ECONOMIC AND SERVICEABLE SEISMIC SYSTEMS

PHASE II – ALL-BOLTED BUCKLING RESTRAINED BRACED FRAMES

AISC REPORT

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1 Summary

The cast-in-place concrete industry enjoys healthy commercial market share when competing against structural steel. Even when a project is comprised of steel for its primary structural frame, the concrete industry has achieved significant advantages with more economical and often times better performing lateral systems. In many cases, steel framed buildings with concrete core wall lateral systems are more the norm than the exception. Current structural steel design provisions for seismic load resisting systems have unfortunately contributed to the economics of this trend rather than mitigated it.

Seismic load resisting systems for structural steel buildings have undergone considerable evolution over the past fifteen years. The overriding theory driving current design approaches is to provide systems that remain stable under relatively large story drifts, while at the same time experiencing controlled inelastic deformations to dissipate energy. As can be seen by various systems described in the "Seismic Provisions for Structural Steel Buildings," March 9, 2005 by the American Institute of Steel Construction, this is primarily accomplished by proportioning elements such that specific major components experience inelastic deformations. Components that connect major lateral load resisting elements, as well as components that are not intended to resist lateral loads, are anticipated to remain substantially elastic and undergo minimal damage. While the idea of isolating large deformations to anticipated components and locations has considerable merit, the current design methods by which this concept is applied poses some possible inefficiencies and shortcomings.

The controlled and predictable yield of major components has resulted in considerable limitations on global and local member geometry. To achieve desired compactness requirements and slenderness ratios, often beam, brace, and column sections gravitate to sectional areas well in excess of that which is required to resist loads derived from the load combinations of the applicable building code. This places considerable force demands on connections, which in seismic applications are typically required to develop the expected yield strength of the primary member. The results are increased material and connection costs.

To develop the expected yield strength of members such as beams or braces, welded connections are typically required. The reason is the area reduction due to holes for bolted connections typically results in inadequate available tensile strength at the net section to achieve the required expected yield strength of the member. Because nearly every component of many seismic connections requires welding, typically some magnitude of welding must occur in the field. In the case of moment frames, often complete penetration field welds are specified. Because field welding is arguably the most expensive process in steel construction, this considerably increases the relative expensive of the steel frame making it less competitive with other lateral load resisting systems. The costs associated with demanding field inspection, such as that typically required for complete penetration welds, adds further expense.

Primary structural components such as beams and columns are extremely expensive by structural standards and difficult to adequately repair or replace, particularly when equipped with fully welded connections. Typically these components, by design, are fully integrated with the overall structural scheme and in most cases are relied upon to carry gravity loads in addition to lateral loads. Therefore, replacement of such components once significantly damaged is often unrealistic, leaving complete demolition and replacement of the building as the only viable option. The resulting expense to the owner or insurer from a significant seismic event could be economically devastating.

New innovations in seismic load resisting systems have recognized the approach of isolating inelastic deformations to primary, permanently attached components may, in fact, be flawed. By instead isolating inelastic deformations to easily accessible, bolted components that can be relatively inexpensively removed and replaced, a serviceable seismic load resisting system can be achieved.

The idea of replaceable fuses, for example, supports this line of thinking. While continuity of major structural members has historically been considered an advantage of cast-in-place concrete, in the case of serviceability after a significant seismic event, the opposite is true. The spalling and cracking of major beams and columns due to major seismic damage almost ensures a cast-in-place concrete structure requires demolition and replacement. Structural steel systems, conversely, possess an inherent advantage over cast-in-place concrete systems in that damaged components can potentially be replaced if the system is properly designed.

> Herein a *serviceable system* is defined as a frame where inelastic deformation has been accommodated in such a way the damaged element can be reasonably removed from the frame after a seismic event and replaced with a similar element, e.g., a buckling restrained brace (BRB). Connections and other members are designed to remain substantially elastic and can therefore be reused.

Serviceable seismic load resisting systems pose many advantages. Components that are relatively easily replaced characteristically exhibit easy initial installation. Therefore, the field labor associated with initial installation of a serviceable system may be reduced over the current labor intensive installation processes described previously. Reduction in field labor typically translates to reduction in overall cost. More importantly, the potential creation of national criteria for serviceable structures sets the stage for a national certification program. Such a program may include pre- and post-service field inspection requirements to evaluate the level of damage sustained to specific components and determine whether replacement is necessary. A structure with enhanced potential to be viably salvageable after a significant seismic event is directly marketable to owners. Furthermore, a building that has met the design and pre-service inspection criteria of a certified serviceable structure program is likely attractive to insurers, who in turn could offer increased coverage and/or reduced premiums to owners for building such a structure. The ultimate result could introduce new structural steel framing options in moderate and high seismic regions.

To adequately address a wide spectrum of building program needs, proposed serviceable connections and components have been developed for moment frame and braced frame systems. To ensure maximized economy, in addition to aforementioned reduced field labor costs, the systems proposed utilize readily available or easily fabricated components designed to carry minimal force levels as required by the applicable building code. The proposed systems are: Fully Bolted Buckling Restrained Braced Frames (BRBF), and Ductile WT Moment Frames.

Part I of the final report addressed the Ductile WT Moment Frame connections, especially the WT behavior. More testing will be necessary on the full connection assemblage to completely validate the concept.

Part II of the final report is this document. It contains the results of two full-scale BRBF one-bay, one-story frames. The frame connections were fully bolted and detailed such that after a major seismic event the brace would absorb most of the inelastic energy. The BRB could be replaced by unbolting the damaged brace and replacing it with a new one. In the present test series, a Star Seismic BRB (WC250) was initially installed in the test frame. The Appendix T of ANSI/AISC 341-05 (AISC 341, 2005) translation/drift test regime was used. The BRB and connections performed well and the system illustrated robust and stable hysteric behavior. The frame was re-plumbed and another brace (WC 200) was installed.

Testing of the second brace again employed the ANSI/AISC 341-05 regime with a two-percent maximum drift. The frame was examined for damage and then tested again under the ANSI/AISC 341-05 regime, only in this test with a three-percent drift. The brace and the connections performed well. The hysteric behavior again was stable for all cycles. Three percent was the limit of the test configuration, so the test series was ended.

This report contains the test description and results for global behavior for the frame and local strains in areas of interest. The information from these tests was used to develop recommendations for proportioning and configuring the members and connections.

In summary, the concept of designing for a serviceable frame after a major seismic event appears to be viable. Connection details can accommodate the significant drift requirements. The replacement of the brace was demonstrated.

2 Background

2.1 Buckling Restrained Braced Frames -- Overview

The performance criteria for seismic design currently adopted by reference in the 2006 International Building Code (IBC) is based upon preserving life safety by avoiding major structural failure or collapse (FEMA, 2003). In order to achieve these criteria, structures are anticipated to experience inelastic deformations within the primary structural system during a significant seismic event (ASCE, 2010). In high seismic regions it is probable that structures will experience such inelastic deformations during the course of its service life. (McManus, 2010)

The inelastic deformations can occur in several ways depending upon the goals and type of system being designed. The buckling restrained braced frame (BRBF) system uses diagonal braces and these elements are design to yield in a predictable and favorable manner.

Figure 1 illustrates a typical BRBF and the BRB application in the two-story X-bracing configuration. These gusset plates are welded to the columns and beams.

a. b.

Figure 1 - BRBF Example Lawrence Berkeley National Lab (Star Seismic)

Figure 2 illustrates a schematic of a BRB which is made of three distinct sections: the core that is design to yield, the transition zone and the extension plate. The steel core and transition are encased in a grouted tube that restrains the core from buckling under compressive loads. Typical cross section details are illustrated in Figure 3. The details for the tested BRBs are changed to accommodate bolting. The BRB details are provided in Appendix A.

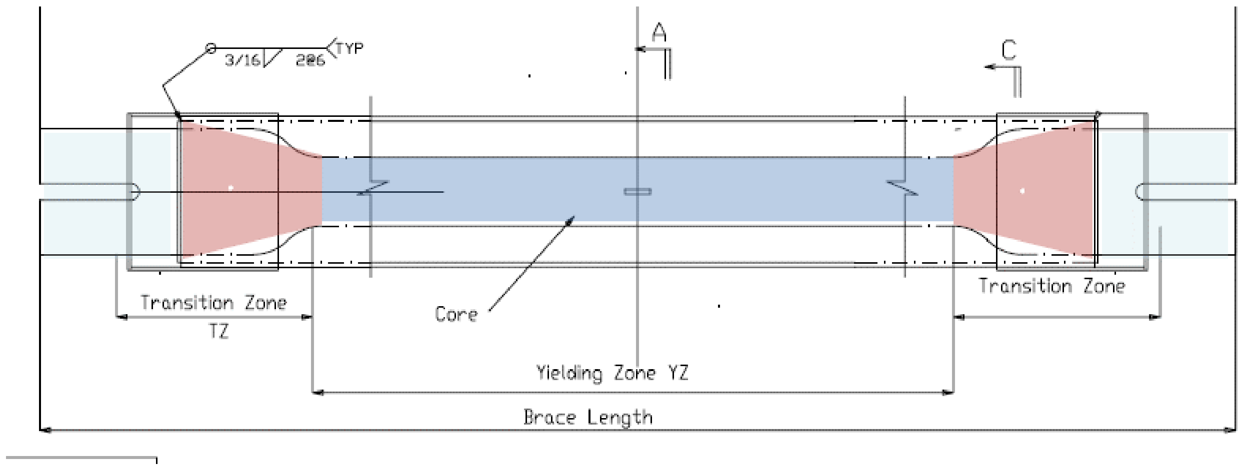

Figure 2 - Schematic of a BRB

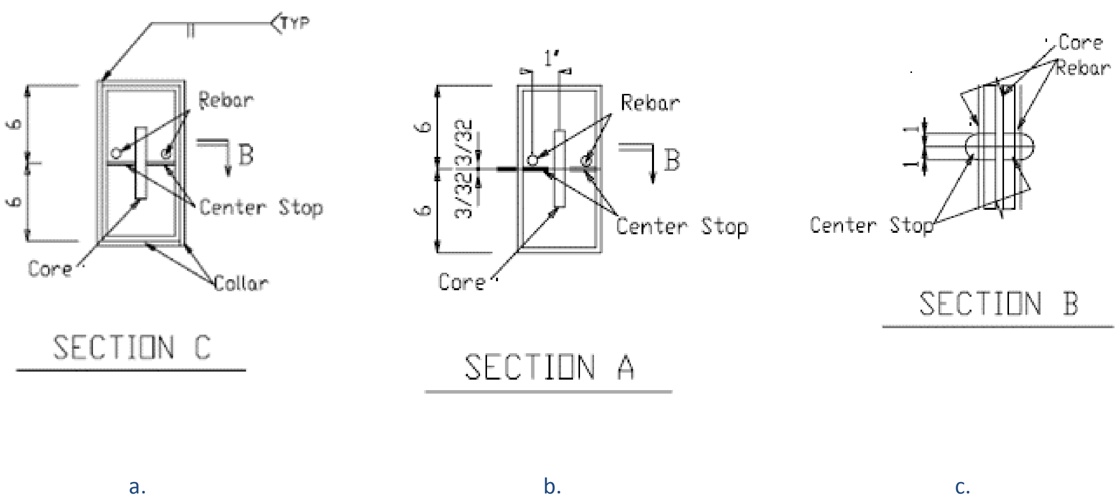

Figure 3 - Typical BRB Section Details

2.2 Serviceable BRBF Seismic Systems

Buckling Restrained Braced Frames (BRBF) are an efficient and well performing lateral system. Because the core of the brace need only be proportioned to provide sufficient stiffness to meet story drift requirements, or to carry the loads from the applicable building code without considerations of buckling, the required strength of the connections to develop the expected yield of these braces is typically far less than that of other types of seismic braced frames. Forces to the connections can therefore be adequately addressed with bolted connections. However, tests of BRBF assemblies to date have consisted primarily of welded connections between the brace and gusset, and almost entirely of welded connections between the gusset and the beam and column. Test results in braced frame systems often result in significant damage at the interface between the gusset and beam or column due to the large rotations induced at the connection under the large story drifts simulated in seismic testing. Therefore, even if the BRBF were bolted to the gusset but welded to the primary members, a serviceable system would not be achieved should damage to the gusset occur during a seismic event.

By bolting the gusset to the brace as well as the beam and column as shown in Figure 4, a serviceable system can be produced. Connection angles can be adequately proportioned for strength but likely offer more flexibility than directly welded connections. The reduced restraint may help to mitigate the damage sometimes observed in welded connections.

Figure 4 - Fully Bolted Buckling Restrained Brace Connection (prior to test)

2.3 Research Goal

The primary goal of this research was to evaluate fully bolted buckling restrained braced frames as serviceable seismic load resisting systems though experimental testing. A secondary goal was to verify fully bolted connections designed using current AISC provisions adequately develop the BRB at code required story drifts. Thirdly was the development of linear and nonlinear analysis procedures that adequately represent the behavior. Recommendations for design as well as linear and non-linear modeling are developed. The intent of the design is to limit inelastic deformation to the BRB, while the connections and other members remain elastic.

3 Fully Bolted Buckling Restrained Braced Frame

3.1 Brace and Frame Design

3.1.1 Beam and Column Design

Primary framing members for the test frame and reaction frame were intended to remain elastic during the tests. Initial design was consistent with simple hand methods that are common in professional practice. The adjusted brace strength of the WC250 in compression was assumed to develop in the brace. The adjusted brace strength in compression is defined within the AISC Seismic Provisions as $\beta\omega R_y P_{ysc}$ where β is the compression strength factor, ω is the strain hardening factor, R_y is the ratio of expected yield stress to minimum specified yield stress, and P_{ysc} is the axial yield strength of the core (AISC 341, 2005). The ratio of compression strength to tension strength, β, was assumed to be 1.14 based on test data from the University of Utah (Romero et al., 2007). From the same data, the hardening factor, ω, was assumed to be 1.58. Because Star Seismic performed tensile coupon tests on the braces provided for the testing herein, R_y, was taken as 1.0. The forces in the primary framing members associated with the assumed adjusted brace strength were calculated using statics and the strength was checked using standard AISC-LRFD procedures. Member were assumed to have pinned ends with an effective length factor, K = 1.0. All wide flange sections were ASTM A992 steel.

Seismic compactness criteria and available sections from the fabricator assisting with the project were also considered in the design. The lightest seismically compact nominal 14 in. by 14 in. (356 mm) wide flange shapes were used for the columns in the test frame (see Figure 5). The high and low ends of the BRB (diagonal orientation) were initially configured such that the actuator force would be delivered to the brace through the upper beam of the test frame. Consequently, the upper beam was initially sized to carry this force. It was also sized based on availability from Puma Steel, flange geometry to adequately receive bolted connections, and flange and web compactness ratios within the maximums allowed by the AISC Seismic Provisions (AISC 341, 2005). However, the brace direction was switched later in design such that the actuator and brace would be in compression at the same time. This was done to ensure the strength of the brace was developed recognizing the strength of the brace and capacity of the actuator were both greater in compression than in tension. With the new configuration, the upper beam of the test frame theoretically became a zero force member.

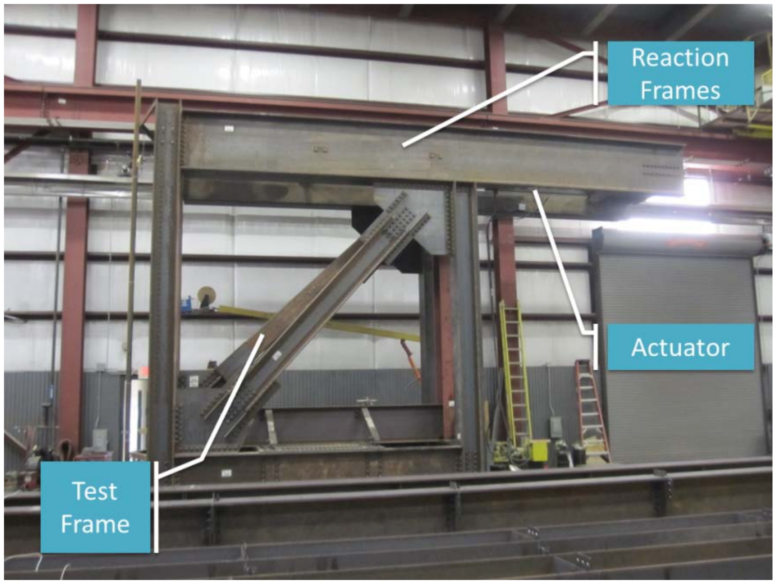

Figure 5 - North View of Frame

The lower beam of the test frame transferred the horizontal component of the brace force through a diaphragm plate to the reaction frame (see Figure 6).

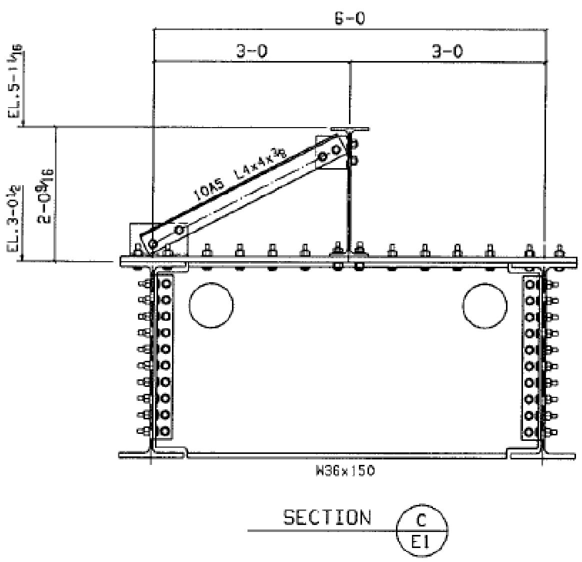

Figure 6 - Section of Diaphragm Plate at Bottom of Test Assemblage

This beam was designed assuming strong axis brace points at the member ends and weak axis brace points at the ends and at third points. Strong axis eccentricity was not considered in the initial design because eccentric forces were assumed to be easily resolved through the frequent bottom flange connections to cross beams within the reaction frame. The lower beam was sized using similar considerations to the upper beam except that the web compactness ratio was slightly above the AISC maximum seismic compactness limit. Exceeding the web seismic compactness ratio was intended to challenge to the beam capacity and ensure, through successful performance, that all compact sections could be assumed to perform adequately. Additionally, the web of the lower beam was slender for shear strength calculations per the AISC specification (AISC 360, 2005).

Primary members within in the reaction frame were also chosen based on material availability, but were primarily intended to provide elastic stiffness several times that of the test frame. Consequently, demand to capacity ratios in the members were relatively small and seismic compactness was not considered. Adequate capacity of all members was verified in later analytical modeling.

3.1.2 Design of BRB-to-Frame Connections

In general, for any bolted joint in the seismic load resisting system (SLRS), the joint can be designed as a bearing type connection if standard holes are used in all plies, but must be constructed as slip-critical. Thus, the bolts must be pretensioned, and the faying surface must meet at least Class A requirements (Class B and C faying surface requirements would also be acceptable). This requirement is intended to limit deformations within the joint during an earthquake. An exception to this requirement is for bolted joints at diagonal brace connections. In this case, oversized holes are permitted in one ply of connected interfaces provided the connection is designed as slip-critical. This exception was added to the 2005 AISC Seismic Provisions based on feedback from erectors, who indicated that fit-up of bolted brace connections was very difficult with standard holes.

Finally, for any bolted joint in the SLRS, the nominal bearing strength cannot be taken greater than $2.4dtF_u$where d is bolt diameter, and t and F_u are the thickness and rupture strength of the material being connected, respectively. Chapter J of the AISC Specification permits the nominal bearing strength to be taken as high as $3.0dtF_u$. However, at this level, significant hole elongation occurs. Consequently, in order to again limit movement at bolted joints during an earthquake, the Seismic Provisions limit the nominal bearing strength.

The uniform force method was used to determine the force distribution in the brace connections. The uniform force method determines force distribution to connection components and primary members based on the geometric extents of the primary members being connected. Further description of this method can be found in the AISC Steel Construction Manual (AISC 13, 2005). Special Case 2, as defined by AISC, was used at the upper brace connection to theoretically eliminate shear to the beam. This addresses multiple force distribution approaches through the testing. The gusset plate at the upper connection was attached to the column web, whereas the gusset was connected to the column flange at the lower connection to incorporate multiple framing conditions into the testing as well.

All plate and angle material was ASTM A36. All bolts were 7/8 in. (22 mm) diameter. ASTM A325 bearing bolts with threads excluded from the faying surfaces were used to connect angles to gusset plates and primary members. ASTM A490 bolts were used to connect the BRB to the gusset plates using slip critical connections. A Class A faying surface preparation was provided with standard holes in the gusset plates and oversized holes in the connection plates on the BRB.

The probable brace forces used for connection design were based on β and ω factors recommended from tests of Star Seismic braces at the University of Utah (Romero et al., 2007), which was discussed previously regarding member design. Star Seismic uses these factors in practice, and the intent was to be consistent with their typical design approach. Standard LRFD ϕ factors were applied in designing for each of the connection limit states.

Governing design limit states of the gusset-to-beam/column connections were bolt shear, prying action, and bolt bearing on the gusset. A490 slip critical bolts in oversized holes were used to connect the braces to the gussets. Thus slip critical bolt shear capacity governed the brace-to-gusset connections. Demand/capacity ratios varied between roughly 0.9 up to 1.1 for these governing limit states. The 10% overstresses were typically on prying action checks in the connection angles.

3.2 Experimental Testing

3.2.1 Test Procedure, Arrangement, and Equipment

Full-scale testing of the braces first involved one trial run on the test specimen without any brace installed. The intent of the trial run was to verify that the data acquisition software would work properly with the instrumentation. Testing of the two buckling restrained braces was done per recommended procedures of AISC 341-05 Appendix T. The initial test regimen was based on a maximum of two percent drift in the test frame and the required cumulative inelastic deformation of 200 times the yield deformation of the brace. To account for deformations external to the brace tendon, such as in connection components and primary members, the yield deformation used for development of the test regimen was conservatively calculated assuming a work point-to-work point tendon length of 246 in. (6250 mm). Coupled with an assumed yield stress of 43 ksi (296 MPa), the yield deformation was approximated as 0.365 in. (9.27 mm). The actual yield deformations calculating using the average yield stress for each tendon from coupon tests and the tendon length from shop drawing details were 0.160 in. (4.06 mm) and 0.166 in. (4.21 mm) for the WC250 and WC200, respectively. Using the larger of these values, the actual cumulative inelastic deformation requirement for the braces is 33.2 in. (843 mm). Upon successful completion of the test on the WC200 brace, the regimen was reconfigured based on a maximum drift of three percent and successively applied to the same WC200 brace and brace connections. The same beams, columns, and beam-to-column connections were used for both tests.

A reaction frame with an actuator rated to produce 600 kips (2850 kN) push force and 450 kips (2140 kN) pull force was constructed to perform testing as diagramed in Figure 7. The reaction frame was arranged so that lateral bracing of the test frame had minimal restraint in the plane of the test. The orientation of the actuator was such that pushing force would put the BRB into compression and pulling would create tension in the brace. The actuator was used to produce translation-controlled loading of the test frame. The accumulated translation of the test frame was calculated from the collection of top translation relative to base translation at the outer test frame column.

The test specimen was instrumented with two string potentiometers, one linear potentiometer, pressure gauges on the actuator to determine load to the test frame, and multiple strain gauges on beams, connection angles, and the gusset plate, see Figure 7.

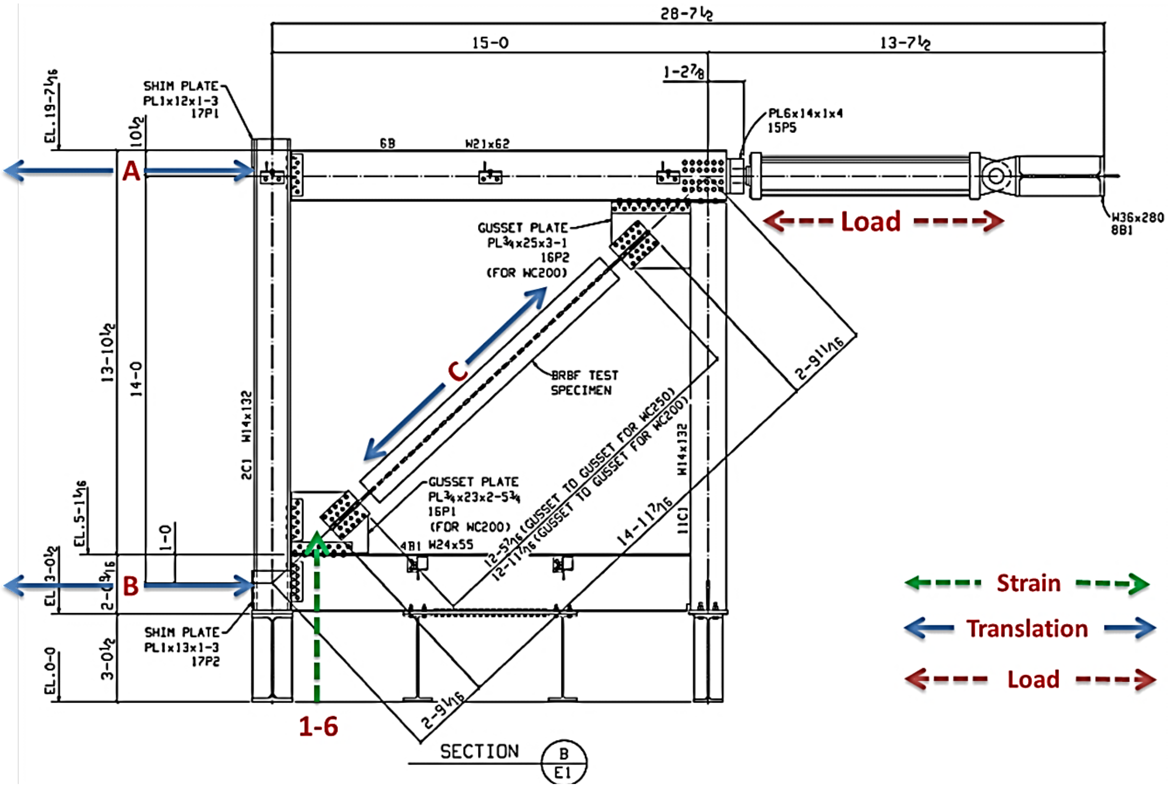

Figure 7 - BRB Test Frame Instrumentation

The first string pot was mounted along the BRB long axis to measure total axial deformation of the brace tendon. The second string pot was mounted at the top of the test specimen on the outer column measuring the total drift. The linear pot was mounted on the outer column as well in order to measure any movement at the bottom, see Figure 8. The string pots were mounted on timber elements. Bolts and hooks were welded to the test frame and reaction frame to receive the timber mounted instrumentation. Nylon cable ties and glue were used to attach the linear pot and the string pot on the outer column. Two clamps were also used to secure the string pot on the top of the outer column to prevent any slip.

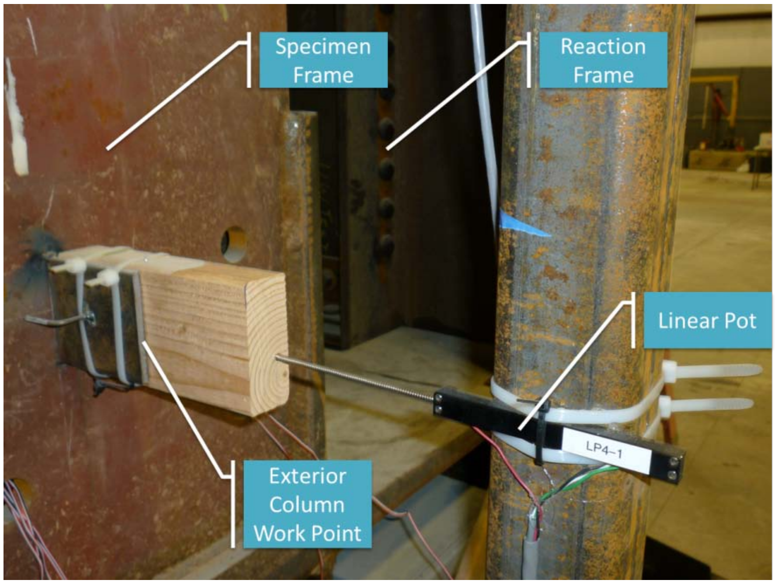

Figure 8 - Bottom of Frame Linear Pot Mounting

The strain gauge orientation for the first test on the WC250 was primarily located around the bottom gusset plate connecting the brace to the beam and column. Strain gauge one (SG_1) was mounted vertically on the gusset plate. SG_2 was mounted on the gusset plate aligned with the brace. SG_3 was mounted horizontally near the same location as one and two with the intent of capturing the in-plane state of stress in the gusset, see Figure 9.

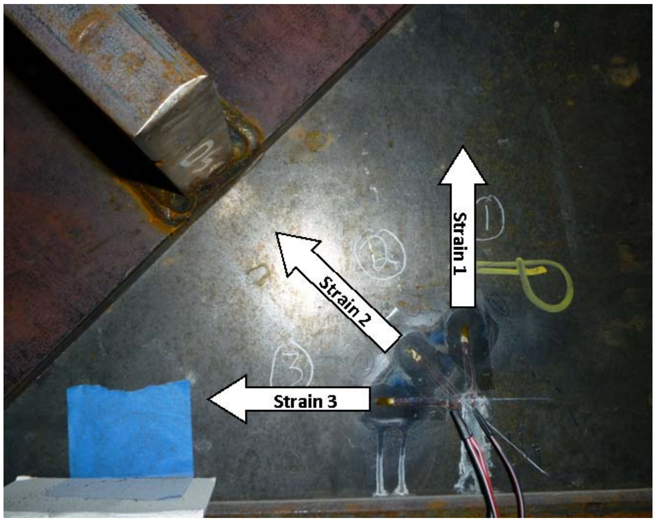

Figure 9 - WC250 Strain Gauges

SG_4 was located on the angle connecting the gusset plate to the bottom beam, and was placed near the outermost bolt hole. SG_5 was placed under the top flange of the bottom beam directly below SG_4, see Figure 10. SG_6 was placed on the outstanding leg of the angle connecting the gusset plate to the column next to the outermost bolt hole similar to SG_4, see Figure 11.

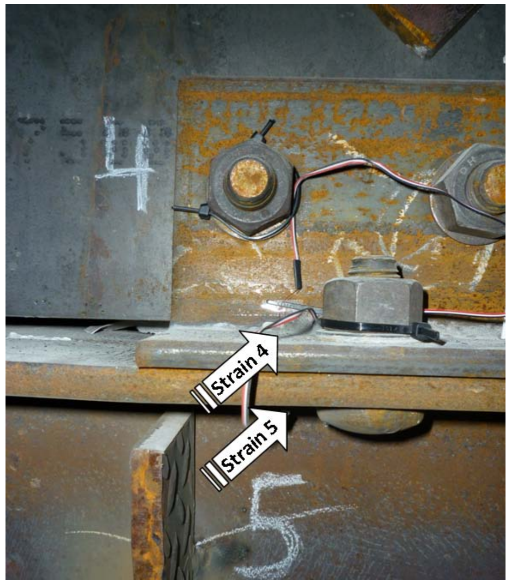

Figure 10 - WC250 Strain Gauges

Figure 11 - WC250 Strain Gauges

For the WC200 test, SG_1through SG_5 were in the same locations as in the WC250 test. However, SG_6 was placed on the web of the bottom beam, see Figure 12.

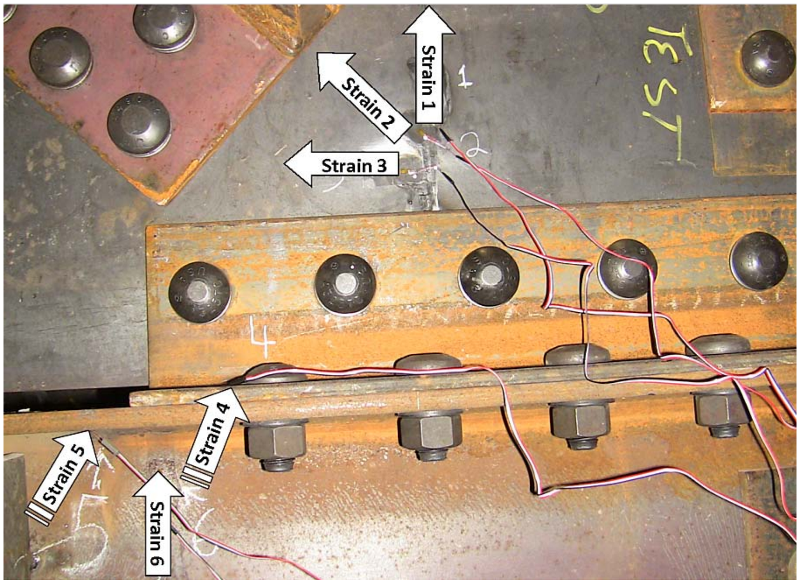

Figure 12 - WC200 Strain Gauges

The initial trial run of the data acquisition software, with gusset plates in place but no brace, provided information to adjust the software, but also unintentionally resulted in pulling the test frame to a drift of nearly 3%, which caused local web yielding and web crippling in the bottom beam in the test frame. Note the beam was intentionally slightly outside the limits for seismic web compactness and the web was slender for shear. The proportions were selected to minimize scrutiny of the sections used upon successful completion, but leaves question as to whether web yielding and crippling would have occurred if a compact section were used. The mistake was the result of an error in the software that pushed the frame passed the target deformation and continued until the program was shut down manually. Also it was determined that the original automated software could not function properly due to high load spikes produced when slip critical bolted joints slipped into and out of bearing. The pressure gauges in the actuator were not designed for dynamic loading, thus would read pressures beyond the recordable limits of the sensors when small, sudden movements in the frame occurred. Based on these limitations, it was decided to conduct the test manually with one computer operator controlling the actuator until the desired test frame displacement was reached. This approach proved to be adequate and was used for all subsequent tests.

The data acquisition software used to collect translation, pressure, and strain data was National Instruments' LabView 2010 Version 10.0.0. All strain gauges used were Vishay Micro-Measurements & SR-4 general purpose strain gauges. The digital string pot used on the braces was Celesco model SR1E with an incremental encoder output signal and a stroke range of 125 in. (3180 mm.) The smaller string pot mounted at the top of the column with a 10 in. (254 mm) stroke was UniMeasure model JX-EP-10 .The linear potentiometer used at the base of the outer column was ETI Systems model LCP12S-100. Details are provided in the associated manuals, see Appendix F.

3.2.2 Test 1 Results – WC250 Brace

Due to "banging" from built-up load and subsequent slip in the joints, much of the information was simply filtered to remove transients. Only data corresponding to the system in motion was filtered. Translation measured along the length of the BRB was not properly collected due to a programming/hardware problem, and thus deemed not representative of brace tendon elongation. There was negligible translation at the base of the test specimen, as expected. The applied load vs. displacement history exhibited stable and repetitive behavior with positive incremental stiffness, see Figure 13.

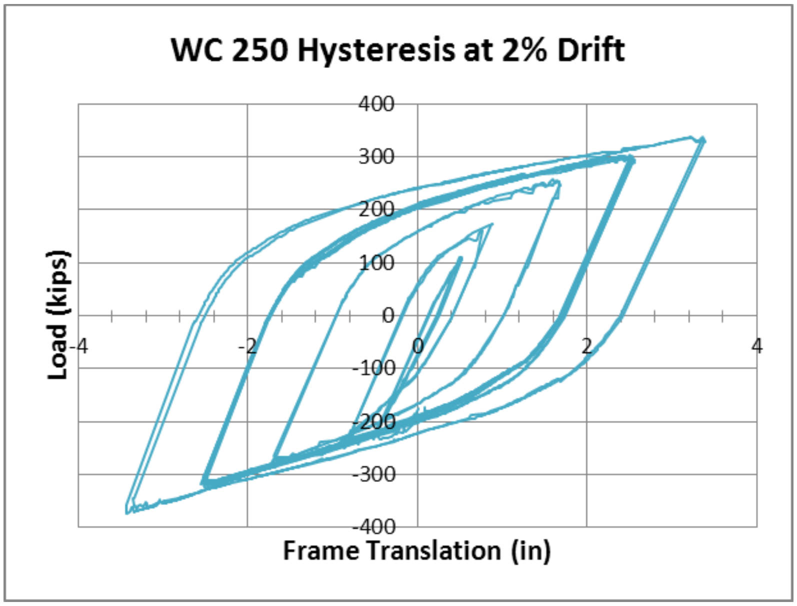

Figure 13 - Test 1 WC250 Hysteresis

The test regime was designed such that the frame accumulate translation would reach 131.6 in. (3343 mm). Actual accumulated frame translation was measured to be 134.5 in. (3416 mm). Because tendon elongation was not properly measured during this test, the ratio of inelastic deformation to frame translation from the WC200 test was used to approximate the cumulative inelastic deformation for the WC250 test. This is reasonable because tendon length and yield stress are similar between the two braces. Using the ratio from the WC200 test, the cumulative inelastic deformation for the WC250 was approximately 64.7 in. (1642 mm), which is nearly 400 times the calculated yield deformation and approximately twice the AISC minimum requirement of 33.2 in. (843 mm).

Strain data are shown in Figure 14 through Figure 20. SG_1 measures strain on the gusset in the vertical direction. The strain shows an asymmetrical response to load. At an assumed steel modulus of 29,000 ksi (200 000 MPa), the max stress in the vertical direction was 7.2 ksi (50 MPa) at a strain of ϵ=247μ. Hereafter, similar data are paired, e.g., (247μ, 7.2 ksi) and the results are discussed in terms of stress.

SG_2 is consistent with the axial forces from the brace into the gusset plate, and matches the hysteresis of the system (symmetric with loading). The max strain and stress are (1300μ, 39 ksi) at SG_2. SG_3 measures the strain in the horizontal direction on the gusset plate along the beam connection. SG_3 exhibited behavior similar to SG_1 with an asymmetric response to loading, (231μ, 6.7 ksi). This asymmetric response is to be expected as the load transferred from the brace to the gusset is 43 degrees from horizontal in relation to SG_1 and SG_3. With this orientation of the brace, the vertical component of strain (SG_1) is affected more by tension forces from the brace and less by compression when the gusset is bearing on the bottom beam. The horizontal strain (SG_3) is more affected by compression forces from the braces.

SG_4 was located along the bottom angle connecting the gusset plate to the bottom beam, positioned perpendicular to the longitudinal beam axis. The gauge was positioned next to a bolt and reported a value largely in excess of 36 ksi (250 MPa) specified yield stress (2100μ, 60.9 ksi) when the brace was in tension and the angles resist forces through bending. Much lower values were present when the brace was in compression and the angles were bearing on the beam flange. At the maximum strain recorded in tension the approximate stress was calculated to be (718 μ, 20.8 ksi). Stress in excess of theoretical yield is not surprising at this location as the stresses vary considerably across the outstanding leg of the angle and concentrations are likely present near bolts.

SG_5 measured strain perpendicular to the length of the bottom beam on the underside of the beam's top flange. The stress does spike close to yield during the two largest displacement cycles at approximately (1840μ, 53.5 ksi) which is reasonable given the higher rotations of the frame at this point and thus more tension near the bolt holes in the top flange. Similar to SG_4, concentrations likely are present near the bolts.

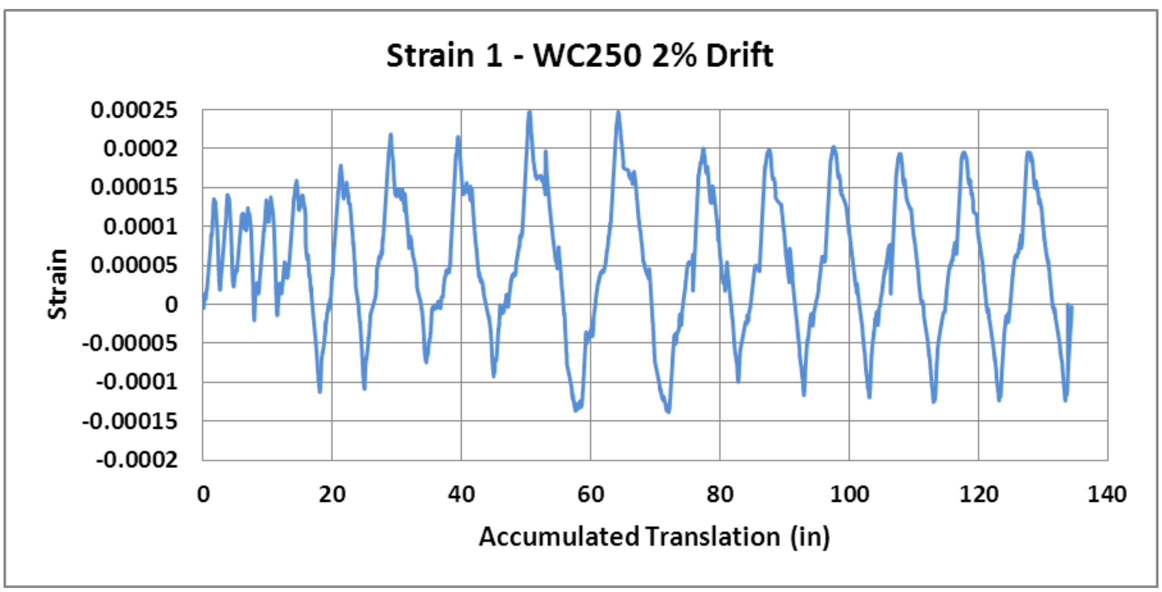

Figure 14 - Test 1 WC250 SG_1

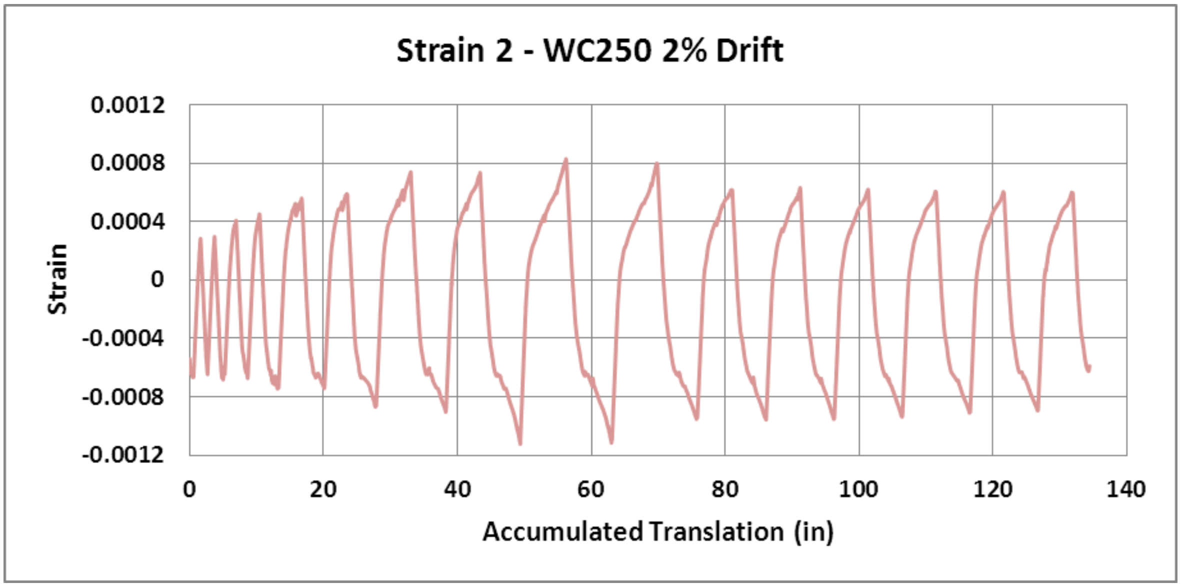

Figure 15 - Test 1 WC250 SG_2

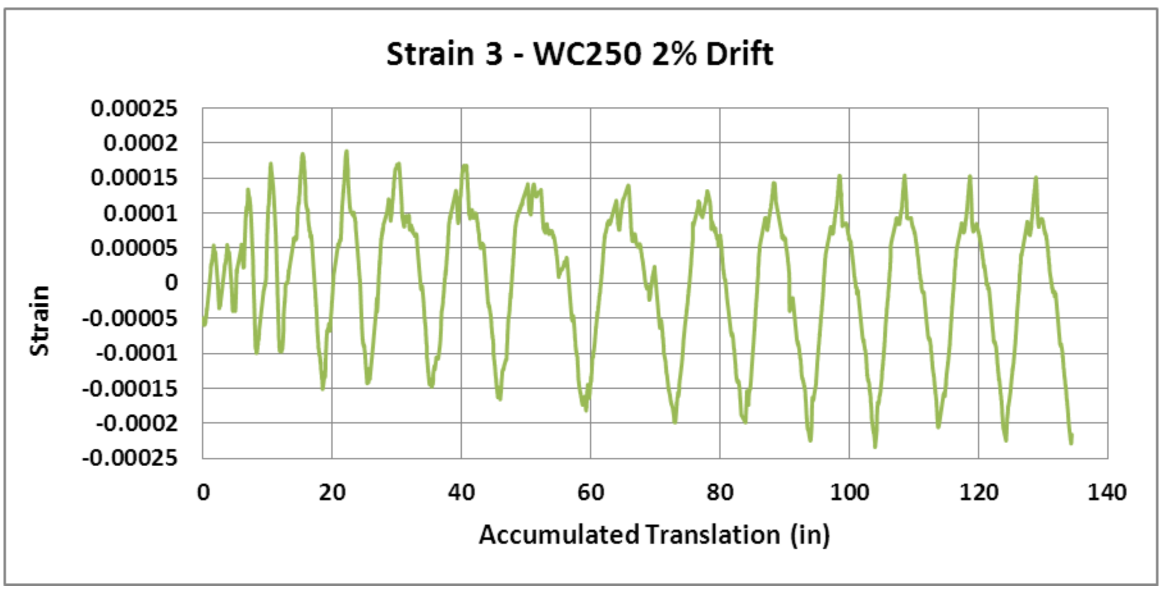

Figure 16 - Test 1 WC250 SG_3

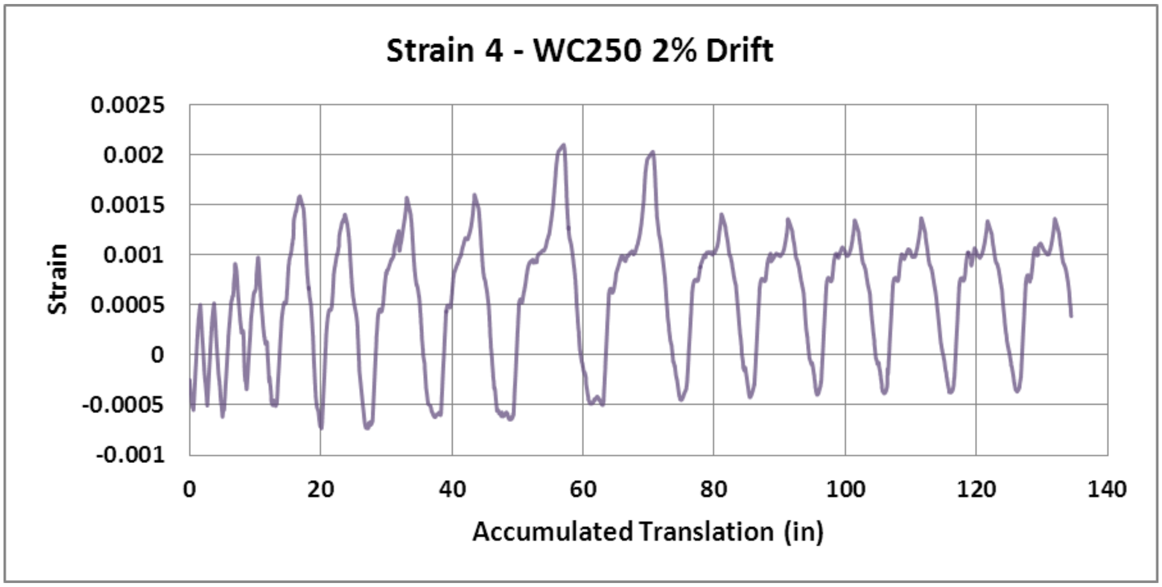

Figure 17 - Test 1 WC250 SG_4

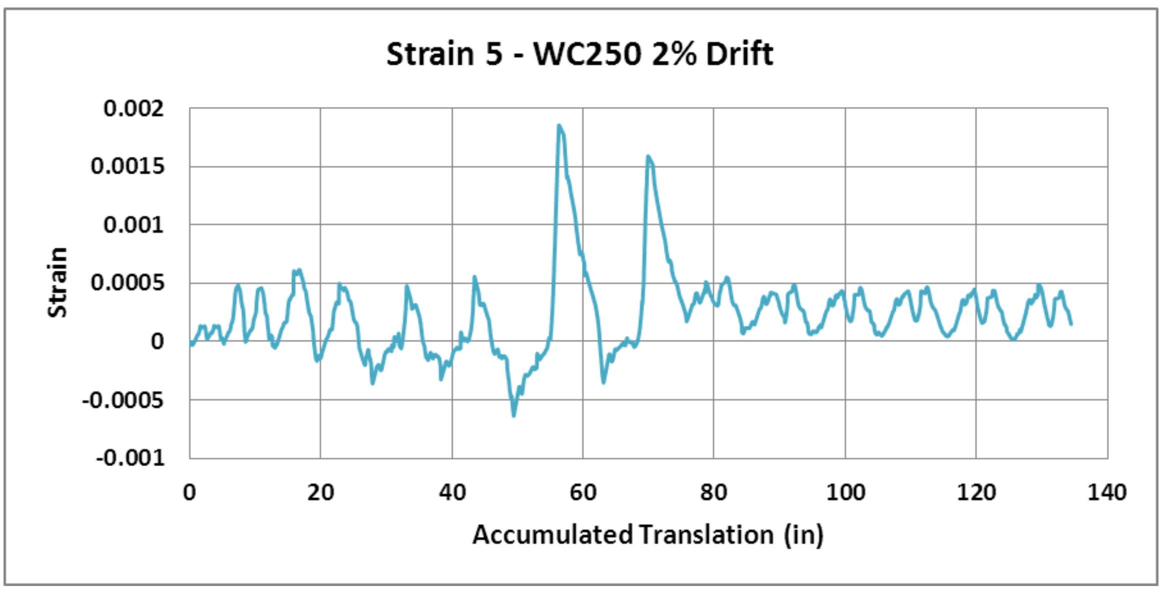

Figure 18 - Test 1 WC250 SG_5

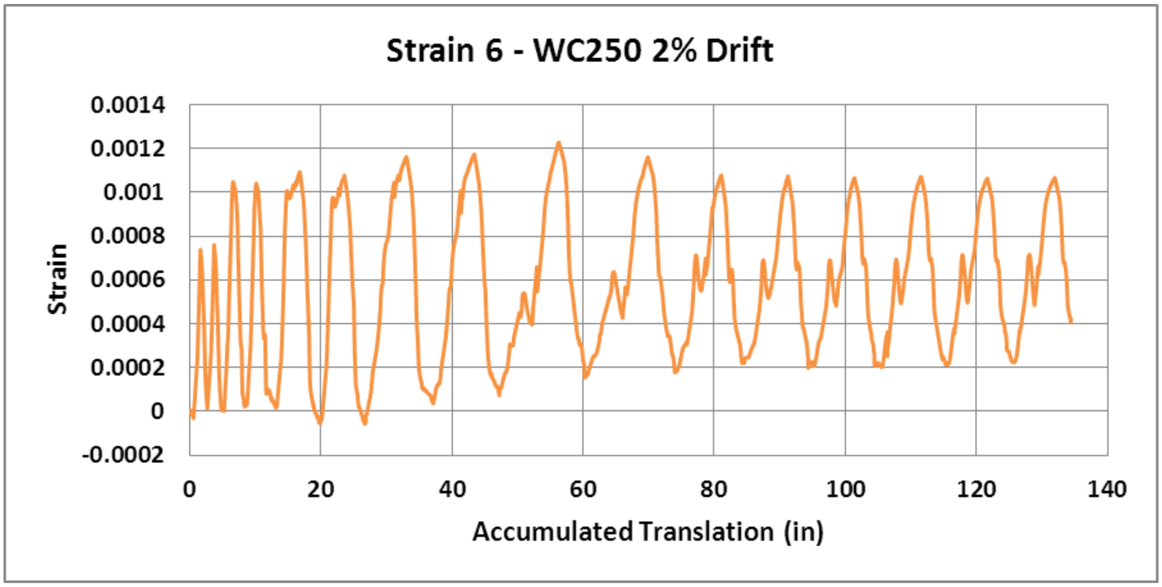

Figure 19 - Test 1 WC250 SG_6

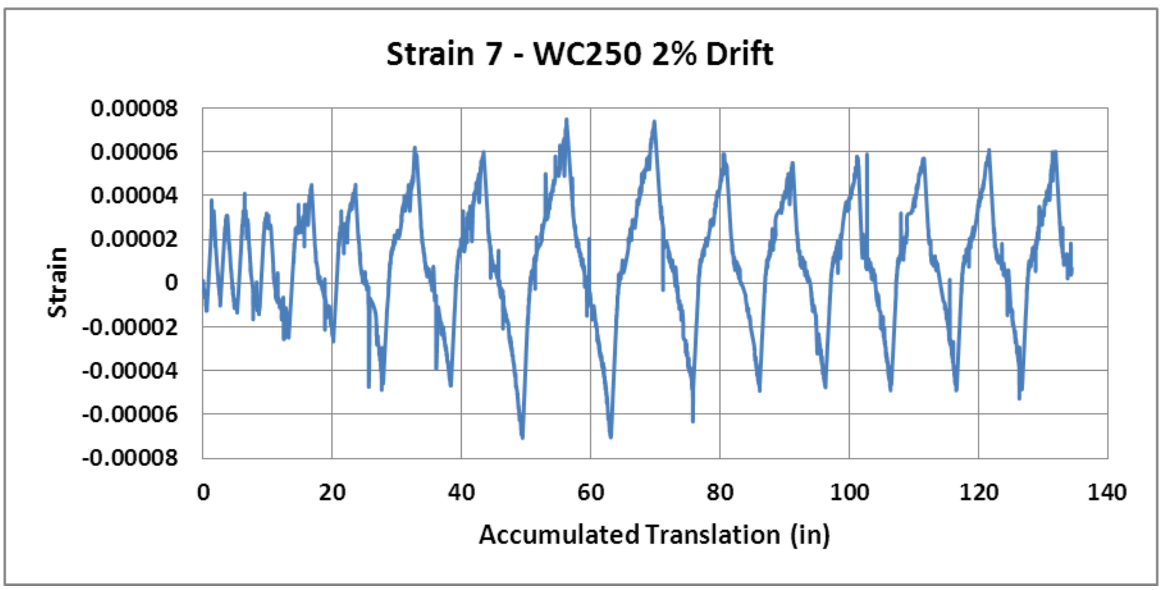

Figure 20 - Test 1 WC250 SG_7

SG_6 measures strain in the angle connecting the gusset plate to the column near the outermost bolt in the horizontal direction. This connection shows similar behavior to SG_4 with higher strain when the brace is in tension and lower strain in compression (bearing on the flange). The approximate stress measured was (1220μ, 35.4 ksi), which indicates lower stress in this element than in the angles connected to the beam or in the beam flange.

SG_7 was only recorded in the WC250 test, and was measured roughly at the work point of the upper beam where the actuator load was applied to the test specimen. Stresses at this point were low, reaching a maximum of near (76μ, 2.2 ksi). This value suggests approximately 40 kip (178 kN), or 12% of the load in the actuator, was transferred to the beam. Thus 88% was resisted by the brace.

The University of Utah reported a maximum force in WC250 during testing to be 404 kips (1797 kN) in tension and 474 kips (2108 kN) compression. This project used a connection design axial force in the brace of 435 kips (1935 kN) in tension and 496 kips (2006 kN) compression. During testing of the WC250 the maximum axial force achieved in the brace was 404 kips (1797 kN) in tension (equal to the University of Utah max) and 451 kips (2006kN) compression (95 percent of University of Utah max).

SG_1 through SG_3 can be used to determine the state of strain (or stress) in the gusset plate along the brace located at the point of coincidence of the gages. See Figure 9 (and Figure 12 for the WC 200). Given the three normal strains at the peak load of 451 kips (2006 kN), the shear strain can be determined to be (229 μ, 6.6 ksi). This corresponds to the maximum principle shear stress of 25.0 ksi and principle normal stresses of 24.9 ksi, and 25.1 ksi, see Appendix D Figure 59 for calculations. The von Mises yield criterion would predict yield at approximately 0.57 x F_y = 20.8 ksi. Therefore the max shear stress in the gusset exceeded the theoretical yield stress at the maximum load during the test.

While the upper connection of the test specimen was not instrumented with strain gauges, visual inspection of the primary members and connection components after the test indicated no noticeable damage. In connecting the gusset plate to the web of the column, the relatively high out-of-plane flexibility of the column web appeared to accommodate frame rotation without distress to connection components or primary members. Consequently, in consideration of a serviceable system, this configuration was demonstrated to be significantly more desirable than connecting to the column flange.

3.2.3 Test 2 Results –WC200 Brace

The WC200 test resulted in similar behavior to the WC250 test. Filtering similar to the previous test was used. Translation along the length of the brace was properly measured in this test and produced usable hysteretic information. The frame translation verse applied load also exhibited stable and repetitive behavior with positive incremental stiffness, see Figure 21.

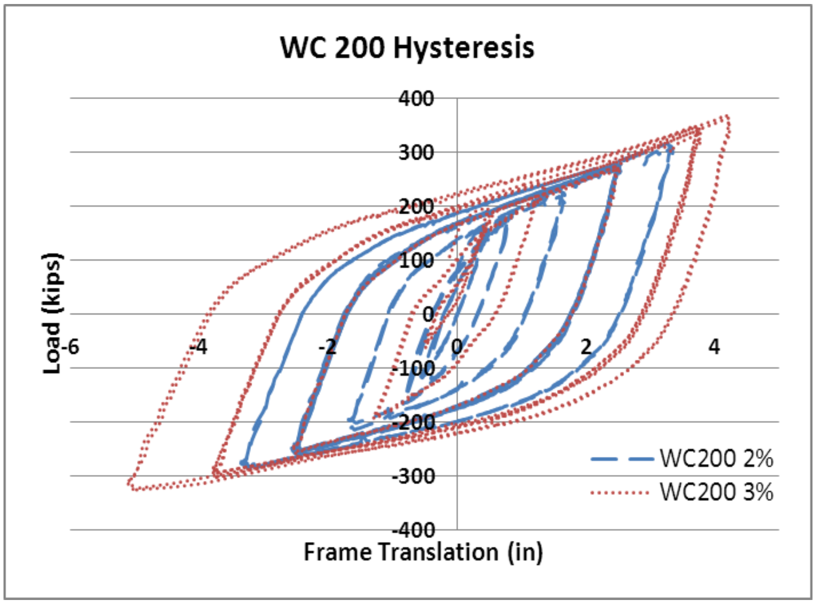

Figure 21 - Test 2 WC 200 Load-Translation

The total brace elongation is illustrated in Figure 22. The second regime of cycles for 3% drift begins at scan 6000. Translation along the brace shows a slightly asymmetric response to loading with larger displacements in tension than in compression during the 2% test, and larger displacements in compression than in tension during peak loads in the 3% test. The maximum elongation during the 2% drift test was 2.1 in. (53 mm) in tension and 1.9 in.(48 mm) in compression. The maximum elongation during the 3% drift test was 2.8 in. (71 mm) in tension and 2.9 in. (74 mm) in compression equal to 2.5% and 2.6% strain, respectively.

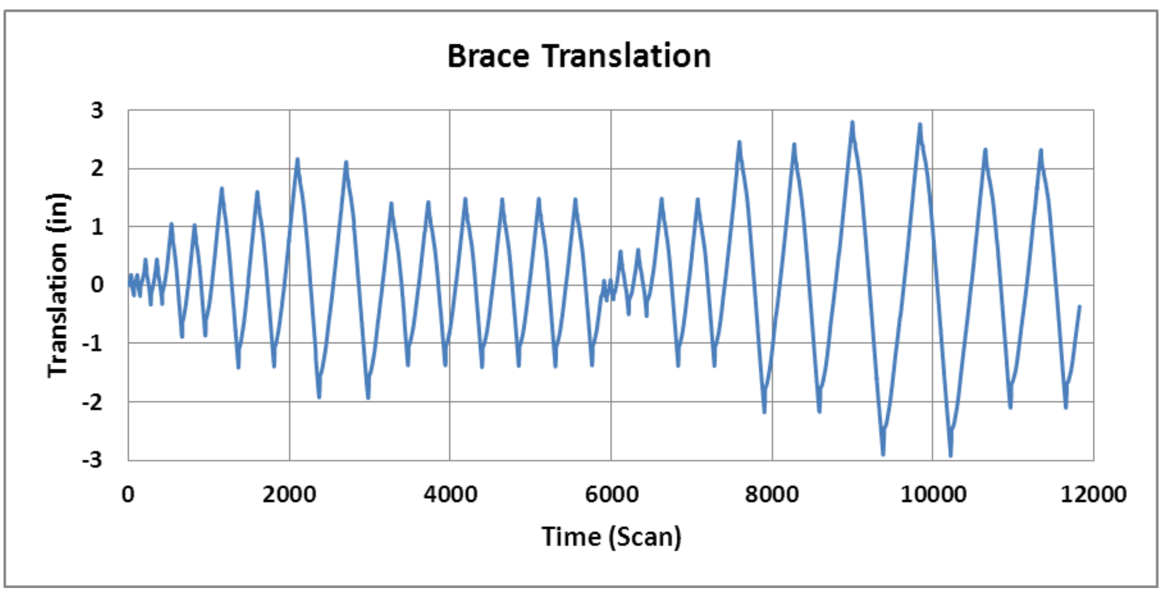

Figure 22 - WC200 Brace Translation

Strain data for the WC200 test shown in Figure 23 through Figure 28 displays the two consecutive tests done with 2% drift first, followed by 3% drift. The second test at 3% drift begins at approximately scan 6000. See Figure 22. The testing regime reached an accumulated frame translation of 133.3 in. (3386 mm) during the 2% drift test, and reached a total of 265.9 in. (6754 mm) by the end of the 3% test. The cumulative inelastic axial brace deformation, as measured by the string pot on the exterior of the brace, was 64.1 in. (1628 mm) for the 2% drift test and 68.4 in. (1737 mm) for the 3% test. Thus the total cumulative inelastic deformation was 132.5 in. (3366 mm), which corresponds to almost 800 times the calculated yield deformation or approximately four times the AISC minimum requirement.

SG_7 at the top of the test frame was not measured in this test because of broken wiring. SG_1 through SG_5 showed behavior similar to that in the WC250 test. SG_6 was at a different location in the WC200 test and measured the stresses in the beam web perpendicular to the long axis of the beam. It was observed by strain at SG_4 that once the connection angle yielded it performed at approximately the same strains during the 2% drift test as when subjected to 3% drift. The "upward ratcheting" of SG_4 is due to yielding. Note that the downward shift is consistent with the yield strain of strain-hardened steel.

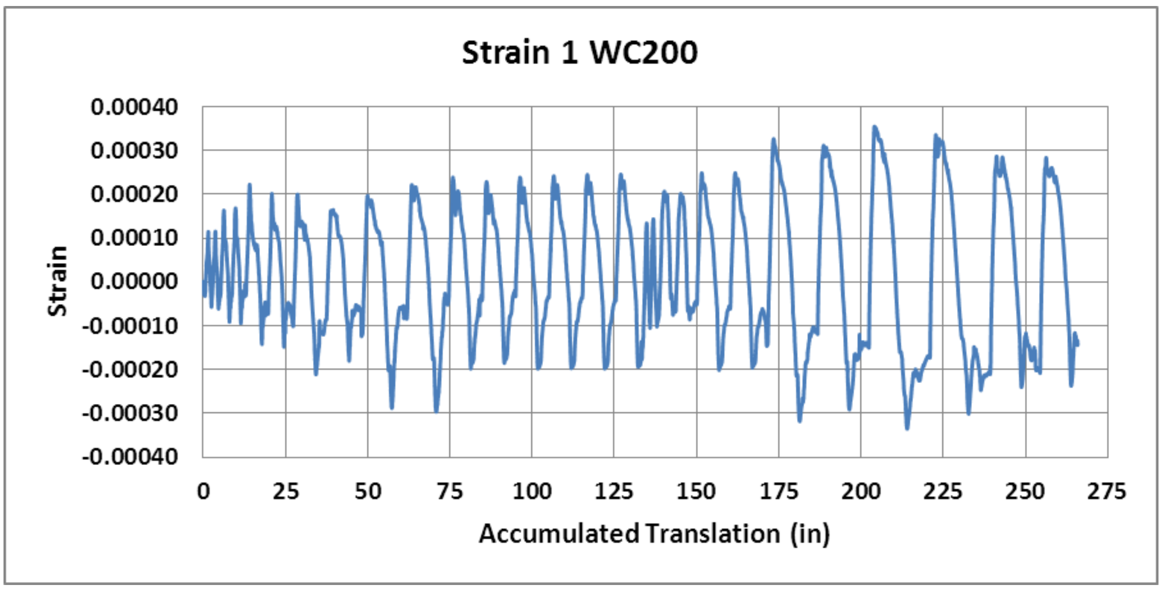

Figure 23 - Test 2 WC200 SG_1

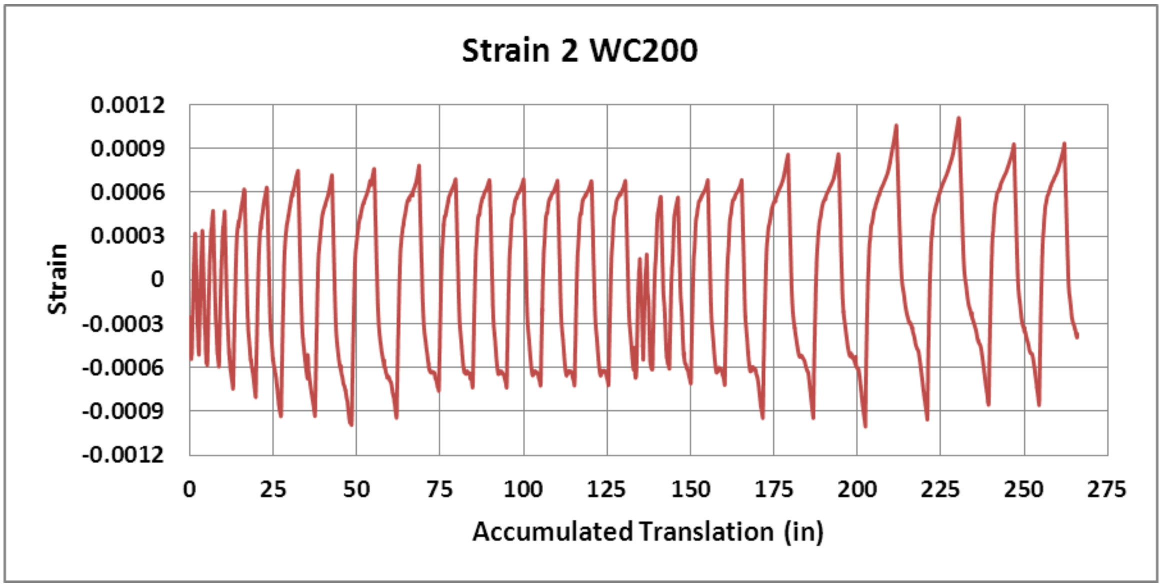

Figure 24 - Test 2 WC200 SG_2

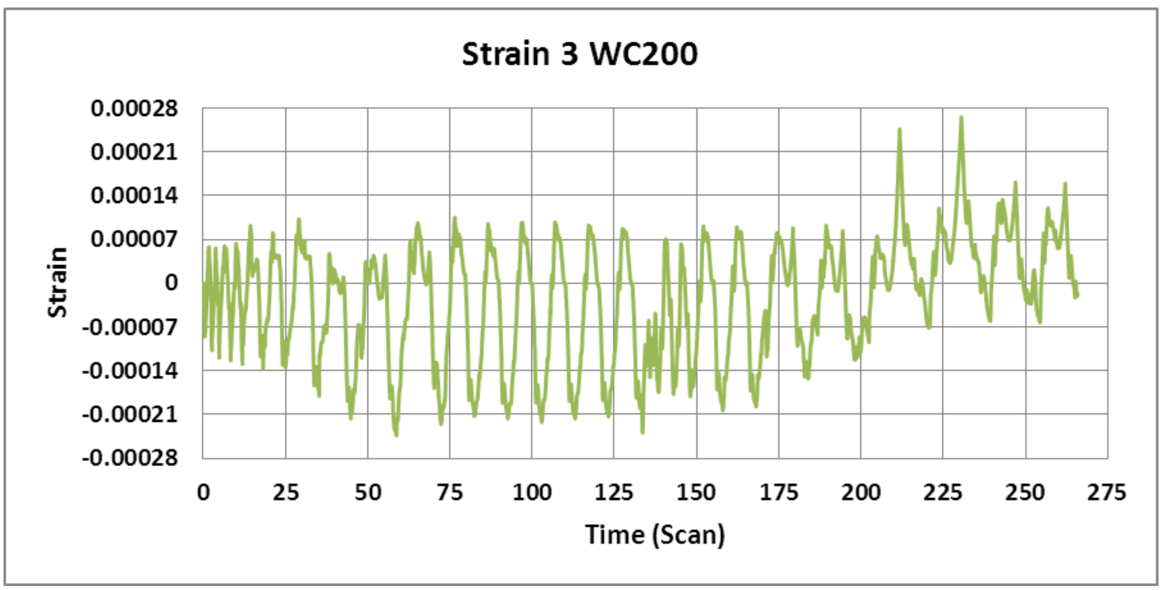

Figure 25 - Test 2 WC200 SG_3

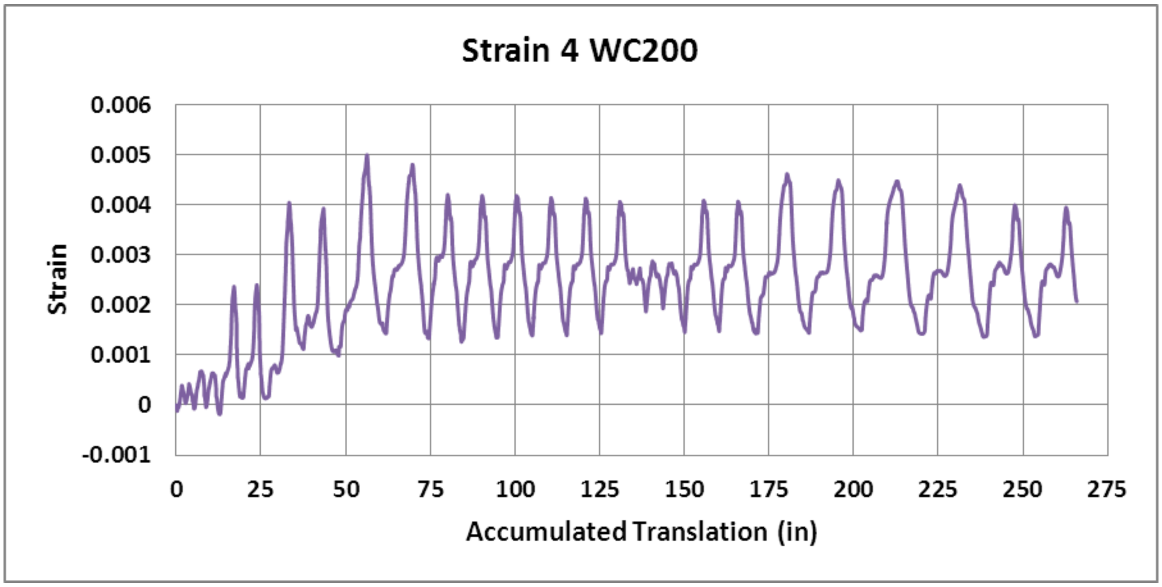

Figure 26 - Test 2 WC200 SG_4

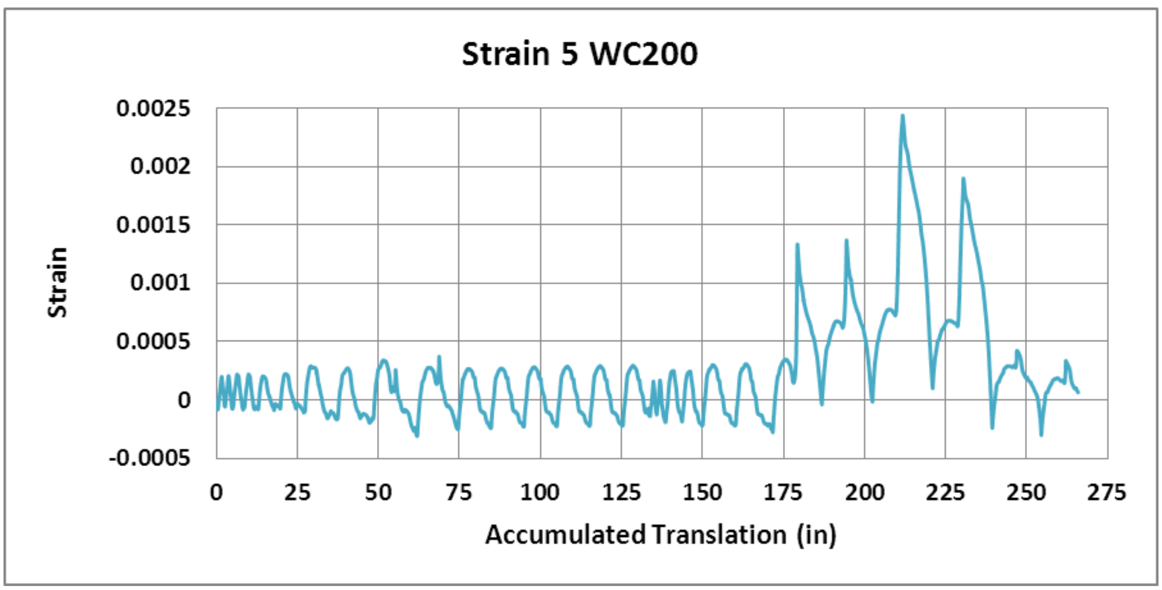

Figure 27 - Test 2 WC200 SG_5

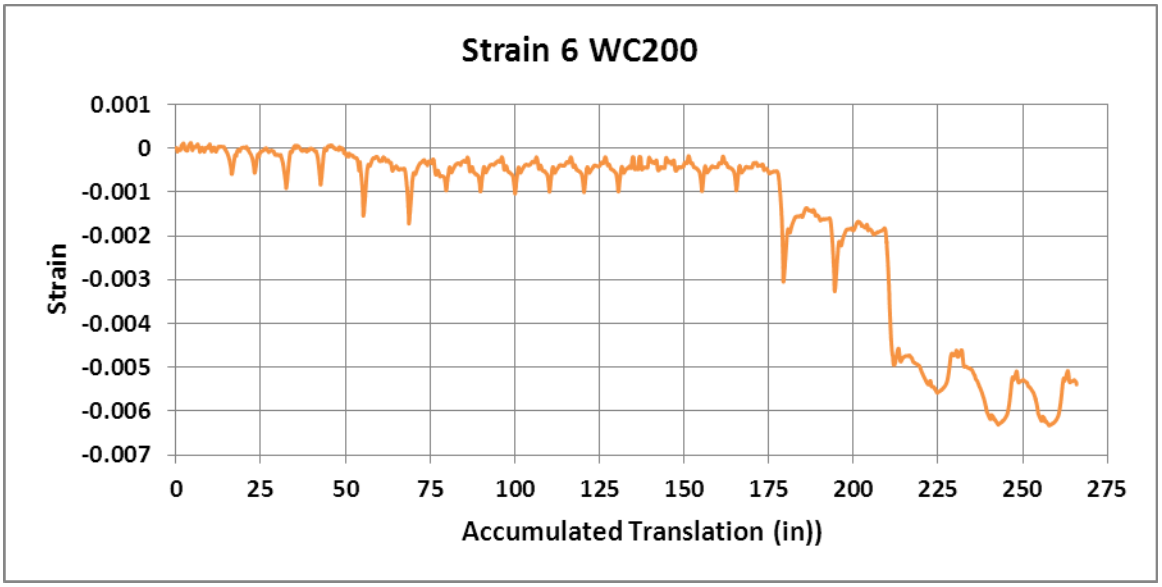

Figure 28 - Test 2 WC200 SG_6

SG_5 and SG_6 showed some interesting behavior in the beam once subjected the 3% drift cycles. It is observed that after an accumulated translation of 175 inches (4445 mm) SG_5 shows the flange close to yield at a stress of 38.6 ksi (268 MPa), and at the same time SG_6 shows that the web is yielding and reaching a strain of over 6000μ. At this cycle the brace was in tension; however because of the frame rotation the angle between the column and beam closes and tends to "pinch" the gusset. This results in compression in the beam web. The web continues to exhibit some nonlinear behavior as it buckled slightly out of plane and thus Figure 28 shows total strain (compression and bending) due to buckling.

Similar to the WC250, post-test visual inspection of the primary members and connection components at the upper connection indicated no noticeable damage. This again suggested connecting one side of the gusset plate to a relatively flexible web of a primary member is desirable in consideration of a serviceable system.

3.3 Numerical Modeling

The objective of analytical numerical modeling is twofold:

a. Use the available BRB design parameters to verify the design of the test frame and reaction frame.
b. Compare the numerical model to the observed test results with no "tuning" of the numerical model or BRB backbone curves.

With testing of the computer-simulated model, the linear and non-linear behavior for the brace and test frame can be verified. Thus methods for both linear and non-linear frame analysis can be developed based upon the test results. With this information, accomplishing the second objective provides valuable modeling parameters for use in designing and evaluating future frame and/or building models. Correct stiffness, yield points, and BRB behavior can be determined for future use.

Material and brace properties used are from previous research and testing performed outside of this project. Tensile strength for the brace cores were reported by MSI Testing and referenced by Star Seismic, which was also used in the analytical numerical modeling. (See Appendix D – Data Sheets for MSI results) The tensile testing results are further discussed in the following section.

Research on the Star Seismic braces was referenced and reviewed prior to initial modeling of the braces and the test frame to verify the given Star Seismic parameters. Full-scale testing of the braces completed by (Romero et al., 2007) provided regression equations to model the backbone curves that were normalized by yield strength. The results from axial tests performed on seven BRBs were compiled into a single plot to develop the tension and compression strain vs. hardening curves, see Figure 29. Figure 30 illustrates typical results for a BRB, in this case a WC250. Note that a WC250 was used in one of the present tests.

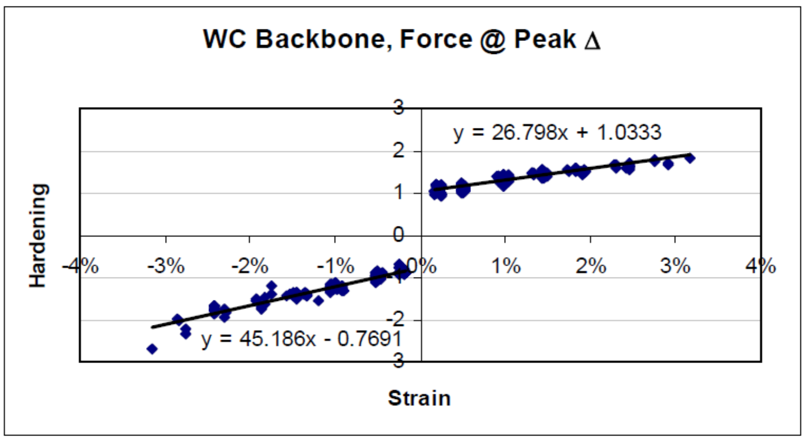

Figure 29 - WC backbone curve (Romero et al. 2007)

The linear regression equations from the resulting curves were established; see EQ 1 and EQ 2.

EQ 1

EQ 2

where EQ 1 is the tension regression equation and ω is the tension hardening (the load at maximum deformation normalized to yield stress). EQ 2 is the compression regression equation and ω_β is the compression hardening.

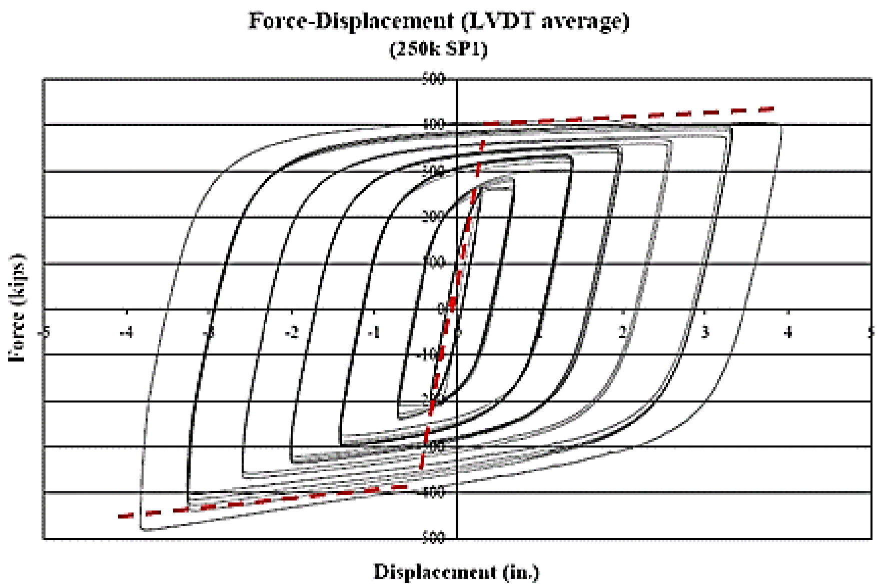

Figure 30 - Typical Load Translation Test Result (WC250) (Romero et al.2007)

The dashed line illustrated in Figure 30 approximates the backbone with a bi-linear function. The normalize version of this function is provided in Eq. 1 and Eq. 2.

Star Seismic provided the University of Utah (Romero et al., 2007) a table with the dimension of the steel core for the braces, which was used to check the accuracy of a spread sheet developed for the research herein, see Table 1.

Table 1 - Dimensions of Steel Core for the Braces (Romero et al. 2007)

			Brace Designation			
			WC150	WC250	WC500	WC780
Specified yield strength, F_y, ksi			41.4	39.9	39.9	39.9
Extension Plate (KP)		Thickness t_{KP}, in	0.75	2	2	4
Extension Plate (KP)		Width b_{kp}, in	9	9	9	18.5
Extension Plate (KP)		Length L_{kp}, in	13	19	23	23
Extension Plate (KP)		Stiffness K_{KP}, kip/in	15,058	27,474	22,696	93,304
Core Plate	# of Plates		1	1	2	4
Core Plate		Thickness t_p, in	0.75	1	1	1
Core Plate		Total Thickness t_T, in	0.75	1	2	4
Core Plate	Transition Zone (TZ)	Width b_{TZ}, in	10	10	10	10
Core Plate	Transition Zone (TZ)	Length L_{TZ}, in	14	14	14	14
Core Plate	Transition Zone (TZ)	Stiffness K_{TZ}, kip/in	15,536	20,714	41,429	82,857
Core Plate	Yielding Zone (YZ)	Width b_{YZ}, in	4.90	5.75	5.75	4.88
Core Plate	Yielding Zone (YZ)	Length L_{YZ}, in	152.7	134.7	134.7	132.6
Core Plate	Yielding Zone (YZ)	Stiffness K_{YZ}, kip/in	698	1,238	2,476	4,269

For the WC200 and WC250 braces provided in this project, the dimensions were calculated from the shop drawings for input into the developed spread sheet. See Appendix A for the shop drawing.

3.3.1 Brace Modeling

In order to verify strength, results from tensile testing on the brace steel cores were provided by MSI Testing Inc. from Salt Lake City, UT (Test Method ASTM 370.) The report was referenced with the Nucor Mill Group of Jewett, TX report for the material properties of the core utilized in the Star Seismic braces. In the case of the steel used for the WC250, MSI Testing concluded that the average yield strength of 43.1 ksi (297.2 MPa), which was greater than that stated by the mill test report of 39.2 ksi (270.3 MPa). Star Seismic noted that the average from the MSI Testing report was used in the design of the braces; thus the same value was used in this project. The same was not observed of the WC200 with an average test value of 43.2 ksi (297.9 MPa) and a mill reported yield strength of 43.5 ksi (300 MPa). Star Seismic used an average of the MSI Testing and the mill report for the WC200 with a value of 43.3 ksi (298.5 MPa).

The brace was first modeled based on the geometric information provided by Star Seismic LLC, and using the brace backbone model (Romero et al., 2007), developed from the University of Utah Full Scale Testing of WC Series Buckling-Restrained Braces.

A backbone curve was developed from the University of Utah test data based on the load at maximum deformation normalized to the yield load for each test specimen. Regression equations were developed to model the Force vs. Translation relationship, including the elastic and inelastic behavior.

The areas and dimensions of the BRB steel core extension plate, transition zone, core plate, and yielding zone were assumed to be proportional to the University of Utah (UT) test specimens. An individual stiffness value for the different zones within the steel core was calculated based on area multiplied by the modulus of elasticity divided by the length. The effective stiffness was then calculated by assuming the individual sections would act as springs in series. See Figure 31.

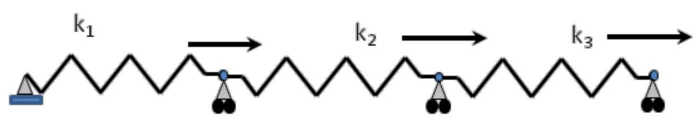

Figure 31 - Springs in Series

The springs represent the transition, core, and extension plates. The equivalent elastic stiffness is computed from:

$$K_{equivalent} = \left[\frac{1}{k_1} + \frac{1}{k_2} + \frac{1}{k_3} \right]^{-1}$$

Given the shop drawings and information, the effective stiffness for the WC200 and WC250 was determined using the assumptions previously stated. The calculated effective stiffness values were used in SAP 2000 v12 with multi-linear links to model the response of each BRB. A multi-linear link and a Wen model were created to ensure that the multi-linear response was accurate when compared to the UT data for validation. (SAP 2000 v12)

Again, the inelastic behavior was modeled using the UT backbone curves. Figure 32 and Figure 33 illustrate the SAP 2000 models of a single BRB using the multi-linear plastic model, the Wen model and data from one of the University of Utah WC250 tests.

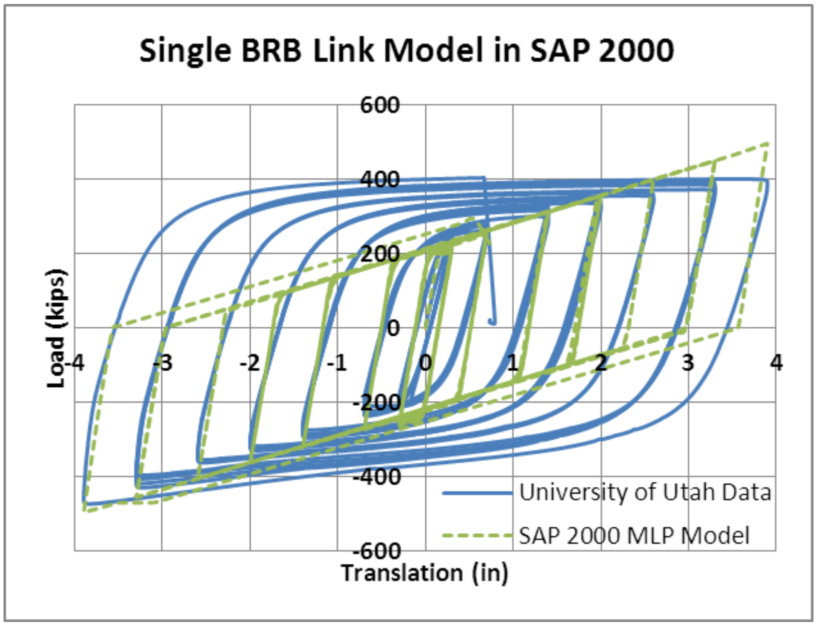

Figure 32 - Single BRB Link Multi-Linear Plastic Model vs. University of Utah Test Data

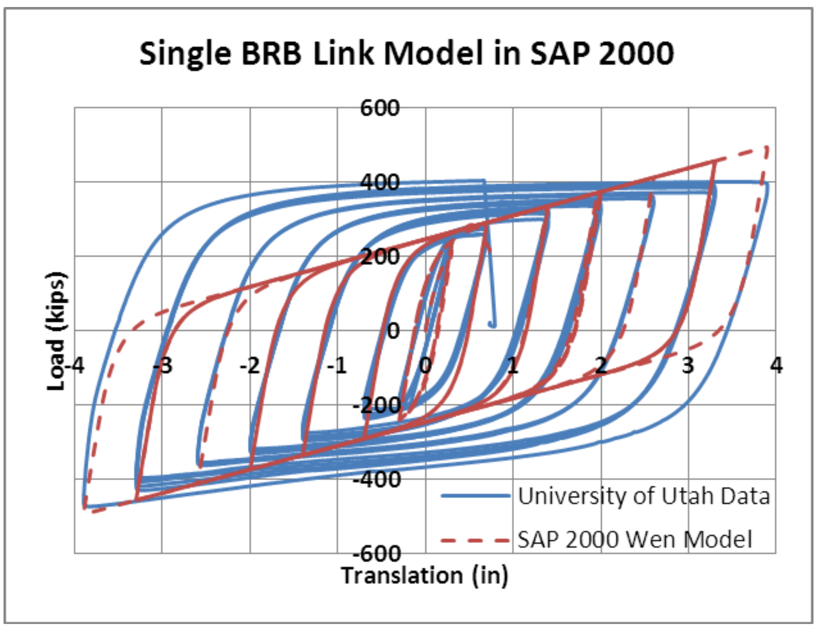

Figure 33 - Single BRB Link Wen Model vs. University of Utah Test Data

By comparison the SAP2000 modeling of single BRBs is more of a coarse approximation of the actual behavior as demonstrated by the University of Utah testing results. Also it is shown that the numerical model does not show any asymmetrical pattern as the actual brace does when loading in compression verses tension.

3.3.2 Full Frame Modeling

Due to the complexity of modeling the entire testing apparatus in SAP 2000 v12 (hereafter SAP 2000), the frame was modeled in multiple steps. First, the geometry of the frame was modeled with undefined shapes and stiffness to determine which frame members would be necessary for the full analytical model see Figure 34. Based on a nominal 100-kip load applied to the top of the frame, each member was analyzed for axial and shear forces to determine its influence on the system during testing. Initial modeling of the angles bracing the test specimen from movement out-of-plane of the load direction, were removed due to an undesirable transfer of shear forces to the test frame in the SAP 2000 model. These angles were connected with single bolt pinned ends in the actual test assemblage, and did not resist any shear forces as they would slip and rotate under frame translations. Constraints were imposed on the nodes where the angles connected to the test specimen as a more effective means of modeling the system. When modeling the large rigid plate connecting the test reaction frames to the test frame, it was determined that deformations in the plate were small enough that the connection could be assumed rigid, the expected result.

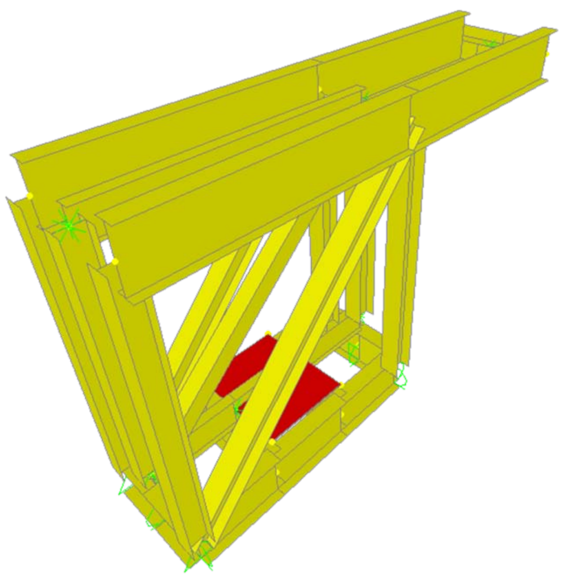

Figure 34 - Initial Full Frame Model in SAP 2000

More load tracking review was done in SAP 2000. By observation, and as expected, it was determined that the majority of the deformation was occurring in the test specimen due to the much greater stiffness of the reaction frames, see Figure 35.

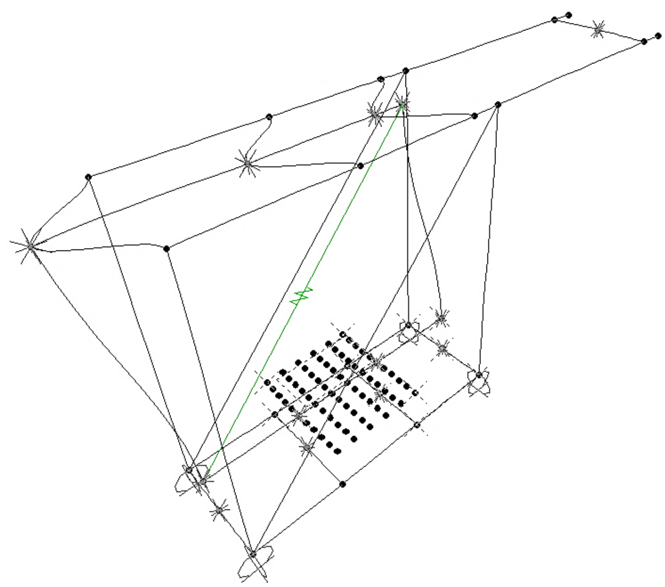

Figure 35 - Full Frame Deformed Shape with 100 Kip Load (SAP 2000)

The next step was to model the test frame alone with constraints on the nodes that would normally be attached to the reaction frame. A few assumptions were made to simplify the model. Connections were assumed to be either rigid or fully pinned, as the actual stiffness of the connections was not fully known. The previously developed links were imported into the test specimen model and placed appropriately, see Figure 36. With the 100 kip load applied to the test specimen, it was determined that the link was working properly when compared to hand computations.

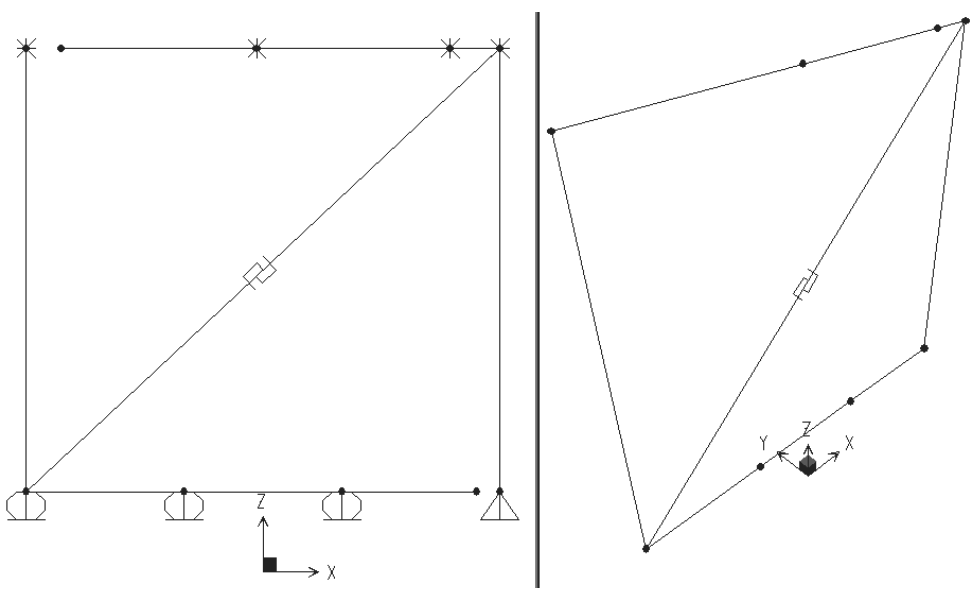

Figure 36 - Simplified Analytical Model (SAP 2000)

3.3.3 Comparison of Numerical Modeling and Experimental Results

By using the link developed in SAP2000, it was possible to run the same time history test on the analytical model as was done on the physical test frame. The target translations for the experimental testing were input into SAP2000 and a displacement controlled loading cycle was run. The results from the multi-linear model of the brace were then plotted against the experimental data for comparison; see Figure 37 and Figure 38. In order to produce a more accurate comparison, the output from the SAP 2000 model was link force, column shear, and axial force in the top beam, which is equivalent to the pressure gauges in the actuator measuring forces on all these elements during the test. Notably the multi-linear model behaved similarly to the experimental model. The WC200 model did predict a slightly higher peak load at maximum positive translation, but at the max negative translation, the model and experimental data are almost identical. The WC250 model is much more in line with the experimental data, and is even slightly conservative at max negative translation having a peak load slightly lower than the experimental data.

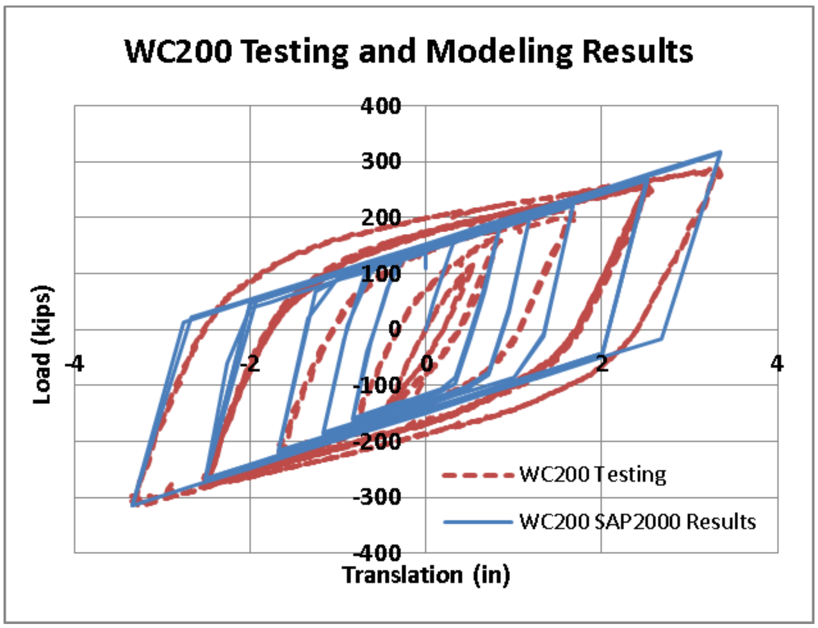

Figure 37 - WC200 Testing and Multi-Linear Plastic Modeling Results

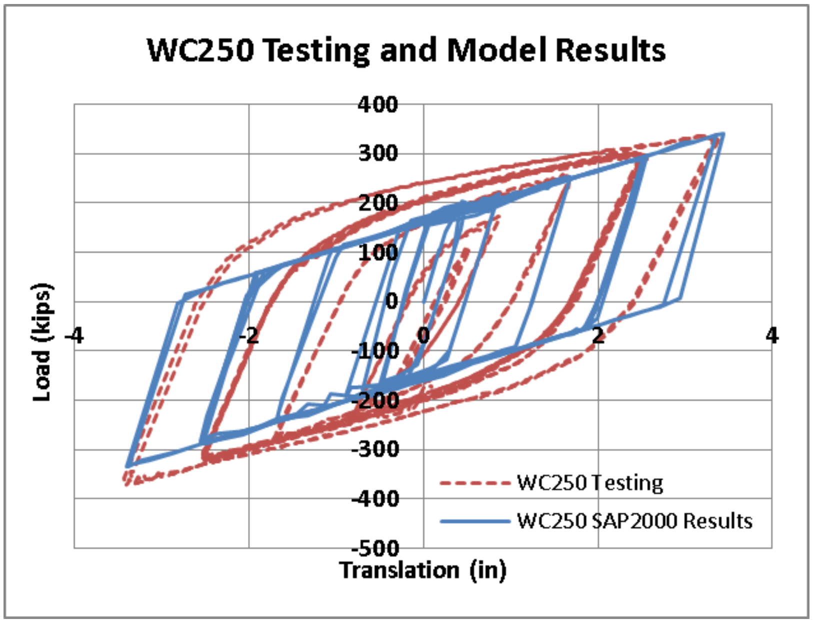

Figure 38 - WC250 Testing and Multi-Linear Plastic Modeling Results

Utilizing Wen modeling of the two braces produced a more accurate hysteresis of the frame behavior than the multi-linear plastic models. The hysteretic loops match more closely with the test data, see Figure 39 and Figure 40, and had a slightly higher value at the maximum displacement similar to the multi-linear plastic model. These similarities suggest that the backbone curve developed from the University of Utah test gives proper values for modeling.

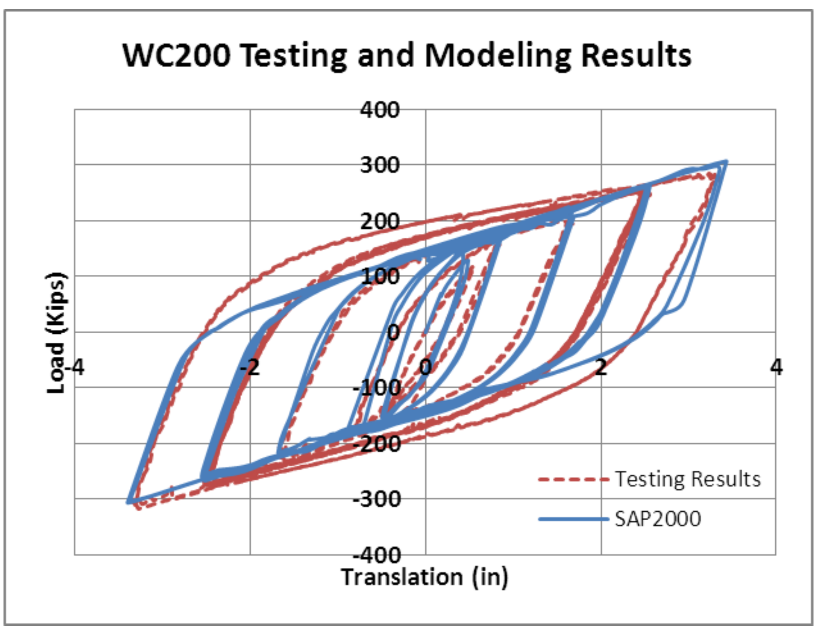

Figure 39 - WC200 Testing and Wen Modeling Results

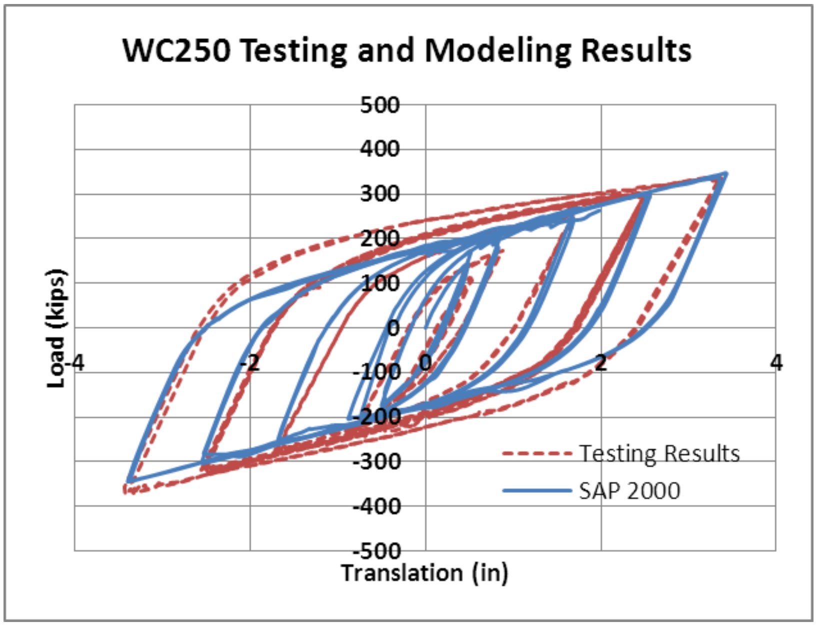

Figure 40 - WC250 Testing and Wen Modeling Results

It should be noted that both the multi-linear and the Wen models are fully symmetrical in their response to loads in tension and compression. This explains the minor offset when comparing the testing results to the SAP 2000 modeling, as the BRB does perform somewhat different in tension vs. compression.

3.4 Conclusions from Modeling and Experimental Testing

- AISC 360 and 341 provisions are appropriate for fully bolted BRBF connections
    - The configurations of connecting gusset plates to the column flange or web demonstrated adequate capacity to carry required loads
    - The use of standard Uniform Force Method and Uniform Force Method - Special Case 2, Minimizing Shear in the Beam-to-Column Connection were shown to be appropriate for connection force distribution
- Rotational stiffness of all-bolted BRBF connections does not attract significant frame load, thus the majority of the load to the frame is delivered to the BRB
- Orienting columns such that the gusset plate is connected to the column web allows for rotation of the gusset connection under large drifts without noticeable damage to the primary beams and columns
- Orienting columns such that the gusset plate is connected to the column flange results in connection restraint against frame rotations that can cause damage to unstiffened primary beams and columns
- Linear and non-linear behavior of BRBF can represented reasonably by analytical modeling using parameters from BRB backbone curves

3.4.1 AISC Acceptance Criteria

According to AISC 341, the required similarities between the brace test specimen and prototype were met in this test due to the full-scale testing apparatus. All of the brace rudiments were met because a full scale brace was used. The cross-sectional shape and orientation of the steel core was exactly how the prototype would be configured. The axial yield strength of the steel core was equal to that of the prototype, and the material for, and method of, separation between the steel core and buckling restraining system were exactly that of the prototype. All connection details and materials used were that of an actual system used in actual building frames.

Loading history and sequence during the testing met or exceeded the AISC requirements outlined in section T6. Plots of applied load versus displacement exhibited stable, repeatable behavior with positive incremental stiffness. The tension testing requirements were met and reported by MSI Testing prior to the BRB testing (see Figure 60 through Figure 63.) Throughout all testing cycles no fracture, brace instability or brace end connection failure occurred.

4 Design Recommendations

The research herein has shown that with proper compression strength and strain hardening adjustment factors for the buckling restrained brace, the connection design provisions of AISC 360 and AISC 341 result in desirable braced frame behavior using fully bolted connections. In addition to the provisions of these documents, the following general recommendations are made to facilitate constructability and maximize connection strength. Furthermore, the following serviceability recommendations are made to promote an easily repairable system in which inelastic damage to the primary beams and columns is minimized.

4.1 General Recommendations

1. Bearing bolts in standard holes or slip critical bolts with oversized holes in one ply of connecting interfaces may be used to connect the ends of buckling restrained braces to gusset plates.
2. Bearing bolts in standard holes should be used to connect gusset plates to double angle connection assemblies, and double angle connection assemblies to primary beams and columns.
3. Bolt rows in the connection angle assemblies may be aligned or staggered. Staggered assemblies are recommended to allow for reduced bolt gauges on the flanges of the primary members.

4.2 Serviceable Recommendations

4. Beam and column flange thickness should exceed connection angle thickness to limit bolt bearing deformations in the primary members.

5. To reduce the possibility of inducing yield in the beam or column flange, the bending capacity of the primary member flange, including the effects of prying action, should exceed that of the outstanding legs of the connection angles. Primary members should be oriented such that at least one side of the gusset plate is connected to the web of either the beam or the column.
    a. Orienting primary members such that the gusset plate is connected to the flange of both the beam and the column results in “pinching” forces between the gusset plate and primary members, which can result in local damage to the primary members. These forces are alleviated by connecting one side of the gusset plate to the web of primary member because of the relative out-of-plane flexibility of the member web.

5 Conclusions

The following are conclusion drawn from the experimental testing and numerical modeling of both the full frame and the individual braces.

- In reference to AISC 341 acceptance criteria, testing of the full scale fully bolted buckling restrained braced frame met all strength requirements, and even exceeded the required testing regimen of two percent drift. The frame design exhibits the ability to withstand multiple seismic events without fracture, brace or primary framing member instability, or brace end connection failure.

- Generally all members in the frame, aside from the non-seismically compact beam, and connections remained elastic, thus the inelastic deformations were substantially limited to the brace.

- The serviceable system was proven through testing of the WC250 brace followed by successive testing of the WC200 brace, through which the frame performed substantially as expected. The ability to easily replace the braces and connection components, and still have full functionality of the frame demonstrates the advantages of the fully bolted design.

- The methods used to develop a numerical model of the buckling restrained braces in SAP2000 were effective, and could be easily adapted to different brace sizes for various systems. Utilization of the link properties in a full frame model accurately predicted behavior of the system. Multi-linear approximation was adequate to model the behavior of the BRB in the frame, but the Wen model provides a more accurate prediction including the nonlinear transition near yield.

6 Acknowledgements

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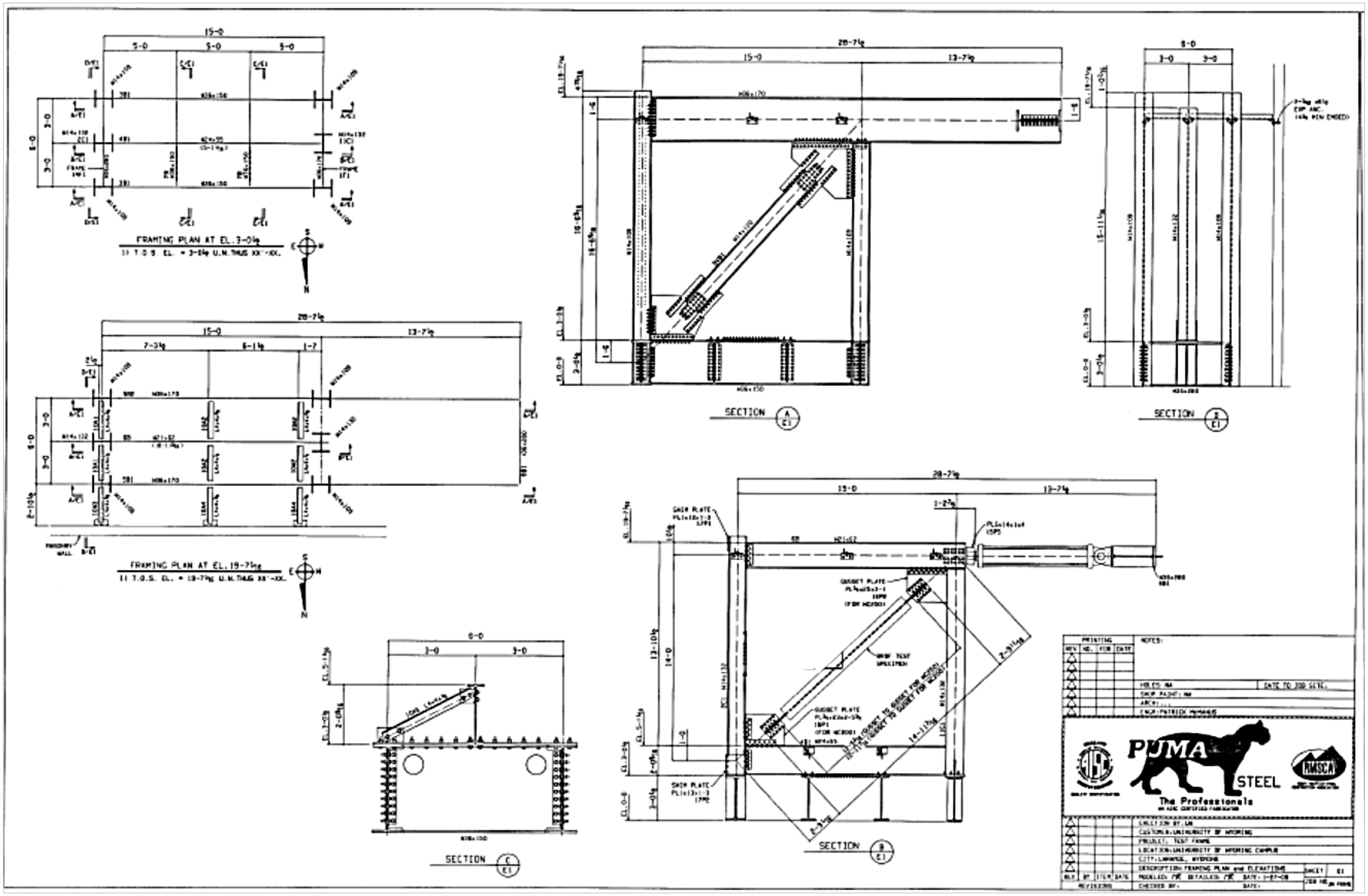
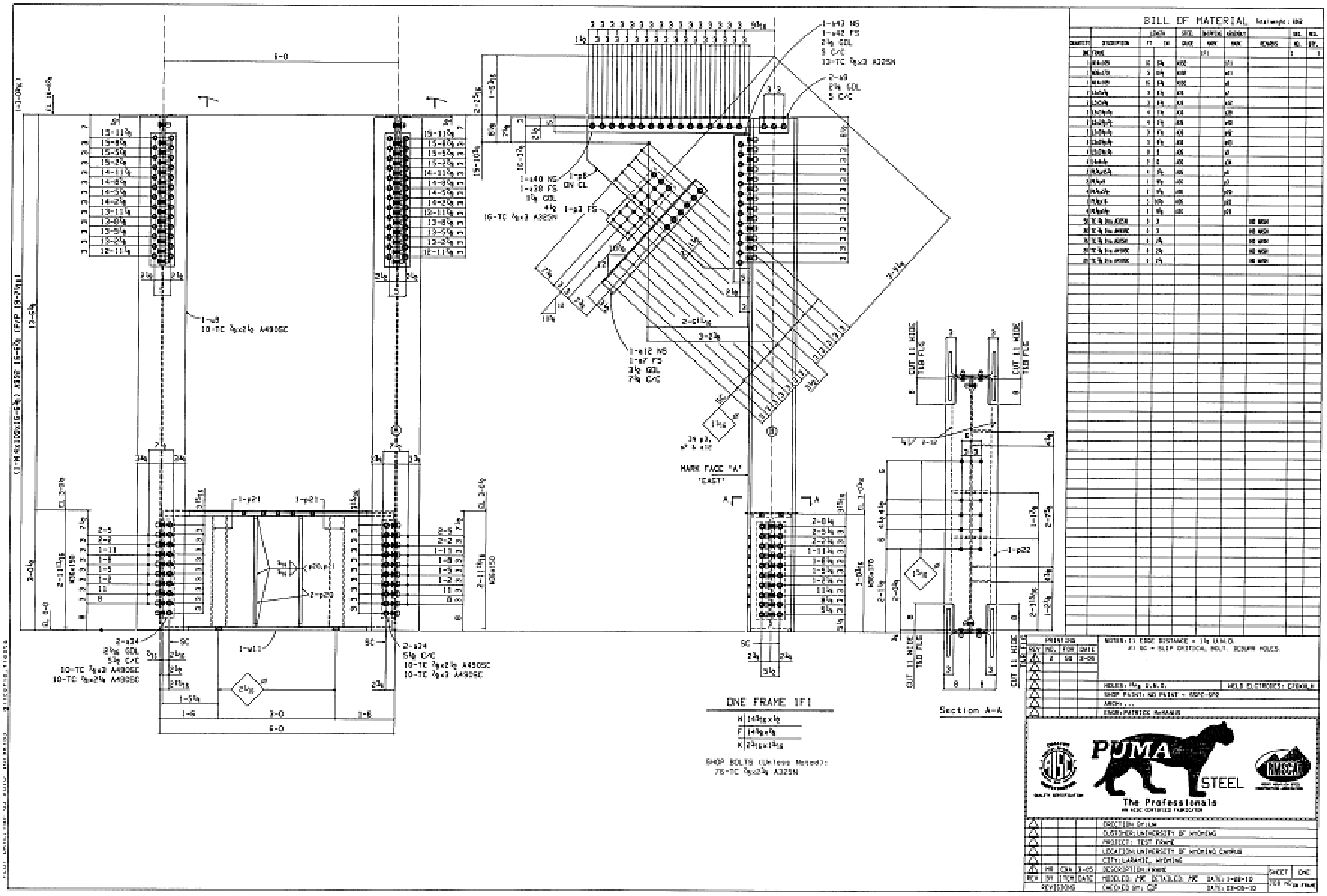
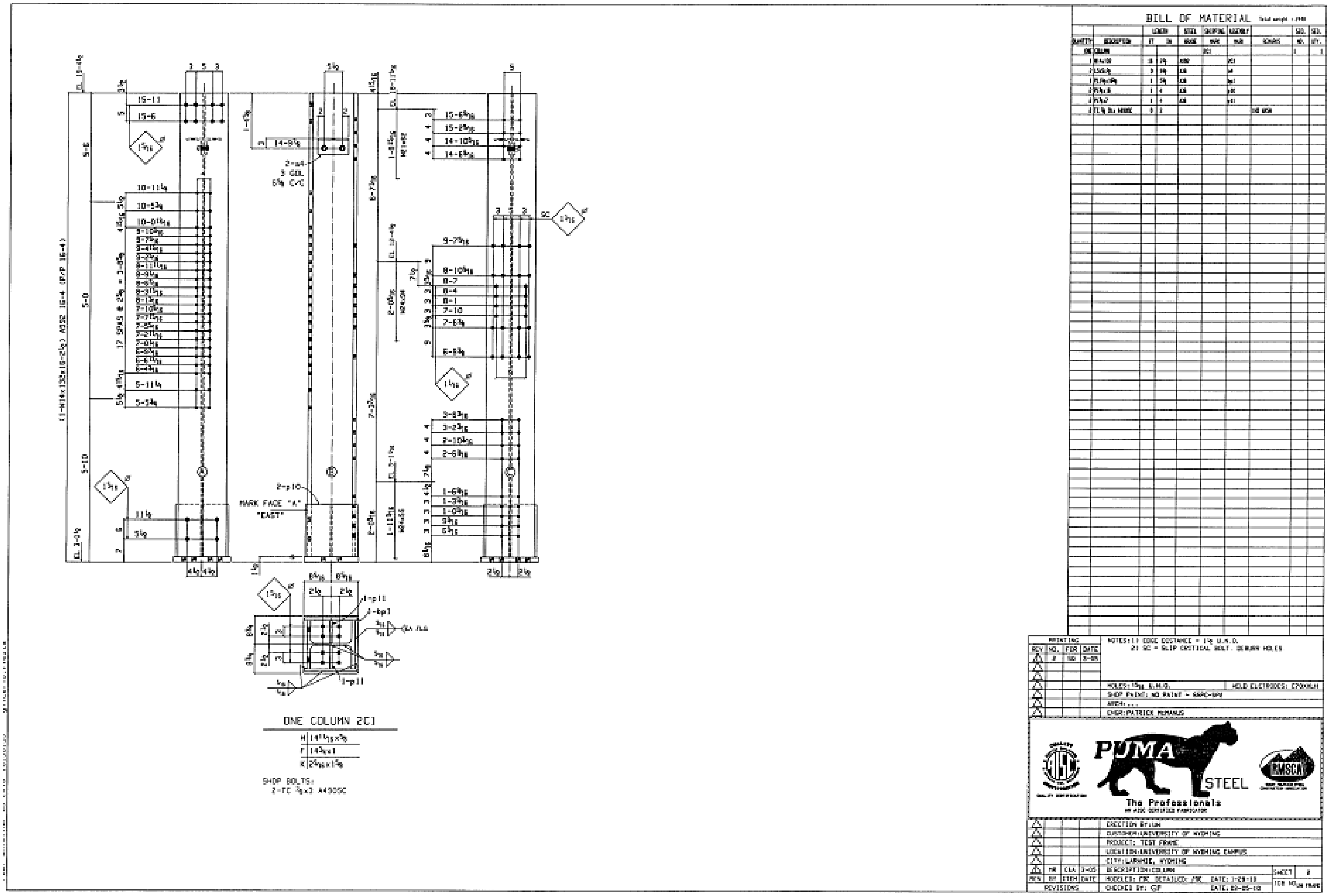
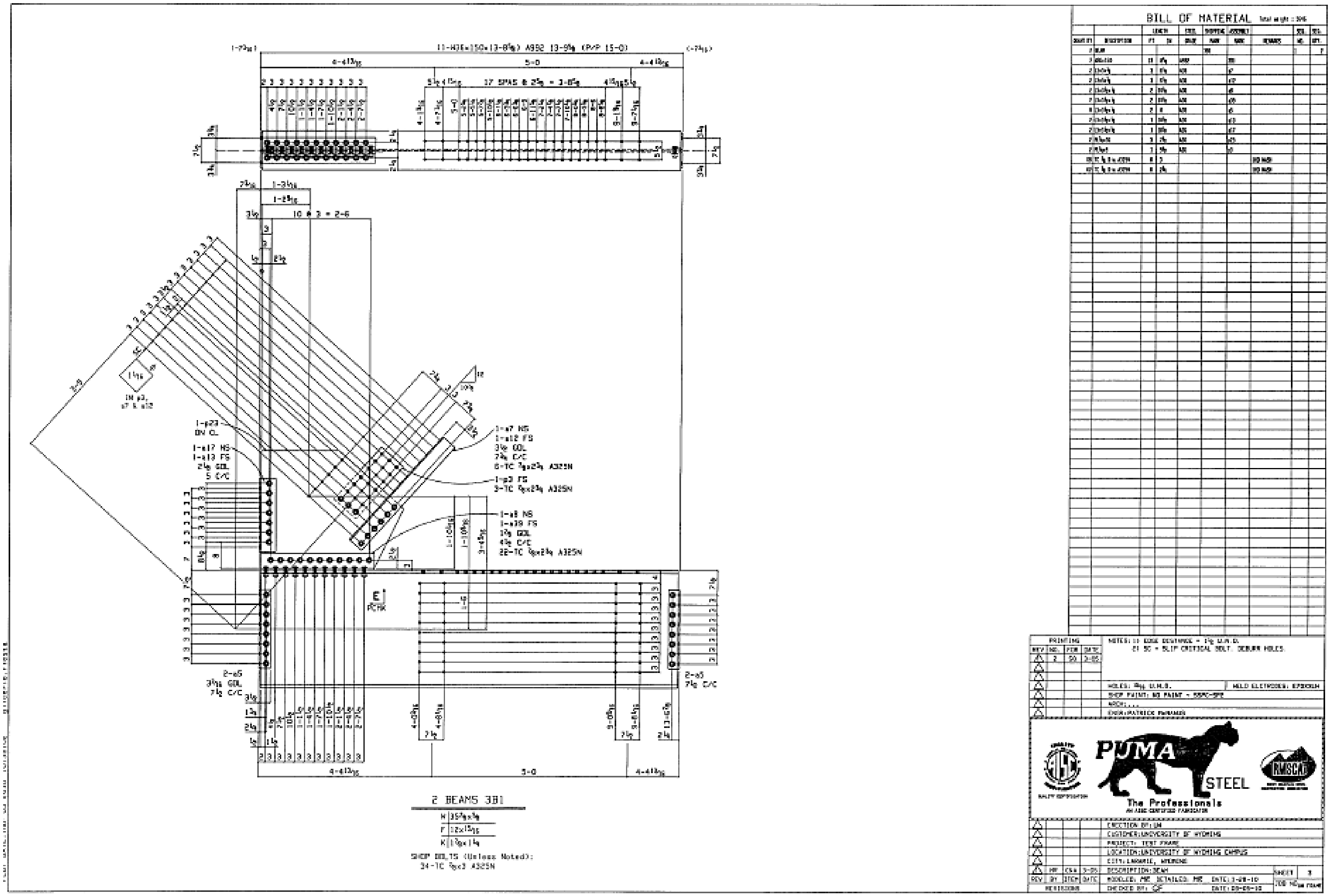
8 Appendix A – Experiment Test Drawings

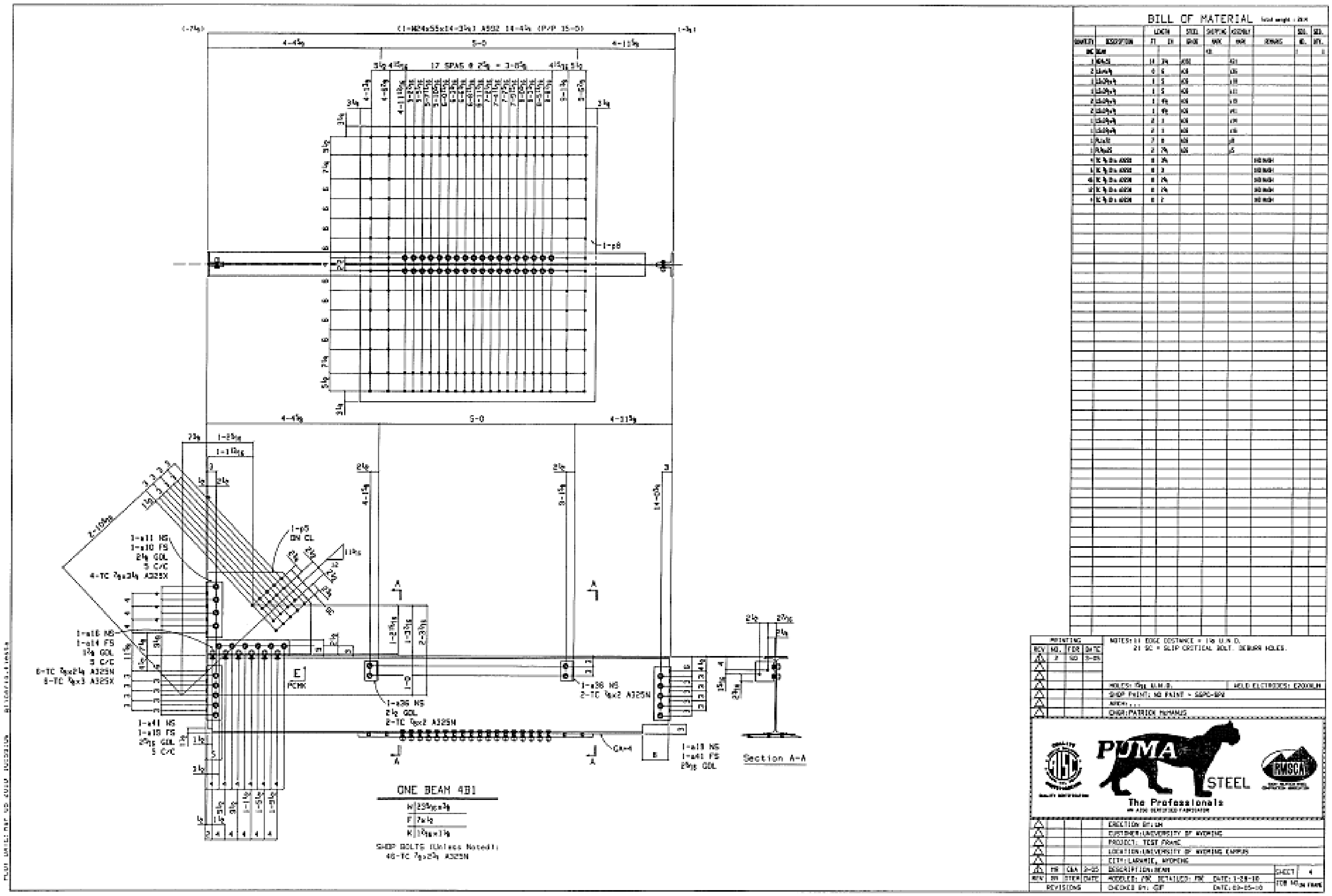


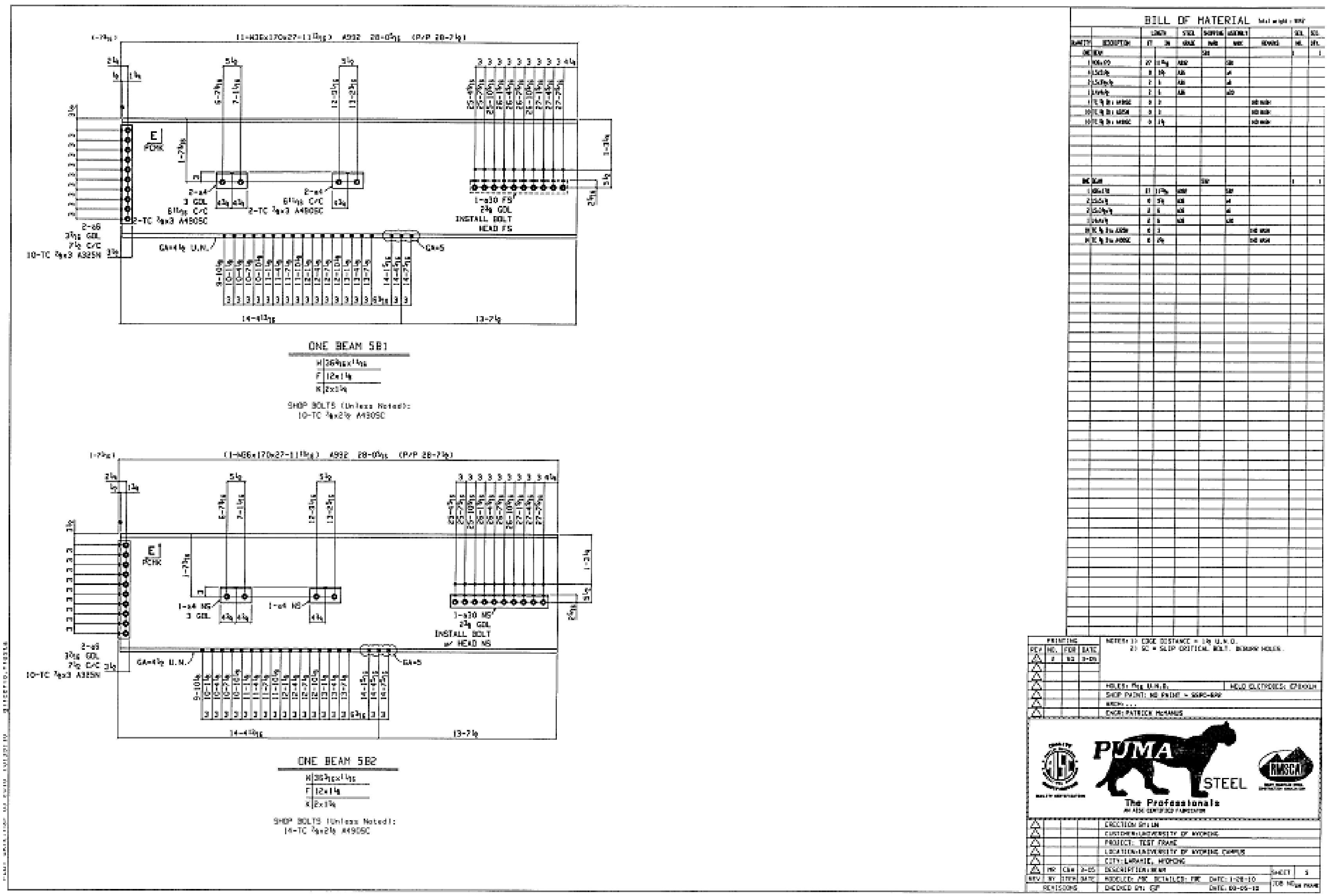

BILL OF MATERIAL
MARK FACE "A" "EAST"
ONE FRAME 1F1
SHOP BOLTS (Unless Noted):
Section A-A
CUT 11 WIDE
PUMA
STEEL
The Professionals
CUSTOMER: UNIVERSITY OF WYOMING
PROJECT: TEST FRAME
LOCATION: UNIVERSITY OF WYOMING CAMPUS
CITY: LARAMIE, WYOMING
DESCRIPTION: FRAME



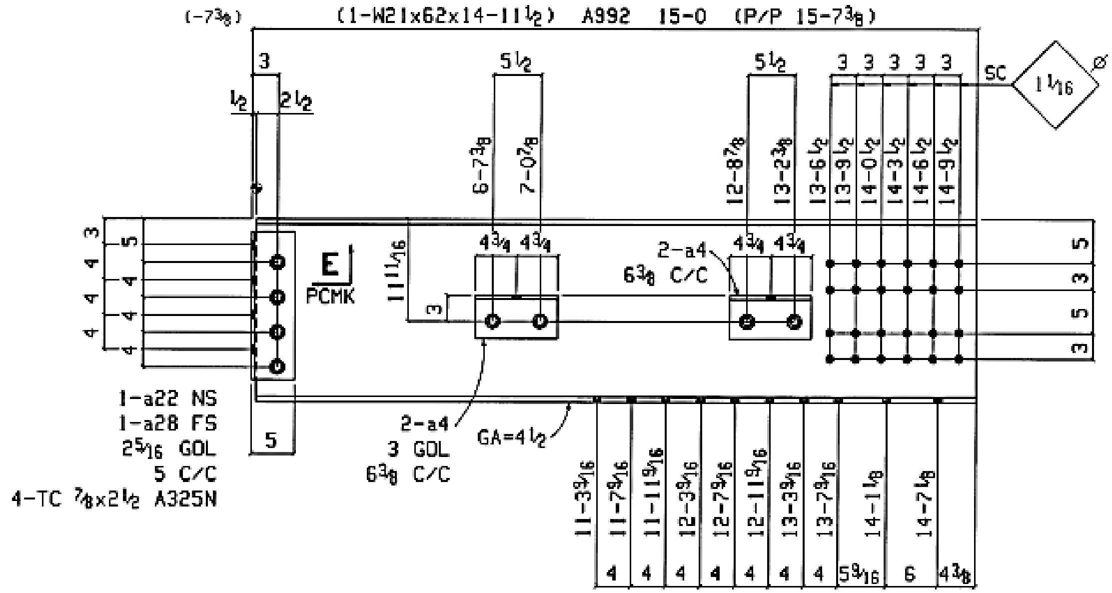
BILL OF MATERIAL
ONE COLUMN 2C1
SHOP BOLTS:
MARK FACE "A" "EAST"
NOTES: 1) EDGE DISTANCE = 1½ U.N.O.
2) SC = SLIP CRITICAL BOLT. DEBURR HOLES
SHOP PAINT: NO PAINT
ENGR: PATRICK MCMANUS
PUMA STEEL
The Professionals
CUSTOMER: UNIVERSITY OF WYOMING
PROJECT: TEST FRAME
LOCATION: UNIVERSITY OF WYOMING CAMPUS
CITY: LARAMIE, WYOMING
DESCRIPTION: COLUMN
MODELED: PMC DETAILED: PMC DATE: 1-29-10
CHECKED BY: CJP DATE: 02-05-10
REVISIONS
SHEET
JOB NO. UW FRAME


BILL OF MATERIAL
2 BEAMS 3B1
SHOP BOLTS (Unless Noted):
PUMA
STEEL
The Professionals


BILL OF MATERIAL
ONE BEAM 4B1
SHOP BOLTS (Unless Noted):
Section A-A
NOTES: 1) EDGE DISTANCE = 1½ U.N.O.
2) SC = SLIP CRITICAL BOLT. DEBURR HOLES.
SHOP PAINT: NO PAINT - SSPC-SP2
ENGR: PATRICK McMANUS
PUMA STEEL
The Professionals
CUSTOMER: UNIVERSITY OF WYOMING
PROJECT: TEST FRAME
LOCATION: UNIVERSITY OF WYOMING CAMPUS
CITY: LARAMIE, WYOMING
DESCRIPTION: BEAM
REVISIONS
SHEET 4


BILL OF MATERIAL
ONE BEAM SB1
SHOP BOLTS (Unless Noted):
10-TC 7/8x2 1/2 A490SC
ONE BEAM SB2
SHOP BOLTS (Unless Noted):
14-TC 7/8x2 1/2 A490SC
NOTES: 1) EDGE DISTANCE = 1 1/2 U.N.O.
2) SC = SLIP CRITICAL BOLT. BEWARE HOLES.
SHOP PAINT: NO PAINT
ENGR: PATRICK McMANUS
PUMA
STEEL
The Professionals
CUSTOMER: UNIVERSITY OF WYOMING
PROJECT: TEST FRAME
LOCATION: UNIVERSITY OF WYOMING CAMPUS
CITY: LARAMIE, WYOMING
DESCRIPTION: BEAM
CHECKED BY: GP
SHEET 5
REVISIONS

PLOT DATE: Mar 05 2010 10:59:13 glicerio.fiesta



ONE BEAM 6B

W	21x⅜
F	8¼x⅝
K	1 5/16 x⅞

SHOP BOLTS (Unless Noted):
4-TC ⅞x2¾ A490SC

BILL OF MATERIAL

Total weight : 1017

QUANTITY	DESCRIPTION	LENGTH FT	LENGTH IN	STEEL GRADE	SHIPPING MARK	ASSEMBLY MARK	REMARKS	SEQ. NO.	SEQ. QTY.
ONE	BEAM				6B			1	1
1	W21x62	14	11½	A992		6B			
4	L5x5x½	0	9½	A36		a4			
1	L5x3½x⅜	1	5	A36		a22			
1	L5x3½x⅜	1	5	A36		a28			
4	TC ⅞ Dia A490SC	0	2¾				1HD WASH		
4	TC ⅞ Dia A325N	0	2½				1HD WASH		

NOTES: 1) EDGE DISTANCE = 1½ U.N.O.
2) SC - SLIP CRITICAL BOLTS. DEBURR HOLES.

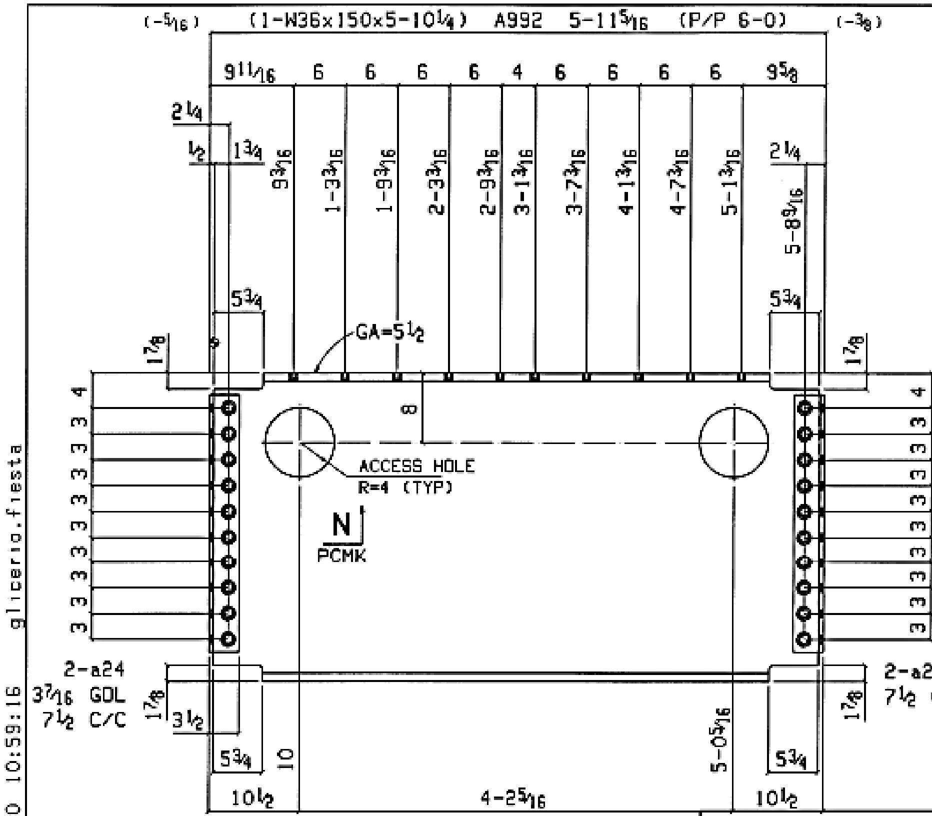
WELDS: E70XXLH HOLES: 15/16 UNO SHOP PAINT: NO PAINT ~ SSPC-SP2

NO.	FOR	DATE	REV	BY	ITEM	DATE
2	S&O	3-05		MR	C&A	3-05

PRINTING REVISIONS

ARCH: . . .
ENGR: PATRICK McMANUS
CUSTOMER: UNIVERSITY OF WYOMING ERECTION BY: UW
PROJECT: TEST FRAME
LOCATION: UNIVERSITY OF WYOMING CAMPUS
CITY: LARAMIE, COLORADO
DESCRIPTION: BEAM
MODELED: MR DETAILED: MR DATE: 1-28-10
CHECKED BY: GF DATE: 03-05-10
SHEET 6
JOB NO UW FRAME

PUMA STEEL The Professionals



2 BEAMS 7B

W | 35⅞x⅝
F | 12x$\frac{15}{16}$
K | 1⅞x1¼

SHOP BOLTS:
20-TC ⅞x3 A325N

BILL OF MATERIAL Total weight : 2073

		LENGTH		STEEL	SHIPPING	ASSEMBLY		SEQ.	SEQ.
QUANTITY	DESCRIPTION	FT	IN	GRADE	MARK	MARK	REMARKS	NO.	QTY.
2	BEAM				7B			1	2
2	W36x150	5	10¼	A992		7B			
8	L5x3½x½	2	6	A36		a24			
40	TC ¾ Dia A325N	0	3				1HD HASH		

NOTES: 1) EDGE DISTANCE = 1½ U.N.O.
2) SC = SLIP CRITICAL BOLTS. DEBURR HOLES.

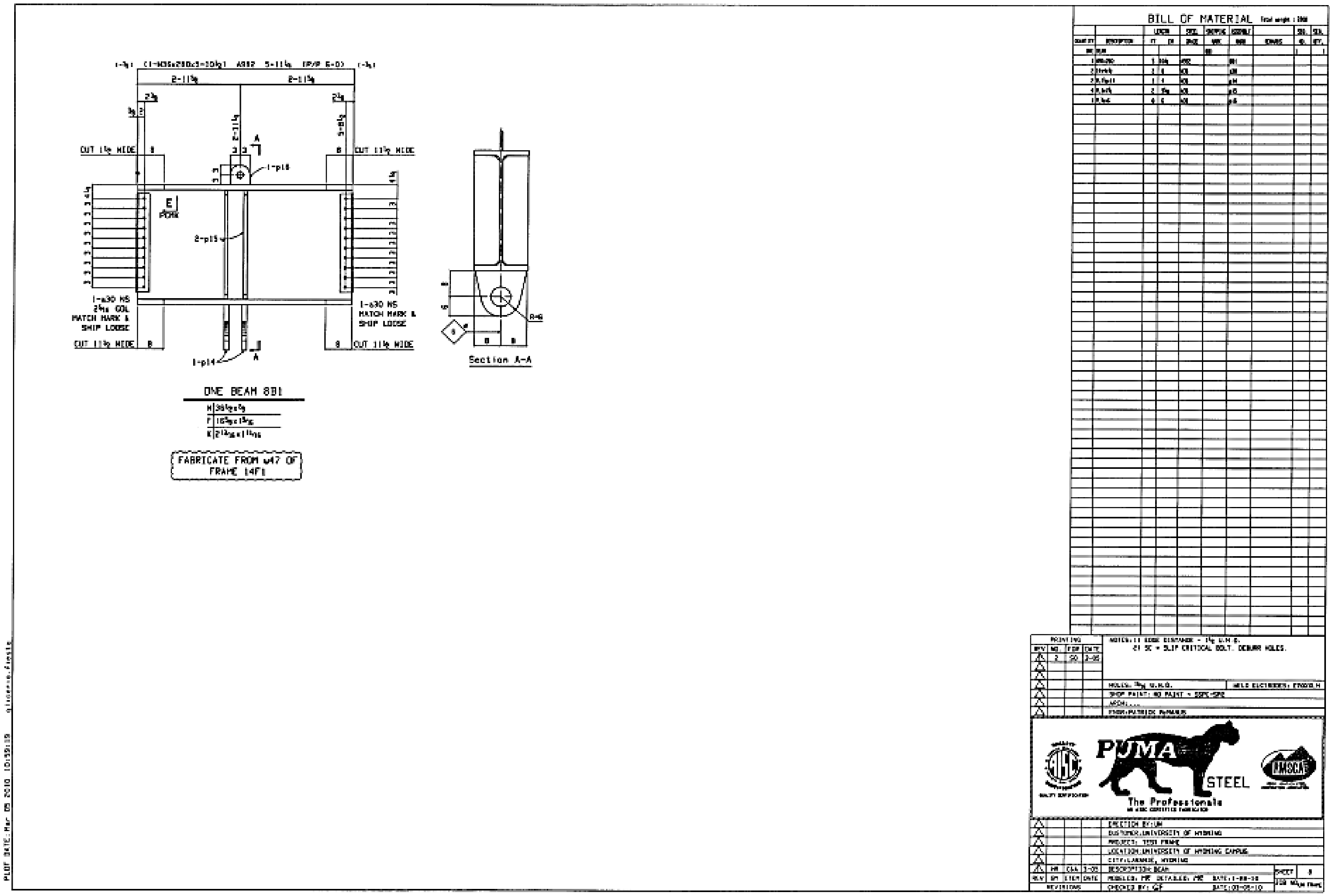
WELDS: E70XXLH HOLES: $\frac{15}{16}$ UNO SHOP PAINT: NO PAINT ~ SSPC-SP2

NO.	FOR	DATE	BY	ITEM	DATE
2	S&Q	3-05	MR	C&A	3-05

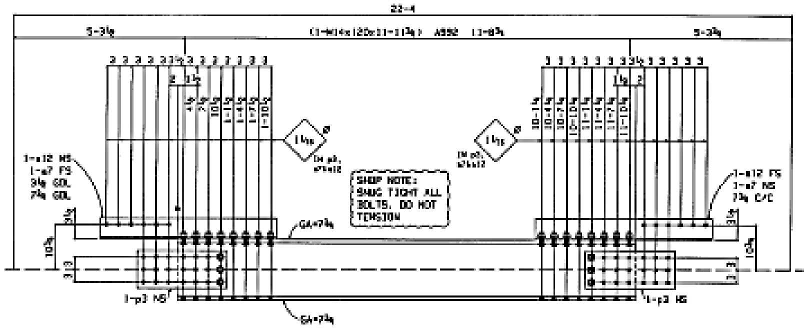
PRINTING REVISIONS

ARCH: ...
ENGR: PATRICK McMANUS
CUSTOMER: UNIVERSITY OF WYOMING ERECTION BY: UW
PROJECT: TEST FRAME
LOCATION: UNIVERSITY OF WYOMING CAMPUS
CITY: LARAMIE, COLORADO
DESCRIPTION: BEAM SHEET 7
MODELED: MR DETAILED: MR DATE: 1-28-10
CHECKED BY: GF DATE: 03-05-10
JOB NO UW FRAME

PUMA STEEL
The Professionals



2-11¾
CUT 1¼ WIDE
1-pl8
E
PCMK
2-pl5
1-ø30 HS
MATCH MARK &
SHIP LOOSE
1-pl4
A
Section A-A
ONE BEAM 8B1
FABRICATE FROM w47 OF
FRAME 14F1
BILL OF MATERIAL
PRINTING
SHOP PAINT: NO PAINT
ARCH:...
PUMA
STEEL
The Professionals
ERECTION BY:UW
CUSTOMER:UNIVERSITY OF WYOMING
PROJECT: TEST FRAME
LOCATION:UNIVERSITY OF WYOMING CAMPUS
CITY:LARAMIE, WYOMING
DESCRIPTION:BEAM
REVISIONS
SHEET



2 VERTICAL BRACES 9VB1

SHOP BOLTS:
38-TC ⅞ø x 2¾ A325N

BILL OF MATERIAL

QUANTITY	DESCRIPTION	LENGTH FT	LENGTH IN	GRADE	SHIPPING MARK	ASSEMBLY MARK	REMARKS	WT.	WT.
2	VERTICAL BRACE				[illegible]			[illegible]	[illegible]
2	[illegible]	11	[illegible]	A992		[illegible]			
4	[illegible]	3	[illegible]	A36		[illegible]			
4	[illegible]	3	[illegible]	A36		[illegible]			
4	[illegible]	1	[illegible]	A36		p3			
[illegible]	[illegible]	0	[illegible]				[illegible]		

NOTES: 1) EDGE DISTANCE = 1½ U.N.O.
2) SC = SLIP CRITICAL BOLT. DEBURR HOLES.

HOLES: [illegible] U.N.O.

WELD ELECTRODES: [illegible]

SHOP PAINT: NO PAINT - [illegible]

ARCH: ...

ENGR: PATRICK [illegible]

PUMA STEEL
The Professionals

ERECTION BY: [illegible]

CUSTOMER: UNIVERSITY OF WYOMING

PROJECT: TEST FRAME

LOCATION: UNIVERSITY OF WYOMING CAMPUS

CITY: LARAMIE, WYOMING

DESCRIPTION: VERTICAL BRACE

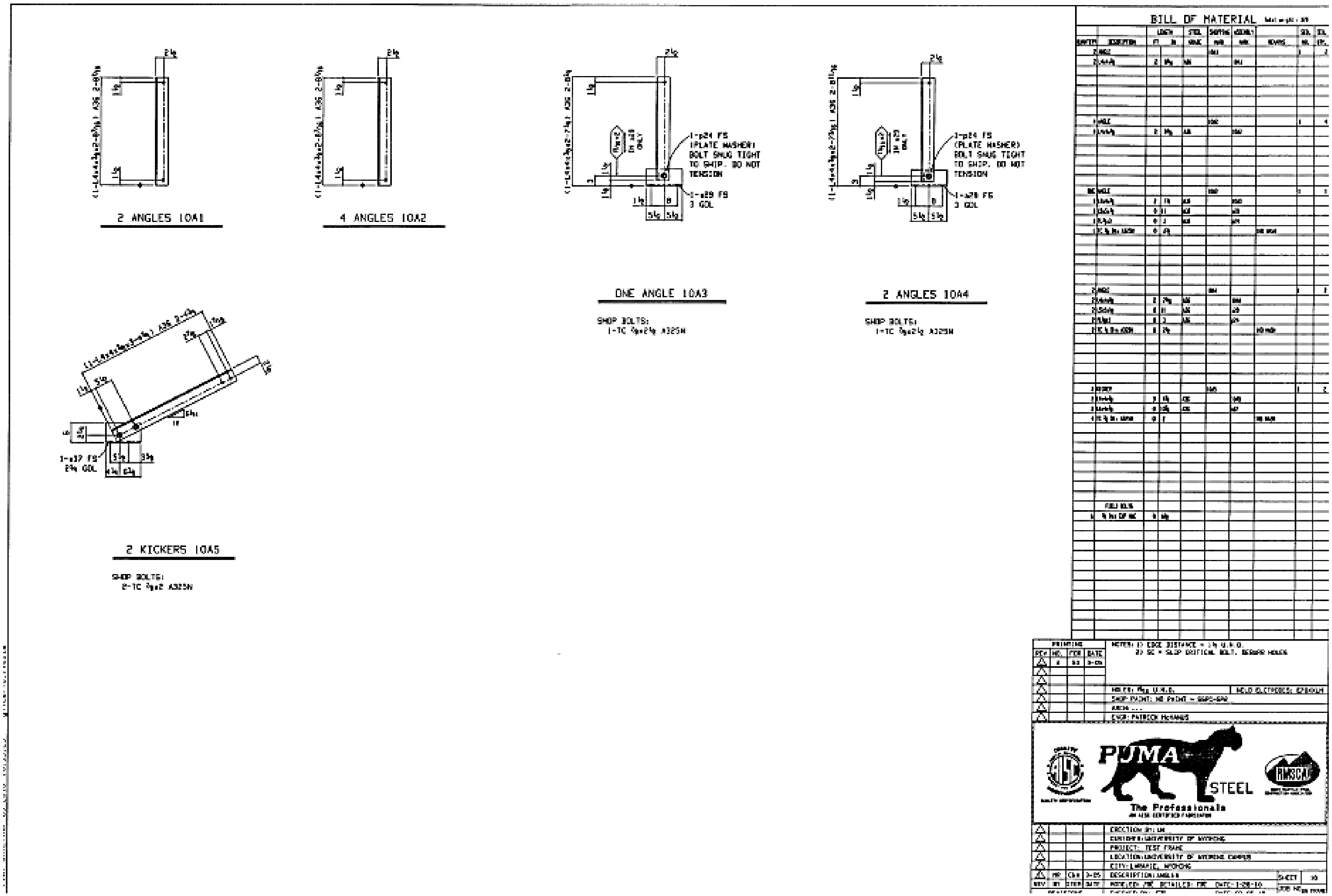
MODELED: MR DETAILED: MR DATE: 1-28-10

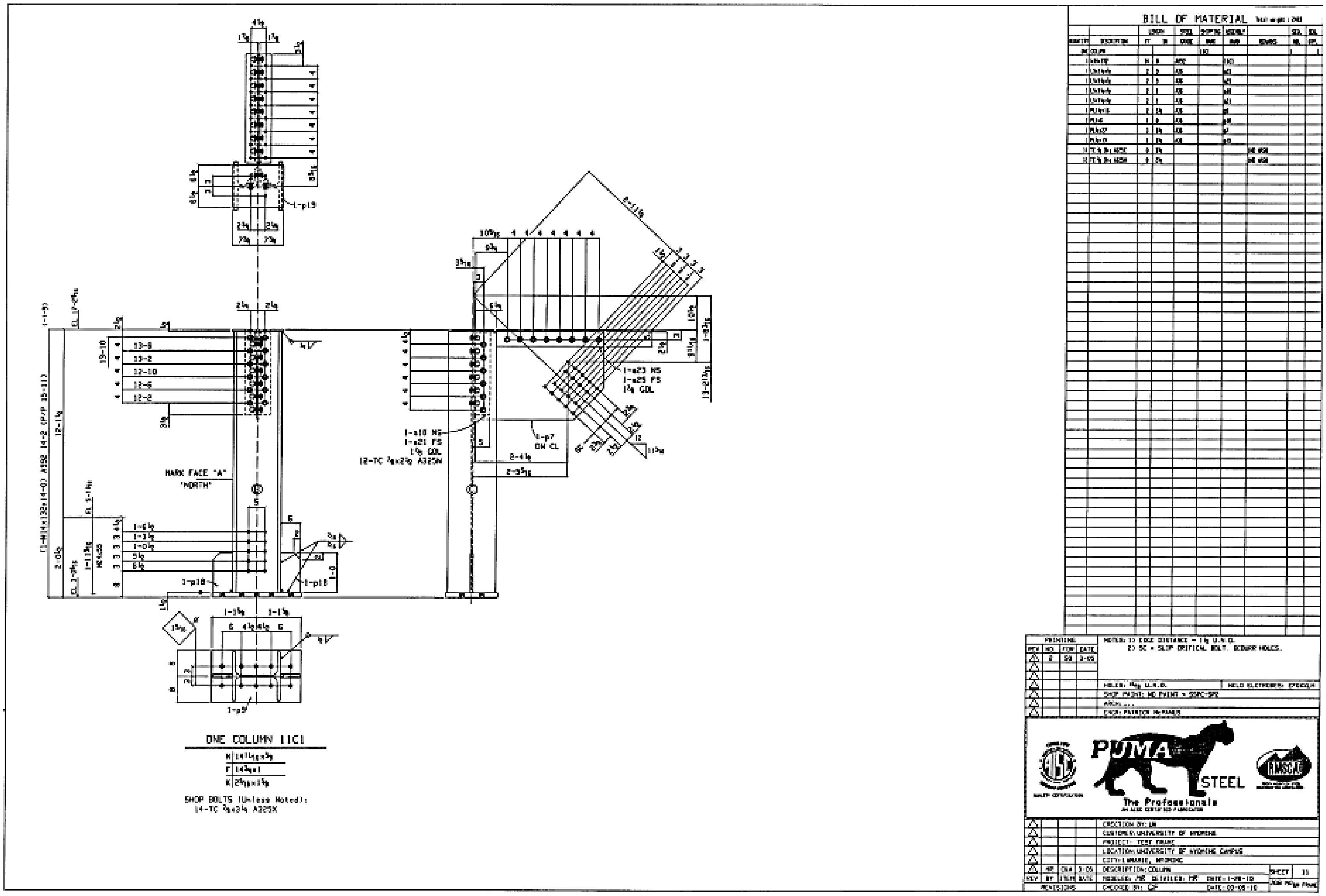
CHECKED BY: CF DATE: [illegible]

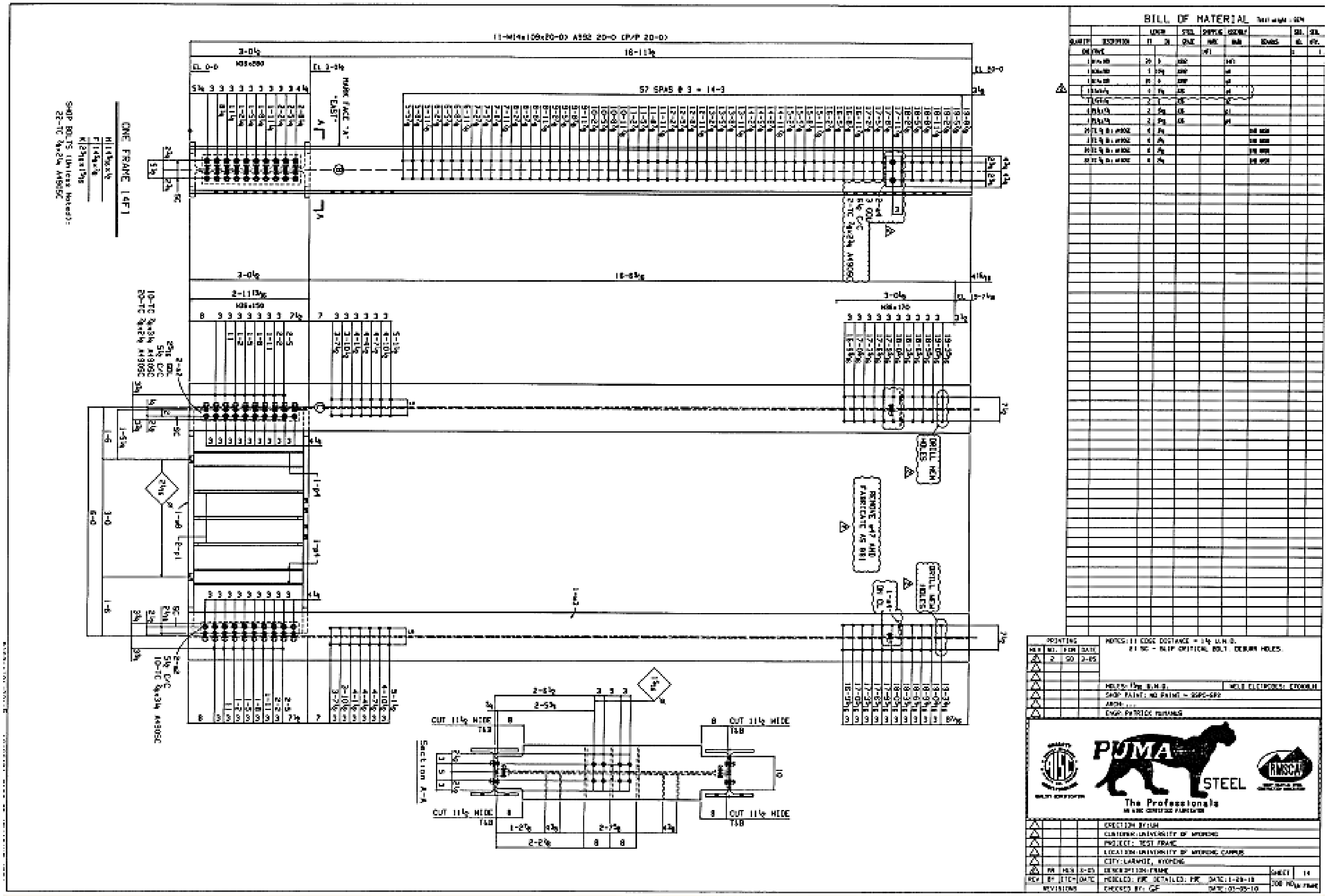
SHEET [illegible]

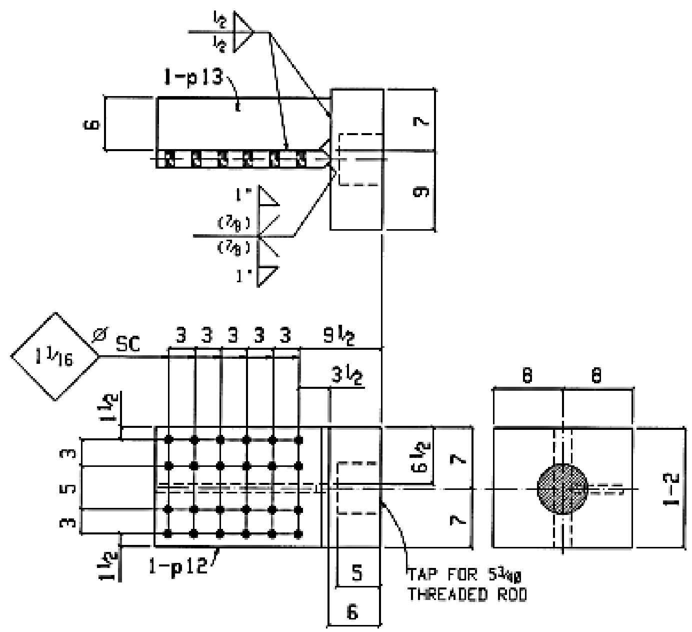
REV	PRINTING NO.	FOR	DATE
	2	[illegible]	[illegible]

PLOT DATE: Mar 05 2010 10:58:32


2 ANGLES 10A1
4 ANGLES 10A2
ONE ANGLE 10A3
SHOP BOLTS:
1-TC 7/8x2 1/2 A325N
2 ANGLES 10A4
SHOP BOLTS:
1-TC 7/8x2 1/2 A325N
2 KICKERS 10A5
SHOP BOLTS:
2-TC 7/8x2 A325N
1-p24 FS
(PLATE WASHER)
BOLT SNUG TIGHT
TO SHIP. DO NOT
TENSION
BILL OF MATERIAL
NOTES: 1) EDGE DISTANCE = 1 1/4 U.N.O.
2) SC = SLIP CRITICAL BOLT, BEARNG HOLES
SHOP PAINT: NO PAINT
ENGR: PATRICK McMANUS
PUMA
STEEL
The Professionals
ERECTION BY: LM
CUSTOMER: UNIVERSITY OF WYOMING
PROJECT: TEST FRAME
LOCATION: UNIVERSITY OF WYOMING CAMPUS
CITY: LARAMIE, WYOMING
DESCRIPTION: ANGLES
SHEET
10


BILL OF MATERIAL
ONE COLUMN [1C]
SHOP BOLTS (Unless Noted):
MARK FACE "A" "NORTH"
NOTES: 1) EDGE DISTANCE = 1½ U.N.O.
2) SC = SLIP CRITICAL BOLT. DEBURR HOLES.
SHOP PAINT: NO PAINT
ERECTION BY: LW
CUSTOMER: UNIVERSITY OF WYOMING
PROJECT: TEST FRAME
LOCATION: UNIVERSITY OF WYOMING CAMPUS
CITY: LARAMIE, WYOMING
DESCRIPTION: COLUMN
PUMA STEEL
The Professionals





ONE PLATE 15P5

(1-PL6x14x1-4) A36

BILL OF MATERIAL Total weight : 574

		LENGTH		STEEL	SHIPPING	ASSEMBLY		SEQ.	SEQ.
QUANTITY	DESCRIPTION	FT	IN	GRADE	MARK	MARK	REMARKS	NO.	QTY.
ONE	PLATE				15P5			1	1
1	PL6x14	1	4	A36		15P5			
1	PL2x14	1	$	A36		p12			
1	PL1x6	1	$	A36		p13			

NOTES: 1) EDGE DISTANCE = 1 1/2 U.N.O.
2) SC = SLIP CRITICAL BOLTS. DEBURR HOLES.

WELDS: E70XXLH HOLES: 15/16 UNO SHOP PAINT: NO PAINT ~ SSPC-SP2

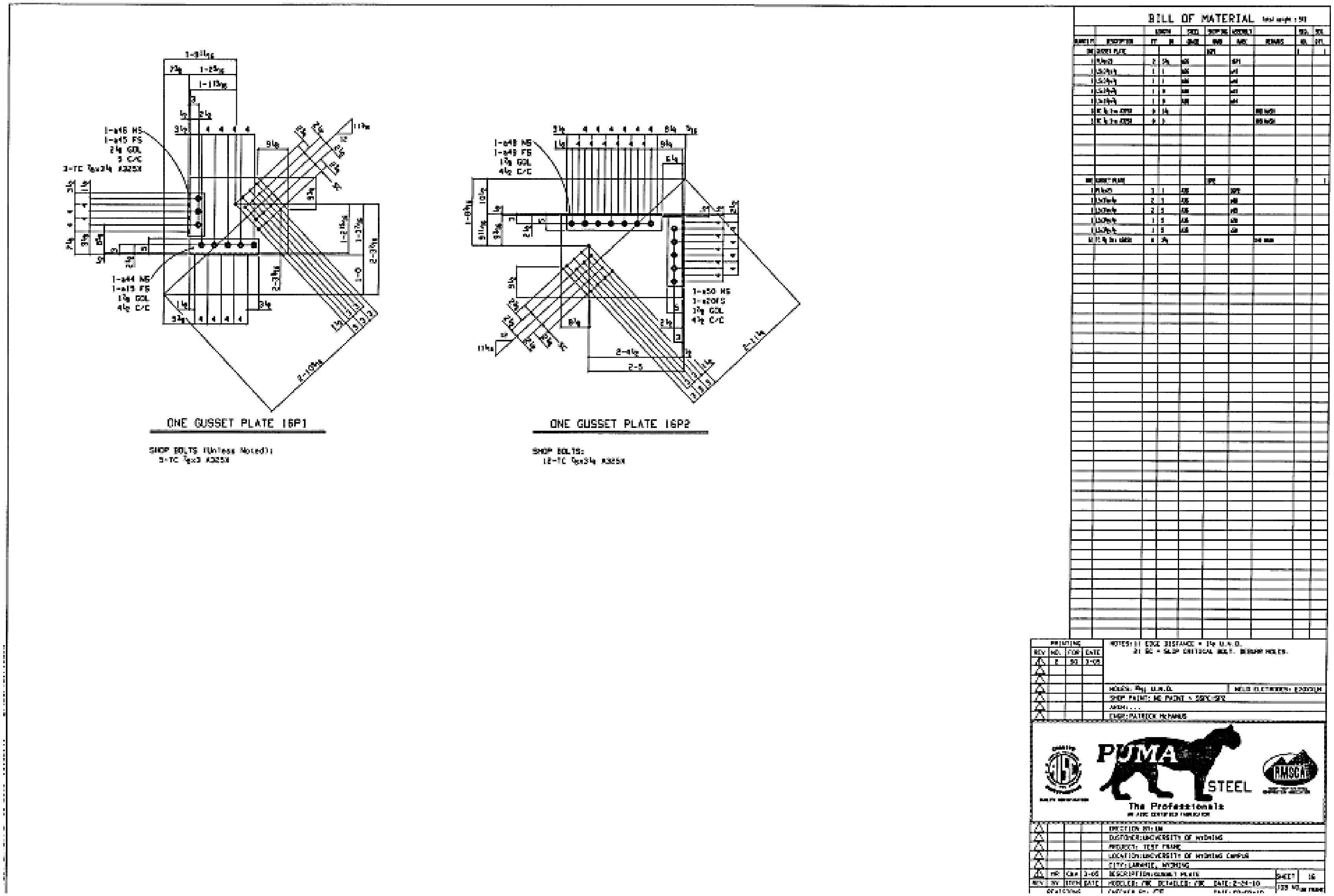
NO.	FOR	DATE	REV	BY	ITEM	DATE
2	S&O	3-05		noy	C&A	3-05

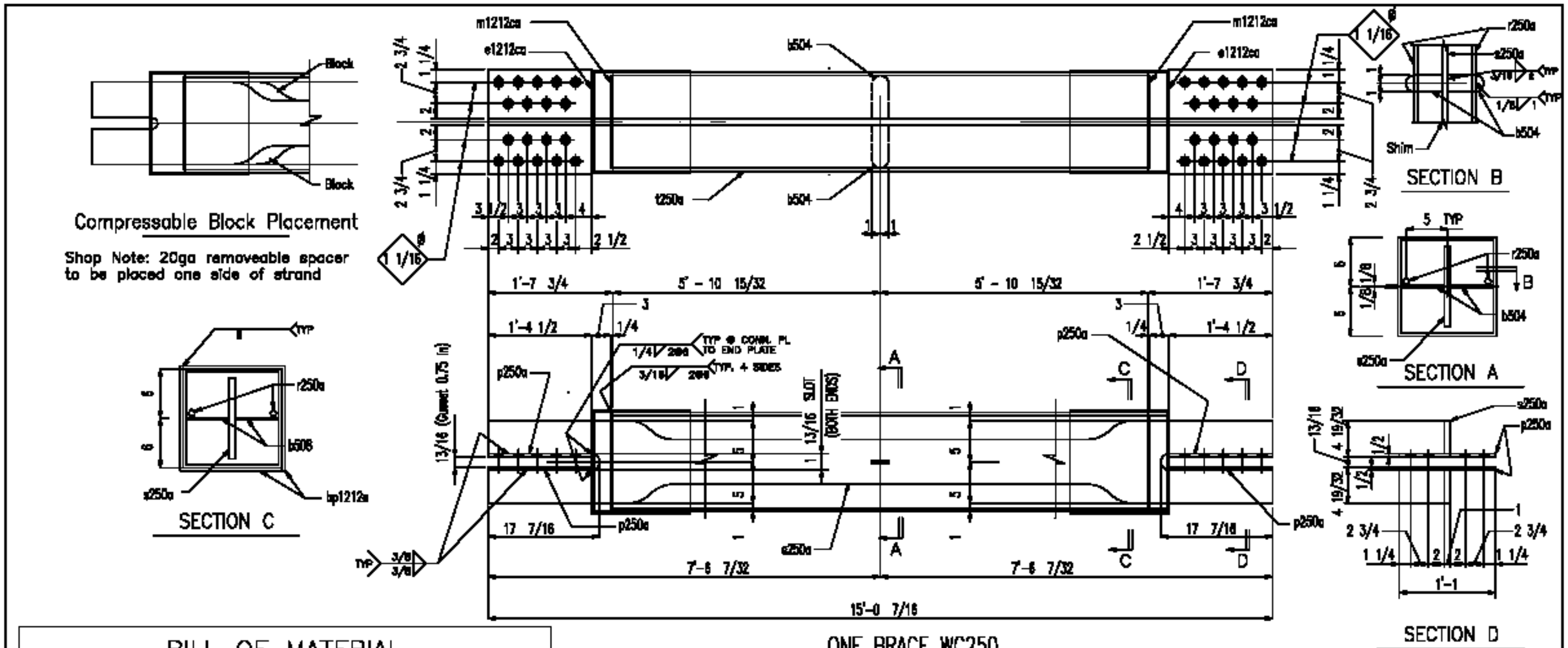
PRINTING REVISIONS

ARCH: ...
ENGR: PATRICK McMANUS
CUSTOMER: UNIVERSITY OF WYOMING ERECTION BY: UW
PROJECT: TEST FRAME
LOCATION: UNIVERSITY OF WYOMING CAMPUS
CITY: LARAMIE, COLORADO
DESCRIPTION: BEAM
MODELED: MR DETAILED: MR DATE: 1-28-10
CHECKED BY: GF DATE: 03-05-10
SHEET 15
JOB NO UW FRAME

PUMA STEEL The Professionals

PLOT DATE: Mar 05 2010 10:59:39 glicerio.fiesta


BILL OF MATERIAL
ONE GUSSET PLATE 16P1
SHOP BOLTS (Unless Noted):
ONE GUSSET PLATE 16P2
SHOP BOLTS:
NOTES:
EDGE DISTANCE
SHOP PAINT: NO PAINT
ENGR: PATRICK McMANUS
PUMA
STEEL
The Professionals
RMSCA
CUSTOMER: UNIVERSITY OF WYOMING
PROJECT: TEST FRAME
LOCATION: UNIVERSITY OF WYOMING CAMPUS
CITY: LARAMIE, WYOMING
DESCRIPTION: GUSSET PLATE
SHEET
16



ONE BRACE WC250

BILL OF MATERIAL

NO. PCS.	DESCRIPTION	LENGTH FT.	LENGTH IN.	MARK	REMARKS
1	Brace	15	0 7/16	WC250	
1	HSS12x12x1/4	11	8 15/16	t250a	A500-Gr B
2	PL 1/4 x 16	2	0 1/4	bp1212a	A36
1	FB 1 x 10	15	0 7/16	s250a	
2	PL 1/4x2	0	5 1/8	b504	A36
2	PL 1/4 x 11 1/2	0	11 1/2	m1212ca	A36
2	#4 rebar	14	3	r250a	#5 rebar optional
2	PL 1/2 x 12 1/8	1	0 1/8	e1212ca	A36
4	PL 1/2 x 13	1	4 1/2	a250a	A36

NO.	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	5/21/2010 SJ	Approval

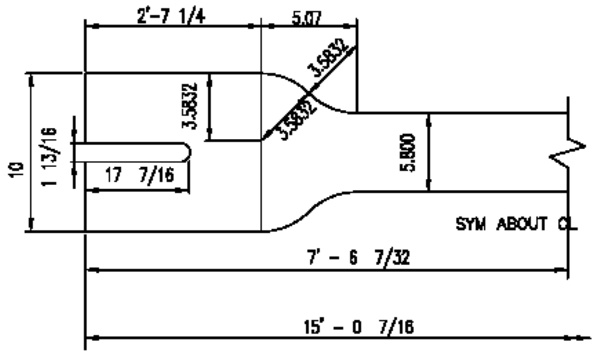
SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

MATERIAL A36 & AS NOTED SHOP PAINT NONE
APPROX BRACE WT ______ EA SURFACE PREPARATION NONE
APPROX CONCR. ______ YRDS EA

SUBJECT WC250
PROJECT PUMA TEST BRACE
LOCATION Laramie, Wyoming
CUSTOMER University of Wyoming
ZONE ______ SS BRB NO. ______
ERECTION DWG ______ ENG. DWG ______
DRAWN BY SJ DATE 5/07/2010 CHECKED BY BP

STAR SEISMIC™
6300 N. SAGEWOOD DR
SUITE H #511
PARK CITY, UT 84098
www.starseismic.net
PHONE: 1-435-940-9222
FAX: 1-435-655-0073

JOB NO 100510 DWG NO WC250



Lysc=117.9375 in

Total Brace Asc= 5.80 in^2

BILL OF MATERIAL

NO. PCS.	DESCRIPTION	LENGTH FT.	LENGTH IN.	MARK	REMARKS
1	STRAND PLATE			s250	
1	FB 1 x 10	15	0 7/16	s250	A36-LY Heat#B10-7775

1 STRAND PLATE s250

(SHOP NOTE: DEBURR ALL CUT EDGES)

NO.	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	5/21/2010 SMJ	Approval

SUBJECT s250

PROJECT Puma Test Brace

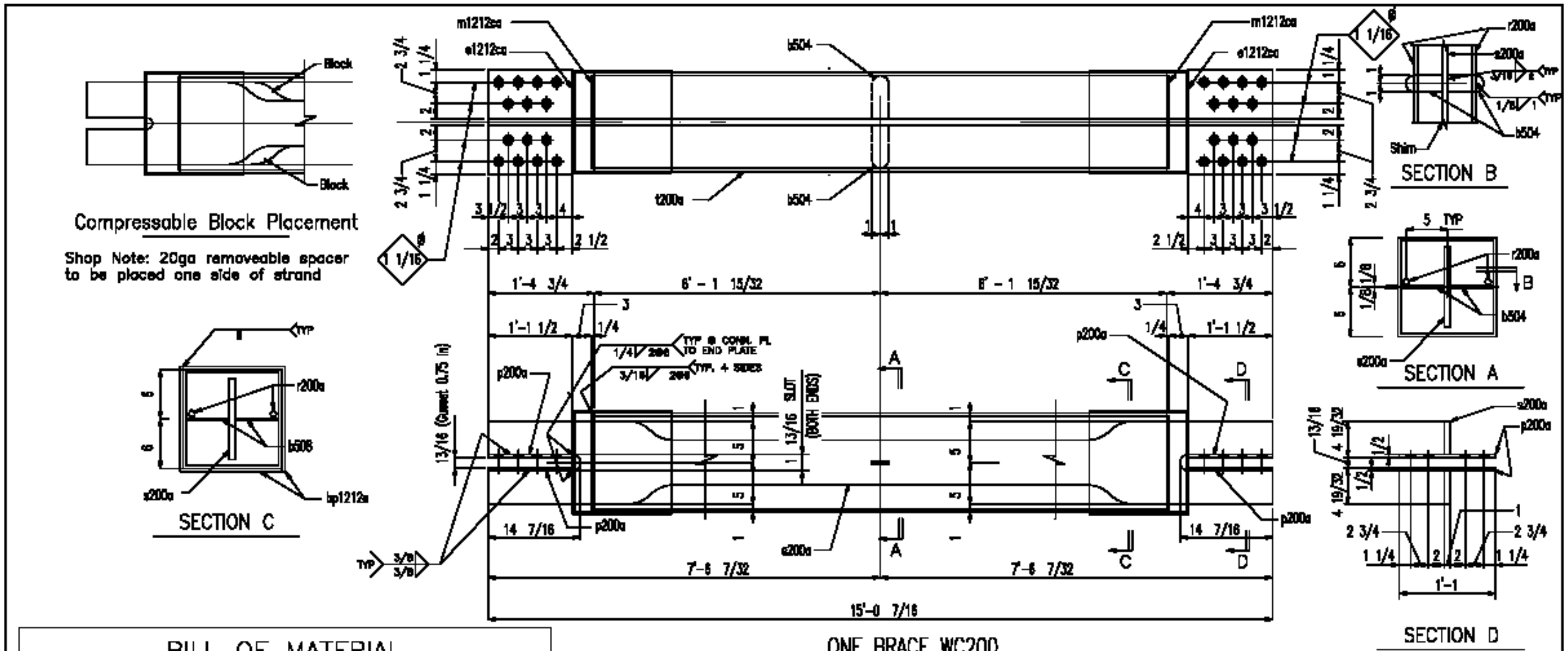
LOCATION Laramie, WY

CUSTOMER U. of Wyoming

DRAWN BY SMJ DATE 5/10/2010 CHECKED BY BP

STAR SEISMIC™
6300 N. SAGEWOOD DR
SUITE H #511
PARK CITY, UT 84098
www.starseismic.net
PHONE: 1-435-940-9222
FAX: 1-435-655-0073

JOB NO 100510 DWG NO s250



ONE BRACE WC200

BILL OF MATERIAL

NO. PCS.	DESCRIPTION	LENGTH FT.	LENGTH IN.	MARK	REMARKS
1	Brace	15	0 7/16	WC200	
1	HSS12x12x1/4	12	2 15/16	t200a	A500-Gr B
2	PL 1/4 x 16	2	0 1/4	bp1212a	A36
1	FB 1 x 10	15	0 7/16	a200a	
2	PL 1/4x2	0	5 1/8	b504	A36
2	PL 1/4 x 11 1/2	0	11 1/2	m1212ca	A36
2	#4 rebar	14	9	r200a	#5 rebar optional
2	PL 1/2 x 12 1/8	1	0 1/8	e1212ca	A36
4	PL 1/2 x 13	1	1 1/2	p200a	A36

NO.	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	5/21/2010 SJ	Approval

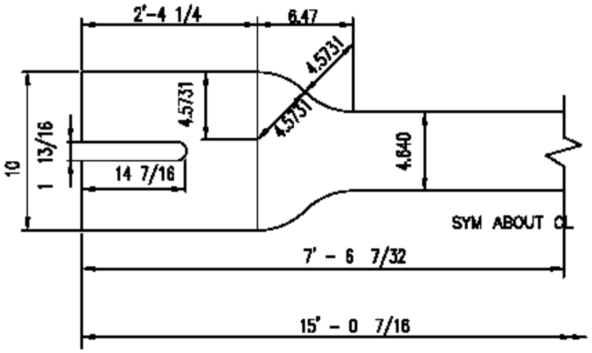
SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

MATERIAL A36 & AS NOTED SHOP PAINT NONE
APPROX BRACE WT ______ EA SURFACE PREPARATION NONE
APPROX CONCR. ______ YRDS EA

SUBJECT WC200
PROJECT PUMA TEST BRACE
LOCATION Laramie, Wyoming
CUSTOMER University of Wyoming
ZONE ______ SS BRB NO. ______
ERECTION DWG ______ ENG. DWG ______
DRAWN BY GP DATE 5/07/2010 CHECKED BY SJ

STAR SEISMIC™
6300 N. SAGEWOOD DR
SUITE H #511
PARK CITY, UT 84098
www.starseismic.net
PHONE: 1-435-940-9222
FAX: 1-435-655-0073

JOB NO 100510 DWG NO WC200



Lysc=123.9375 in

Total Brace Asc= 4.64 in^2

1 STRAND PLATE s200

SHOP NOTE: DEBURR ALL CUT EDGES

BILL OF MATERIAL

NO. PCS.	DESCRIPTION	LENGTH FT.	LENGTH IN.	MARK	REMARKS
1	STRAND PLATE			s200	
1	FB 1 x 10	15	0 7/16	s200	A36-LY Heat#B10-7775

NO.	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	5/21/2010 SMJ	Approval

SUBJECT s200
PROJECT Puma Test Brace
LOCATION Laramie, WY
CUSTOMER U. of Wyoming

DRAWN BY SMJ DATE 5/10/2010 CHECKED BY BP

STAR SEISMIC™
6300 N. SAGEWOOD DR
SUITE H #511
PARK CITY, UT 84098
www.starseismic.net
PHONE: 1-435-940-9222
FAX: 1-435-655-0073

JOB NO 100510 DWG NO s200

9 Appendix B – Experimental Testing Results

9.1 Test 1 Results – WC250 Figures

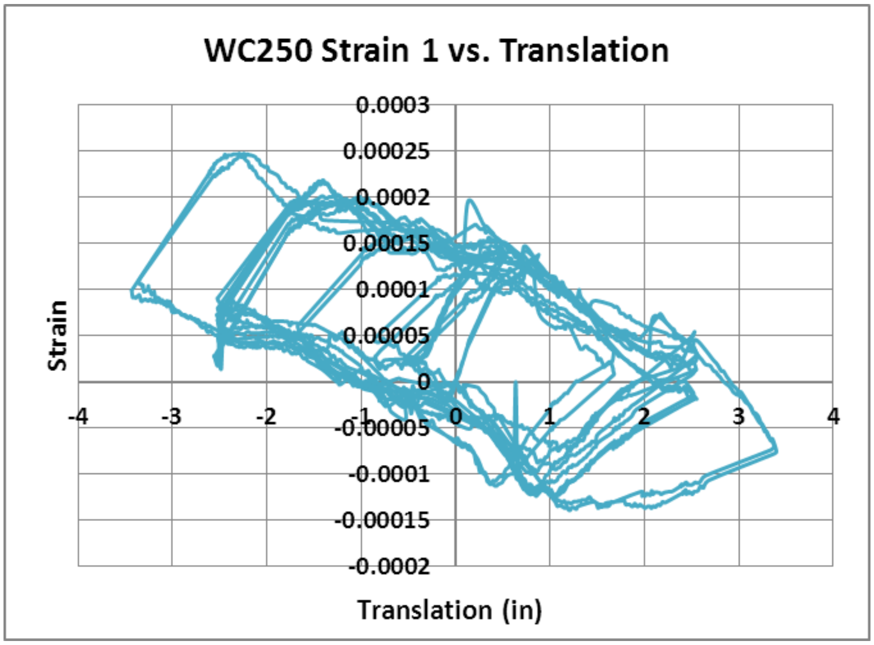


Figure 41 - WC250 Strain 1 vs. Translation

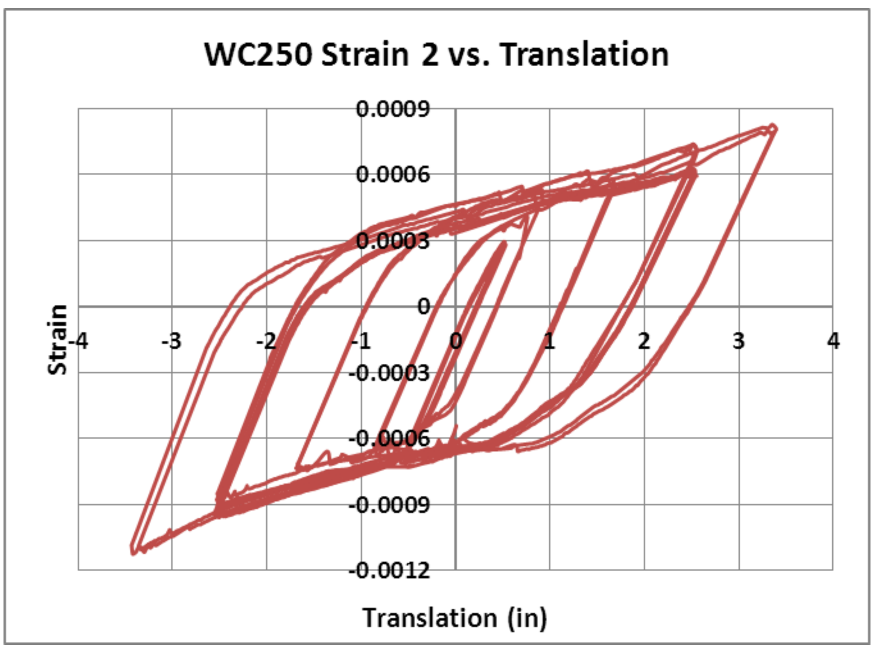


Figure 42 - WC250 Strain 2 vs. Translation

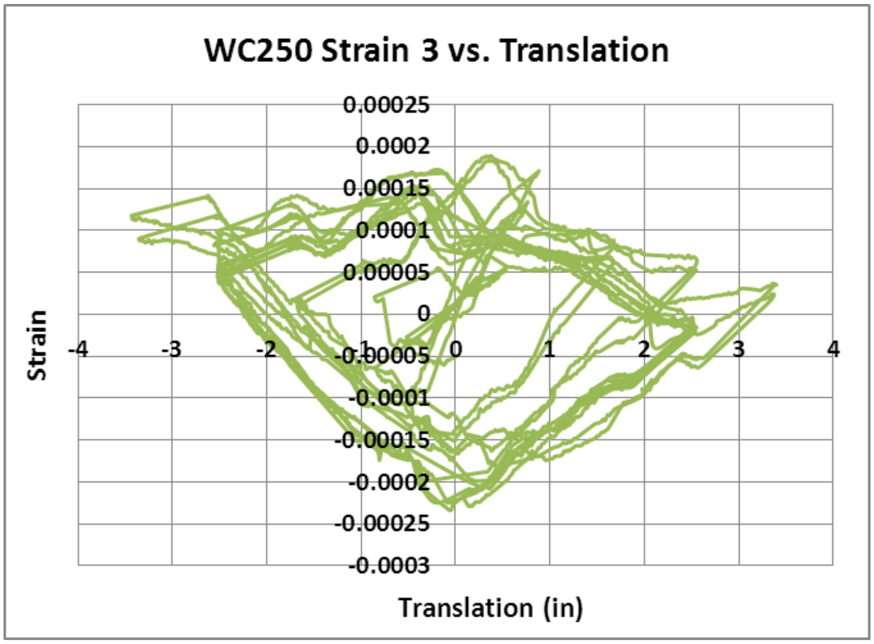


Figure 43 - WC250 Strain 3 vs. Translation

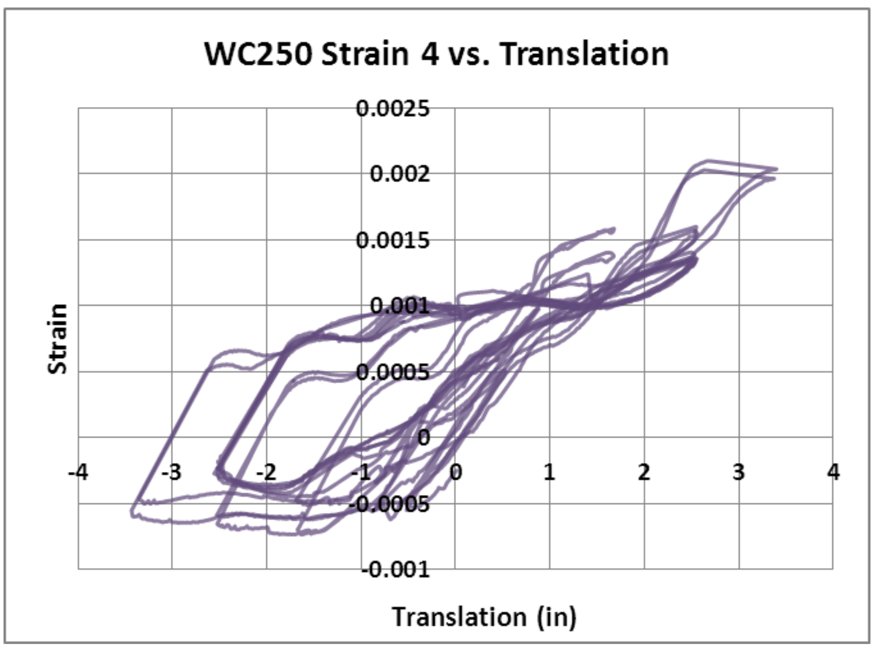


Figure 44 - WC250 Strain 4 vs. Translation

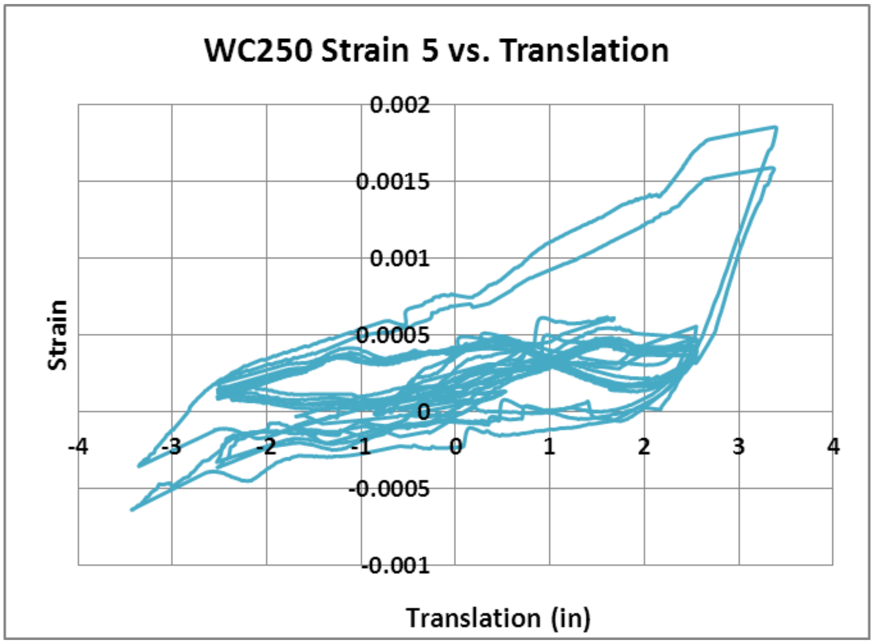


Figure 45 - WC250 Strain 5 vs. Translation

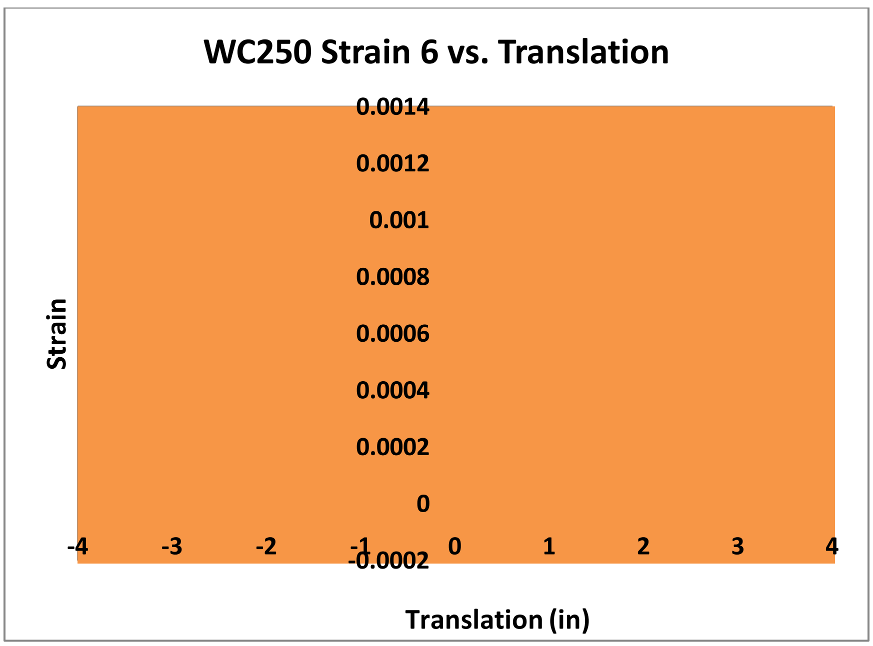


Figure 46 - WC250 Strain 6 vs. Translation

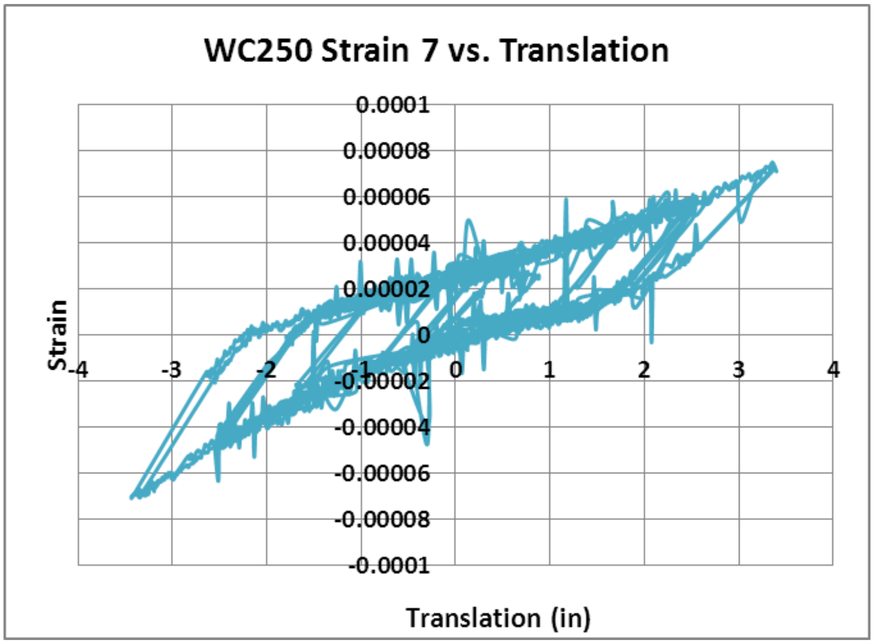


Figure 47 - WC250 Strain 7 vs. Translation

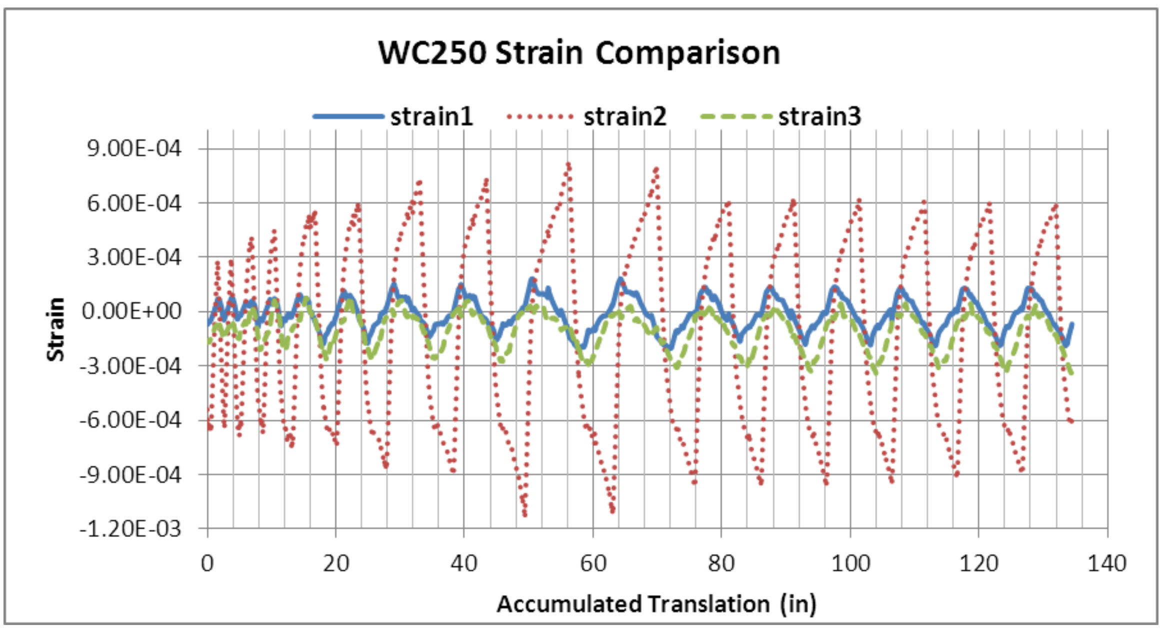


Figure 48 - WC250 Strain Comparison

9.2 Test 2 Results – WC200 Figures

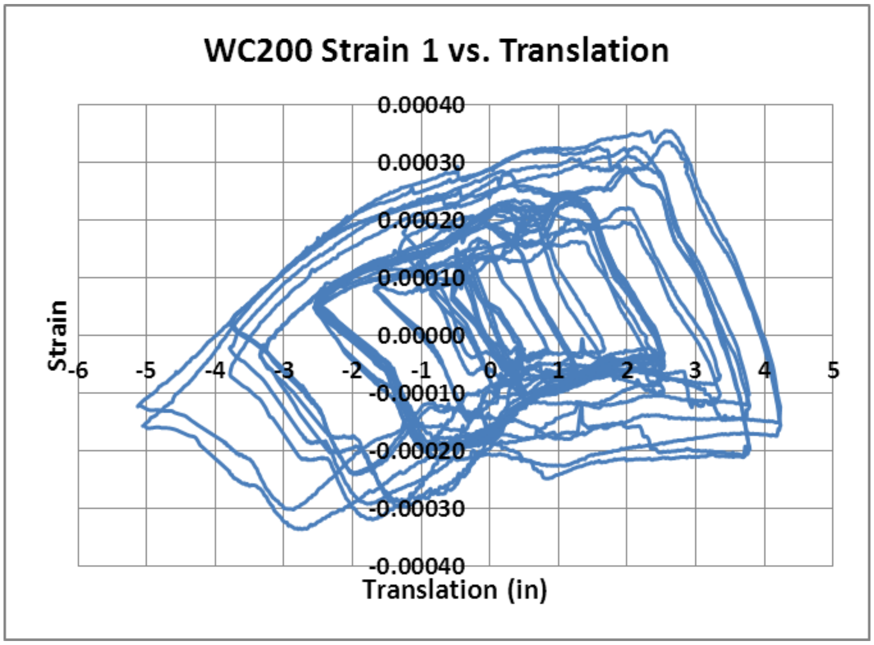


Figure 49 - WC200 Strain 1 vs. Translation

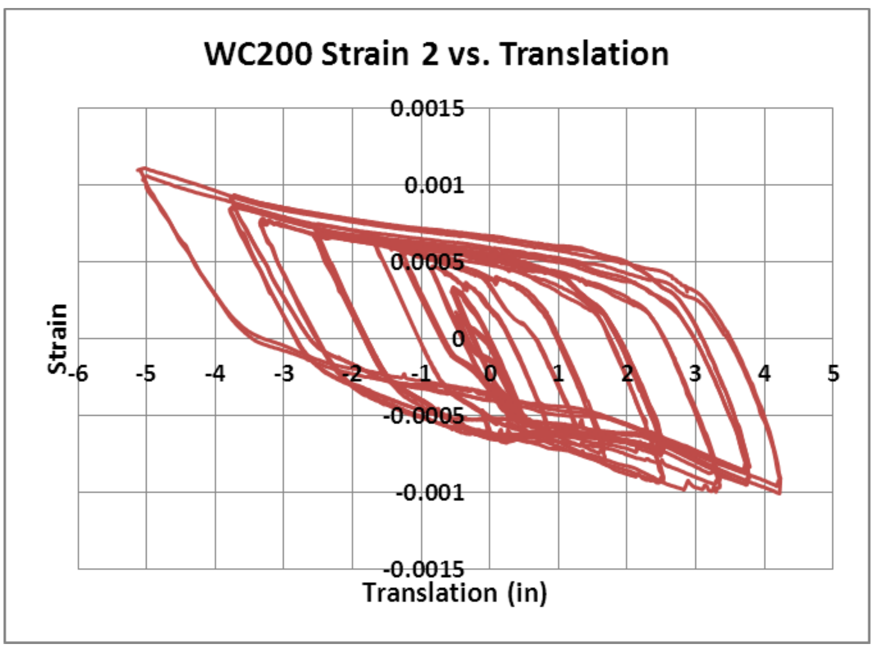


Figure 50 - WC200 Strain 2 vs. Translation

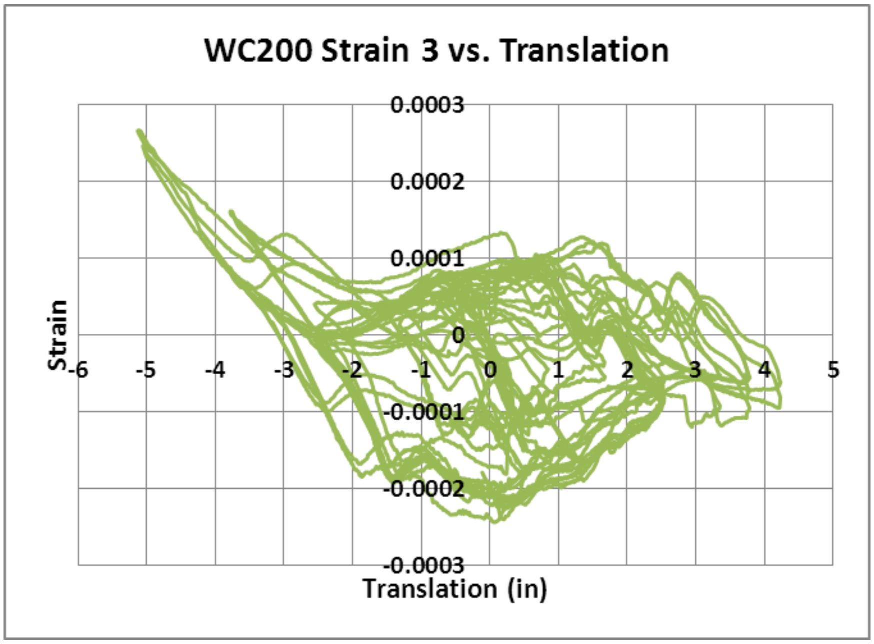


Figure 51 - WC200 Strain 3 vs. Translation

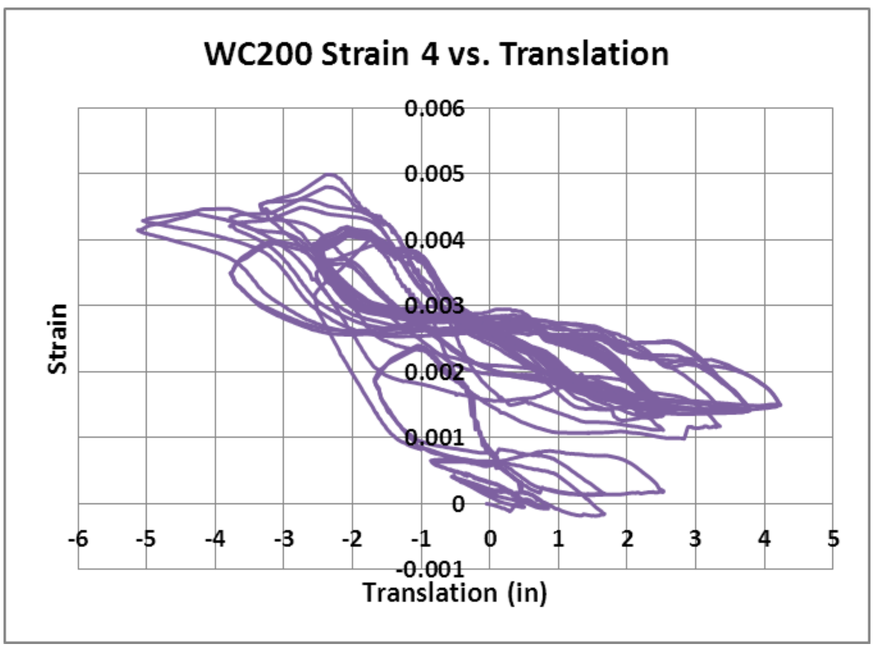


Figure 52 - WC200 Strain 4 vs. Translation

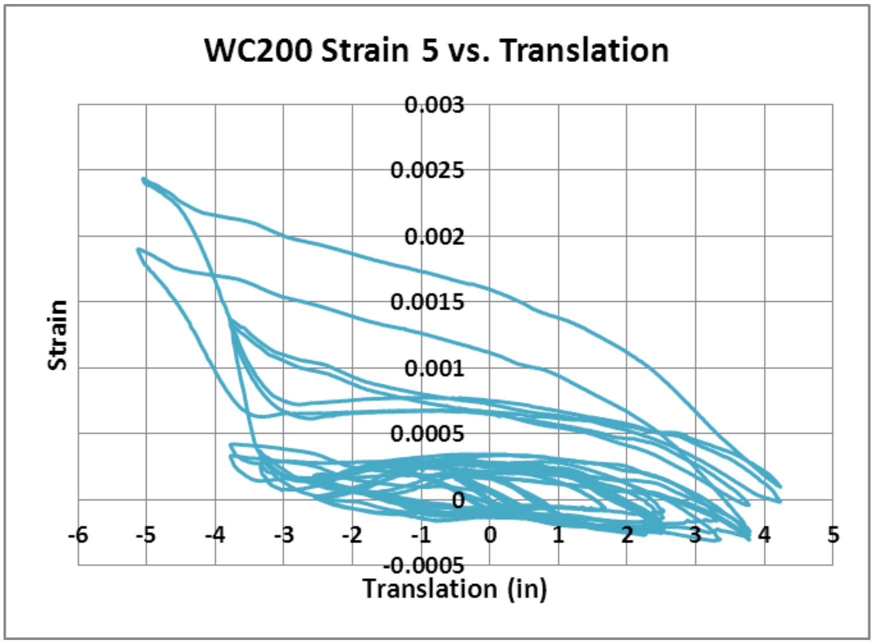


Figure 53 - WC200 Strain 5 vs. Translation

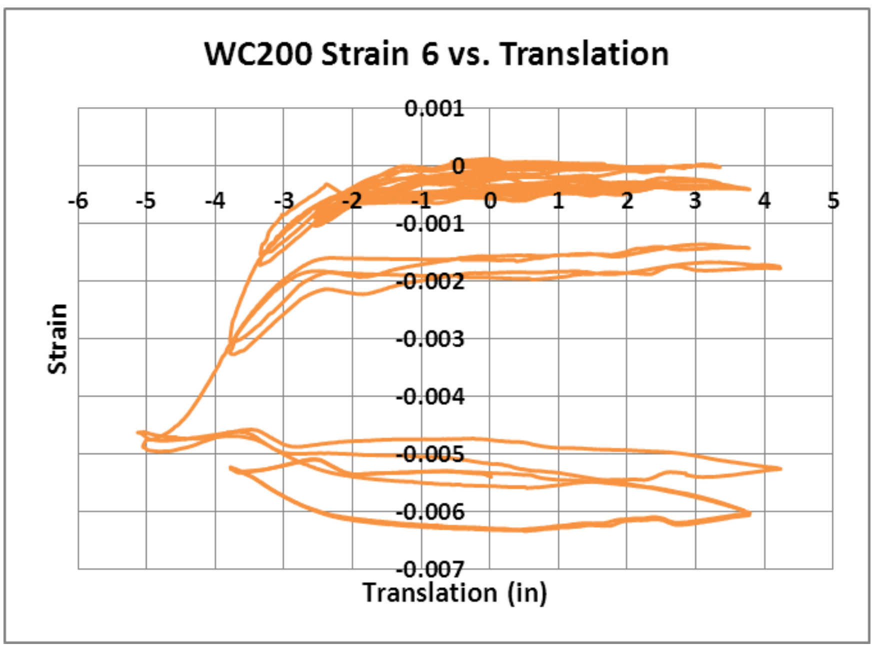
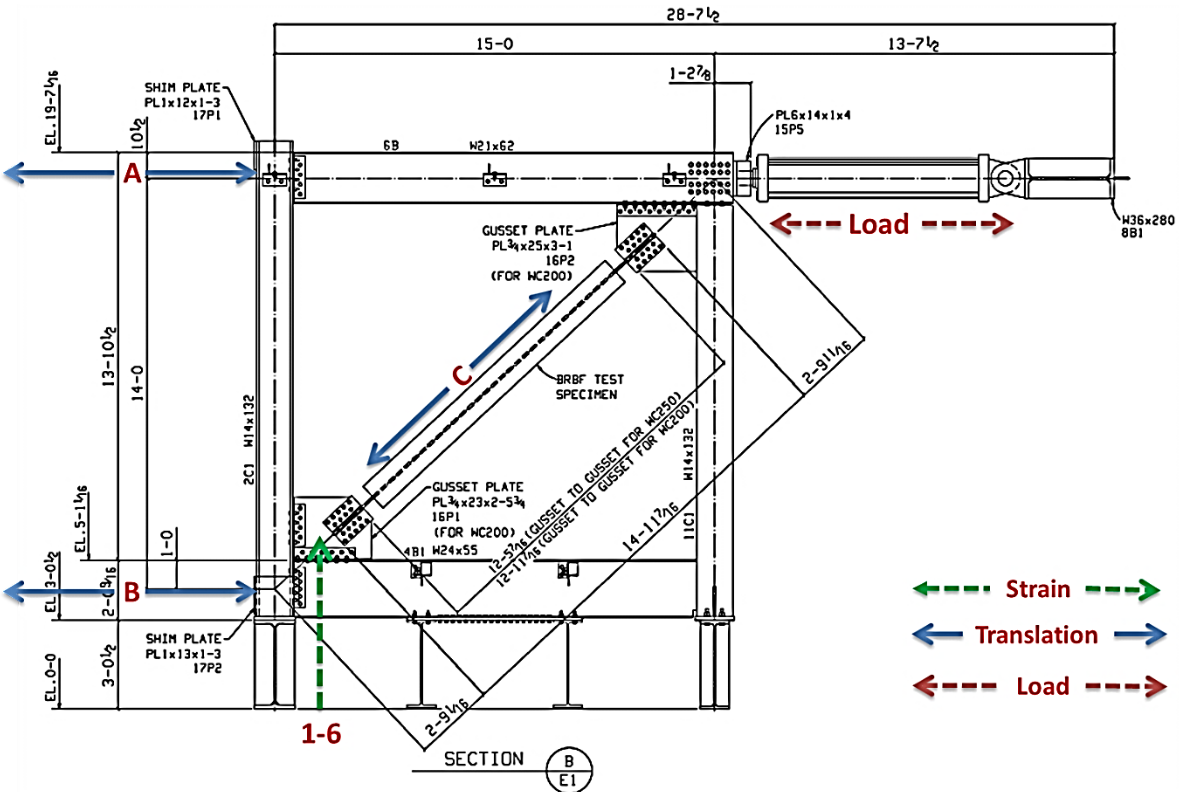


Figure 54 - WC200 Strain 6 vs. Translation

10 Appendix C – Experimental Testing Pictures



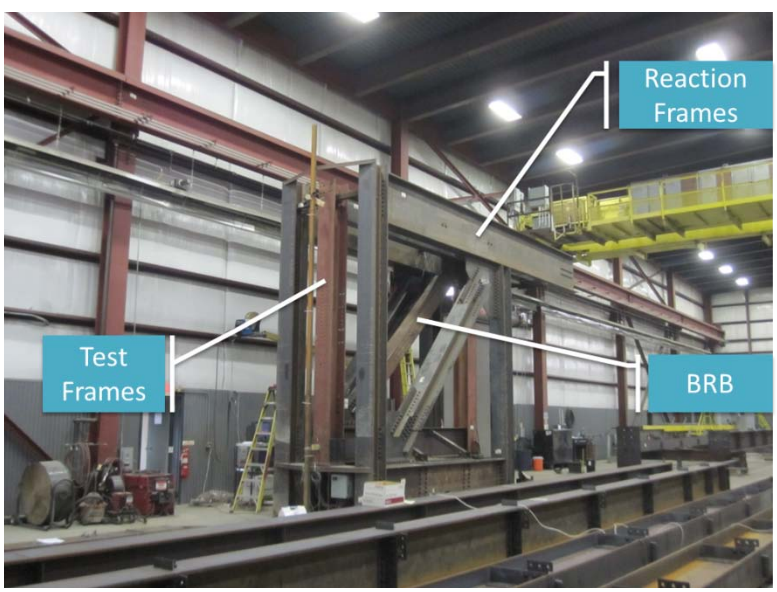


Figure 55 - East View of Test Frame

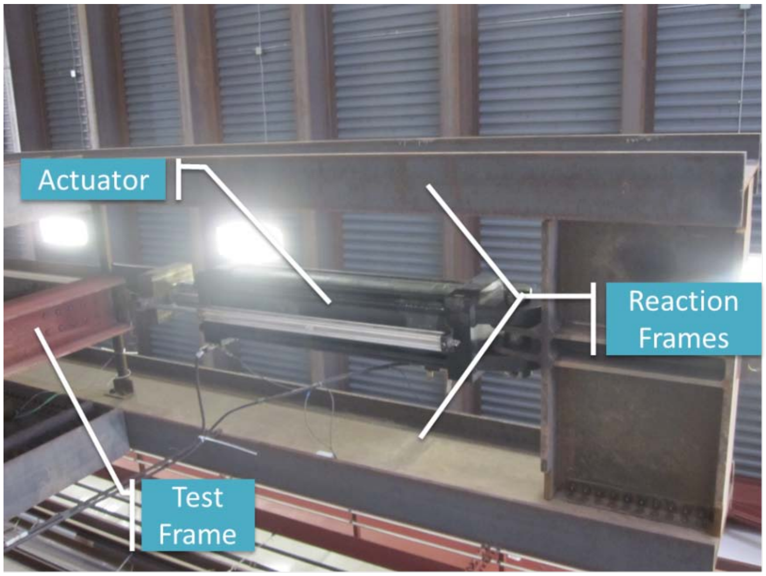


Figure 56 - View From Below Actuator

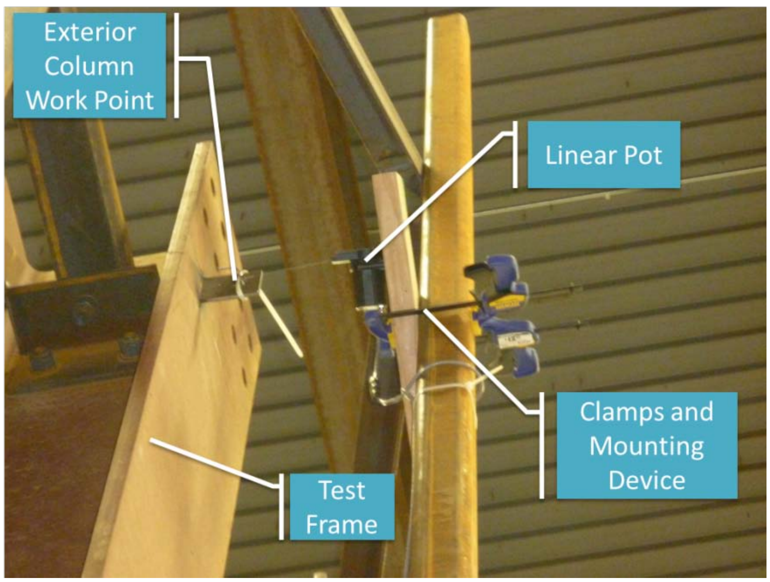


Figure 57 - Top of Frame String Pot Mounting

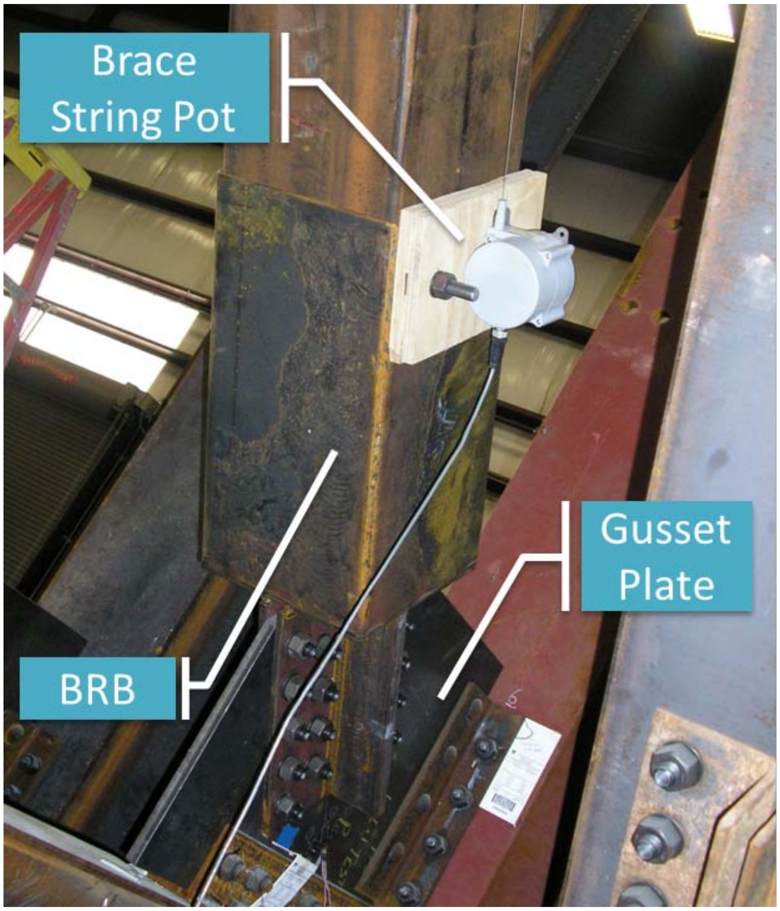
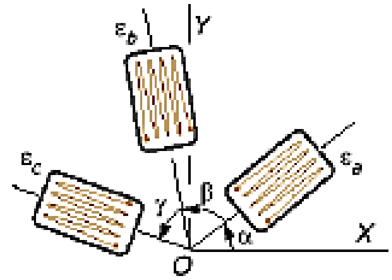


Figure 58 - Brace String Pot Mounting

11 Appendix D – Data Sheets

Strain gauge rosette



$$\varepsilon_a := \varepsilon_x \cdot \cos(\alpha)^2 + \varepsilon_y \cdot \sin(\alpha)^2 + \varepsilon_{xy} \cdot \sin(\alpha) \cdot \cos(\alpha)$$

$$\varepsilon_b := \varepsilon_x \cdot \cos(\alpha + \beta)^2 + \varepsilon_y \cdot \sin(\alpha + \beta)^2 + \varepsilon_{xy} \cdot \sin(\alpha + \beta) \cdot \cos(\alpha + \beta)$$

$$\varepsilon_c := \varepsilon_x \cdot \cos(\alpha + \beta + \gamma)^2 + \varepsilon_y \cdot \sin(\alpha + \beta + \gamma)^2 + \varepsilon_{xy} \cdot \sin(\alpha + \beta + \gamma) \cdot \cos(\alpha + \beta + \gamma)$$

Given: Stain data for SG 1-3 at peak load of 451 kip

$$\alpha := 0 \quad \beta := 43 \cdot \left(\frac{\pi}{180}\right) \quad \gamma := 47 \cdot \left(\frac{\pi}{180}\right) \quad \varepsilon_a := 0.0000261 \quad \varepsilon_b := -0.001117 \quad \varepsilon_c := -0.0000293$$

Given

$$\varepsilon_a = \varepsilon_x \cdot \cos(\alpha)^2 + \varepsilon_y \cdot \sin(\alpha)^2 + \gamma_{xy} \cdot \sin(\alpha) \cdot \cos(\alpha)$$

$$\varepsilon_b = \varepsilon_x \cdot \cos(\alpha + \beta)^2 + \varepsilon_y \cdot \sin(\alpha + \beta)^2 + \gamma_{xy} \cdot \sin(\alpha + \beta) \cdot \cos(\alpha + \beta)$$

$$\varepsilon_c = \varepsilon_x \cdot \cos(\alpha + \beta + \gamma)^2 + \varepsilon_y \cdot \sin(\alpha + \beta + \gamma)^2 + \gamma_{xy} \cdot \sin(\alpha + \beta + \gamma) \cdot \cos(\alpha + \beta + \gamma)$$

Strain Matrix

$$\text{Find}(\varepsilon_x, \varepsilon_y, \gamma_{xy}) \rightarrow \begin{pmatrix} 26.1 \times 10^{-6} \\ -29.3 \times 10^{-6} \\ -2.240121 \times 10^{-3} \end{pmatrix} \qquad \varepsilon := \begin{pmatrix} 26.1 \times 10^{-6} \\ -29.3 \times 10^{-6} \\ -2.240121 \times 10^{-3} \end{pmatrix}$$

Principal Strains * Assuming plane stress

$$\varepsilon_{max} := \frac{(\varepsilon_1 + \varepsilon_2)}{2} + \sqrt{\frac{(\varepsilon_1 - \varepsilon_2)^2}{4} + \left(\frac{\varepsilon_3}{2}\right)^2} = 1.119 \times 10^{-3}$$

$$\varepsilon_{min} := \frac{(\varepsilon_1 + \varepsilon_2)}{2} - \sqrt{\frac{(\varepsilon_1 - \varepsilon_2)^2}{4} + \left(\frac{\varepsilon_3}{2}\right)^2} = -1.122 \times 10^{-3}$$

Principal Stresses $\quad E := 29000\text{ksi} \quad \nu := 0.3 \quad G := \frac{E}{2 \cdot (1 + \nu)} = 11154 \cdot \text{ksi}$

$$\sigma_{max} := \frac{E \cdot (\varepsilon_{max} + \nu \cdot \varepsilon_{min})}{(1 - \nu^2)} = 24.927 \cdot \text{ksi} \qquad \sigma_{min} := \frac{E \cdot (\nu \cdot \varepsilon_{max} + \varepsilon_{min})}{(1 - \nu^2)} = -25.06 \cdot \text{ksi}$$

$$\tau_{max} := \frac{(\sigma_{max} - \sigma_{min})}{2} = 24.994 \cdot \text{ksi}$$

Figure 59 - Principal Stress and Strain Calculations for WC250 Peak Load

MSI TESTING, INC.

"SERVING THE INDUSTRY FOR 30 YEARS"

50 WEST LOUISE AVE. • SALT LAKE CITY, UT 84115

TELEPHONE (801) 484-1789

REPORT of ANALYSIS

Star Seismic
3070 Rassmussen Road Suite 260
Park City, Utah 84098

April 20, 2009
Project # 09-113
P.O. # Verbal Carter
Page 1 of 2

Attn : Carter Mickey

The following are the results of the testing done on the sample(s) you provided:

Sample #	Description :	
1	910-1216	1" Plate
2	910-1214	3/4" Plate
3	910-1210	3/4" Plate
4	910-1213	1" Plate
5	910-1212	3/4" Plate
6	910-1214	1" Plate

Longitudinal Tensile Test: Test Method in accordance with ASTM A370 .

Sample	Dia-meter	Area	Tensile Load (LBS)	Tensile Strength (PSI)	Yield Load (LBS)	Yield Strength (PSI)	Elong. %	Reduction of Area
1-1	.501	.1971	12,588	64,000	8,150	41,400	34	68
1-2	.498	.1948	12,242	63,000	7,600	39,000	32	68
1-3	.490	.1886	12,729	67,500	7,426	39,400	31.5	64
2-1	.494	.1917	12,920	67,500	7,984	41,700	32.5	68
2-2	.499	.1956	12,,801	65,500	8,267	42,300	34.5	65
2-3	.498	.1948	12,872	66,000	8,905	45,700	32	68
3-1	.491	.1893	11,,961	63,000	7,232	38,200	29.5	66
3-2	.497	.1940	12,408	64,000	8,696	44,800	32	66
3-3	.498	.1948	12,415	63,500	7,462	43,400	34	64

Star Seismic Note:
Average Fysc of mill and MSI testing = 43.3 ksi. Use 43.3 ksi in design.

Figure 60 - WC200 Steel Core Tensile Test

SOLD TO: STAR SIESMIC LLC
3070 RASMUSSEN RD
STE 260
PARK CITY, UT 84098-

SHIP TO: CRANE CONSTRUCTION NW, INC.
669 West 200 South, Bldg #2
SALT LAKE CITY, UT 84101-

NUCOR
BAR MILL GROUP
PLYMOUTH DIVISION

CERTIFIED MILL TEST REPORT Page: 1
NUCOR - PLYMOUTH IS AN I.S.O. 9001 AND AN A.B.S. CERTIFIED MILL

Ship from:
Nucor Steel - Utah
W Cemetery Road
PLYMOUTH, UT 84330
435-458-2300

Date: 1-Apr-2009
B.L. Number: 321848
Load Number: 141175

Material Safety Data Sheets are available at www.nucorbar.com or by contacting your inside sales representative.

NBMG-08 March 24, 2009

HEAT NUM. *	DESCRIPTION	PHYSICAL TESTS					CHEMICAL TESTS						
		YIELD P.S.I.	TENSILE P.S.I.	ELONG % IN 8"	BEND	WT% / DEF	C / Ni	Mn / Cr	P / Mo	S / V	Si / Cb	Cu / Sn	C.E.
PO# =>	90305												
PL0910121201	Nucor Steel - Utah	43,667	63,349	31.0%			.10	.57	.008	.027	.28	.27	
	3/4x10" FL 45'	301MPa	437MPa				.10	.09	.031	.007	.001	.010	
	A36 YLD 38-44	43,283	63,224	32.0%									
	ASTM A36 Low Yeild	298MPa	436MPa										
PO# =>	90305												
PL0910121402	Nucor Steel - Utah	43,562	63,113	29.0%			.11	.58	.009	.025	.24	.29	
	3/4x10" FL 45'	300MPa	435MPa				.08	.08	.023	.007	.001	.010	
	A36 YLD 38-44	43,452	62,927	29.0%									
	ASTM A36 Low Yeild	300MPa	434MPa										

CMTR COMPLIES WITH DIN EN 10204 - 3.1.B

I HEREBY CERTIFY THAT THE ABOVE FIGURES ARE CORRECT AS CONTAINED IN THE RECORDS OF THE CORPORATION.

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.

QUALITY ASSURANCE: Scott Laurenti

Figure 61 - WC200 Steel Core Tensile Test

MSI TESTING, INC.

"SERVING THE INDUSTRY FOR 30 YEARS"
50 WEST LOUISE AVE. • SALT LAKE CITY, UT 84115
TELEPHONE (801) 484-1789

REPORT ANALYSIS

Star Seismic
3070 Rassmussen Road Suite 260
Park City, Utah 84098

October 13, 2008
Project # 08-269
P.O. #

Attn : Carter Mickey

The following are the results of the testing done on the sample(s) you provided:

Sample #	Description :
1	1" Plate Heat # [redacted] 810-7776
2	1" Plate Heat # [redacted] 810-7777
3	1" Plate Heat # [redacted] 810-7775
4	3/4" Plate Heat # [redacted] 810-7775

Tensile Test: Test Method ASTM A370 .

Sample	Dia-meter	Area	Tensile Load (LBS)	Tensile Strength (PSI)	Yield Load (LBS)	Yield Strength (PSI)	Elong. %	Reduction of Area
1-1	.501	.1971	13,146	66,500	8,564	43,500	35	61
1-2	.500	.1963	13,054	66,500	8,487	43,200	38	62
1-3	.503	.1987	13,231	66,500	8,323	41,900	32	54
2-1	.492	.1901	12,606	66,500	8,264	43,500	33	71
2-2	.497	.1940	12,844	66,000	8,366	43,100	35	68
2-3	.489	.1878	12,499	66,500	8,051	42,900	33	68
3-1	.494	.1917	12,878	67,000	8,249	43,000	34	67
3-2	.493	.1909	12,790	67,000	8,165	42,800	35	68
3-3	.500	.1963	13,199	67,000	8,554	43,600	37	67
4-1	.502	.1979	13,531	68,500	8,870	44,800	35	70
4-2	.498	.1948	13,294	68,000	8,817	45,300	35	68
4-3	.498	.1948	13,122	67,500	8,930	45,900	31	67

Yield .2% offset.
Elongation 2"

Craig Griffiths
Craig Griffiths (President MSI Testing, Inc.)

```
Star Seismic Note:
Mill Fysc range differs from MSI range.  MSI testing average of 43.1 used.
```

Figure 62 - WC250 Steel Core Tensile Test

SOLD TO: PKM STEEL
PO BOX 920
SALINA, KS 67410-0000

SHIP TO: PKM STEEL
STAR SEISMIC
C/O METRO GROUP TRANSLOAD
YARD 5 TRACK 430
SALT LAKE CITY, UT 84101-

NUCOR
BAR MILL GROUP
JEWETT DIVISION

CERTIFIED MILL TEST REPORT

Page: 1

Ship from:
Nucor Steel - Texas
8812 Hwy 79 W
JEWETT, TX 75846
903-626-4461

Date: 27-Aug-2008
B.L. Number: 497602
Load Number: 117155

Material Safety Data Sheets are available at www.nucorbar.com or by contacting your inside sales representative.

NBMG-08 May 16, 2008

HEAT NUM. *	DESCRIPTION	PHYSICAL TESTS					CHEMICAL TESTS						
		YIELD P.S.I.	TENSILE P.S.I.	ELONG % IN 8"	BEND	WT% / DEF	C / Ni	Mn / Cr	P / Mo	S / V	Si / Cb	Cu / Sn	C.E.
PO# =>	71024-8083												
JW0810777402	Nucor Steel - Texas	42,300	61,800	24.0%			.10	.85	.018	.020	.20	.49	
JW08107774C	3/4x10 Flat 38'	292MPa	426MPa				.14	.16	.036	.003	.001	.008	
	A36 PLT LW YLD	41,300	62,100	21.0%									
	ASTM A36 Low Yield Plate	285MPa	428MPa										
PO# =>	71024-8083												
JW0810777501	Nucor Steel - Texas	39,900	61,900	23.0%			.09	.87	.020	.040	.23	.38	
JW08107775B	1x10 Flat 38'	275MPa	427MPa				.17	.16	.039	.004	.001	.007	
	A36 PLT LW YLD	38,500	64,000	23.0%									
	ASTM A36 Low Yield Plate	265MPa	441MPa										
PO# =>	71024-8083												
JW0810777601	Nucor Steel - Texas	40,300	58,100	25.0%			.09	.84	.018	.030	.22	.41	
JW08107776	1x10 Flat 38'	278MPa	401MPa				.13	.15	.033	.003	.001	.009	
	A36 PLT LW YLD	41,700	61,900	25.0%									
	ASTM A36 Low Yield Plate	288MPa	427MPa										
PO# =>	71024-8083												
JW0810777601	Nucor Steel - Texas	40,300	58,100	25.0%			.09	.84	.018	.030	.22	.41	
JW08107776	1x10 Flat 45'	278MPa	401MPa				.13	.15	.033	.003	.001	.009	
	A36 PLT LW YLD	41,700	61,900	25.0%									
	ASTM A36 Low Yield Plate	288MPa	427MPa										
PO# =>	71024-8083												
JW0810777701	Nucor Steel - Texas	39,600	65,500	25.0%			.10	.81	.014	.020	.19	.34	
JW08107777	1x10 Flat 38'	273MPa	452MPa				.14	.13	.033	.003	.001	.006	
	A36 PLT LW YLD	38,800	59,300	25.0%									
	ASTM A36 Low Yield Plate	268MPa	409MPa										

I HEREBY CERTIFY THAT THE ABOVE FIGURES ARE CORRECT AS CONTAINED IN THE RECORDS OF THE CORPORATION.

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.

1" Heat #810-7775

QUALITY ASSURANCE: Ben Cave

Ben R Cave

Figure 63 - WC250 Steel Core Tensile Test

SR1E

Industrial Low Cost String Pot

Incremental Encoder Output Signal
Linear Position Measurement up to 125 Inches (3 meters)
Designed for Outdoor / Wet environments

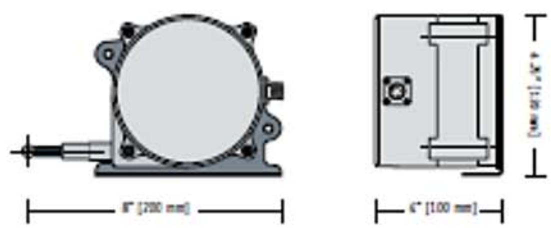
The SR1E is rugged, low-cost, high performance string pot built to withstand wet environments and outdoor applications. Designed for construction equipment and factory use, the SR1E is the perfect low-cost solution for OEM and stocking distributors.

At the heart of this sensor is a robust incremental encoder that delivers a linear resolution of 101 pulses per inch. The SR1E ships with an industry standard push-pull encoder driver that can be powered by 5-30 VDC. (Other resolutions and complimentary channels are available, please consult factory). Each sensor ships with a 4-pin, field installable, M12 connector and an additional 13 ft. (4 m) cordset is also available. Just like the rest of our SR1 series, the SR1E is in stock for quick delivery.

Full Stroke Range	125 inches (3175 mm)
Output Signal	Incremental encoder
Resolution	101 ±2 pulses per inch
Accuracy	±.1% FS.
Repeatability	±.05% FS.
Environmental Suitability	NEMA 6, IP67

SPECIFICATIONS

Input Voltage	5-30 VDC
Input Current	100 mA max., no load
Sensor	Incremental encoder
Output Driver Type	push-pull (note: Vin = Vout)
Output Driver Current	20 mA max., source/sink
Maximum Velocity	80 inches (2 meters) per second
Maximum Acceleration	10 G (retraction)
Operating Temperature	-4° to 185° F (-20° to 85° C)
Enclosure	polycarbonate
Measuring Cable	.034-inch dia. nylon-coated stainless
Electrical Connection	M12 Connector (mating plug included)
Weight	2.5 lbs. (1.3 Kg)



Ordering Information

	part number	description
	SR1E-125	125-inch measurement range 101 pulses per inch includes 4 pin M12 connector
13 ft (4 m) 	9036810-0040	Optional Cordset w/ 4-pin M12 connector

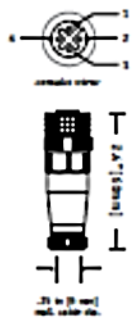
Consult factory for alternate resolution and differential output signals.

Celesco Transducer Products, Inc.
20630 Plummer Street • Chatsworth, CA 91311
tel: 800.423.5483 • +1.818.701.2750 • fax: +1.818.701.2799
celesco.com • info@celesco.com

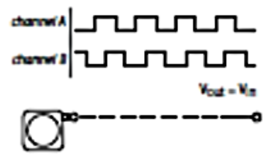
Figure 64 - Celesco String Pot Data Sheet

Electrical Connection

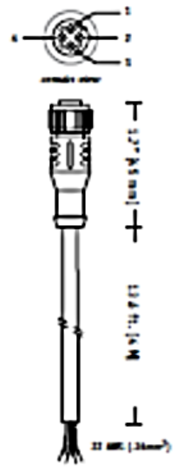
Field Installable Connector



pin	signal
1	5...30 VDC
2	common
3	channel A
4	channel B



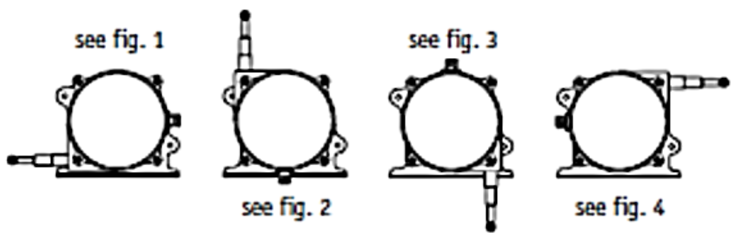
Cord Set Connections



pin	conductor	signal
1	brown	5...30 VDC
2	white	common
3	blue	channel A
4	black	channel B

cable specifications	
length:	13 ft. (4m)
wire size:	22 AWG (.34mm²)
cable material:	PVC
cable color:	gray

Cable Exit Direction Options



Changing the Measuring Cable Exit and Electrical Connector Direction

Changing Measuring Cable Exit

To change the direction of the measuring cable, remove the 4 mounting bracket screws and rotate bracket to one of four available positions. See figures 1 - 4 on the following pages for mounting dimensions.

Changing Electrical Connector Direction

To change the position of the electrical connector, remove the 4 rear cover screws and carefully separate rear cover from the sensor body.

Rotate the rear cover to desired position being careful to not tangle the wiring harness that runs to the connector.

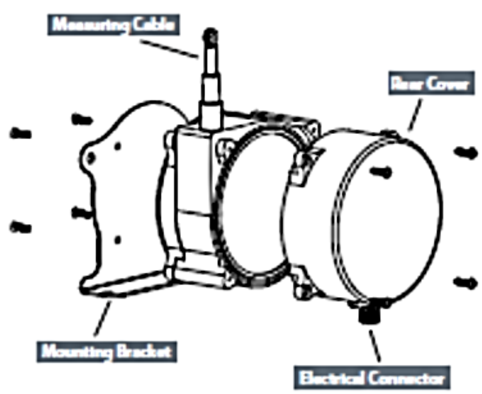
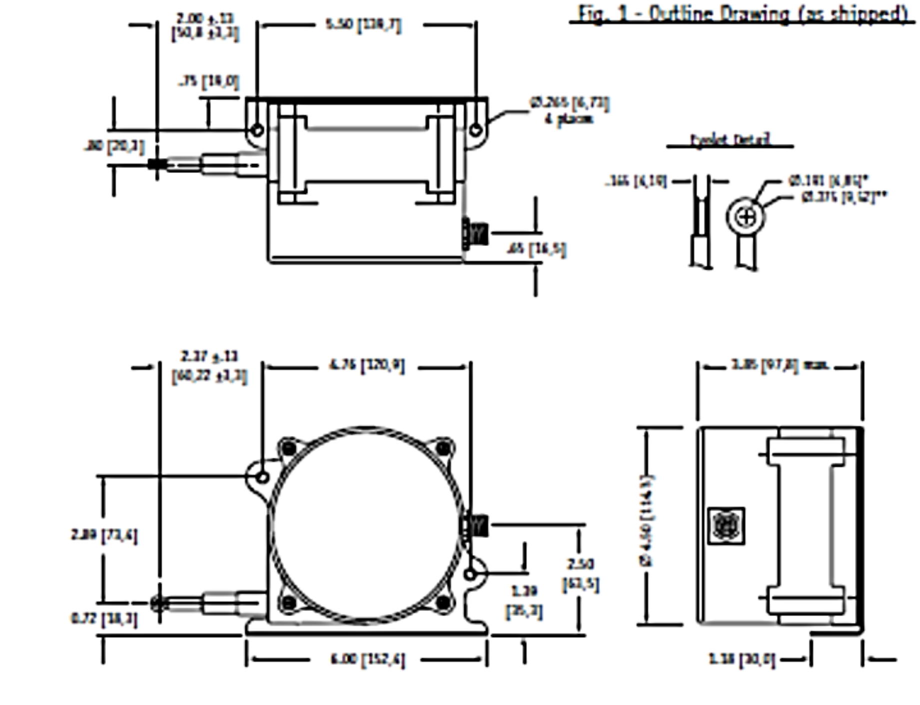


Fig. 1 - Outline Drawing (as shipped)



units are in inches [mm] tolerances are ± .06 [1,0] unless otherwise noted

Fig. 2 - "Up" Cable Exit Direction

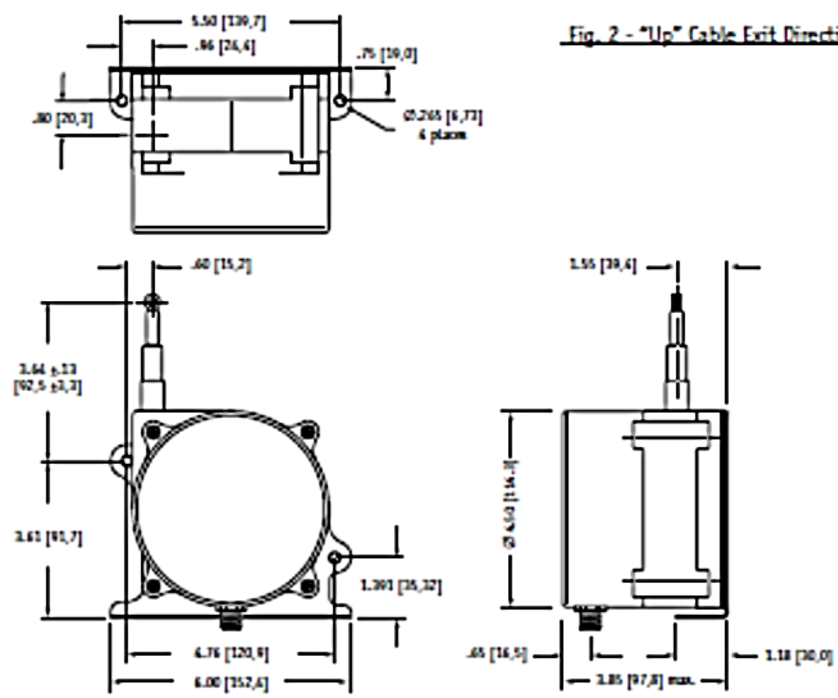


Fig. 3 - "Down" Cable Exit Direction

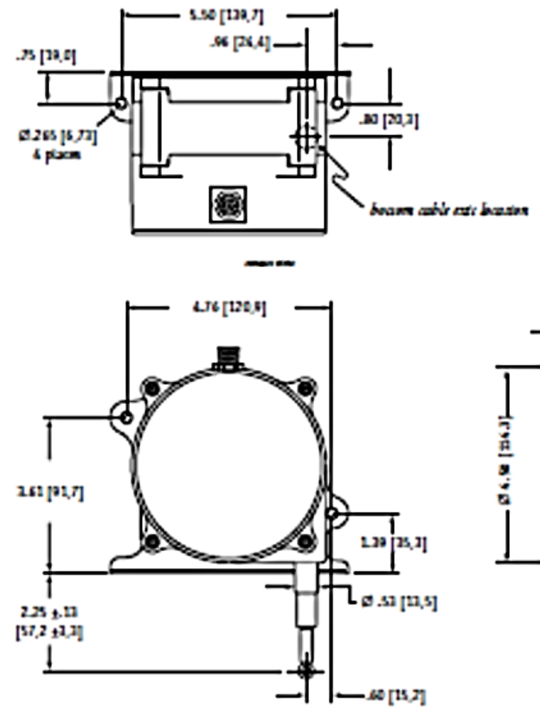
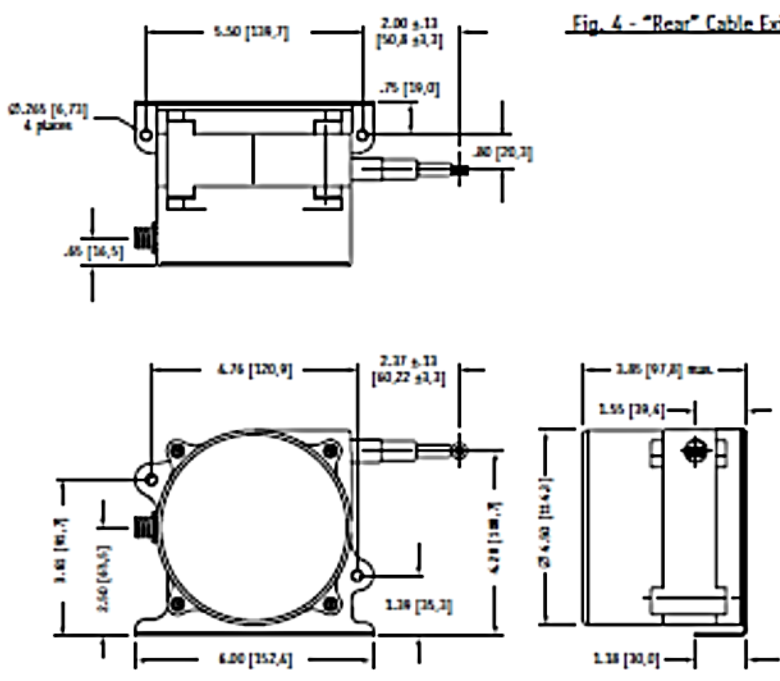


Fig. 4 - "Rear" Cable Exit Direction



version 1.0 Last updated August 4, 2009

JX-EP SERIES
DIGITAL OUTPUT

The UniMeasure JX-EP series linear position transducer with digital output is oriented for use in moderate duty applications in hostile wet or dry environments. The chemical resistant thermoplastic case of the transducer with integral dust wiper is factory configurable to NEMA 12 (IP-52) for dust protection or to NEMA 4X (IP-65) for applications where exposure to washdown, rain, oil and other liquids may occur. The sealed case is achieved through the use of o-rings and a low friction shaft seal. The wire rope exit direction may be specified at time of order or may be user adjusted at time of installation. The standard electrical connection includes a sealed bulkhead fitting and multi-conductor electrical cable. An optional cable to cable connector with mating connector may be added to the electrical cable. Alternatively, the cable to cable connector may be ordered without the mating connector. The mating connector with a length of electrical cable attached may be ordered as a separate item. As a convenience, optional connector locations on the transducer body are offered. The standard electrical output of the unit is a TTL level two channel square wave in quadrature. Optional outputs include line driver and push-pull circuits.

SPECIFICATIONS

General

- Measurement Range .... See Range Table below
- Sensing Device .... Digital Encoder
- Nominal Resolution
  - 10" range.................. 445 counts/inch, 17.5 counts/mm
  - 15", 30" range.................. 327 counts/inch, 12.9 counts/mm
  - 20", 40" range.................. 246 counts/inch, 9.7 counts/mm
  - 25", 50" range.................. 198 counts/inch, 7.8 counts/mm
  - 60" range.................. 166 counts/inch, 6.5 counts/mm
  - 80" range.................. 126 counts/inch, 5.0 counts/mm
- Linearity .... ±0.10% Full Scale
- Repeatability (in times 1 counting mode).................. ±1 Count, ranges to 25"; ±2 Counts, ranges 30" to 80"
- Construction .... Thermoplastic Body
- Wire Rope .... Ø.018 (0.46 mm) Jacketed Stainless Steel
- Wire Rope Tension.................. See Supplemental Data[1]
- Weight .... 6.3 oz. (180 gm)
- Connection .... 24 AWG Shielded Electrical Cable

Life (to wire rope replacement)

- Ranges 10" to 25".................. 1,000,000 full stroke cycles
- Ranges 30" to 80".................. 500,000 full stroke cycles

Electrical

- Excitation Voltage .... 5.00 ±0.25 VDC
- Output .... 2 channel square wave in quadrature TTL level current sinking with 65 KΩ pullups

Environmental

- Operating Temperature .... -40°C to 70°C
- Storage Temperature.................. -40°C to 80°C
- Operating Humidity .... 95% R.H. non-condensing IP-52 case 100% R.H. IP-65 case
- Vibration.................. 20 G's maximum
- Ingress Protection.................. NEMA 12 or 4X, IP-52 or 65

FOOTNOTES TO SPECIFICATIONS
1. Supplemental Data section located at end of JX Series pages.

Model Number Configuration

JX-EP- __ - ___ - __ 0 - ___

Range

10	10"	(250 mm)
15	15"	(380 mm)
20	20"	(500 mm)
25	25"	625 mm)
30	30"	(750 mm)
40	40"	(1000 mm)
50	50"	(1250 mm)
60	60"	(1500 mm)
80	80"	(2000 mm)

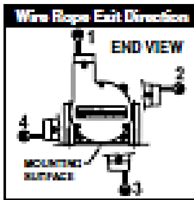
Wire Rope

N.......0.018" nylon jacketed stainless steel

Wire Rope Tension

1.......Standard (See Table A and Table B for values)

Wire Rope Exit Direction



Ingress Protection

1.......IP-65 (NEMA 4X) dust and spraying water protection
2.......IP-52 (NEMA 12) dust and dripping liquids

Electrical Output

1.......5 VDC current sinking 2 channel
2.......5 VDC current sinking differential line drive
3.......5 VDC push-pull differential line drive
4.......8 to 28 VDC current sinking differential line drive
5.......8 to 28 VDC push-pull differential line drive

See TABLE 3, Electrical Output Options, Page 12 for waveform and output stage configuration.

Connector

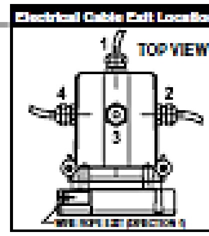
0.......No connector
C.......Cable to cable connector with mating connector
K.......Cable to cable connector with NO mating connector

For Option "K", mating connector with electrical cable is available as P/N 10026-(x) where 'x' is required length in meters

Electrical Cable

P.......150 mm (6") pigtail
1.......1 m (3')
2.......2 m (6.5')
3.......3 m (10')
4.......4 m (13.5')
5.......5 m (16.5')

Electrical Cable Exit Location



NOTE
1) Shaded options available at additional cost.

Example
JX-EP-50-N11-110-42U

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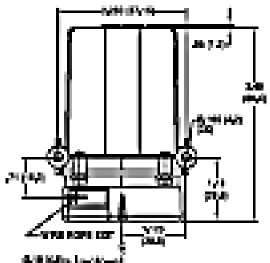
Figure 65 - UniMeasure String Pot Data Sheet

JX SERIES
SUPPLEMENTAL DATA

DIMENSIONAL INFORMATION

Ranges to 50" (1250 mm)



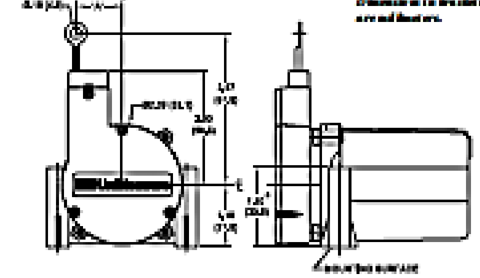
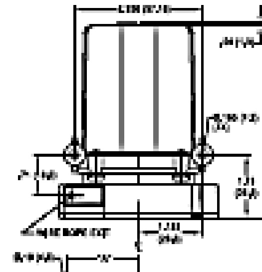


Table A

Range Designator	Dim "A" (in)	Dim "A" (mm)	Nominal Wire Rope Tension (oz)	Nominal Wire Rope Tension (N)
2, 10	.25	6.4	16	4.4
2.5, 15, 20	.50	12.7	14	3.9
3.0, 20, 40	.66	16.7	11	3.1
4.7, 25, 50	.82	20.8	8	2.2

Dimensions in brackets are millimeters.

Ranges 60" (1.5 m) and 80" (2 m)



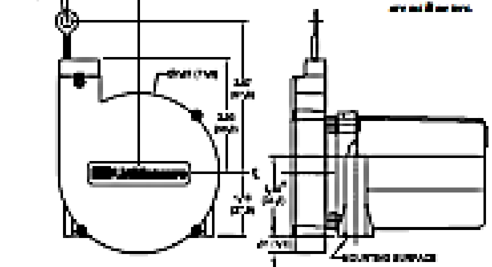


Table B

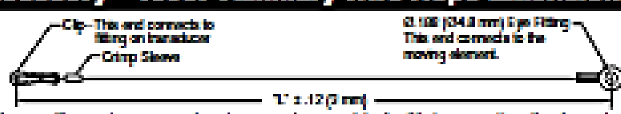
Range Designator	Dim "A" (in)	Dim "A" (mm)	Nominal Wire Rope Tension (oz)	Nominal Wire Rope Tension (N)
60	.98	24.9	10	2.8
80	1.28	32.5	8.6	2.4

Dimensions in brackets are millimeters.

TABLE 3—Electrical Output Options—JX-EP Series

Option	Output Type	Output Stage	Waveform
1	**5 VDC TTL Two Channel Current Sinking** Two channels in quadrature with 65KΩ internal pullup resistors. 5 VDC input voltage	+5 VDC; 65KΩ; Vout; COMMON	A, B
2	**5 VDC TTL Current Sinking Differential Line Drive** Current sinking line drive output. 2KΩ internal pullup resistors. 5 VDC input voltage	+5 VDC; 2KΩ; Vout; 7406	A, $\bar{A}$, B, $\bar{B}$
3	**5 VDC Push-Pull Differential Line Drive** Push-Pull, current sourcing and current sinking output. 5 VDC input voltage. Output is compliant with requirements of TIA/EIA-422-B.	+5 VDC; AM26C31; Vout; COMMON	
4	**8 to 28 VDC Current Sinking Differential Line Drive** Current sinking line drive output with 10KΩ internal pullup resistors. 8 to 28 VDC input voltage	+8 to +28 VDC; 10KΩ; Vout; 7406	
5	**8 to 28 VDC Push-Pull Differential Line Drive** Push-Pull, current sourcing and current sinking output. 8 to 28 VDC input voltage.	+8 to +28 VDC; 7272; Vout; COMMON	

Accessory—10067 Auxiliary Wire Rope Extension Kit



The auxiliary wire rope extension may be used to facilitate mounting the transducer remotely from the measurement point. The clip on the extension attaches to the eye fitting on the transducer. The eye fitting on the opposite end which is identical to the fitting on the transducer mounts to the moving element. The extension kit is also available with the clip end unterminated for situations where it is more convenient to size the wire rope length during installation. The clip and crimp sleeve are included as loose parts for user termination.

10067-__CM-__

*......... Completed kit (no designator required)
U......... Unterminated clip end (clip and crimp sleeve included in kit)

* Leave blank. No designator required.

Dimension "L"
Specify dimension "L" in centimeters to the nearest whole centimeter.

Note 1. 1 cm = 0.394", 1 inch = 2.54 cm
Note 2. Shortest length "L" is 5 cm (approximately 2").

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