: _____

Assembly _____ **Make** _____

STEEL _____ REBAR _____

ANCHORS _____

WT. _____ SPECIAL INSTRUCTIONS _____

FINISH PRIME GALV. TAPE EDGES YES NO

CONCRETE TECHNOLOGY CORP.
HARDWARE ORDER

DATE _____
RE-ISSUE _____
SHEET _____ OF _____

JOB: _____ JOB NO.: _____

Assembly _____ **Make** _____

STEEL _____ REBAR _____

ANCHORS _____

WT. _____ SPECIAL INSTRUCTIONS _____

FINISH PRIME GALV. TAPE EDGES YES NO

CONCRETE TECHNOLOGY CORP.
HARDWARE ORDER

DATE _____
RE-ISSUE _____
SHEET _____ OF _____

JOB: _____ JOB NO.: _____

Assembly _____ **Make** _____

STEEL _____ REBAR _____

ANCHORS _____

WT. _____ SPECIAL INSTRUCTIONS _____

FINISH PRIME GALV. TAPE EDGES YES NO

CONCRETE TECHNOLOGY CORP.
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HARDWARE ORDER

DATE _____
RE-ISSUE _____
SHEET _____ OF _____

JOB: _____ JOB NO.: _____

Assembly _____ **Make** _____

STEEL _____ REBAR _____

ANCHORS _____

WT. _____ SPECIAL INSTRUCTIONS _____

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1	FOR CONST.	DEOB	AM	3/17/15
0	SUBMITTAL	DEOB	AM	3/10/15
No.	Status	By	App'd	Date

CONTRACTOR : _____

Design _____
Drawn DEOB
Checked AM
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UW PCI BIG BEAM COMPETITION
UNIVERSITY OF WASHINGTON
SEATTLE, WASHINGTON

HARDWARE

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Date MARCH, 2015
Sh. No. _____ of _____
Dwg. No. H-1
Job. No. 15X05 A