DEVELOPMENT AND VALIDATION OF A TWELVE BOLT EXTENDED STIFFENED END-PLATE MOMENT CONNECTION

Trevor A. Szabo

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Matthew R. Eatherton, Chair
Thomas M. Murray
Roberto T. Leon

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ABSTRACT

Three end-plate moment connection configurations are prequalified for special moment frames for seismic applications in AISC 358-10. The eight bolt extended stiffened connection is the strongest of the three configurations, but it can only develop approximately 30 percent of currently available hot-rolled beam sections. The strength of this configuration is limited by bolt strength. There is a need for a stronger end-plate moment connection, hence the reason for the development and validation of a twelve bolt configuration.

Equations were developed for the design procedure using various analytical methods, which included yield line analysis and an effective tee stub model. An experimental program was conducted, which consisted of the full-scale cyclic testing of four end-plate moment connections. The intention of the testing was to develop and validate the design procedure, and prequalify a new twelve bolt configuration. A displacement-controlled loading protocol was applied according to AISC 341-10. The experimental results showed that the model for thick end-plate behavior is conservative by 6.7%, the model for end-plate yielding is conservative by 8.8%, and the model for bolt tension rupture with prying conservatively predicts by 18.5%. The specimens that were designed to form a plastic hinge in the beam fractured in a brittle manner. The deep beam specimen fractured in the first 2% story drift cycle, and the shallow beam specimen fractured in the second 3% story drift cycle. The fracture of the prequalification specimens was determined to have been caused by stiffeners of high yield stress relative to the beam yield stress.

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GENERAL AUDIENCE ABSTRACT

End-plate moment connections are a common way to create a rigid joint between beams and columns. Before using a moment connection in a steel building to resist horizontal earthquake loads, each connection configuration must be tested at full-scale and meet performance criteria prescribed in the applicable building code (in this case, the *Seismic Provisions for Structural Steel Buildings* published by the American Institute of Steel Construction).

Three end-plate moment connection configurations have been previously “prequalified” for high seismic regions, which means that sufficient previous testing has shown adequate performance. The eight bolt end-plate moment connection is the strongest of the three configurations, but it can only develop approximately 30 percent of currently available hot-rolled steel beam sections. The strength of this configuration is limited by bolt strength. There is a need for stronger end-plate moment connections, which motivated the development and validation of a twelve bolt configuration in this thesis.

Equations were developed for the design of the twelve-bolt end-plate moment connection including equations to predict when the bolts would fracture and when the end-plate would yield. An experimental program was conducted, which consisted of the full-scale cyclic testing of four end-plate moment connections. The intention of the testing was to validate the design procedure and demonstrate that the connection could withstand significant inelastic rotation. The connection assembly was cycled back and forth according to a displacement protocol prescribed in the *Seismic Provisions for Steel Buildings*. The experimental results showed that the equations were able to predict bolt rupture within 6.7% of the applied moment at fracture, the equation for end-plate yielding was conservative by 8.8%, and the equation for bolt fracture with prying action was conservative by 18.5%. The specimens that were intended to show the connection could withstand significant inelasticity fractured in an unexpected brittle manner. The deep beam

specimen fractured in the first 2% story drift cycle, and the shallow beam specimen fractured in the second 3% story drift cycle, neither of which reach the target of 4% story drift. The fractures were determined to have been caused by stiffeners that had too high a yield stress relative to the beam yield stress.

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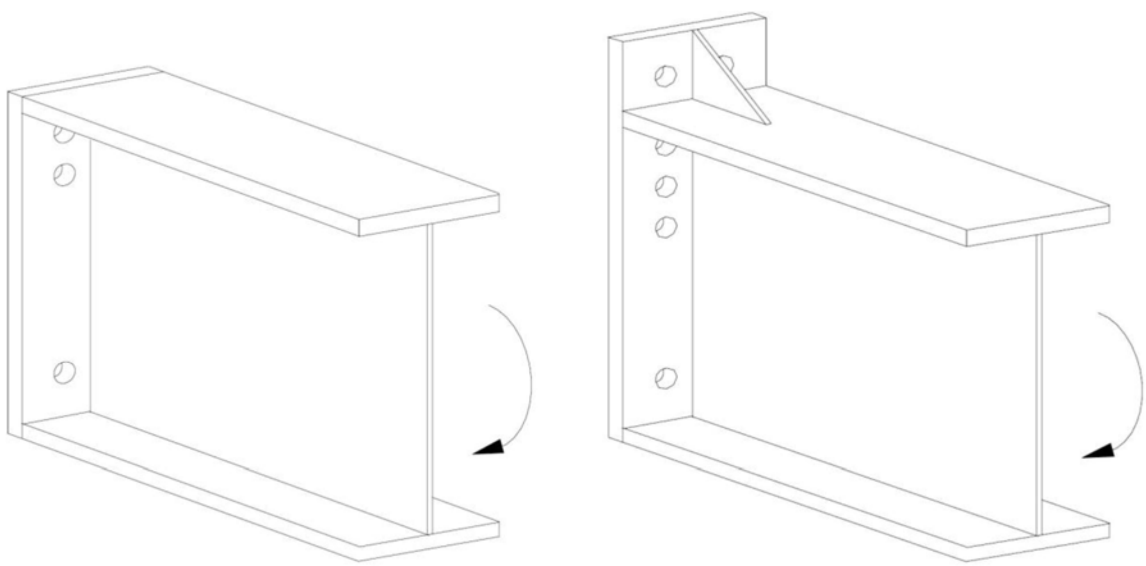
1. INTRODUCTION

1.1. Background

Bolted end-plate moment connections are a popular type of connection used in the metal building industry. End-plate moment connections involve a beam section that has a plate welded to the end, which has rows of bolts that allow it to be attached to an adjacent member. This adjacent member can be a column flange, a column web or another beam with an end-plate as a splice. There are many different configurations of bolts that have been studied. End-plate moment connections can be classified in multiple different ways: flush vs. extended, number of bolts at the tension flange, number of bolts per row, stiffened vs. unstiffened and type of loading. The greatest number of bolts are placed around the tension flange. In connections that see load reversal, the same bolt pattern is placed at both flanges. An example of a flush unstiffened, and an extended stiffened end-plate connection can be seen below in Figure 1-1.



a) Four-Bolt Flush Unstiffened *b) Multiple Row Extended 1/3 Stiffened*

Figure 1-1 Example of a Flush and an Extended End-Plate Connection (Murray and Shoemaker 2002)

In the Northridge Earthquake in 1994, there were many unexpected brittle failures in beam-to-column connections. The buildings that saw failures ranged in height, from one story to 26 stories, and age, in the middle of being erected to 30 years old (FEMA 2000). These failures brought particular attention to the welds, which were complete joint penetration (CJP) welds from the beam flange to the column flange. The webs were bolted to the column flange with a shear tab. These welds were done in the field with a backing bar, which was left in place. Cracks typically initiated where the backing bar met the column due to stress concentrations (FEMA 2000). As a result of the issues posed by these moment connections, the advantages of end-plate moment connections seemed even greater.

End-plate moment connections are less prone to these types of fractures for several reason. The welds are done in shop, which yields greater quality welds because the welds are performed under controlled conditions and in the most favorable positions. Also, the root of the weld is always on the inside of the flange. This leads to greater ductility because the root is lesser quality material, and the root is a stress concentration. By having the root on the inside of the flange, the root will see less strain demand, which helps mitigate the stress concentration posed by the presence of a root.

Some of the advantages of end-plate moment connections are the ease of erection, shop welds, and greater ductility (Adey et al. 2000). Bolted connections are easier to construct and typically faster than field welded connections. Shop welds can also be performed ahead of time. The advantages of end-plate moment connections also include the reasons previously mentioned relating to fracture potential.

End-plate moment connections typically use pretensioned bolts. The AISC Specification for Structural Steel Buildings presents a table with the minimum pretension level depending on the bolt diameter (AISC 2010b). Tests were done on end-plate moment connections with snug tight bolts and connections with pretensioned bolts. Snug tight bolted connections were found to be acceptable (Kline et al. 1990). However, pretensioned bolts provided stiffer connections (Kline et al. 1990).

Research started on end-plate moment connections in the 1950's (Meng 1996). A significant amount of the research on this category of connection has been done over the past twenty years. Presently, research has been done on end plate moment connections that have four to twelve bolts at the tension flange with different patterns of bolts, stiffened and unstiffened, and under monotonic and cyclic loading (Adey et al. 1997; Blumenbaum 2004; Borgsmiller 1995; Sumner 2003).

1.2. Motivation and Proposed Solution

Currently in the AISC's *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC 358-10), AISC's *Extended End-Plate Moment Connections Seismic and Wind Applications* (AISC Design Guide 4) and AISC's *Flush and Extended Multiple-Row Moment End-Plate Connections* (AISC Design Guide 16), out of all of the end-plate moment connections that are detailed, the most amount of bolts at the tension flange is eight bolts (AISC 2011; Murray and Sumner 2014; Murray and Shoemaker 2002). If a firm that is designing a steel moment frame building wants to use an end-plate moment connection geometry other than the ones presented in AISC 358 for a special moment resisting frame (SMRF), the firm needs to test the connection according to certain testing provisions (AISC 2010a). A decent range of prequalified connections are listed. The connections that are currently prequalified are shown below in Figure 1-2. However, as the spans keep getting pushed further and deeper members are required, connections with more bolts are also required. This presents a need for a prequalified connection for SMRF with twelve bolts at the tension flange.

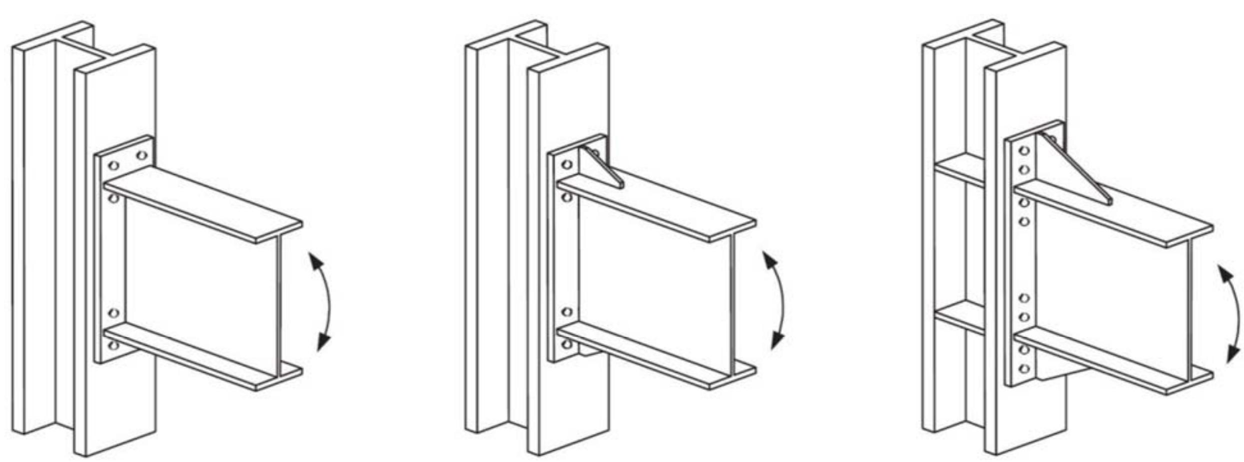


Figure 1-2 Extended End-Plate Connections (Murray and Sumner 2014)

The proposed solution for this limited connection size is an extended end-plate moment connection with twelve bolts at the tension flange. The geometry of this proposed connection can be seen below in Figure 1-3.

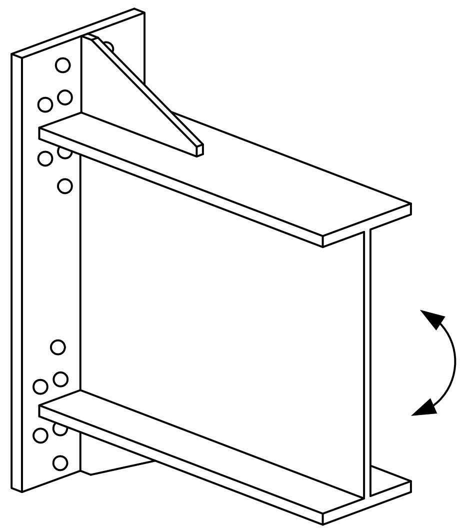


Figure 1-3 Twelve-Bolt Extended Stiffened End-Plate Configuration

For all steel special and intermediate moment frames using end-plate moment connections, the beam flange to end-plate welds are required to be complete joint penetration (CJP) welds and are considered demand critical welds, which means that they must meet certain criteria defined in the code (AISC 2010a, 2011). CJP weld are generally more expensive and time consuming than other types of welds, such as fillet or partial penetration (PJP) welds. Inspection using ultrasonic testing to determine the size of any flaws in the weld is required following the completion of the CJP welds in material 5/16 inches thick or greater (AISC 2010a). PJP welds or fillet weld can be cheaper and quicker, and ultrasonic testing is not required. Thus, PJP would be preferred.

1.3. Objective and Scope of Research

The goal of this research was to:

- Propose design procedure for a new configuration of end-plate moment connection with twelve bolts at the tension flange: twelve-bolt extended stiffened (12ES)
- Verify these design procedures through full-scale testing
- Provide supporting evidence for the adequacy of partial joint penetration welds with reinforcing fillet welds for the beam to end-plate welds in end-plate moment connections

Design procedures for the 12-bolt extended stiffened end-plate moment connection were developed for thick-plate and thin-plate behavior. The design procedure for thick-plate behavior was based on previous work, where each of the bolts at the tension flange are assumed to reach full strength simultaneously. The development of the design procedure for thin-plate behavior started with yield line analysis, which was used to analytically determine the moment capacity associated with end-plate yielding. Several yield line mechanisms were considered, however, one mechanism was found to be the controlling case. After an end-plate yields, prying action occurs in the bolts. The moment capacity associated with bolt tension rupture with prying action was determined by using an effective tee stub model.

Four full-scale beams were tested under quasi-static cyclic loading. These specimens were the twelve-bolt extended stiffened configuration. The general end-plate configuration that was tested is shown in Figure 1-3 above. Each specimen used different end-plate thickness, size bolt, and bolt spacing. Two of these twelve bolt connections were 44 inch deep, compact beam specimens and the other two were 24 inch deep, compact beam specimens. One of the deep specimens was designed to fail the bolts in bolt tension rupture without prying action in order to validate the thick end-plate behavior. One of the shallow specimens was designed for end-plate yielding and failure of the bolts in bolt tension rupture with prying action in order to validate the thin end-plate behavior. The other two specimens, one deep and one shallow, were designed to form a plastic hinge in the beam in order to prequalify the connection for use in SMRF.

The two specimens that were used to validate the thick and thin design procedures used PJP welds with reinforcing fillet welds for the beam flange to end-plate welds. These specimens were designed to fail under lesser loads than their respective plastic hinging specimens, which means the full capacity of a CJP weld was not necessary at this location. Although these welds were not designed to fail, they still provide support for the use PJP welds with reinforcing fillet welds in place of CJP welds.

During the testing of the two specimens that were designed to form a plastic hinge, each beam failed in a brittle manner. An investigation was implemented to determine the cause of these fractures. This investigation involved determining material properties of the beam material, examining previous cyclic testing of extended stiffened end-plate moment connections, and performing an idealized cross-section analysis, which involved assessing the yielding that occurred in the stiffeners.

1.4. Thesis Organization

This document is organized into the following chapters:

- Chapter 1 introduces the topic of research in general with background information, the motivation and objectives of this research.
- Chapter 2 summarizes the literature on the three topics encompassing this research, which includes high capacity end-plate moment connections, partial joint penetration welds, and the expected seismic behavior of full metal buildings.
- Chapter 3 discusses how yield line analysis was applied, the different yield line mechanisms that were examined, and the approach that was used to determine the controlling yield line mechanism.
- Chapter 4 outlines the proposed design procedure for this new twelve-bolt end-plate moment connection.
- Chapter 5 details the test setup, the specimens, the instrumentation and the testing procedure.
- Chapter 6 discusses the experimental results of each specimen.
- Chapter 7 explores the potential issues that could have led to brittle fracture, and investigation measures that were conducted to determine the cause(s) of the fractures.
- Chapter 8 provides a summary, and conclusions for the new twelve-bolt end-plate moment connection.

2. LITERATURE REVIEW

This chapter contains a summary of research on three different topics: high capacity end-plate moment connections, the use of partial joint penetration welds in moment connections, and the expected seismic behavior of full metal buildings. These three topics encompass the research presented in this document.

2.1. High Capacity End-Plate Moment Connections

Currently in AISC 358-10, AISC Design Guide 4 and Design Guide 16, the most amount of bolts at the tension flange is eight bolts. To increase the capacity of end plate moment connections, more bolts need to be added to the tension flange. Four separate known sets of testing were previously conducted to explore end plate moment connections with twelve bolts at the tension flange and sixteen bolts at the tension flange. The first set involved one specimen, which was conducted by Rodkey and Murray in 1993 at Virginia Tech. The connection consisted of four rows of bolts on the inside of the flange and one row outside the flange with two bolts in each row (Rodkey and Murray 1993). This connection can be seen below in Figure 2-1.

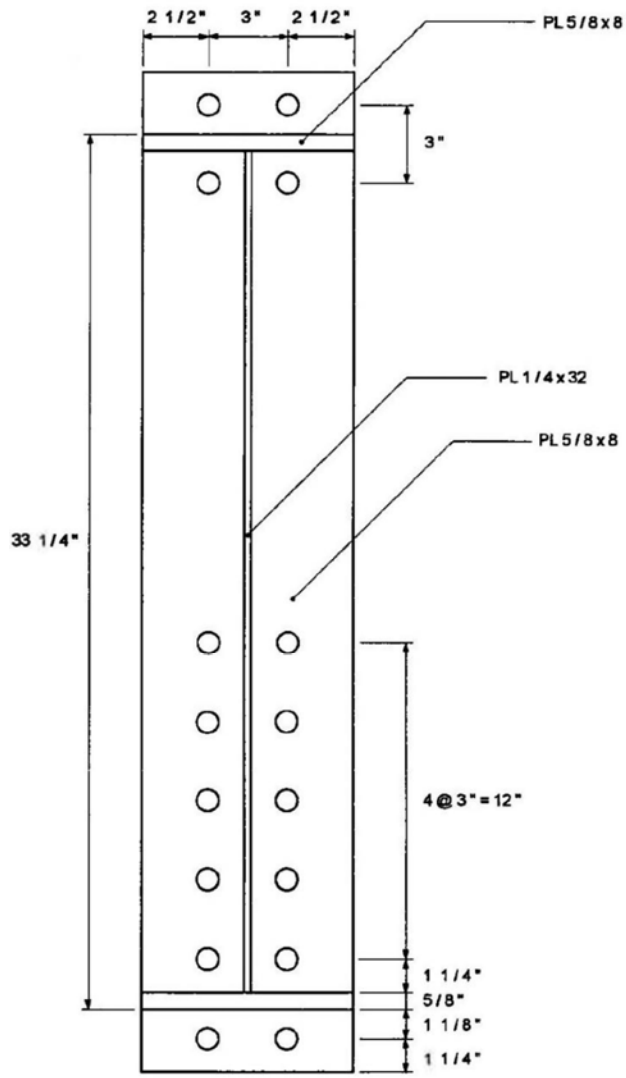


Figure 2-1 Twelve-Bolt Extended Unstiffened Connection (Rodkey and Murray 1993)

The specimen was a splice connection, which was loaded monotonically under pure moment without shear (Rodkey and Murray 1993). The failure mode was bolt tension rupture of a bolt in the row outside the flange. During the testing, bolts instrumented with strain gauges showed that the three inner most rows did not see an increase in force due to the applied moment (Rodkey and Murray 1993). This indicates that those bolt locations are not effective at resisting the applied moment. The tension force was not being well distributed to all of the bolts. The outer most bolts were taking most of the force.

The second set of testing was conducted by Sumner and Murray in 2001 at Virginia Tech. The specimens were splice connections, which were loaded monotonically under pure moment without shear (Sumner and Murray 2001). Three specimens were tested with 12 bolts at the tension flange and one specimen was tested with ten bolts at the tension flange (Sumner and Murray 2001). The twelve bolt specimens consisted of two rows of bolts inside the flange and one row outside the flange with four bolts in each row. This geometry can be seen below in Figure 2-2. The ten bolt specimen used the same geometry of bolts. However, the two outer most bolts in the inner most row were removed. One twelve bolt specimen was designed for thin end-plate behavior and the remaining specimens were designed for thick end-plate behavior (Sumner and Murray 2001).

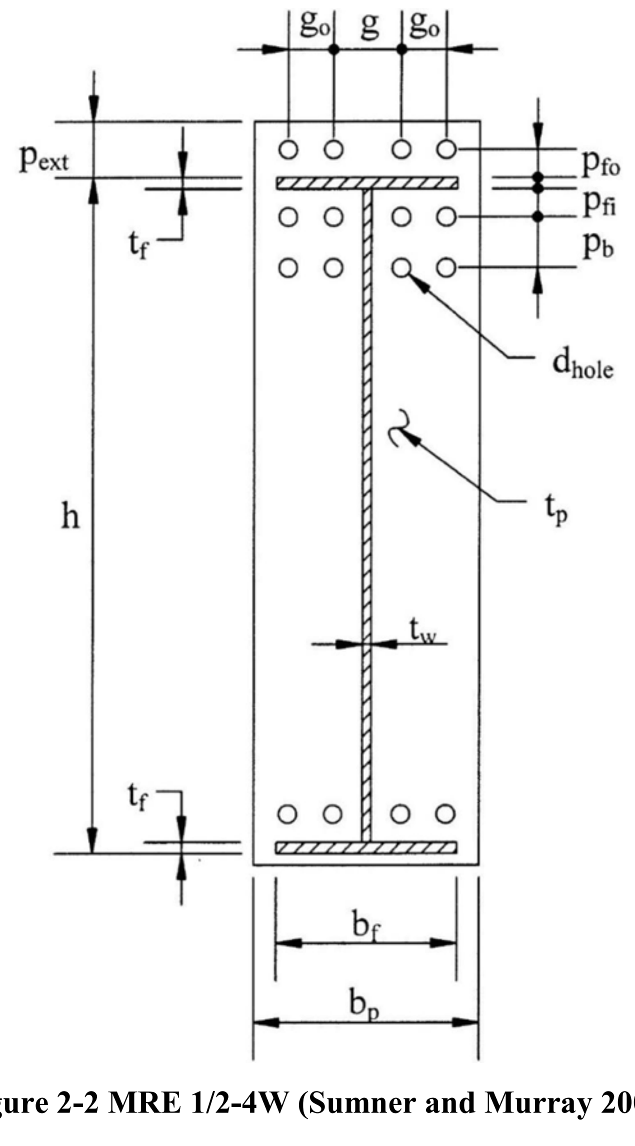


Figure 2-2 MRE 1/2-4W (Sumner and Murray 2001)

The thin end-plate specimen experienced end-plate yielding. The thick end-plate specimens failed due to bolt tension rupture and did not experience any end-plate yielding (Sumner and Murray 2001). It was assumed that the two outermost bolts in the innermost row did not contribute in the bolt force model. The ten bolt specimen was used to test this assumption. In one of the twelve-bolt thick end-plate specimens, the force in these two bolts did not increase as the applied moment increased. This shows that those two bolts are ineffective. However, the ten-bolt specimen showed approximately a 13 percent reduction in connection strength (Sumner and Murray 2001). Although the two bolts in question were ineffective, they still contributed a little to the overall connection strength.

The third set of testing was done by Schnupp and Murray in 2003 at Virginia Tech (Schnupp and Murray 2003). The testing was cyclic and involved two unsymmetrical built-up specimens, which each had 16 bolts at each flange, which consisted of rows of four bolts. The connection geometry for this testing can be seen below in Figure 2-3. The bolts were 1-1/2" A490, which were pretensioned to the minimum as specified by the AISC LRFD Specification. For the second specimen, a 1-1/4" A325 bolt had to be used in one location because the 1-1/2" A490 bolt would not fit due to fabrication tolerances (Schnupp and Murray 2003). For each specimen, four bolts at each flange were instrumented with bolt strain gauges. These locations are indicated below in Figure 2-3 by the black bolt locations. The second specimen had only three instrumented bolts at one of the flanges due to one of the instrumented bolts getting damaged.

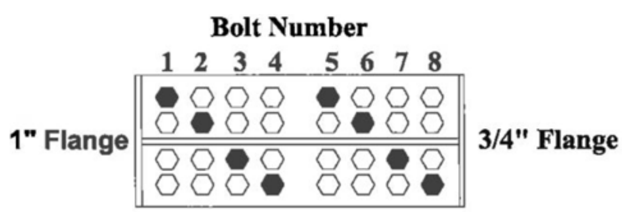
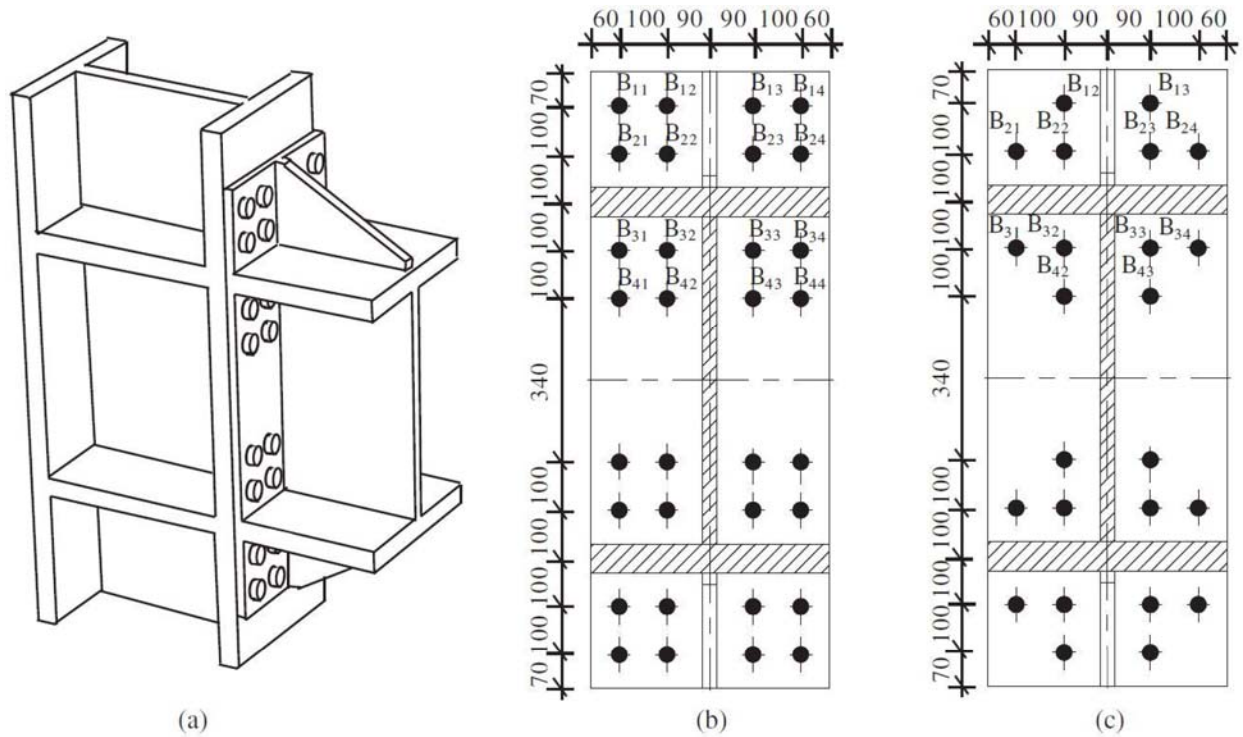


Figure 2-3 Flush 16-Bolt End-Plate Moment Connection (Schnupp and Murray 2003)

During the testing of the first specimen, the beam experienced web and flange local buckling (Schnupp and Murray 2003). The test was ended after the failure of a bolt at the intersection of the 3/4" thick flange and web. Some bolts were found to be loose and the threads on some other bolts were found to have been stripped. However, it was suspected that the stripped threads were due to the tightening process (Schnupp and Murray 2003). After inspecting the column, it was found that the weld between the column web and flange in the panel zone ruptured. A new column was fabricated with larger welds was fabricated for the second test. The connection was able to achieve over 4% story drift, and the beam was able to undergo significant levels of inelasticity (Schnuup and Murray 2003).

The testing of the second specimen resulted in failure due to web and flange local buckling (Schnupp and Murray 2003). After the first signs of local buckling, the moment capacity of the specimen decreased with each cycle. The test was concluded when the moment capacity was less than 60 percent of its ultimate moment capacity. Inspection of the bolts showed that all bolts were tight, and no threads were stripped. This specimen was able to achieve just over 3% story drift, which was significantly less than the first specimen (Schnupp and Murray 2003).

The fourth set of testing was conducted in 2016 by Gang Shi, Xuesen Chen and Dongyang Wang at Tsinghua University (Shi et al. 2017). The testing involved four extended stiffened end-plate moment connections: three 16-bolt specimens and one 12-bolt specimen. The configuration of these two types of connections can be seen below in Figure 2-4. All four of these specimens were designed for thin end-plate behavior. The three parameters of interest, which were varied between each specimen, were end-plate thickness, bolt size, and bolt layout (Shi et al. 2017). The first specimen was the base specimen, and the following specimens each differed from the first specimen by changing one of those three parameters of interest. All of the bolts were pretensioned with a torque wrench, and were instrumented with bolt strain gauges. The specimens were tested monotonically until failure in a cantilever beam-to-column setup. The beam and column for each specimen were the same section, a welded H800 x 500 x 60 x 30.



Configurations of ultra-large capacity end-plate joints: (a) sketch; (b) bolt layout A; (c) bolt layout B (units: mm).

Figure 2-4 Ultra-Large Capacity End-Plate Joint Specimens (Shi et al. 2017)

The first specimen had 16 bolts at each flange, 32 mm thick end-plate, and M30 bolts. The second specimen had a thinner end-plate, which was 25 mm thick. The third specimen had smaller bolts, which were M27. The fourth specimen had 12 bolts at each flange. All of these specimens failed in a similar fashion: end-plate yielding followed by bolt tension rupture or necking (Shi et al. 2017). The end-plates separated the most at the centerline of the stiffener and at the centerline of the flange. The deformation of the end-plate was bell-curve at these locations. The end-plate curved back to be in contact with the column flange moving away from these locations of maximum separation. End-plate separation was the greatest for the second specimen, which had the thinnest end-plate (Shi et al. 2017).

From this testing, some conclusions can be made about the contributions of the bolts to the moment capacity of the connection based on their location. Comparing specimen 1 and specimen 4 shows that the corner bolts, which were not included in the configuration for specimen 4, do not provide a significant contribution to the overall moment capacity of the connection. The moment capacities only differed by 3.7% (Shi et al. 2017). This is further supported by the bolt strain gauge data. Also based on the bolt strain gauge data, the bolts near the flange contributed the most to the overall moment capacity of the connection. Generally, the bolts outside of the flange contribute more than the corresponding bolt on the inside the flange. Also, bolts near the stiffener or the web contribute more than the bolts near the edges of the end-plate (Shi et al. 2017).

2.2. Partial Joint Penetration Welds

Currently in AISC 341-10, for all steel frames, a CJP weld is required to be used for beam flange to column welds. These welds are deemed demand critical weld, which means that they must meet certain criteria as defined in the code (AISC 2010a). PJP welds with reinforcing fillet welds can be a viable alternative to CJP welds. There has been reluctance to allow PJP welds because there are sharp notches at the roots of the welds (Kurobane 2004). These sharp notches introduce stress concentrations and could lead to a non-ductile failure. However, recent research has shown otherwise.

In 2009, Chen and Wang conducted 30 extended end-plate moment connection tests. Twenty-four of which were monotonically loaded and the remaining specimens were cyclically loaded. For the beam flange to end-plate welds, some specimens used CJP groove welds, some used only fillet welds, and some used different variations of combined PJP and fillet welds (Chen and Wang 2009). Some of the PJP welded specimens were oriented on a gradient, meaning that the beam was not perpendicular to the end-plate. The different configurations of flange-to-end-plate welds can be seen below in Figure 2-5. After all of the testing, none of the beam flange to end-plate welds failed under either type of loading and the predominant failure mode was local buckling of the beams (Chen and Wang 2009). It was also found that the gradient did not have a noticeable effect on the performance of the welds. However, after a finite element analysis parametric study, thin end-plates and end-plate with a greater bolt gauge were shown to deteriorate the resistance of the welds (Chen and Wang 2009). Overall, the testing shows that any type of weld can be designed on an equal capacity principle and meet satisfactory performance criteria.

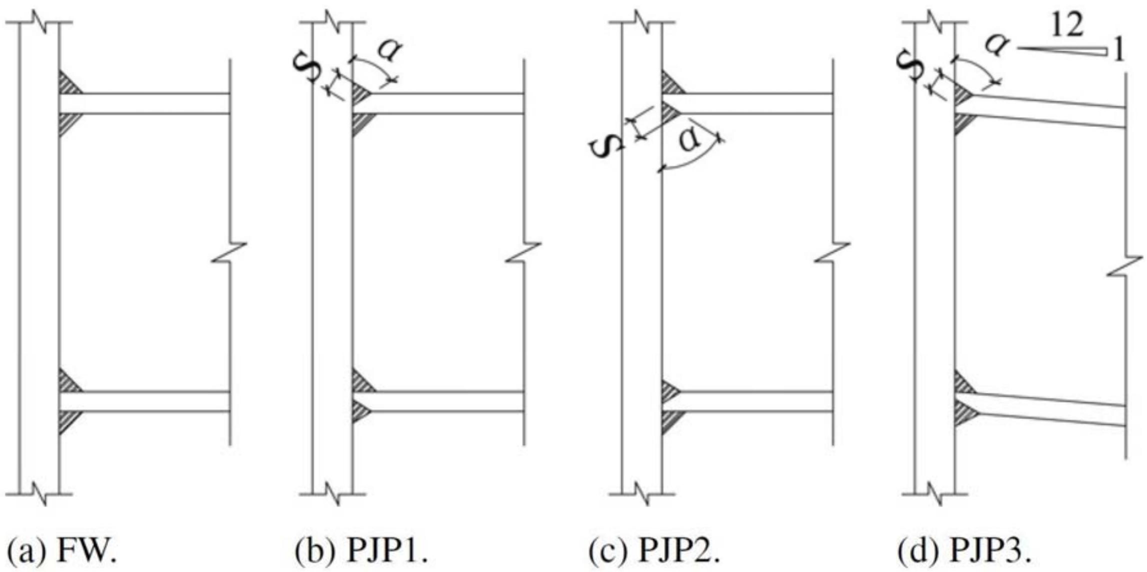


Figure 2-5 Non-Complete Penetration Flange-to-End-Plate Welds (Chen and Wang 2009)

Kurobane and Azuma examined the applicability of PJP groove welds in beam-to-column connections under seismic loading in 2004. The beams were welded directly to the column flange. The beam flange welds consisted of a single-sided 45 degree PJP groove weld with weld built-up on top of the PJP weld and a fillet weld below the flange (Kurobane and Azuma 2004). The connection geometry is displayed below in Figure 2-6. The testing consisted of four full-sized specimens. The cantilever beams were loaded to form a plastic hinge at the end of the beam near the connection (Kurobane and Azuma 2004). All of the connections showed sufficient capacity, except for one connection which failed due to a lack of penetration (Kurobane and Azuma 2004). Although ductile cracks initiated at the weld toes at the edges of the flanges, it was determined that brittle fracture was unlikely to start from the roots of the PJP welds (Kurobane and Azuma 2004). Criteria found to be important were joint detailing, welding position and other welding conditions (Kurobane and Azuma 2004).

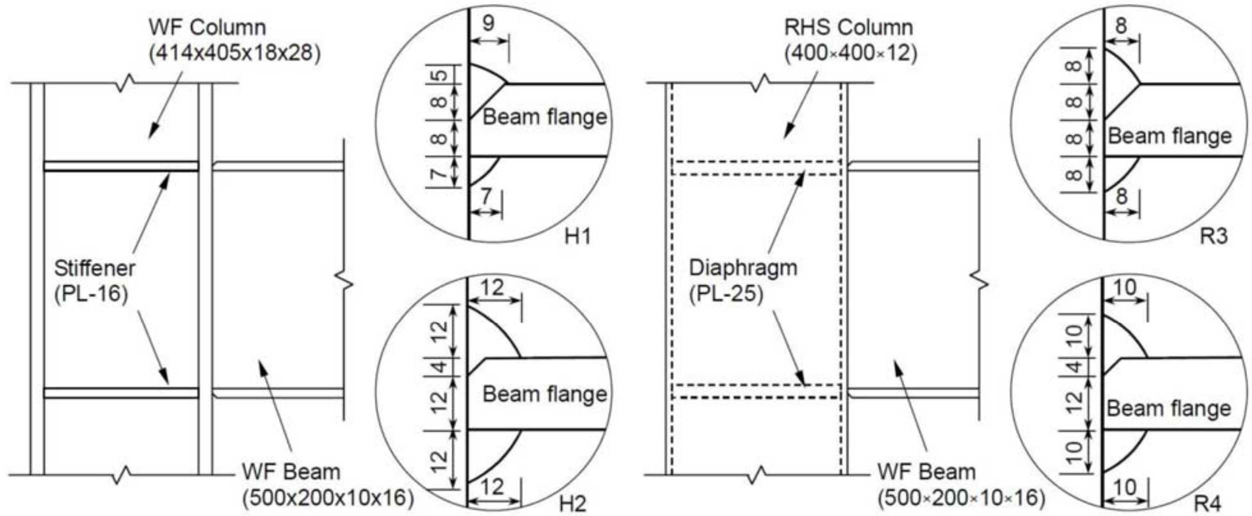


Figure 2-6 Beam-to-Column Test Connection (Kurobane and Azuma 2004)

In 2016, similar testing was done at the University of Oklahoma by Sherry (Sherry 2016). Two end-plate moment connections were tested under cyclic loading as a beam-to-column connection (Sherry 2016). These two connections used built-up PJP welds for the beam flange to end-plate welds, which includes fillet welds inside and outside the flange. In the end, the beams reach over 85% of their plastic capacity during testing and the welds did not fail (Sherry 2016). After testing, the connections were taken apart to measure the actual size of the welds and the actual penetration. It was found that there was a significant amount of lack of penetration in the flange welds. Even with this lack of penetration, it was determined that these modified PJP weld met all of the criteria to prequalify them for intermediate moment frames (Sherry 2016).

Similar to end-plate moment connections is column baseplates. Both connections are bolted end-plate connections that experience moment and shear. Since there was a lack of data on the fracture resistance of column baseplate connection welds, six 2/3-scale column baseplate specimens were tested by Myers et al. in 2008. The specimens were loaded cyclically under general protocol and near-fault protocol (Myers et al. 2008). Four specimens were detailed with CJP welds at the flanges, and two specimens with PJP welds and reinforcing fillet welds at the flanges (Myers et al. 2008). The CJP specimens included weld access holes, whereas the PJP specimens did not include weld access holes. The two types of connection details can be seen below in Figure 2-7. The specimens were loaded until the specimen failed. The PJP specimens were more ductile and sustained higher drifts; 8%-9% drift for the PJP specimens compared to

5%-6% drift for the CJP specimens (Myers et al. 2008). The performance of all of the connections exceeded the expected drift demands for a maximum considered earthquake, which is 4%-5% drift (Myers et al. 2008).

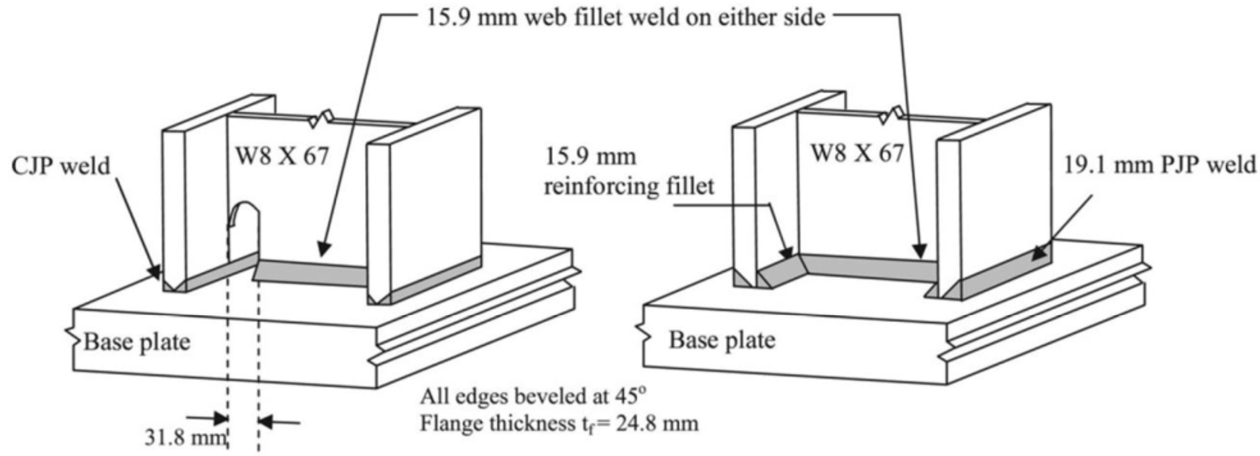


Figure 2-7 Column Baseplate Connections (Myers et al. 2008)

Another type of moment connection is a column splice, which is also required by AISC 341-10 to use CJP welds at the flanges. Although column splices are a different type of moment connection than end-plate moment connections, the welds in both types of connections experience moment and shear. However, the demands are typically less in column splices compared to beam-column connections (Shaw 2013). In 2013, five full-scale column spice tests were conducted at the University of California Davis by Shaw. These specimens were connected with an erection plate and welded together with PJP welds at the flanges (Shaw et al. 2015). The columns were loaded in a series of cycles about their strong axis. After the cyclic loading portion, the specimens were loaded monotonically until failure (Shaw et al. 2015). The test setup can be seen in Figure 2-8 below. All specimens were able to complete the cyclic loading portion and continue on to the monotonic loading portion (Shaw et al. 2015). All specimens performed very well. However, some factors could not be taken into account, such as buildings taller than 20 stories, near ground fault motions, vertical accelerations and bidirectional shaking (Shaw et al. 2015). Despite this, the study was considered to be conservative compared to field conditions (Shaw et al. 2015).

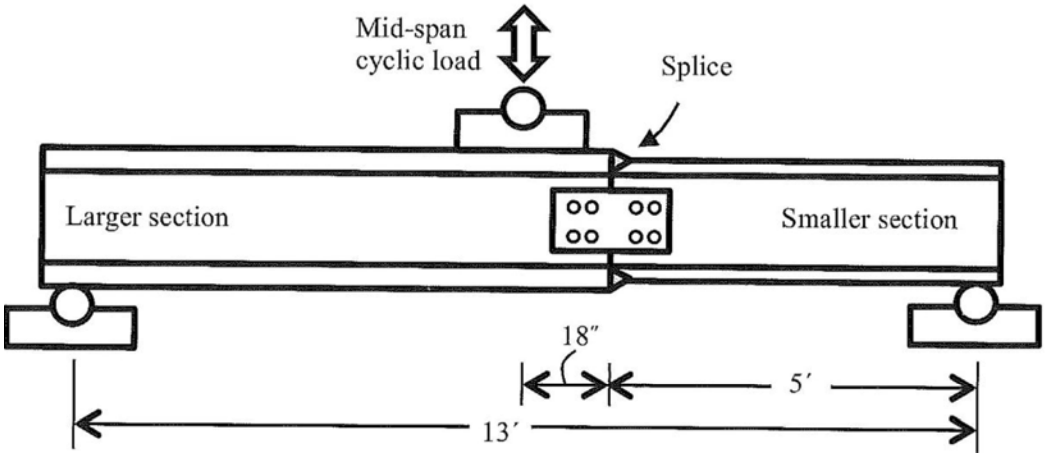


Figure 2-8 Column Splice Test Setup (Shaw et al. 2015)

2.3. Expected Seismic Behavior of Full Metal Buildings

All of the tests previously discussed only explore the behavior of two isolated members at a particular connection. However, at the University of California San Diego, three full-scale metal building systems were tested on a shake table to simulate earthquake events (Smith and Uang 2010). The first specimen consisted of a single story metal building system (MBS) frame with metal side wall panels, which represents the majority of MBS buildings (Smith and Uang 2010). The second specimen had heavy concrete sidewall panels added. Both of these specimens used built-up non-compact or slender sections (Smith and Uang 2010). The third specimen was a MBS frame with a mezzanine attached to one side and a concrete wall attached to the other (Smith and Uang). The third specimen was built with sections with compact flanges, unlike the first two specimens. The test setup can be seen below in Figure 2-9.

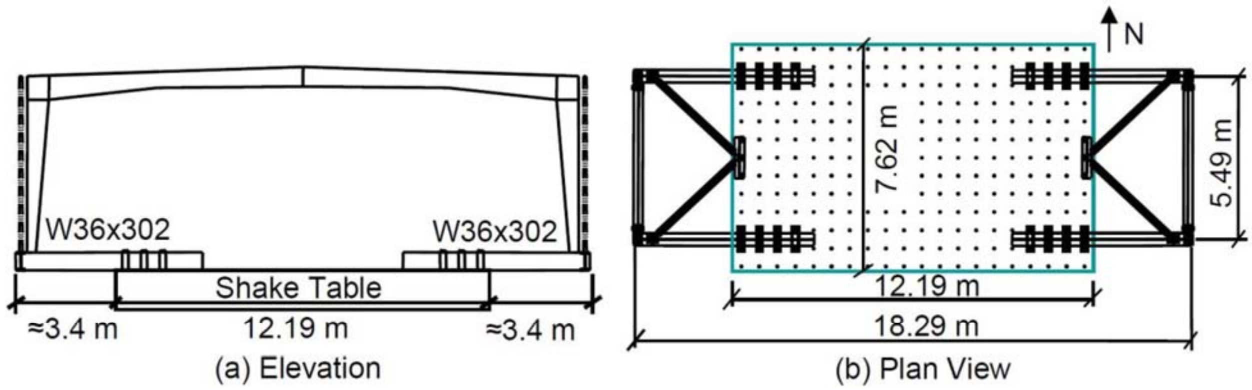


Figure 2-9 MBS Test Setup

Each specimen was tested with white noise and impulse motions to determine dynamic characteristics of the system (Smith and Uang 2010). Five different ground motions were selected from FEMA P695, which were 1979 Imperial Valley, 1989 Loma Prieta, 1992 Landers, 1994 Northridge and 199 Chi-Chi (Smith and Uang 2010). Each of these were scaled to a low percentage of the Design Basis Earthquake (DBE) for each specimen. For each specimen, only Imperial Valley was gradually scaled up until failure was reached (Smith and Uang 2010). Each specimen were well instrumented with over 300 channels for data acquisition, which included strain gauges, rosettes, displacement transducers, string potentiometers and accelerometers (Smith and Uang 2010).

The first specimen performed very well and showed a large system overstrength. Although the system was not very ductile, the system was able to handle a base shear of about 290% of the DBE without damage (Smith and Uang 2010). Lateral torsional buckling was observed in some of the rafter, and there was indication of low cycle fatigue on the account of rupturing of a bottom flange (Smith and Uang 2010). The second and third specimens did not perform as well as the first specimen. Specimen two also saw lateral torsional buckling in the rafters (Smith and Uang 2010). This specimen did not exhibit ductile behavior, but had significant overstrength. However, it was not as much overstrength as the first test (Smith and Uang 2010). The third specimen behaved differently than the other two specimens. The webs of the panel zones eventually tore due to elastic shear buckling and low cycle fatigue (Smith and Uang 2010). After this damage, the specimen produced larger hysteresis loops. The system remained elastic throughout the testing and only experienced damage in the panel zones (Smith and Uang 2010). This specimen exhibited the most amount of energy dissipation, but showed less overstrength than the first specimen (Smith and Uang 2010). These tests showed that systems with more mass are designed with a lower factor of safety. The safety factors for specimens two and three were both below 1.00. All specimens remained standing at the end of the testing, however, more studies are recommended to develop better design criteria (Smith and Uang 2010).

3. YIELD LINE ANALYSIS

Yield line analysis is a method used to determine the plastic flexural failure mechanism and capacity at the ultimate limit state. It is commonly applied to steel connections and reinforced concrete two-way slabs. The moment strength associated with end-plate flexure in an end-plate moment connection is the item of interest in this instance. Generally, there are two different methods for yield line analysis: an upper bound which applies kinematics, and a lower bound which applies equilibrium. The upper bound approach was applied in this situation. The lower bound approach is difficult to apply to end-plate moment connections because there are so many facets and equilibrium must be satisfied for each facet. The upper bound virtual work approach is commonly used for steel connections.

3.1. Assumptions

Since yield line analysis can become immensely complex as the number of faces and the number of yield lines increases, it is important to make some assumptions. It is also important state them ahead of time in order to stay consistent between analyses. The assumptions that were made for this set of yield line analyses were as follows:

- Yield lines meet at the center of bolt holes.
- Yield lines are straight, not curved.
- The center of the bolt holes are points of zero displacement.
- Gross cross-sectional area is used instead of net cross-sectional area.
- The panels do not experience additional deformation while the yield lines undergo plastic rotation.
- The inelasticity is concentrated in the yield lines.
- The rotation angles are small.
- The axial forces in the beam are small.
- The yield lines meet at the center of the web (thickness of the web is neglected).

3.2. Yield Line Mechanism Cases

Six different yield line mechanisms were analyzed for this particular end-plate moment connection. Each yield line mechanism, Case 1 through Case 6, is depicted respectively in the following figures below: Figure 3-1, Figure 3-2, Figure 3-3, Figure 3-4, Figure 3-5 and Figure 3-6. The dashed lines represent yield lines, and the circled numbers are the labels for each panel. Case 1 and Case 2 use the same yield line pattern, if the 4 outer bolts are removed, as the two yield line mechanisms listed in AISC 358-10 for the 8-bolt extended stiffened end-plate moment connection. Yield line mechanisms that are valid for that end-plate connection also apply to this new 12-bolt end-plate moment connection. The derivation of the yield line for Case 6 is shown in Appendix N. The derivation of the yield line for Case 3 is shown in Appendix L, and the derivation for Case 5 is in Appendix M.

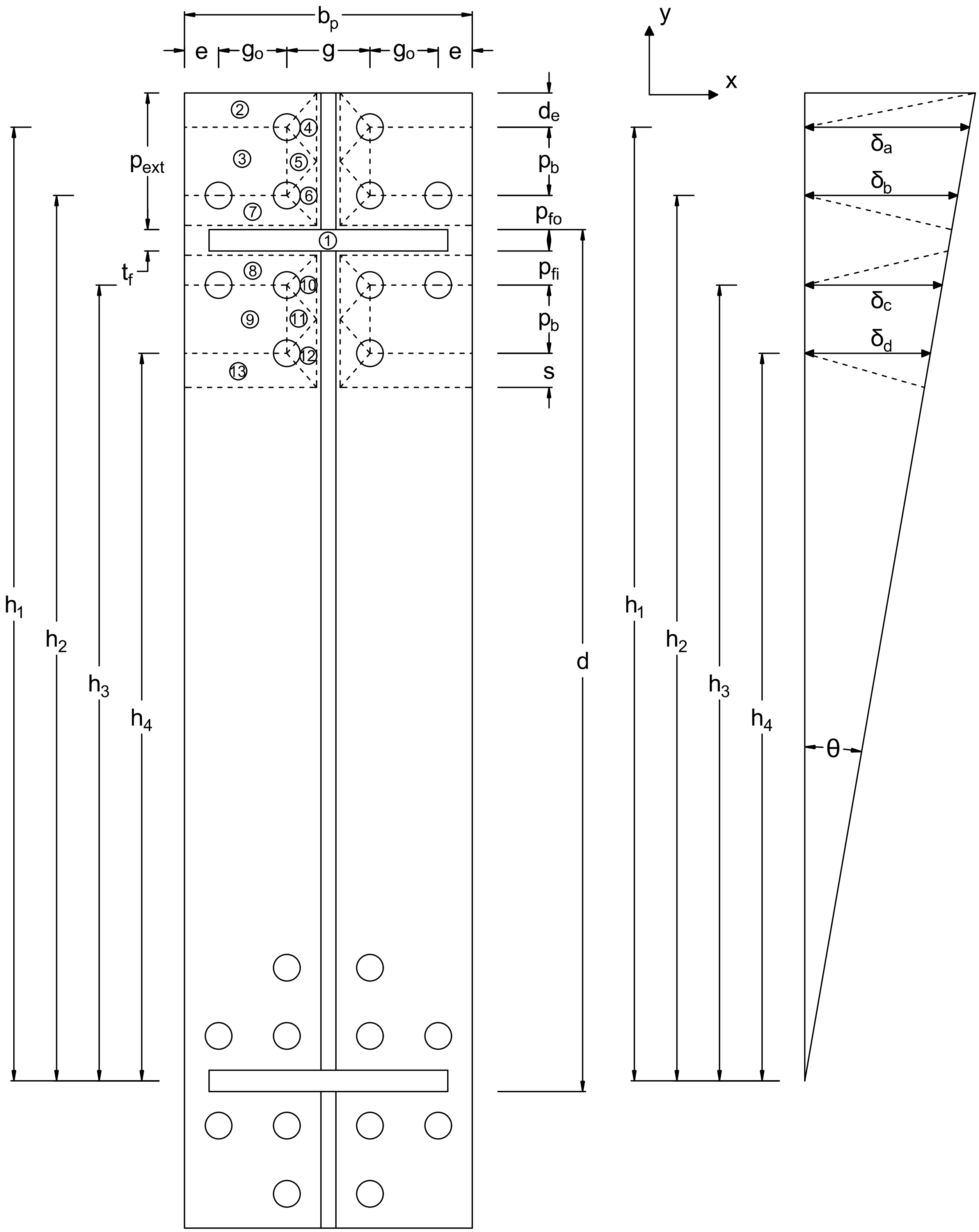


Figure 3-1 Yield Line Mechanism: Case 1

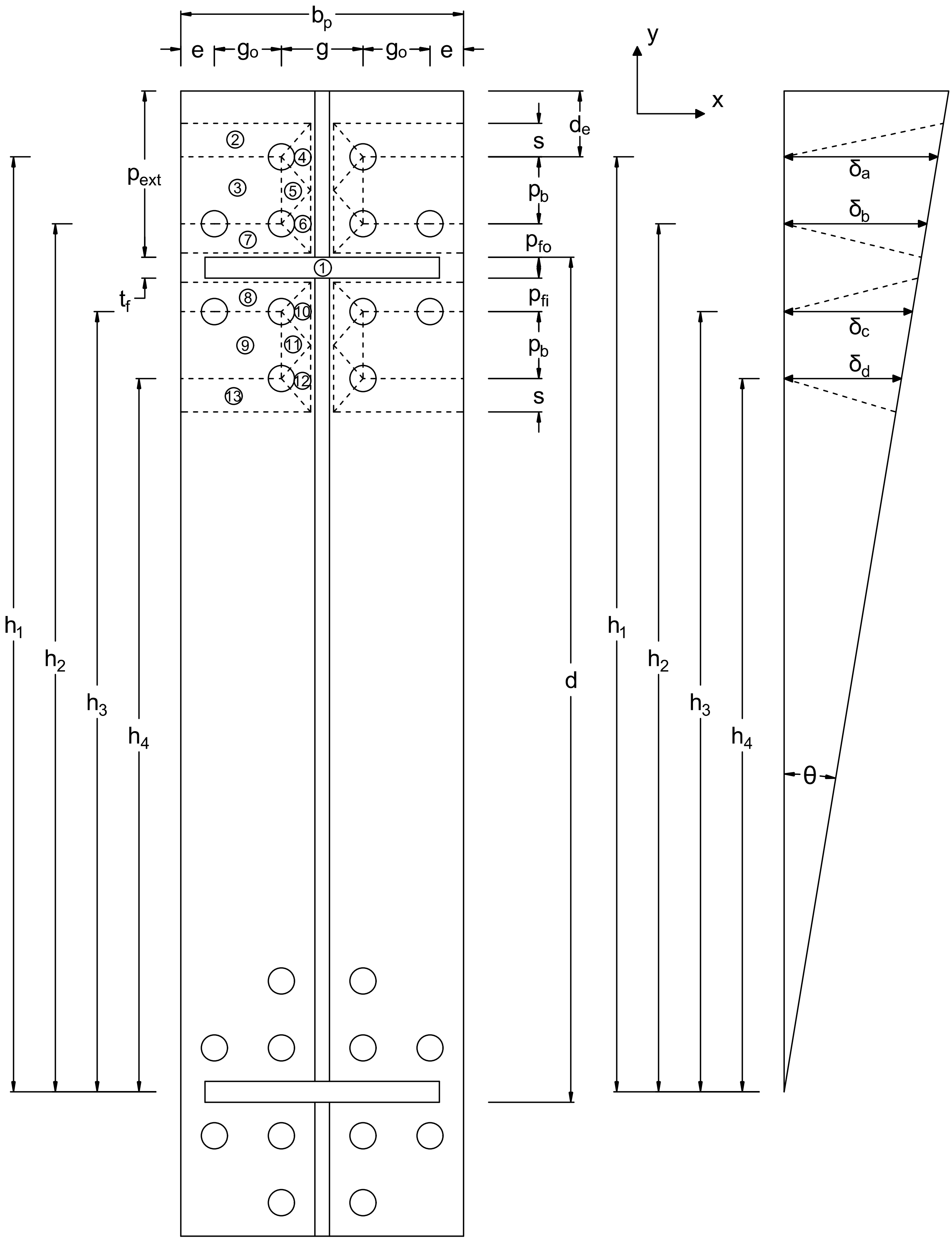


Figure 3-2 Yield Line Mechanism: Case 2

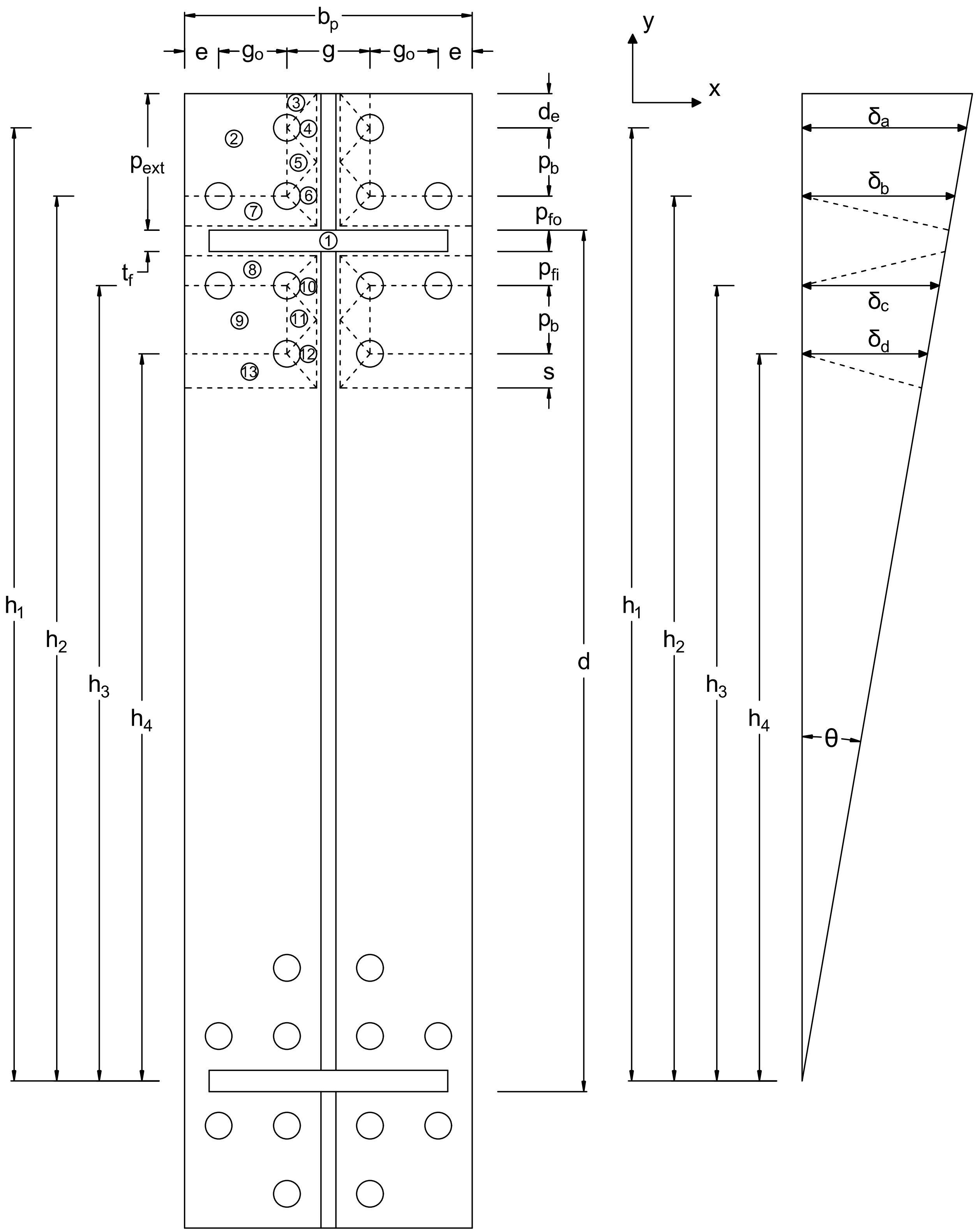


Figure 3-3 Yield Line Mechanism: Case 3

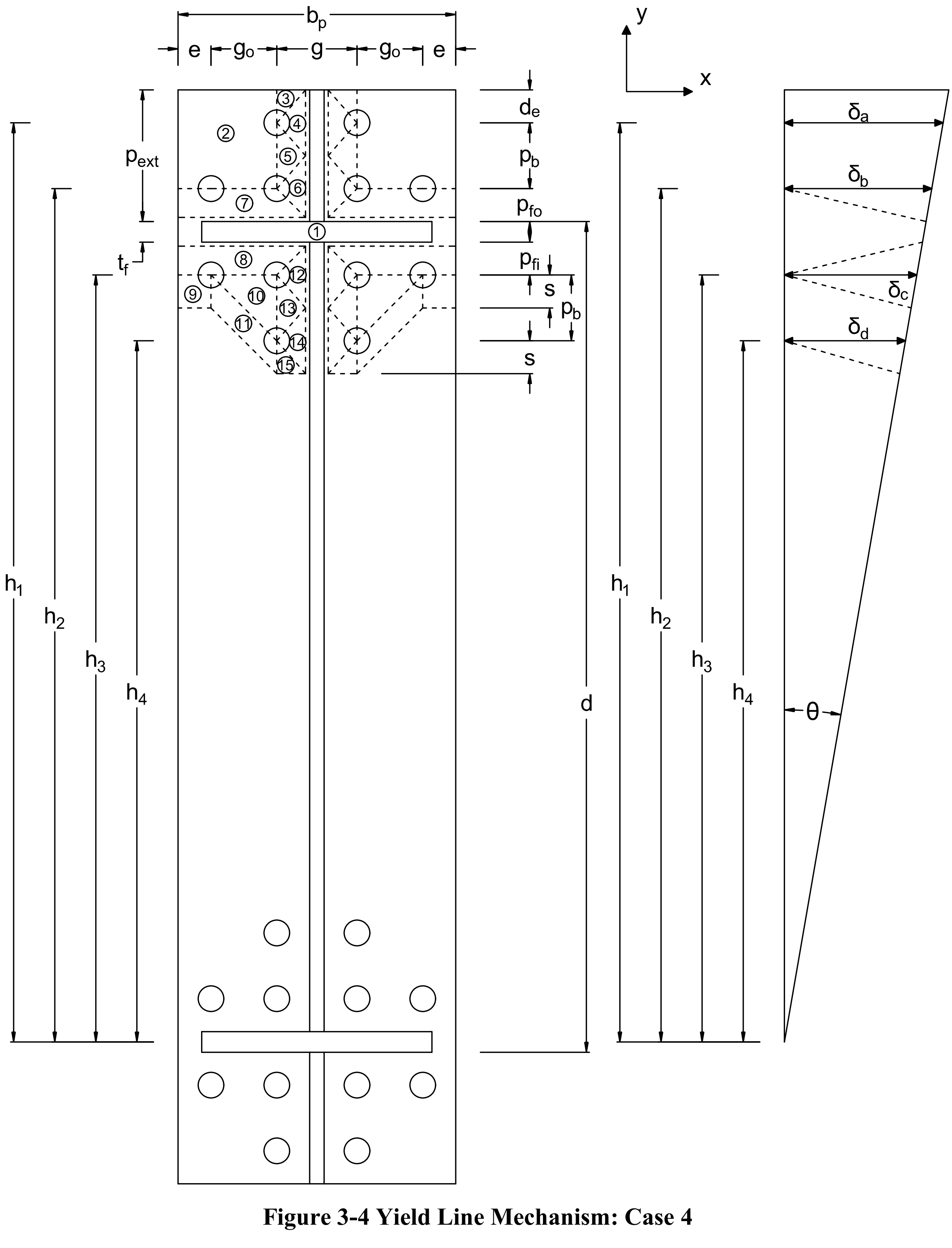


Figure 3-4 Yield Line Mechanism: Case 4

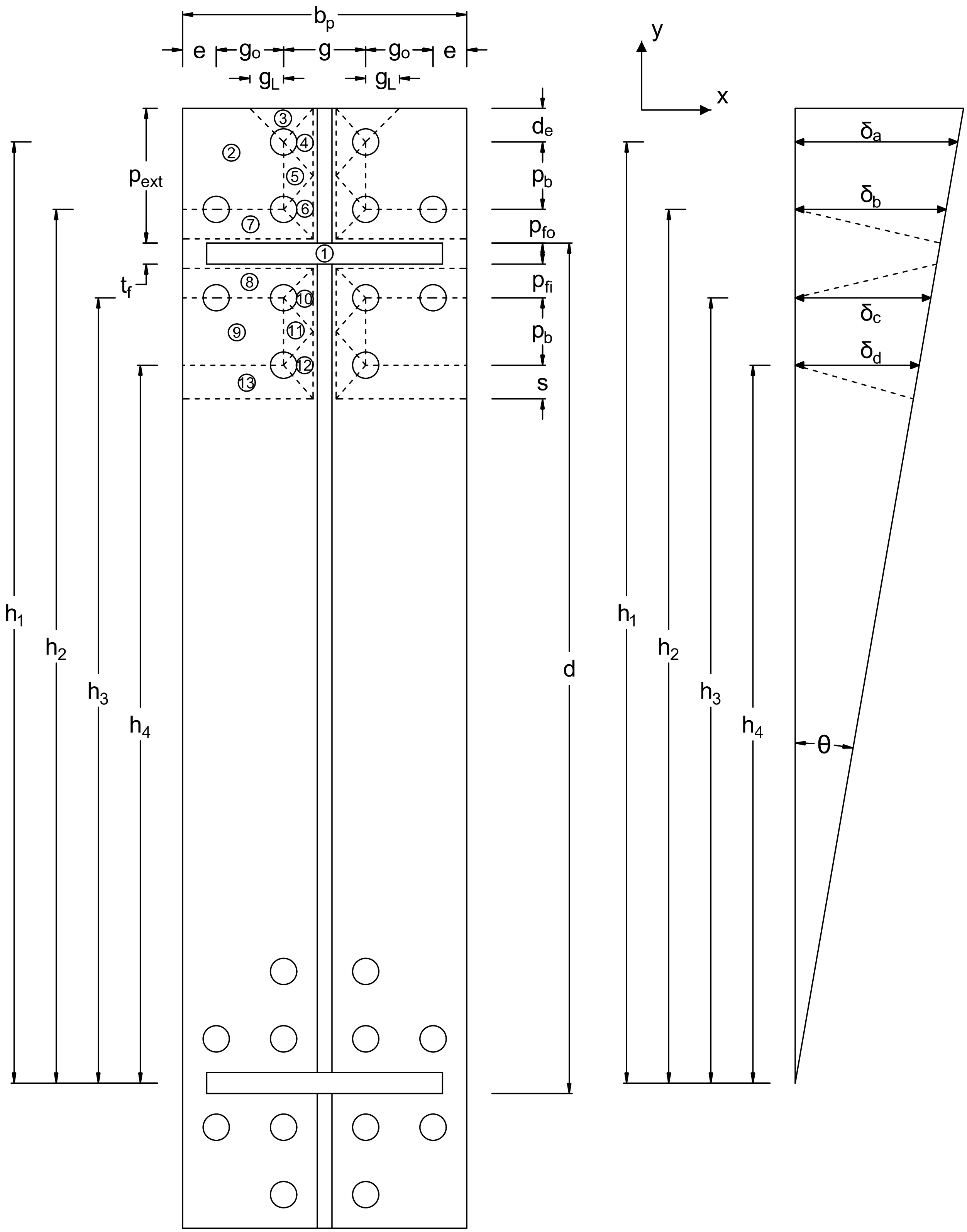


Figure 3-5 Yield Line Mechanism: Case 5

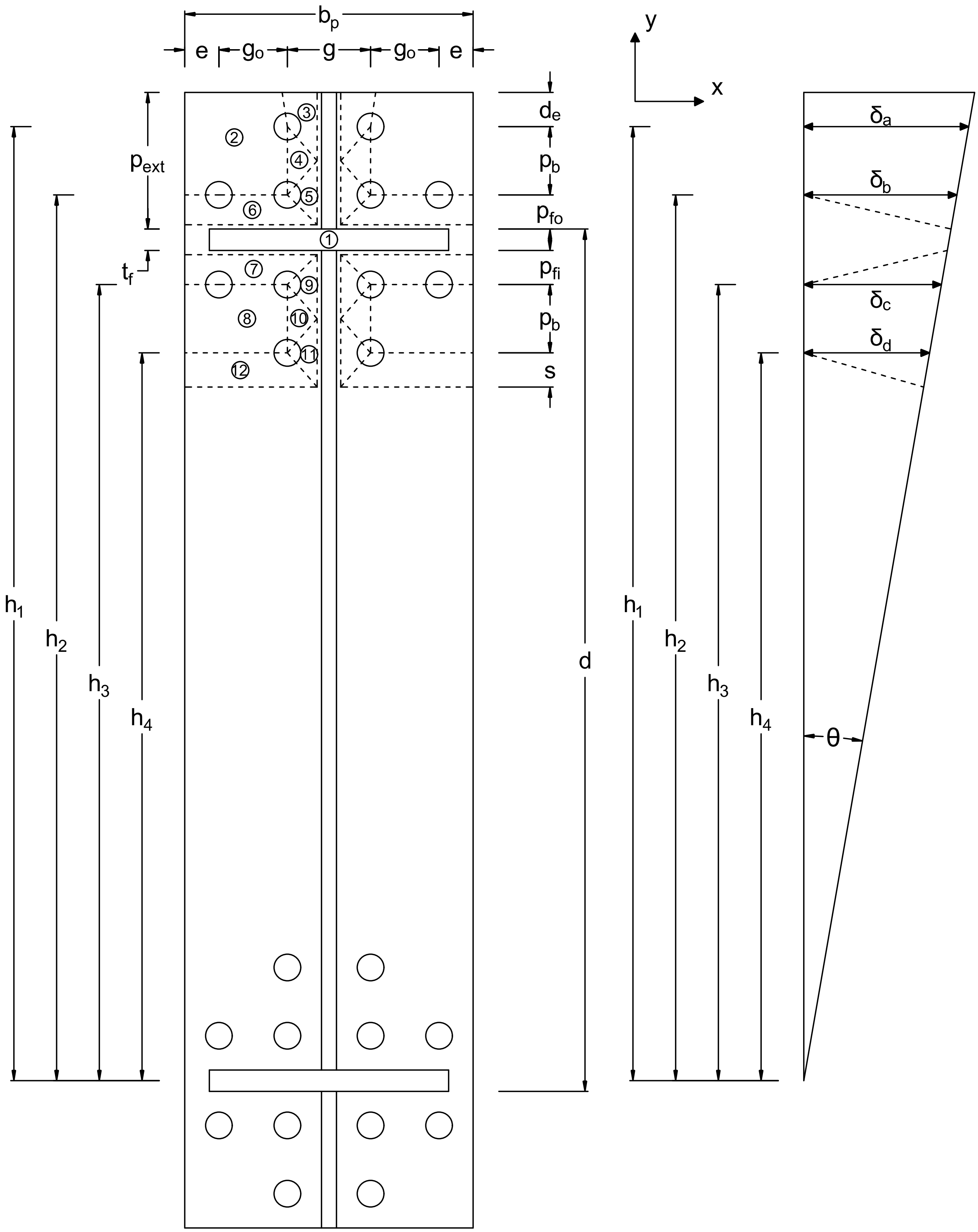


Figure 3-6 Yield Line Mechanism: Case 6

3.3. Determination of Controlling Yield Line Mechanism

All six of these yield line mechanism cases are similar. Case 1, Case 2, Case 3, Case 5, and Case 6 all assume the same yield line pattern for inside of the flange. Case 4 assumes the same yield line pattern as Case 3 outside the flange, but a different yield line mechanism inside the flange. By inspection, Case 4 was determined to not be the controlling case. The yield line pattern inside the flange of Case 4 can be directly compared to that of Case 3. In Figure 3-4, since the yield lines between panel 10, panel 11 and panel 1 are at an angle with respect to the x-axis, Case 4 has a longer length of yield lines than Case 3. This also means that those panels develop rotations about the x-axis and y-axis. Those similar yield lines in Case 3 are parallel to the x-axis, which means rotations only about the x-axis. Case 4 would result in greater stored energy, which leads to a greater yield line parameter, Y, and thus a greater moment strength associated with end-plate flexure As a result, Case 4 was not examined further like the other cases.

Since Case 1, Case 2, Case 3, Case 5, and Case 6 all assume the same yield line mechanism for inside of the flange, the area of concern for comparison is outside the flange. Even within the region of outside the flange for these cases, the difference between them lies within the yield lines between panels 2, 3 and 4. Since each of the yield lines between these panels are related to the dimension 'd_e', each yield line parameter, Y, can be compared by varying 'd_e'. A plot of the yield line parameter for each case based on the dimensions of Specimen 12ES-1.25-1.50-44 is shown below in Figure 3-7. A plot of the yield line parameter for each case based on the dimensions of Specimen 12ES-1.125-1.25-24 is shown below in Figure 3-8. More information about these specimens can be found below in Chapter 5.

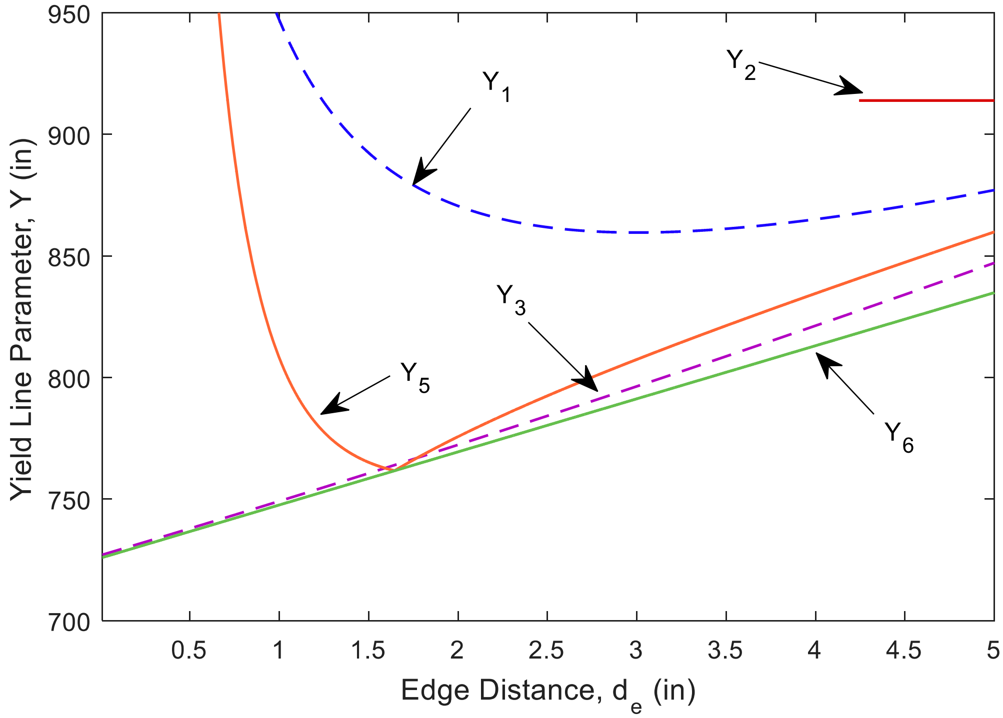


Figure 3-7 Comparison of Yield Line Parameters Based on Specimen 12ES-1.25-1.50-44

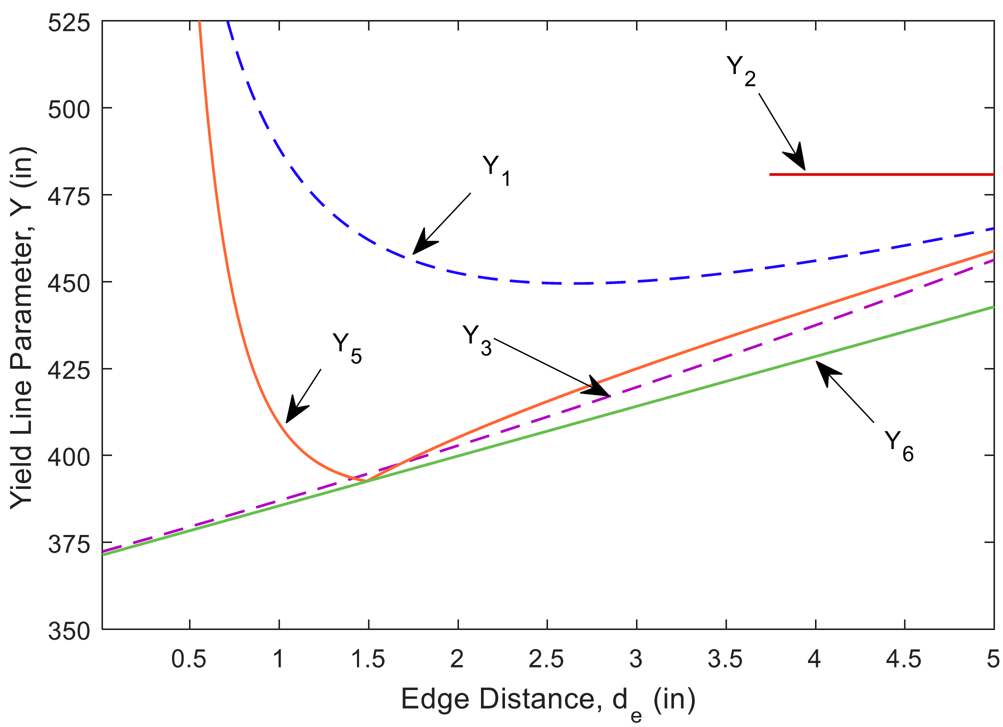


Figure 3-8 Comparison of Yield Line Parameters Based on Specimen 12ES-1.125-1.25-24

Both Figure 3-7 and Figure 3-8 show the shift in the yield line parameters between a deep beam and a shallow beam respectively. Figure 3-7 is based on a deep beam which uses larger bolts than the shallow beam, on which Figure 3-8 is based. The difference in bolt sizes had the largest influence on the bolt spacing dimensions, which also affect the yield line parameters. Since the yield line analysis that was applied is an upper bound method, the yield line mechanism that controls is the one that results in the smallest yield line parameter, Y. Looking at both Figure 3-7 and Figure 3-8, Case 5 is less than Case 3 over a very brief segment between 'd_e' of 1" and 2". However, this is across a negligible range, and Case 5 is negligibly less than Case 3. Case 2 starts controlling at a "d_e" value much greater than the range shown. However, based on the minimum edge distance for different bolt diameters as prescribed in AISC 360-10 Specification for Structural Steel Buildings, which is shown below in Table 3-1, this edge distance where Case 2 start to control is outside the range of typical edge distances (AISC 2010b).

Table 3-1 Minimum Edge Distance (AISC 360-10 Table J3.4)

Minimum Edge Distance from Center of Standard Hole to Edge of Connected Part (in)	
Bolt Diameter (in)	**Minimum Edge Distance**
1/2	3/4
5/8	7/8
3/4	1
7/8	1-1/8
1	1-1/4
1-1/8	1-1/2
1-1/4	1-5/8
Over 1-1/4	1-1/4 x d

In Figure 3-3, the yield line between panel 3 and panel 4 of Case 3 was removed to create the yield line pattern of Case 6, as shown in Figure 3-6. This is a simplification. In Case 6, it was assumed that the yield line between panel 3 and panel 2 would splay outward in the x-direction. The yield line would splay outward enough to cause the rotation about the x-axis in panel 3 to be equal to the rotation about the x-axis in panel 4. Thus, the yield line that was removed would never form. In Figure 3-6, the yield line between panel 2 and panel 3 is shown splayed outward.

However, for the derivation of Case 6, the yield line between panel 3 and panel 2 was assumed to remain parallel to the y-axis for ease of calculations. If the yield line was at an angle to the y-axis, then the yield line would have a stored energy component due to rotation about the x-axis and due to rotation about the y-axis. By assuming that the yield line is parallel to the y-axis, the stored energy component due to rotation about the x-axis is eliminated. Panel 3 was still assumed to remain a plane. In both Figure 3-7 and Figure 3-8, Case 6 is the minimum across the entire range shown, which means that Case 6 is the controlling yield line mechanism.

4. PROPOSED DESIGN PROCEDURE

4.1. Yield Line Mechanism

The process for how the controlling yield line mechanism was determined was described in Chapter 3. In the end, one yield line mechanism was established to be the controlling case for all geometries, with dimensions in the typical range, of this new twelve-bolt end-plate moment connection configuration. Since these yield line patterns were similar, the dimension of interest was for the edge distance, 'd_e'. The typical range for this was 3/4" to 2-1/2". The controlling yield line pattern produced the lowest capacity across that range for both a 24" deep beam and a 44" deep beam. The derivation of the controlling yield line mechanism can be found in Appendix N. The moment strength associated with end-plate flexure in an end-plate moment connection is directly proportional to the yield line parameter, Y.

$$Y = \frac{b_p}{2}\left[\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2}\right] + \frac{1}{2g}\left[h_1(4d_e + 3p_b) + h_2(p_b + 4p_{fo}) + h_3(4p_{fi} + 3p_b) + h_4(p_b + 4s)\right] + \frac{5}{4}g \quad \text{(Eq. 4.1)}$$

$$\phi M_{pl} = \phi F_{py} t_p^2 Y \quad \text{(Eq. 4.2)}$$

$$\phi = 0.9$$

M_{pl} is the moment strength associated with end-plate flexure.

4.2. Bolt Force Model

4.2.1. Thick Plate Model

The thick plate model assumes that the end-plate is thick enough that the end-plate does not deform, each bolt is in direct tension, and each bolt reaches full strength simultaneously. The moment capacity associated with bolt tension rupture without prying action is the sum of the strength of each bolt times its respective lever arm.

$$\phi M_{np} = \phi[2P_t(h_1 + 2h_2 + 2h_3 + h_4)] \qquad \text{(Eq. 4.3)}$$

$$\phi = 0.75$$

P_t is the bolt nominal tensile capacity.

4.2.2. Thin Plate Model

The thin plate model assumes that the end-plate is thin enough that the end-plate yields before the bolts rupture. This leads to deformations in the end-plate, which cause prying forces on the bolts. These prying forces induce a moment in the bolts, which reduces the tensile capacity of the bolts. The prying forces are based on modeling the end-plate as the flange of an effective tee section. The flange of the effective tee section for each bolt position is shown below in Figure 4-1.

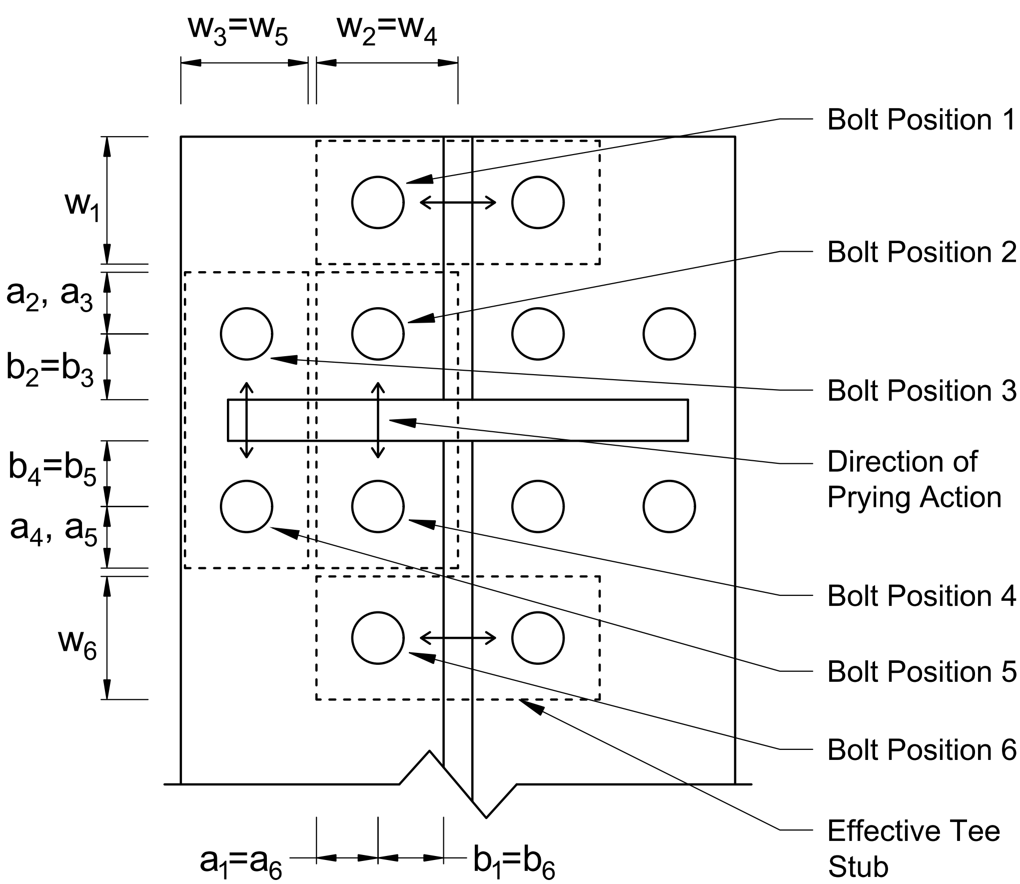


Figure 4-1 Prying Action of Effective Tee Stubs

The maximum calculated distance from the bolt to the edge of effective tee stub is calculated below in Eq. 4.4.

$$a_{calc} = 3.62\left(\frac{t_p}{d_b}\right)^3 - 0.085 \qquad \text{(Eq. 4.4)}$$

t_p is the end-plate thickness.
d_b is the bolt diameter.

For each bolt position, i, the dimensions of the effective tee stub section can be calculated based on the equations presented below, which are based on the procedure presented in the AISC Steel Design Guide Series 16 (Murray and Shoemaker 2002). w_i is the effective width of the end-plate that is acting in the tee stub model. The bolt hole is taken into account with w_i', which is the effective net width of the end-plate. a_i is the distance from the bolt to the reaction force. b_i is the distance from the bolt to the tee stem. The equation for maximum prying force, $Q_{max,i}$, includes a radical term. In the radical, the end-plate yield stress is reduced by the average shear stress calculated using the bolt force associated with the plastic mechanism, F_i'. If the term in the radical is negative, then the end-plate geometry is not appropriate for thin end-plate behavior. The maximum bolt force, P_{qi}, is based on the maximum of the bolt rupture strength factoring in prying, and the bolt pretension force, T_b, because if the bolt pretension is larger than the bolt rupture strength

Table 4-1 Equations for Effective Tee Stub Dimensions

Bolt Position, i	w_i	a_i	b_i
1	$d_e + \frac{p_b}{2}$	$min\left(a_{calc}, \frac{b_p - g}{2}\right)$	$\frac{g - t_w}{2}$
2	$\frac{g + g_o}{2}$	$\min(a_{calc}, p_b)$	p_{fo}
3	$\frac{b_p - g - g_o}{2}$	$\min(a_{calc}, p_b + d_e)$	p_{fo}
4	$\frac{g + g_o}{2}$	$\min(a_{calc}, p_b)$	p_{fi}
5	$\frac{b_p - g - g_o}{2}$	a_{calc}	p_{fi}
6	$\frac{p_b}{2} + s$	$min\left(a_{calc}, \frac{b_p - g}{2}\right)$	$\frac{g - t_w}{2}$

$$s = \frac{1}{2}\sqrt{b_p g} \qquad \text{(Eq. 4.5)}$$

$$w_i' = w_i - \left(d_b + \frac{1}{16}\right) \qquad \text{(Eq. 4.6)}$$

$$F_i' = \frac{1}{b_i}\left[\frac{t_p^2 F_{py}}{4}(0.85 w_i + 0.8 w_i') + \frac{\pi d_b^3 F_{nt}}{32}\right] \qquad \text{(Eq. 4.7)}$$

$$Q_{max,i} = \frac{w_i' t_p^2}{4 a_i}\sqrt{F_{py}^2 - 3\left(\frac{F_i'}{w_i' t_p}\right)^2} \qquad \text{(Eq. 4.8)}$$

$$P_{qi} = max\{P_t - Q_{max,i},\ T_b\} \qquad \text{(Eq. 4.9)}$$

F_{py} is the yield strength of the end-plate material.

F_{nt} is the bolt nominal tensile yield stress.

T_b is the bolt pretension force.

The flexural strength for bolt tension rupture with prying action, M_q, is given by Eq. 4.10. In this equation, the subscript for the dimension "h" only goes to four because there are four rows of bolts, whereas the subscripts related to bolt location go to six because there are six bolt locations. Some bolt locations do not achieve their full bolt tension rupture strength before the first bolt fractures. To account for this, a reduction factor, α_i, is applied. These reduction factors are α_1=1.0, α_2=1.0, α_3=0.5, α_4=1.0, α_5=0.75, and α_6=1.0

$$\phi M_q = \phi \sum_{i=1}^{6} \alpha_i n_i P_{qi} h_j$$

$$= \phi\left[2P_{q1}h_1 + 2P_{q2}h_2 + 2(0.5)P_{q3}h_2 + 2P_{q4}h_3 + 2(0.75)P_{q5}h_3 + 2P_{q6}h_4\right]$$

$$= \phi\left[2P_{q1}h_1 + 2P_{q2}h_2 + P_{q3}h_2 + 2P_{q4}h_3 + 1.5P_{q5}h_3 + 2P_{q6}h_4\right] \qquad \text{(Eq. 4.10)}$$

$$\phi = 0.75$$

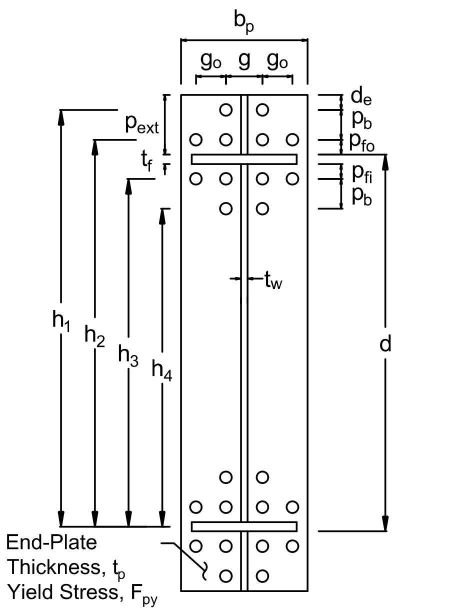
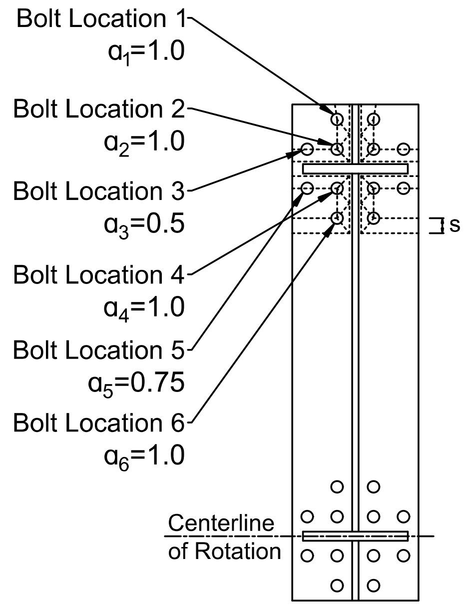
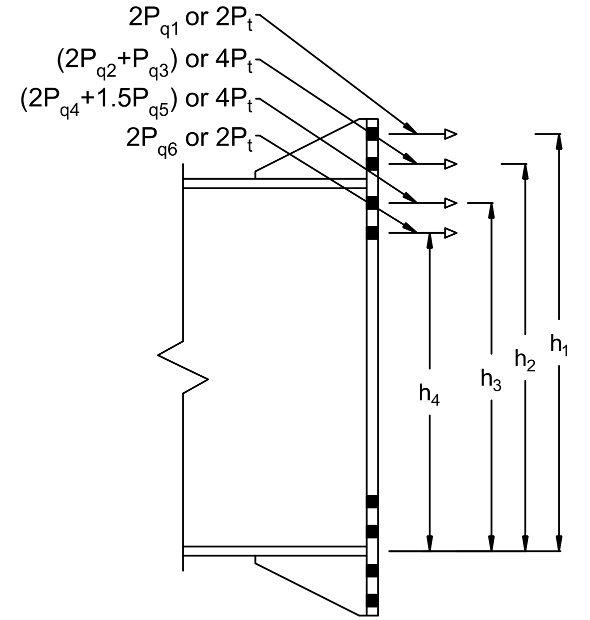
4.3. Seismic Design Requirements

The remaining portion of the design should be completed according to the design procedure listed for end-plate moment connections in the AISC *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC 2011). The requirements from this document include checking the following: beam stiffener local buckling, welding details, strength of the panel zone, column flange flexural yielding, and the column-beam moment ratio in conformance with the strong-column / weak-beam principle.

4.4. Summary

Table 4-2 below gives a summary of the yield line mechanism, the thick plate model, and the thin plate model. The connection will exhibit thick end-plate behavior if $M_{pl} > 1.11M_{np}$. This requirement is based on conclusions from previous testing where prying forces in the bolts were not significant until at least ninety percent of the yield line end-plate strength is achieved (Murray and Sumner 2014).

Table 4-2 Summary of Design Equations

Geometry	Yield Line Mechanism	Bolt Force Model
		

End-Plate Yield	$\phi M_n = \phi M_{pl} = \phi F_{py} t_p^2 Y \qquad \phi = 0.9 \qquad s = \frac{1}{2}\sqrt{b_p g}$ $Y = \frac{b_p}{2}\left[\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2}\right] + \frac{1}{2g}\left[h_1(4d_e + 3p_b) + h_2(p_b + 4p_{fo}) + h_3(4p_{fi} + 3p_b) + h_4(p_b + 4s)\right] + \frac{5}{4}g$
Bolt Rupture with Prying Action	$\phi M_n = \phi M_q = \phi \sum_{i=1}^{6} \alpha_i n_i P_{qi} h_j$ $= \phi\left[2P_{q1}h_1 + 2P_{q2}h_2 + P_{q3}h_2 + 2P_{q4}h_3 + 1.5P_{q5}h_3 + 2P_{q6}h_4\right]$ $P_{qi} = max\begin{Bmatrix} P_t - Q_{max,i} \\ T_b \end{Bmatrix} \qquad \phi = 0.75$ P_t = nominal bolt tension strength $Q_{max,i}$ = prying force for bolt location i T_b = bolt pretension force n_i = number of bolts in bolt location i
Bolt Rupture without Prying Action	$\phi M_n = \phi M_{np} = \phi P_t \sum_i n_i h_i = \phi\left[2P_t(h_1 + 2h_2 + 2h_3 + h_4)\right] \qquad \phi = 0.75$

5. EXPERIMENTAL TESTING

A total of four specimens, which were a new twelve bolt end-plate moment connection configuration, were tested at the Thomas M. Murray Structures Laboratory at Virginia Tech. These specimens were tested with the intention of qualifying the connection, and validating the proposed design procedure. Two specimens were a deep beam, which was 44 inches deep. The other two specimens were a shallow beam, which was 24 inches deep. The deep beam specimens were designed for thick end-plate behavior. One of which was designed to fail due to bolt tension rupture without prying action, and the other was designed to fail due to plastic hinging of the beam. One shallow beam specimen was designed for thin end-plate behavior, and thus designed to fail due to bolt tension rupture with prying action. The other shallow beam specimen was designed for thick end-plate behavior, and designed to fail due to plastic hinging of the beam. The specimens where the bolts were designed to fail were used for the purpose of validating the proposed design procedures. The following sections detail the experimental testing program.

5.1. Test Specimens

All of the beams and columns were built-up steel sections. Two beams and two columns were fabricated for these tests. Each beam was used for two connection specimens by using each end of each beam. The beams and their end-plates were specified to be fabricated from a combination of A529 Gr. 55 and A572 Gr. 55 plate material. The beam flanges, and the end-plates for two of the specimens were A529 Gr. 55 steel. The remaining material for the beams and end-plates was A572 Gr. 55 steel. The mill test reports for the beam material can be seen in Appendix A.

All specimens were connected to the same size columns, which were 42 inches deep. The columns were designed based on the strong column – weak beam criterion, and were designed to have minimal panel zone deformation. The material for the columns and their end-plates were specified to have a yield strength of 50 ksi. The mill test reports for the column material can be found in Appendix B. The cross-sections for the girder specimens are shown below in Figure 5-1 and the cross-sections for the column specimens are shown in Figure 5-2 below. Each girder specimen was used for two specimens, one end-plate on each end of the beam. A naming convention was given to each specimen, which is “Connection Type – Bolt Diameter – End-Plate

Thickness – Rafter Depth". Girder 1 included End-Plate 12ES-0.75-1.00-44 and End-Plate 12ES-1.25-1.50-44. Girder 2 included End-Plate 12ES-0.875-0.75-24 and End-Plate 12ES-1.125-1.25-24.

The deep beam specimen was tested with Column 1 and the shallow beam was tested with Column 2. Specimen 12ES-0.875-0.75-24 was designed for thin end-plate behavior, whereas the rest were designed for thick end-plate behavior. Specimen 12ES-0.75-1.00-44 was designed to fail due to bolt tension rupture without prying action. Specimen 12ES-1.25-1.50-44 was designed to fail via plastic hinging of the beam. The details of End-Plate 12ES-0.75-1.00-44 and End-Plate 12ES-1.25-1.50-44 are shown in Figure 5-3 below. Specimen 12ES-0.875-0.75-24 was designed to fail due to end-plate yielding and bolt tension rupture with prying action. Specimen 12ES-1.125-1.25-24 was designed to fail via plastic hinging of the beam. The details of End-Plate 12ES-0.875-0.75-24 and End-Plate 12ES-1.125-1.25-24 are shown below in Figure 5-4. The test matrix is shown below in Table 5-1. The fabrication drawings for the beams, columns and end-plates are shown in Appendix D. The design calculation for the beams and columns are in Appendix G.

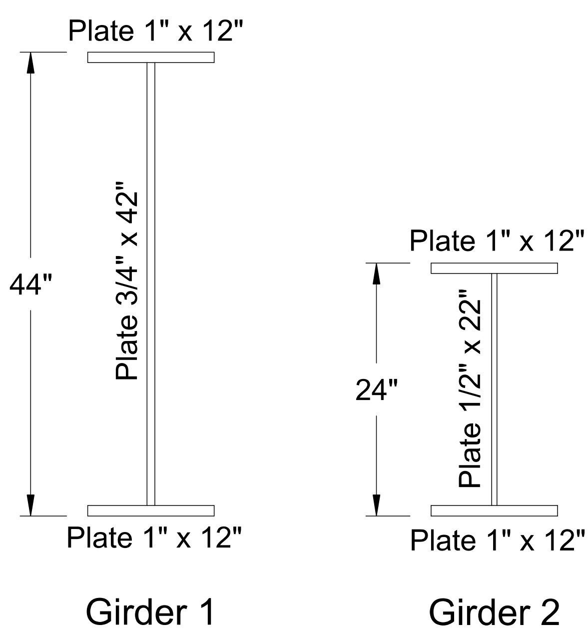


Figure 5-1 Girder Cross-Sections

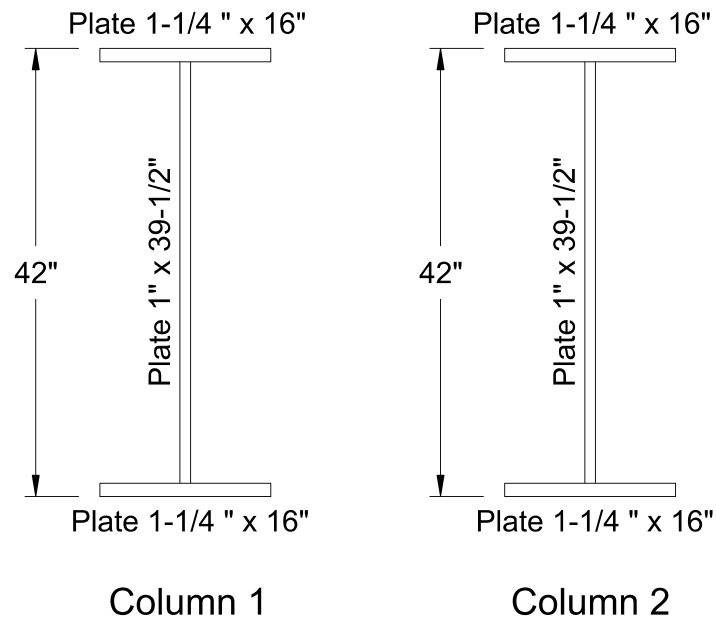


Figure 5-2 Column Cross-Sections

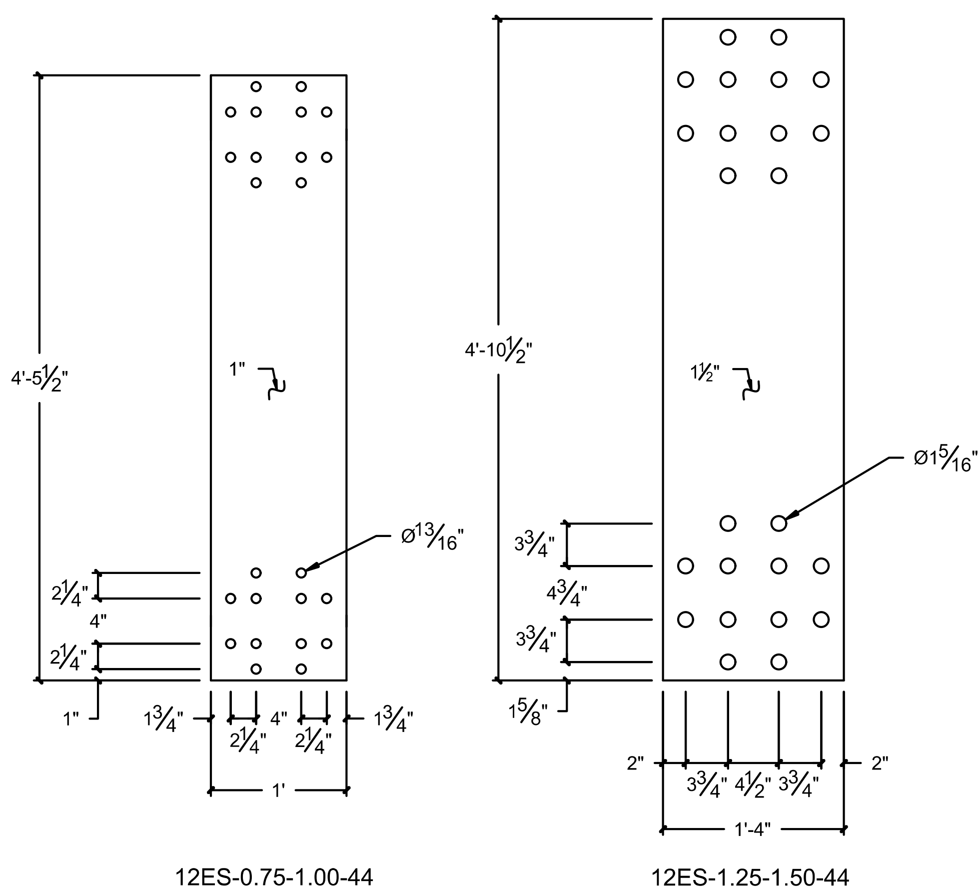


Figure 5-3 Deep Beam End-Plates

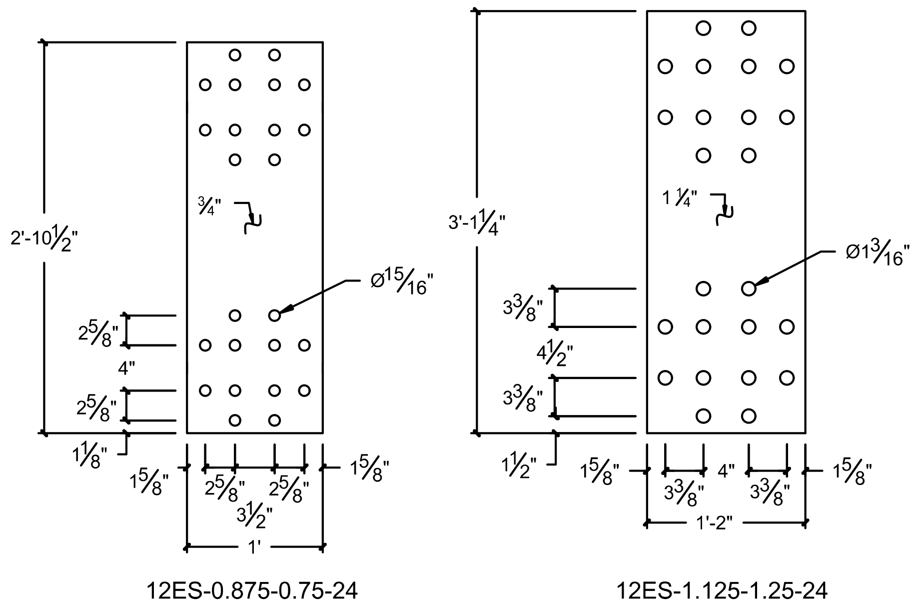


Figure 5-4 Shallow Beam End-Plates

Table 5-1 Test Matrix

Specimen	Beam Depth, d (in)	Bolts	End-Plate Thickness, t_p (in)	End-Plate Width, b_p (in)	Purpose of Test	M_{pl} (k-ft)	M_{np} (k-ft)	M_p (k-ft)
12ES-0.75-1.00-44	44	3/4" A490	1	12	Thick End-Plate Behavior	3097	2147	3881
12ES-1.25-1.50-44	44	1-1/4" A490	1-1/2	16	Plastic Hinging	7967	5963	3881
12ES-0.875-0.75-24	24	7/8" A490	3/4	12	Thin End-Plate Behavior	1010	1563	1542
12ES-1.125-1.25-24	24	1-1/8" A490	1-1/4	14	Plastic Hinging	2891	2583	1542

Note: M_{pl}, M_{np}, and M_p are based on nominal material properties

5.1.1. Measured Dimensions

For each specimen, various dimensions were measured and compared to the detailed dimension. Each dimension was measured three times. The small dimensions were measured with a Vernier caliper, and the others with a tape measure. These dimensions include the beam depth, the beam flange width, the beam flange thickness, beam web thickness, the end-plate width, and the end-plate thickness. The measured dimensions for Specimen 12ES-0.75-1.00-44, Specimen 12ES-1.25-1.50-44, Specimen 12ES-0.875-0.75-24, and Specimen 12ES-1.125-1.25-24 can be found below in Table 5-2, Table 5-3, Table 5-4, and Table 5-5 respectively.

Table 5-2 Measured Dimensions for Specimen 12ES-0.75-1.00-44

Specimen 12ES-0.75-1.00-44	Measured (in)			Average (in)	Detailed (in)
Beam Depth (d)	43.94	44.00	44.06	44.00	44
Flange Width (b_f)	11.94	11.94	11.94	11.94	12
Flange Thickness (t_f)	1.010	1.011	1.005	1.009	1
Web Thickness (t_w)	0.755	0.751	0.754	0.753	0.75
End-Plate Width (b_p)	11.88	11.88	11.88	11.88	12
End-Plate Thickness (t_p)	1.020	1.015	1.013	1.016	1

Table 5-3 Measured Dimensions for Specimen 12ES-1.25-1.50-44

Specimen 12ES-1.25-1.50-44	Measured (in)			Average (in)	Detailed (in)
Beam Depth (d)	43.75	43.94	43.81	43.83	44
Flange Width (b_f)	11.94	11.94	11.94	11.94	12
Flange Thickness (t_f)	1.017	1.012	1.011	1.013	1
Web Thickness (t_w)	0.748	0.755	0.754	0.752	0.75
End-Plate Width (b_p)	16.00	15.94	15.94	15.96	16
End-Plate Thickness (t_p)	1.531	1.526	1.528	1.528	1.5

Table 5-4 Measured Dimensions for Specimen 12ES-0.875-0.75-24

Specimen 12ES-0.875-0.75-24	Measured (in)			Average (in)	Detailed (in)
Beam Depth (d)	24.00	24.00	24.00	24.00	24
Flange Width (b_f)	11.94	11.94	11.94	11.94	12
Flange Thickness (t_f)	1.021	1.022	1.023	1.022	1
Web Thickness (t_w)	0.510	0.505	0.508	0.508	0.5
End-Plate Width (b_p)	12.00	12.00	12.00	12.00	12
End-Plate Thickness (t_p)	0.767	0.765	0.766	0.766	0.75

Table 5-5 Measured Dimensions for Specimen 12ES-1.125-1.25-24

Specimen 12ES-1.125-1.25-24	Measured (in)			Average (in)	Detailed (in)
Beam Depth (d)	24.00	24.00	24.00	24.00	24
Flange Width (b_f)	12.00	11.94	11.94	11.96	12
Flange Thickness (t_f)	1.018	1.016	1.029	1.021	1
Web Thickness (t_w)	0.515	0.506	0.504	0.508	0.5
End-Plate Width (b_p)	14.00	14.00	14.00	14.00	14
End-Plate Thickness (t_p)	1.273	1.255	1.262	1.263	1.25

For each end-plate, multiple dimensions were measured using a digital Vernier caliper. The dimensions that were measured are shown below in Figure 5-5. Each dimension references the edge of the hole, rather than typical detailing to the center of the hole. Several measurements were recorded for each dimension for each end-plate. The average of each dimension was calculated. The average bolt-hole diameter was then used with the other average measured dimension values to calculate the appropriate detailed value, which was compared to the actual detailed value. For all end-plates, all calculated detailed dimensions matched well with the actual detailed dimensions. The measured and calculated dimensions for End-Plate 12ES-0.75-1.00-44, End-Plate 12ES-1.25-1.50-44, End-Plate 12ES-0.875-0.75-24, and End-Plate 12ES-1.125-1.25-24 can be found below in Table 5-6, Table 5-7, Table 5-8, and Table 5-9 respectively.

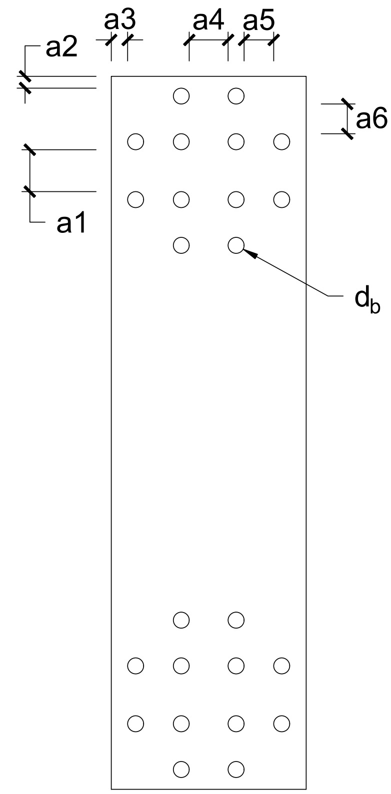


Figure 5-5 Measured End-Plate Dimensions

Table 5-6 Measured & Calculated Dimensions for End-Plate 12ES-0.75-1.00-44

inches	d_b	a1	a2	a3	a4	a5	a6
Measurements	0.808	3.173	0.689	1.315	3.203	1.494	1.427
	0.806	3.138	0.677	1.324	3.205	1.462	1.470
	0.810	3.189		1.299	3.160	1.442	1.450
Average	**0.808**	**3.167**	**0.683**	**1.313**	**3.189**	**1.466**	**1.449**
	d_b	**d_e**	**e**	**g**	**g_o**	**p_b**	
Actual*	0.808	1.087	1.717	3.997	2.274	2.257	
Detailed	0.8125	1.000	1.750	4.000	2.250	2.250	

*Based on average values above

Table 5-7 Measured & Calculated Dimensions for End-Plate 12ES-1.25-1.50-44

inches	d_b	a1	a2	a3	a4	a5	a6
Measurements	1.346	3.428	0.967	1.272	3.121	2.382	2.409
	1.346	3.395	0.966	1.294	3.116	2.369	2.412
	1.351	3.406		1.348	3.132	2.381	2.381
Average	**1.348**	**3.410**	**0.967**	**1.305**	**3.123**	**2.377**	**2.401**
	d_b	d_e	e	g	g_o	p_b	
Actual*	1.348	1.640	1.979	4.471	3.725	3.748	
Detailed	1.3125	1.625	2.000	4.500	3.750	3.750	

*Based on average values above

Table 5-8 Measured & Calculated Dimensions for End-Plate 12ES-0.875-0.75-24

inches	d_b	a1	a2	a3	a4	a5	a6
Measurements	0.984	3.050	0.592	1.118	2.548	1.607	1.655
	0.991	3.005	0.591	1.217	2.528	1.633	1.637
	1.005	3.016		1.224	2.534	1.628	1.650
Average	**0.993**	**3.024**	**0.592**	**1.186**	**2.537**	**1.623**	**1.647**
	d_b	d_e	e	g	g_o	p_b	
Actual*	0.993	1.088	1.683	3.530	2.616	2.641	
Detailed	0.9375	1.125	1.625	3.500	2.625	2.625	

*Based on average values above

Table 5-9 Measured & Calculated Dimensions for End-Plate 12ES-1.125-1.25-24

inches	d_b	a1	a2	a3	a4	a5	a6
Measurements	1.248	3.254	0.885	0.930	2.808	2.133	2.131
	1.254	3.227	0.833	0.933	2.764	2.153	2.134
	1.250	3.240		1.090	2.749	2.137	2.179
Average	**1.251**	**3.240**	**0.859**	**0.984**	**2.774**	**2.141**	**2.148**
	d_b	d_e	e	g	g_o	p_b	
Actual*	1.251	1.484	1.610	4.024	3.392	3.399	
Detailed	1.1875	1.500	1.625	4.000	3.375	3.375	

*Based on average values above

5.1.2. Tensile Coupon Testing

Tensile testing was conducted for the beam material. The coupons for the beam flange material and the beam web material were cut from the actual beam specimens in the region that remained elastic during the tests. Extra plates from the same heat as each end-plate were shipped with the beams. These extra plates provided at least three test coupons for each end-plate material. These coupon tests provide the full load-displacement curves, and a comparison against the mill test reports. The tension coupons were tested using a SATEC 300 kip Universal Testing Machine, which was displacement controlled at a rate of 0.25 inches per minute. The geometry of the coupons was determined according to ASTM A370-16 *Standard Test Methods and Definitions for Mechanical Testing of Steel Products* (ASTM 2016). Tensile coupons with a total length of 18 inches, a reduced width of 1-1/2 inches, and an 8 inch gauge length were used. Marks were punched into each tensile coupon in order to determine the elongation after the specimen ruptured.

The thickness of each coupon was the same as the thickness of its respective end-plate, flange, or web. The measured initial dimensions of each tensile coupon can be found below in Table 5-10. The measured dimensions of the neck of each coupon after the specimen ruptured can be seen below in Table 5-11. The elongation of each coupon specimen can be found below in Table 5-12. For each coupon specimen, the initial dimensions, the final dimensions, the initial gauge length, and the final gauge length were measured with a digital Vernier caliper.

The stress versus strain curves for the flange material coupons can be seen below in Figure 5-6. The stress versus strain curves for the shallow beam web material coupons can be seen below in Figure 5-7. The stress versus strain curves for the deep beam web material coupons can be seen below in Figure 5-8. The stress versus strain curves for the End-Plate 12ES-0.75-1.00-44 material coupons can be seen below in Figure 5-9. The stress versus strain curves for the End-Plate 12ES-1.25-1.50-44 material coupons can be seen below in Figure 5-10. The stress versus strain curves for the End-Plate 12ES-0.875-0.75-24 material coupons can be seen below in Figure 5-11. The stress versus strain curves for the End-Plate 12ES-1.125-1.25-24 material coupons can be seen below in Figure 5-12. A summary of the strengths of the beam material from the tensile coupon tests can be found below in Table 5-13.

The mill test reports for the beam material can be found in Appendix A. A summary of the information located in the mill test reports for the beam material can be found below in Table 5-14. The mill test reports for the column material can be found in Appendix B. A summary of this information located in the mill test reports for the column material is shown below in Table 5-15.

Table 5-10 Initial Tensile Coupon Dimensions

Description	Label	Initial Width (in)				Initial Thickness (in)				Initial Area
		Measured			Average	Measured			Average	(in^2)
Shallow Web	24W	1.521	1.520	1.522	1.521	0.490	0.494	0.496	0.493	0.750
Deep Web	44W	1.519	1.520	1.520	1.520	0.760	0.768	0.762	0.763	1.160
Flanges	24FF	1.509	1.507	1.508	1.508	1.011	1.009	1.007	1.009	1.522
	24F	1.514	1.521	1.515	1.517	1.009	1.010	1.010	1.010	1.531
	44F1	1.508	1.515	1.513	1.512	1.001	0.999	0.998	0.999	1.511
	44F2	1.516	1.524	1.519	1.520	1.002	0.999	1.000	1.000	1.520
End-Plate 12ES-0.75-1.00-44	A1	1.508	1.506	1.508	1.507	1.009	1.004	1.004	1.006	1.516
	A2	1.505	1.505	1.509	1.506	1.000	1.005	1.005	1.003	1.511
	A3	1.512	1.512	1.515	1.513	1.006	1.012	1.008	1.009	1.526
	A4	1.514	1.515	1.516	1.515	1.009	1.011	1.005	1.008	1.528
End-Plate 12ES-1.25-1.50-44	B1	1.512	1.510	1.512	1.511	1.517	1.517	1.524	1.519	2.296
	B2	1.512	1.517	1.508	1.512	1.522	1.520	1.523	1.522	2.301
	B3	1.510	1.510	1.519	1.513	1.518	1.515	1.520	1.518	2.296
End-Plate 12ES-0.875-0.75-24	C1	1.513	1.516	1.512	1.514	0.756	0.755	0.758	0.756	1.145
	C2	1.519	1.515	1.516	1.517	0.754	0.753	0.757	0.755	1.145
	C3	1.515	1.514	1.514	1.514	0.755	0.753	0.752	0.753	1.141
	C4	1.509	1.513	1.510	1.511	0.754	0.753	0.754	0.754	1.139
End-Plate 12ES-1.125-1.25-24	D1	1.508	1.496	1.505	1.503	1.276	1.274	1.273	1.274	1.915
	D2	1.522	1.520	1.528	1.523	1.273	1.275	1.274	1.274	1.941
	D3	1.505	1.505	1.505	1.505	1.276	1.275	1.274	1.275	1.919

Table 5-11 Final Tensile Coupon Dimensions

Description	Label	Neck Width (in)				Neck Thickness (in)				Neck Area
		Measured			Average	Measured			Average	(in^2)
Shallow Web	**24W**	1.145	1.146	1.145	1.145	0.331	0.329	0.329	0.330	0.378
Deep Web	**44W**	1.057	1.069	1.063	1.063	0.507	0.503	0.509	0.506	0.538
Flanges	**24FF**	1.133	1.120	1.119	1.124	0.714	0.713	0.680	0.702	0.789
	24F	1.204	1.202	1.195	1.200	0.774	0.768	0.772	0.771	0.926
	44F1	1.114	1.133	1.129	1.125	0.699	0.697	0.693	0.696	0.784
	44F2	1.135	1.125	1.128	1.129	0.682	0.720	0.715	0.706	0.797
End-Plate 12ES-0.75-1.00-44	**A1**	1.126	1.123	1.125	1.125	0.699	0.712	0.708	0.706	0.794
	A2	1.137	1.129	1.125	1.130	0.737	0.715	0.734	0.729	0.824
	A3	1.108	1.099	1.106	1.104	0.714	0.700	0.704	0.706	0.780
	A4	1.101	1.085	1.097	1.094	0.700	0.693	0.702	0.698	0.764
End-Plate 12ES-1.25-1.50-44	**B1**	1.017	1.000	1.004	1.007	0.971	0.969	0.967	0.969	0.976
	B2	0.997	0.996	0.997	0.997	0.948	0.953	0.956	0.952	0.949
	B3	0.998	1.016	1.014	1.009	0.957	0.954	0.951	0.954	0.963
End-Plate 12ES-0.875-0.75-24	**C1**	1.117	1.119	1.115	1.117	0.529	0.530	0.516	0.525	0.586
	C2	1.062	1.070	1.068	1.067	0.516	0.524	0.505	0.515	0.549
	C3	1.113	1.121	1.124	1.119	0.532	0.530	0.524	0.529	0.592
	C4	1.019	1.089	1.105	1.071	0.501	0.514	0.521	0.512	0.548
End-Plate 12ES-1.125-1.25-24	**D1**	1.041	1.057	1.044	1.047	0.882	0.855	0.868	0.868	0.909
	D2	1.056	1.048	1.053	1.052	0.864	0.860	0.857	0.860	0.905
	D3	1.071	1.064	1.068	1.068	0.876	0.867	0.877	0.873	0.932

Table 5-12 Tensile Coupon Elongations

Description	Label	Initial Gauge (in)				Final Gauge (in)				Elongation (%)
		Measured			Average	Measured			Average	
Shallow Web	24W	8.017	8.010	8.015	8.014	9.594	9.602	9.596	9.597	19.8
Deep Web	44W	8.076	8.072	8.078	8.075	9.998	9.987	9.992	9.992	23.7
Flanges	24FF	8.046	8.050	8.047	8.048	9.942	9.946	9.939	9.942	23.5
	24F	8.039	8.034	8.033	8.035	9.936	9.934	9.942	9.937	23.7
	44F1	8.035	8.034	8.042	8.037	9.993	9.992	9.986	9.990	24.3
	44F2	8.085	8.084	8.086	8.085	9.957	9.961	9.967	9.962	23.2
End-Plate 12ES-0.75-1.00-44	A1	8.025	8.027	8.026	8.026	9.961	9.964	9.968	9.964	24.2
	A2	8.018	8.014	8.019	8.017	9.932	9.938	9.936	9.935	23.9
	A3	8.040	8.042	8.039	8.040	9.989	9.992	9.994	9.992	24.3
	A4	8.003	8.002	8.000	8.002	9.934	9.938	9.935	9.936	24.2
End-Plate 12ES-1.25-1.50-44	B1	8.030	8.032	8.028	8.030	10.217	10.216	10.217	10.217	27.2
	B2	8.081	8.071	8.078	8.077	10.219	10.221	10.220	10.220	26.5
	B3	8.064	8.069	8.067	8.067	10.222	10.225	10.221	10.223	26.7
End-Plate 12ES-0.875-0.75-24	C1	8.000	8.002	8.004	8.002	9.853	9.848	9.855	9.852	23.1
	C2	8.018	8.020	8.019	8.019	9.923	9.931	9.928	9.927	23.8
	C3	8.039	8.046	8.038	8.041	9.912	9.919	9.918	9.916	23.3
	C4	8.004	8.002	8.005	8.004	9.886	9.889	9.890	9.888	23.5
End-Plate 12ES-1.125-1.25-24	D1	8.053	8.059	8.063	8.058	9.945	9.952	9.954	9.950	23.5
	D2	8.057	8.062	8.064	8.061	10.079	10.091	10.082	10.084	25.1
	D3	8.001	7.997	7.999	7.999	10.032	10.040	10.037	10.036	25.5

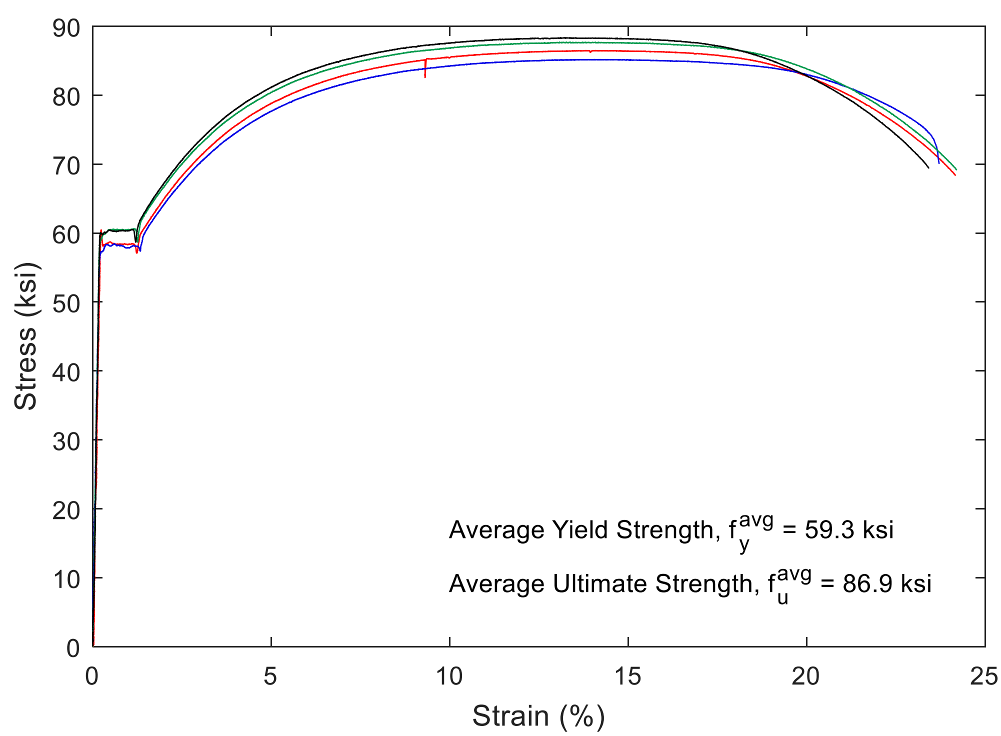


Figure 5-6 Results of Tensile Testing for Flange Material

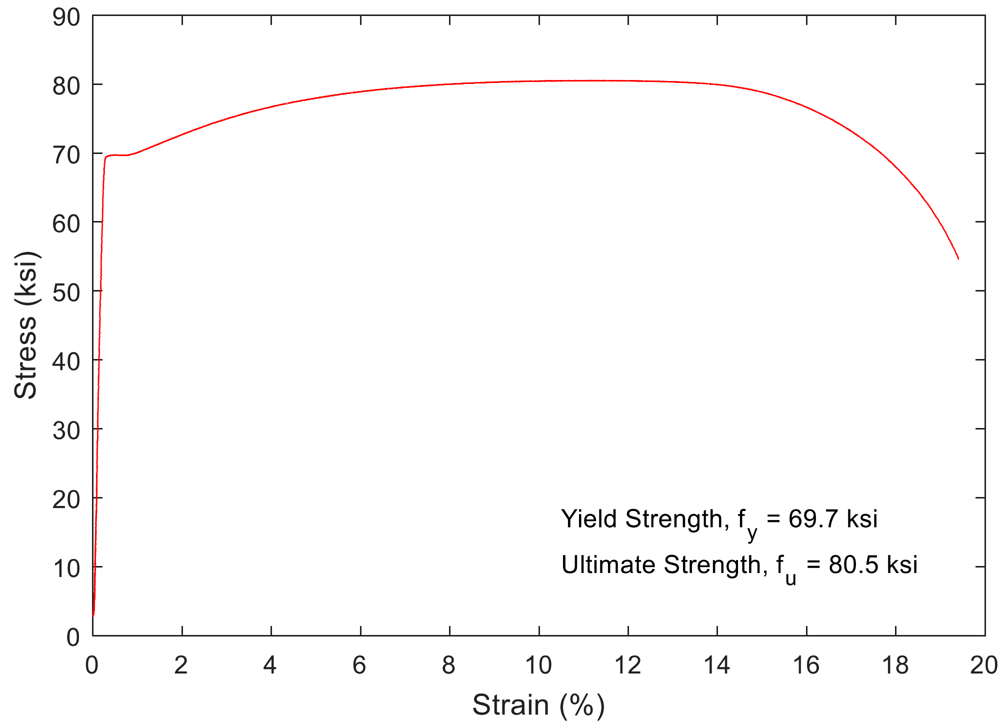


Figure 5-7 Results of Tensile Testing for Shallow Web Material

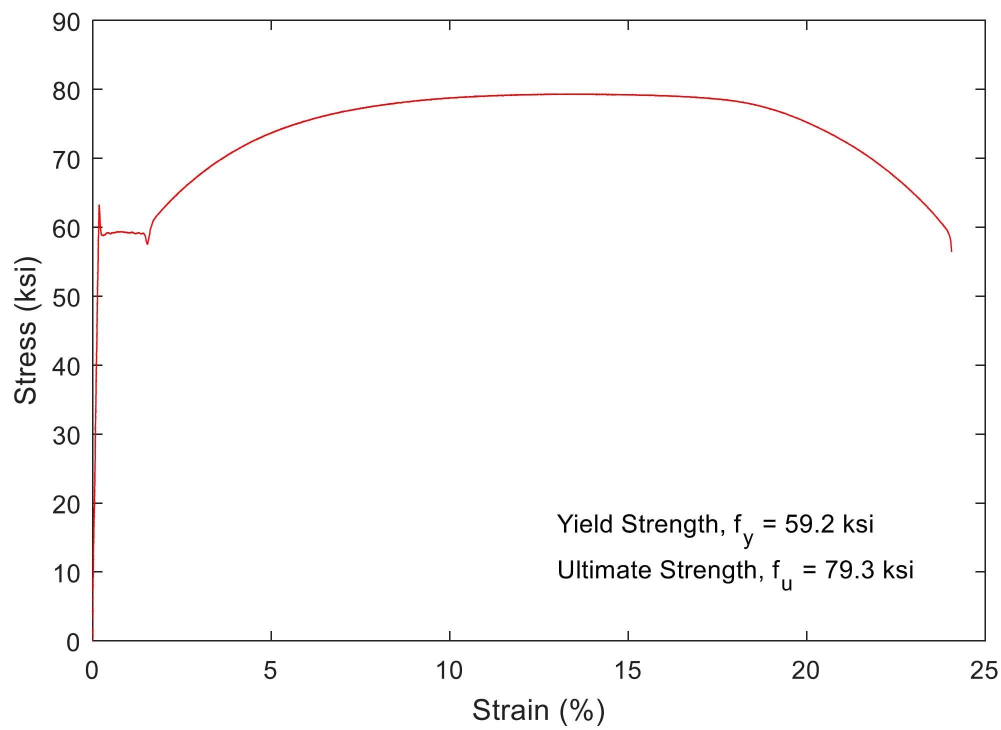


Figure 5-8 Results of Tensile Testing for Deep Web Material

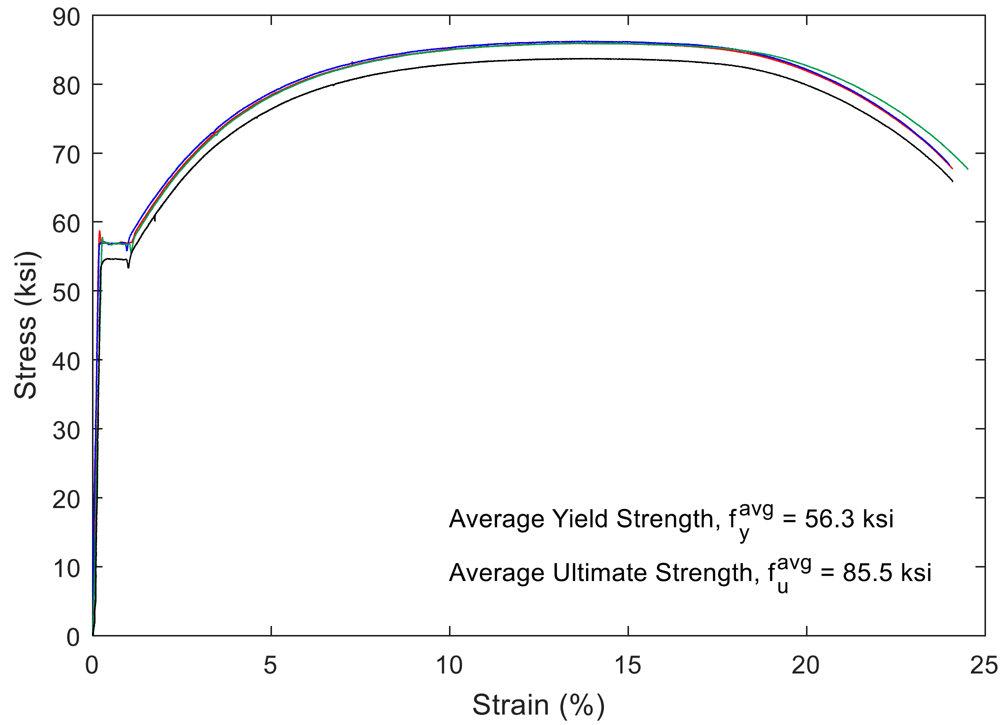


Figure 5-9 Results of Tensile Testing for End-Plate 12ES-0.75-1.00-44 Material

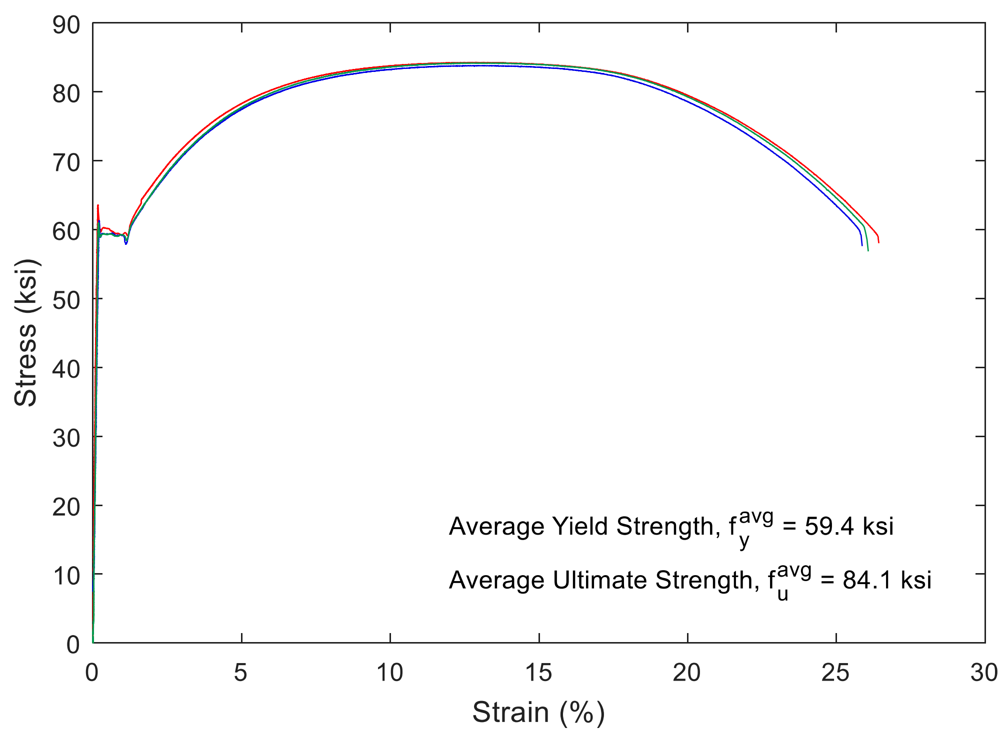


Figure 5-10 Results of Tensile Testing for End-Plate 12ES-1.25-1.50-44 Material

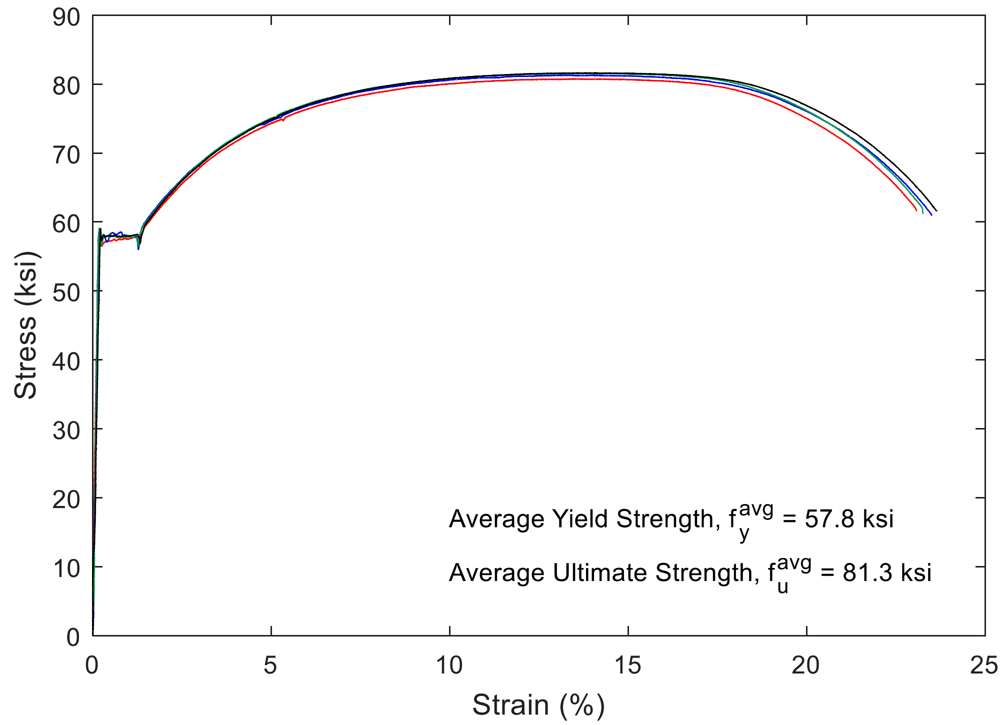


Figure 5-11 Results of Tensile Testing for End-Plate 12ES-0.875-0.75-24 Material

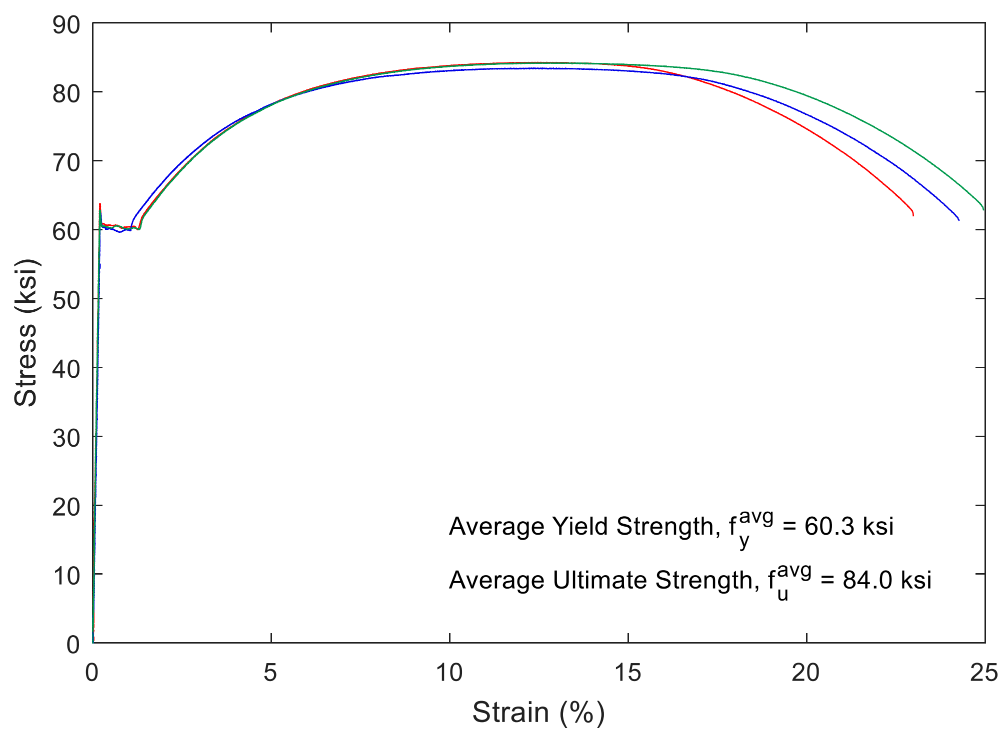


Figure 5-12 Results of Tensile Testing for End-Plate 12ES-1.125-1.25-24 Material

Table 5-13 Summary of Tensile Coupon Testing for Beam Material

Description	Label	Yield Strength, f_y (ksi)		Ultimate Strength, f_u (ksi)	
		Measured	Average	Measured	Average
Shallow Web	24W	69.7	69.7	80.50	80.5
Deep Web	44W	59.2	59.2	79.30	79.3
Flanges	24FF	58.4	59.3	86.50	86.9
	24F	58.1		85.20	
	44F1	60.5		87.70	
	44F2	60.2		88.30	
End-Plate 12ES-0.75-1.00-44	A1	56.9	56.3	85.9	85.5
	A2	56.9		86.2	
	A3	56.9		85.9	
	A4	54.6		83.7	
End-Plate 12ES-1.25-1.50-44	B1	59.7	59.4	84.30	84.1
	B2	59.3		83.80	
	B3	59.2		84.20	
End-Plate 12ES-0.875-0.75-24	C1	57.5	57.8	80.80	81.3
	C2	58.0		81.30	
	C3	57.8		81.60	
	C4	58.0		81.60	
End-Plate 12ES-1.125-1.25-24	D1	60.5	60.3	84.30	84.0
	D2	60.0		83.40	
	D3	60.3		84.20	

Table 5-14 Summary of Beam Material Mill Test Reports

Description	Size (in)	Heat No.	Yield (ksi)*	UTS (ksi)*	Elong. (%)*	
					2"	8"
End-Plate 12ES-0.875-0.75-24	0.75x12x480	55044867	61.80	82.00		18.5
			60.90	83.20		18.9
			61.35	**82.60**		**18.70**
End-Plate 12ES-1.25-1.50-44	1.5x96x240	5506374	66.50	91.30		22.1
			57.10	83.90		21.9
			61.80	**87.60**		**22.00**
End-Plate 12ES-1.125-1.25-24	1.25x96x240	B6Q3803	58.40	83.20		25.1
			58.40	83.20		25.1
			58.40	83.20		25.1
			59.30	83.10		26.8
			59.30	83.10		26.8
			58.76	**83.16**		**25.78**
Deep Web	0.75x96x480	B6Q3899	55.20	67.90	44.1	
			61.10	68.40	39.9	
			55.20	67.90	44.1	
			61.10	68.40	39.9	
			55.20	67.90	44.1	
			61.10	68.40	39.9	
			55.20	67.90	44.1	
			61.10	68.40	39.9	
			58.15	**68.15**	**42.00**	
Stiffeners	0.75x96x480	B6Q3964	62.80	83.90	36.9	
			64.40	79.80	33.1	
			62.80	83.90	36.9	
			64.40	79.80	33.1	
			63.60	**81.85**	**35.00**	
Shallow Web & Stiffeners	0.485x60x241	A507756	73.40	81.70		28
			73.40	**81.70**		**28.00**
Flanges & End-Plate 12ES-0.75-1.00-44	1x12x480	1042587	60.00	81.20		22
			59.50	80.30		22
			59.75	**80.75**		**22.00**

*Bolded numbers are the average value of respective set

Table 5-15 Summary of Column Material Mill Test Reports

Description	Size (in)	Heat No.	Yield (ksi)*	UTS (ksi)*	Elong. (%)*
					8"
Flange	1.25x96x240	6503350	67.90	85.00	22.6
			55.50	79.20	20.2
			61.70	**82.10**	**21.4**
Web & End-Plates	1x74x673	4501876	55.20	80.70	17.0
			53.30	80.00	20.6
			54.25	**80.35**	**18.8**

*Bolded numbers are the average value of respective set

Between all sets of beam material, the stress strain curves were similar in shape except for the web material for the 24 inch deep beam, which is labeled above as “shallow web”. The yield strength that was obtained for the shallow web material was about 70 ksi, whereas all of the other material had a yield strength of about 60 ksi. For the shallow web material, with an ultimate strength of 80.5 ksi, the ratio of F_y to F_u is 0.866. This falls outside the maximum value of 0.85 specified in the ASTM for steels that are commonly used for seismic applications, such as A992 (ASTM 2011). The ASTM for A529 and the ASTM for A572 do not have a maximum value for the ratio of F_y to F_u specified (ASTM 2014b, 2015). For all of the material other than the shallow web material, the ratio of F_y to F_u ranged from about 0.65 to 0.75.

5.1.3. Charpy V-Notch Testing

Charpy V-notch coupons were made from beam flange material. The material was cut from the one of the beam specimens in the region that remained elastic during the tests. The coupons were standard size, and the geometry of the coupons was determined according to ASTM A370-16 *Standard Test Methods and Definitions for Mechanical Testing of Steel Products* (ASTM 2016). A summary of the results of the Charpy V-notch testing is shown below in Table 5-16.

Table 5-16 Summary of Charpy V-Notch Testing Results

Specimen	Temperature(^{o}F)	Fracture Toughness (ft-lb)	Average (ft-lb)
1	60	45	37.1
2	60	31	
3	59	27	
4	60	55	
5	58	27.5	

5.1.4. Bolt Tensile Testing

Three bolts of each size were tested using a SATEC 300 kip Universal Testing Machine. These bolt tests provide the ultimate tensile capacity of the bolts, and a comparison against the mill test reports. The tensile testing was displacement controlled, and the displacement rate was 0.024 inches per minute. A summary of the results from the bolt tensile testing can be seen below in Table 5-17. The mill test reports for the bolts can be found in Appendix C. A summary of this information located in the mill test reports for the bolts is shown below in Table 5-18.

Table 5-17 Results from Bolt Tensile Testing

Bolt Diameter (in)	Peak Force (kip)			Average Force (kip)	Stress Area* (in^2)	Average Strength (ksi)
3/4	55.04	54.32	54.33	54.6	0.334	163.4
7/8	75.34	76.66	77.25	76.4	0.462	165.4
1-1/8	119.26	122.04	122.28	121.2	0.763	158.8
1-1/4	159.83	159.79	162.19	160.6	0.969	165.7

* ASTM A490-14a

Table 5-18 Summary of Bolt Mill Test Reports

Specimen	Bolt Size (in)	Tensile Strength* (ksi)
12ES-0.75-1.00-44	3/4 x 3-1/2	159.0
12ES-0.875-0.75-24	7/8 x 3-1/2	170.2
12ES-1.125-1.25-24	1-1/8 x 4-1/4	163.0
12ES-1.25-1.50-44	1-1/4 x 4-1/2	162.2

*Reported average values

5.1.5. Specimens Welds

The fabrication drawings for the beam specimens and columns specimens can be found in Appendix D. The welding procedure specifications (WPS) for the beam specimens can be found in Appendix E. The WPS for the column specimens can be found in Appendix F. These drawings show the specimens as they were designed.

5.1.5.1. Beam Web to Flange Welds

During the fabrication process, a minor change had to be made to one of the welds on each of the beam specimens. For the deep beam, the weld from the web to flange along the middle portion of the beam changed from a single sided 3/8" fillet weld to a double sided 5/16" fillet weld. For the shallow beam, the weld from the web to flange along the middle portion of the beam changed from a single sided 1/4" fillet weld to a double sided 1/4" fillet weld. These changes are reflected in the design drawings in red ink in Appendix D.

5.1.5.2. Beam to End-Plate Welds

The beam to end-plate welds for Specimen 12ES-0.75-1.00-44, Specimen 12ES-1.25-1.50-44, Specimen 12ES-0.875-0.75-24, and Specimen 12ES-1.125-1.25-24 are shown below in

Figure 5-13, Figure 5-14, Figure 5-15, and Figure 5-16 respectively. Specimen 12ES-0.75-1.00-44 and Specimen 12ES-0.875-0.75-24 were designed and fabricated with built-up partial joint penetration (PJP) welds from the beam flange to the end-plate. These specimens were designed to fail the bolts, whereas Specimen 12ES-1.25-1.50-44 and Specimen 12ES-1.125-1.25-24 were designed to form a plastic hinge in the beam. Since the plastic hinging specimens were expected to experience higher loads, they were detailed as CJP welds with 5/16" reinforcing fillet weld on the inside face of the flange for the beam flange to end-plate welds, which meets the requirements of AISC 341-10 and AISC 358-10. Since PJP welds are not currently allowed in AISC 358, the performance of these welds can be further supported with these specimens (AISC 2011). The PJP welds were designed and detailed according to AWS Structural Welding Code – Steel (AWS 2015).

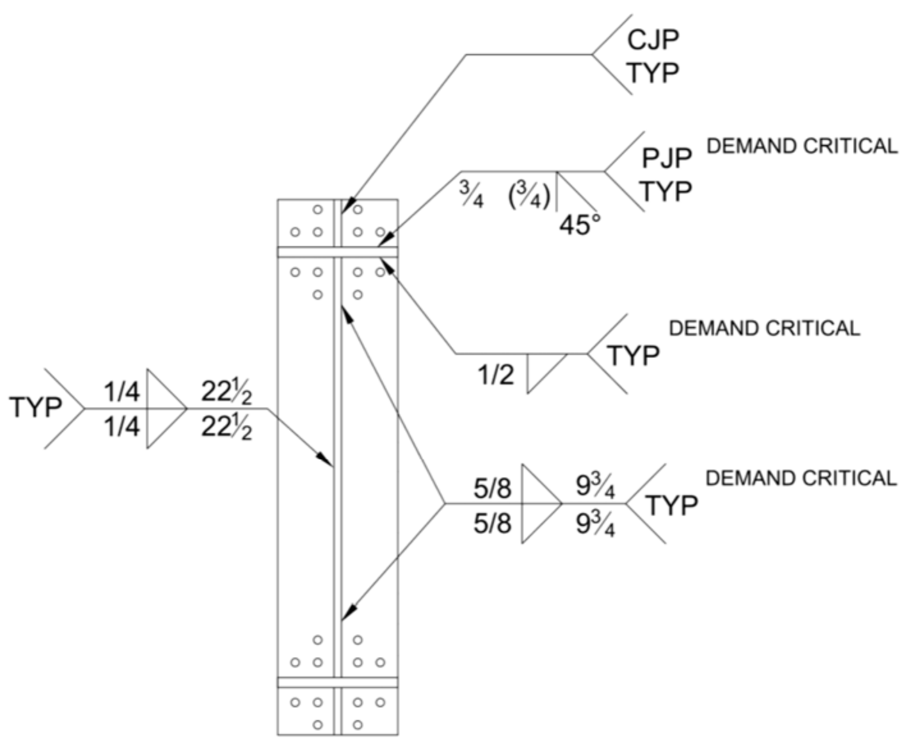


Figure 5-13 Beam to End-Plate Welds for Specimen 12ES-0.75-1.00-44

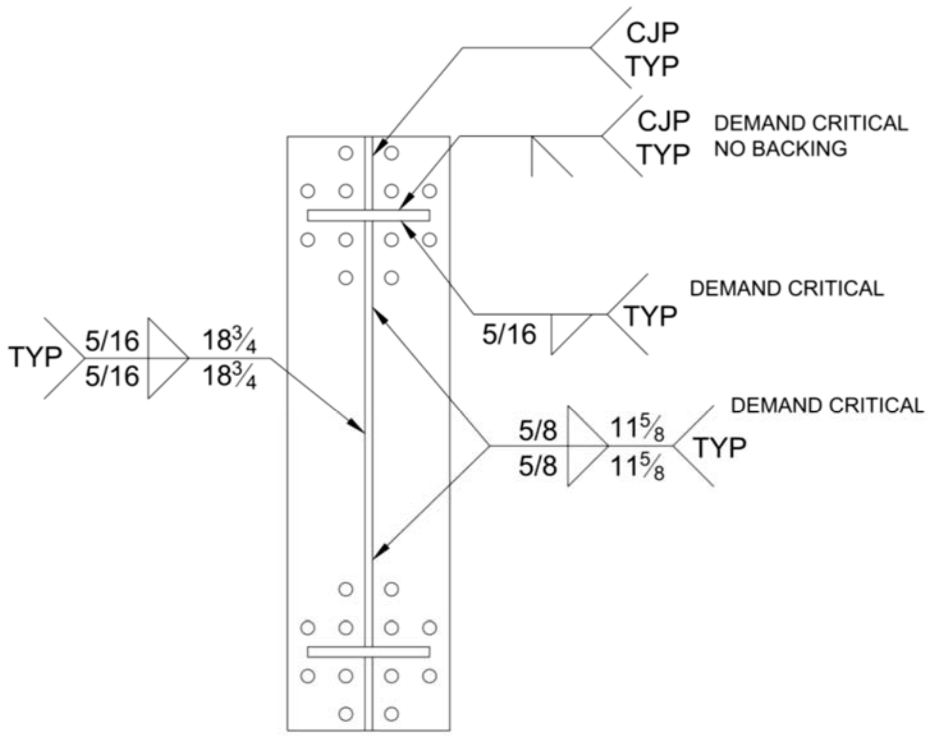


Figure 5-14 Beam to End-Plate Welds for Specimen 12ES-1.25-1.50-44

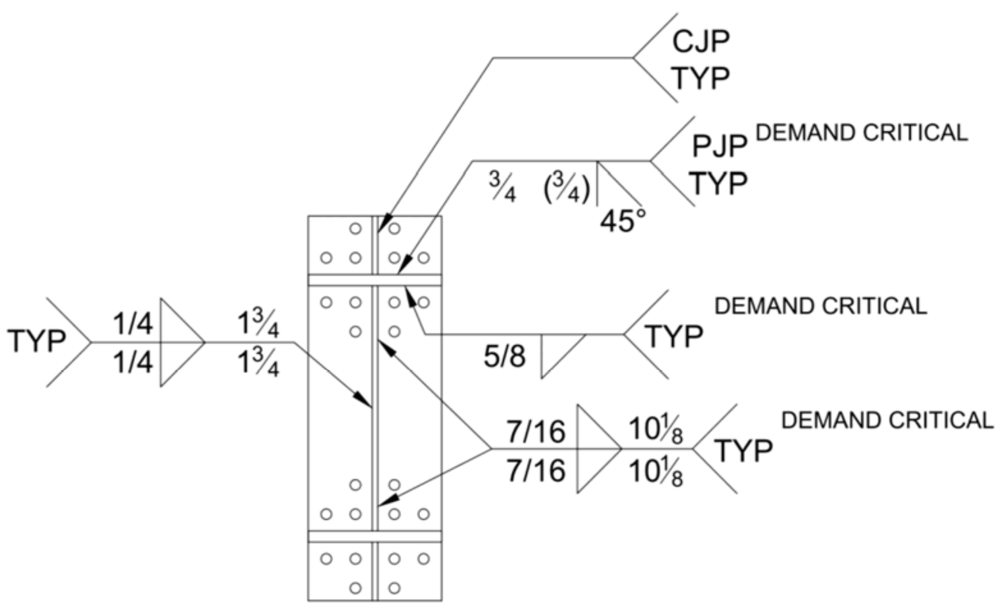


Figure 5-15 Beam to End-Plate Welds for Specimen 12ES-0.875-0.75-24

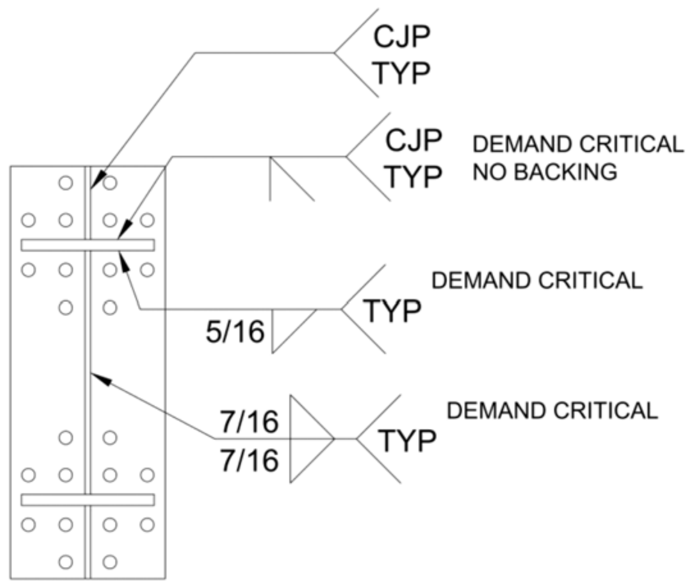


Figure 5-16 Beam to End-Plate Welds for Specimen 12ES-1.125-1.25-24

5.1.6. Specimen 12ES-1.125-1.25-24 Stiffener Correction

A correction was made to the stiffeners on Specimen 12ES-1.125-1.25-24. The stiffeners were not fabricated to match the geometry that was detailed in the design drawings. The geometry in the design drawings is the same as the geometry specified in AISC 358-10. The fabricated geometry of the stiffeners, and the correction that was made are noted in Appendix R.

5.2. Test Setup

Figure 5-17 below includes most elements of the test setup: the test frame, a beam specimen, a column specimen, the hydraulic actuators, and the lateral bracing. The instrumentation is not included in this figure. Drawings with the locations of the instrumentation can be found in the instrumentation plan in Appendix J. The test setup drawing for each specimen are in Appendix H. These tests were set up so that the specimens were in the horizontal plane. The centerline of the specimens was 22 inches from the ground. The lateral bracing was placed to prevent lateral torsional buckling, and to minimize the effects of self-weight.

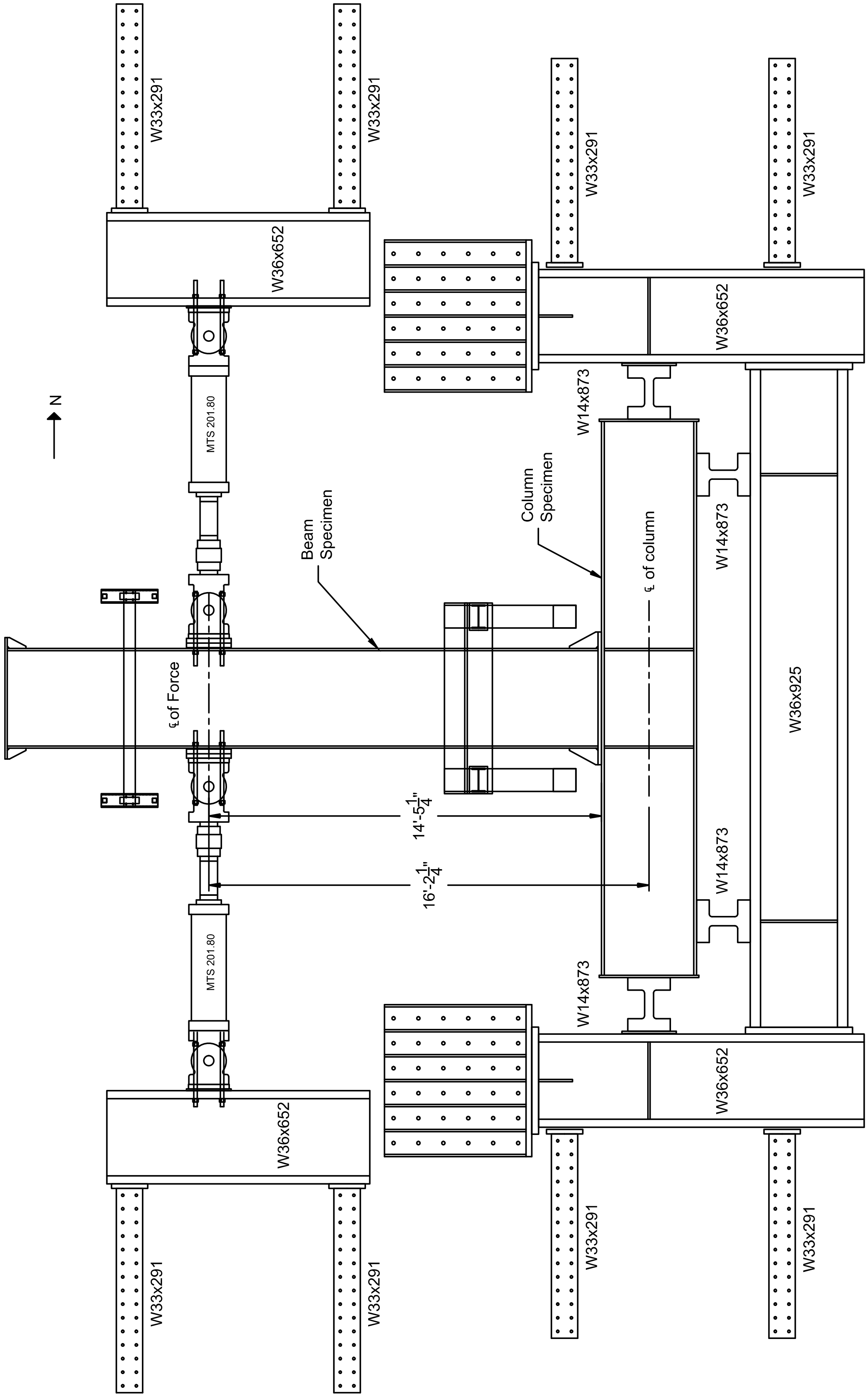


Figure 5-17 General Test Setup

5.2.1. Test Frame

The test frame is shown below in Figure 5-18, which includes the size of each member. Some modifications were made to the test frame during construction due to fit-up issues, specifically the square reaction blocks. These modifications are discussed later in this section. Using a Hydratight hydraulic torque wrench and turn-of-nut method, all bolts in the test frame were pretensioned to at least the minimum level as specified in AISC 360-10. This wrench is shown in Figure 5-19 below. This was done to eliminate the potential slip in the connections of the test frame. The members with bolt holes that run in the north-south direction were bolted to beams that were embedded in the strong floor. These strong floor beams were designed to resist loading along their longitudinal axis, which is in the north-south direction. The north-south W33x291 reaction beams were designed to resist the axial force in the column.

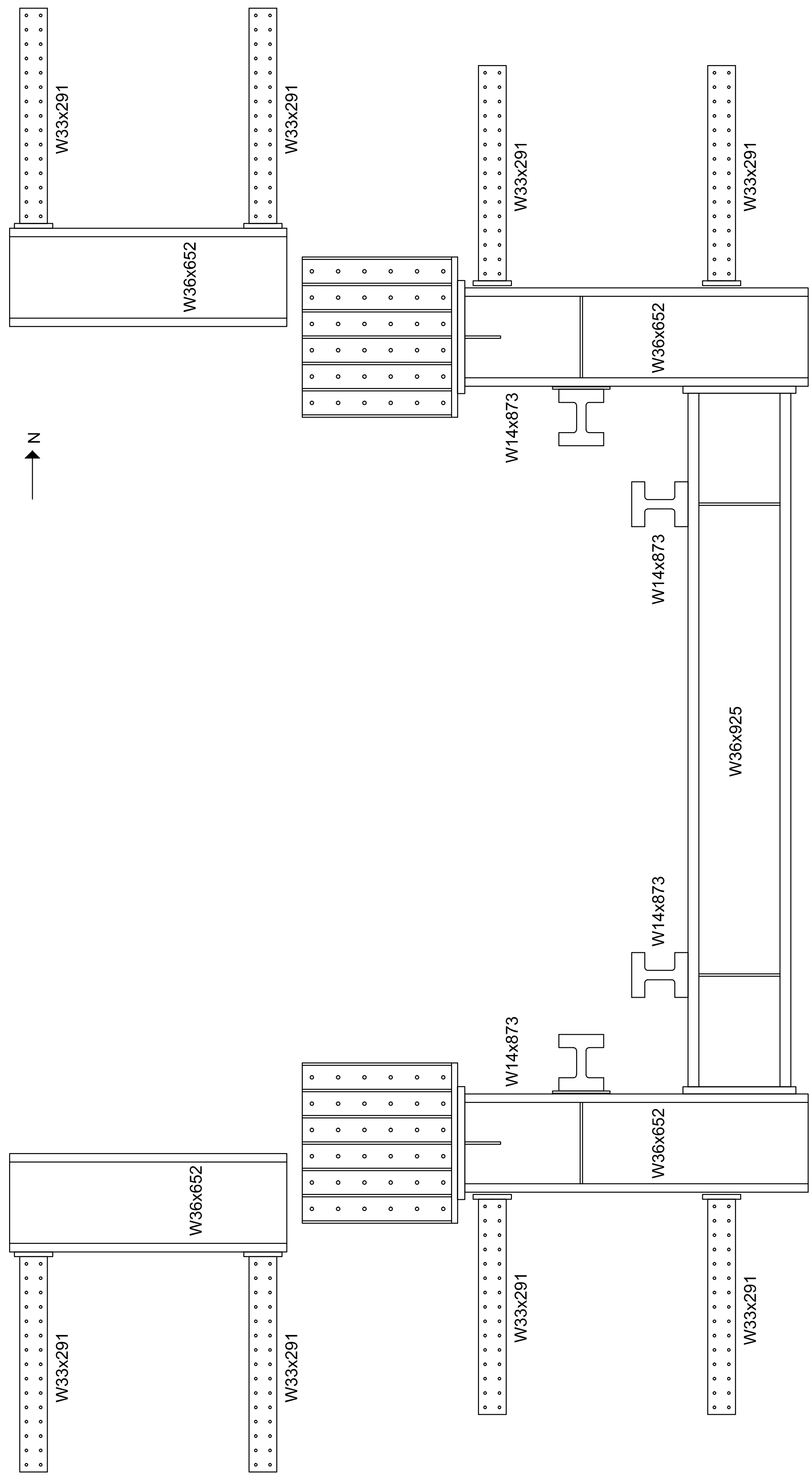


Figure 5-18 Test Frame As Designed

Figure 5-19 Hydratight Hydraulic Torque Wrench

The reaction blocks at the west end of the U-shape part of the test frame were designed to resist the shear in the column. Each block was fabricated from 2” thick steel plate and was anchored to the strong floor with 36 pretensioned 1-3/8” DYWIDAG THREADBARS. In order to anchor to the strong floor, thirty-six 3 inch diameter holes were drilled in the concrete strong floor. DYWIDAG THREADBARS were placed in the hole along with a DYWIDAG hex-nut, which was placed flush with the bottom of the threadbar. The hole with the threadbar and coupler inside was filled with high strength epoxy grout up to 9 inches below ground level. After the epoxy grout cured, an 8-3/4 inch long DYWIDAG coupler with 2-3/4 inch outer diameter was placed on the threadbar in the strong floor so that the top of the coupler did not extend above the ground level. A cut view of this embedded threadbar and coupler is shown in Figure 5-20. Another threadbar was then attached to the coupler. The threadbars extending above the ground were pretensioned to the ground to prevent the wedge reaction blocks from slipping during the tests. The threadbars were pretensioned with an Enerpac Holl-O-Cylinder center-hole jack, which is shown in Figure 5-21.

The wedge reaction block on the south end of the test frame was not sitting fully flush on the floor. To fix this, the block was raised and set on a temporary concrete bearing pad. Formwork was set around the wedge block, which was then raised a few feet above the ground.

Steel plate was placed on the ground under the wedge block to ensure the wedge block sat at the correct elevation. A cementitious grout was mixed and then pour to make the bearing pad. The reaction block was lower immediately after all of the grout was poured so that the grout was still fluid enough to voids and to allow any excess be pushed out. A picture of this reaction block is shown in Figure 5-22 below.

The wedge reaction block on the north end of the test frame did not fit in place due to an unexpected offset the W36x652 placement. This was fixed by shifting the reaction block back by a row of holes, meaning only 30 threadbars would be available to fix the reaction block to the strong floor. The remaining gap between the reaction block and the test frame was filled by two W-sections. A picture of this reaction block is shown in Figure 5-23 below.

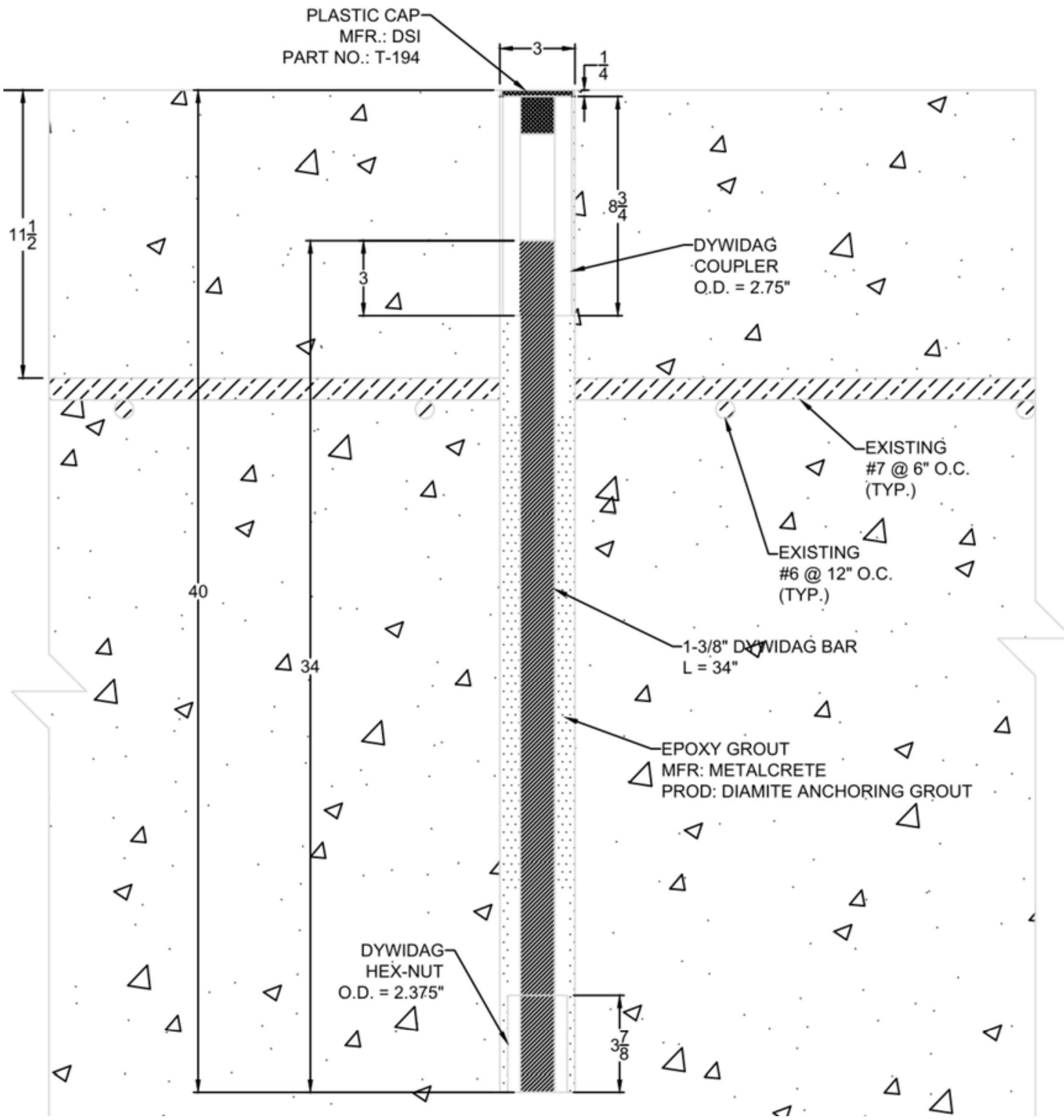


Figure 5-20 Cut View of DYWIDAG THREADBAR Embedded In Strong Floor

Figure 5-21 Enerpac Holl-O-Cylinder Center-Hole Jack

Figure 5-22 Wedge Reaction Block on South End

Figure 5-23 Wedge Reaction Block on North End

5.2.2. Lateral Bracing

The locations with a bolded "X" in Figure 5-24 below are the points of lateral bracing. The lateral bracing near the connection was required to be closer to the column for the shallow beam specimens than for the deep beam specimens. The lateral bracing at the free end remained in the same location for all specimens. The elevation view of the lateral bracing frame at the free end is shown in Figure 5-25. Since the most amount of deflection in the beam is seen at the location of this frame, the beam specimen flanges each ride on a Hilman roller, which provide a minimum level of friction. An elevation view of the lateral bracing frame at the connection end is shown in Figure 5-26 and Figure 5-27. This frame was designed so that the brace point can be shifted as needed for each specimen. There was very little deflection in the beam at this location comparatively. As a result, the frame was designed based on direct contact with the beam specimen. The coefficient of friction at the points of contact was reduced by the application of grease.

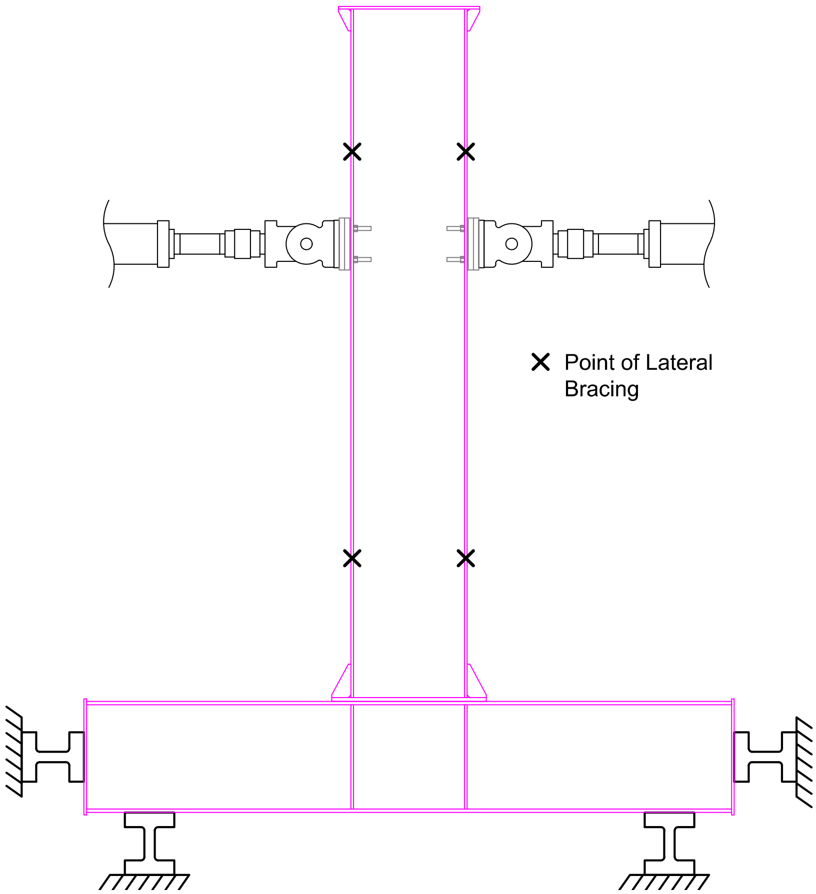


Figure 5-24 Lateral Bracing Locations

The locations of the lateral bracing frames were determined by comparing the applied moment to the nominal moment capacity of the beam controlled by the lateral torsional buckling limit state as determined in Chapter F of AISC 360-10. If the applied moment was greater than the nominal moment capacity, then the unbraced length for that region must be less than or equal to L_p. If the applied moment was less than the nominal moment capacity, then the unbraced length was increased until the nominal moment capacity controlled by lateral torsional buckling equaled the applied moment. The exact C_b was calculated for each unbraced length, if applicable, and was used appropriately. All of the calculations for determining the lateral bracing locations for each specimen are placed in Appendix I.

The lateral bracing frame at the free end and the lateral bracing frame at the connection end were both used for a previous project, which used the same test setup. Since the beams tested for that project experienced much greater moments than were expected for these tests, it was assumed that these lateral bracing frames would work for these tests.

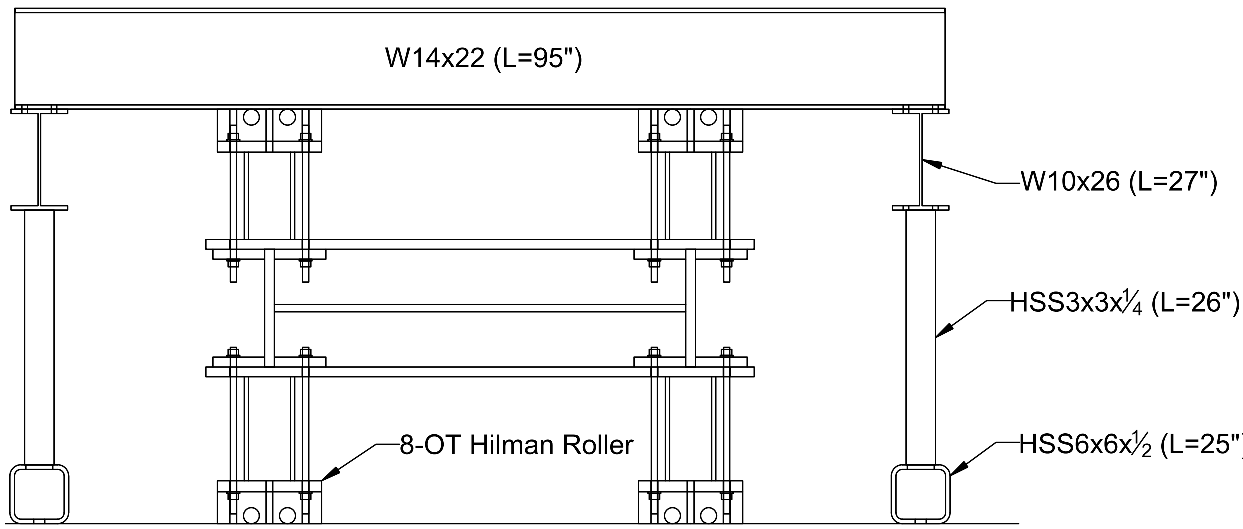


Figure 5-25 Elevation View of Lateral Bracing at Free End of Specimen

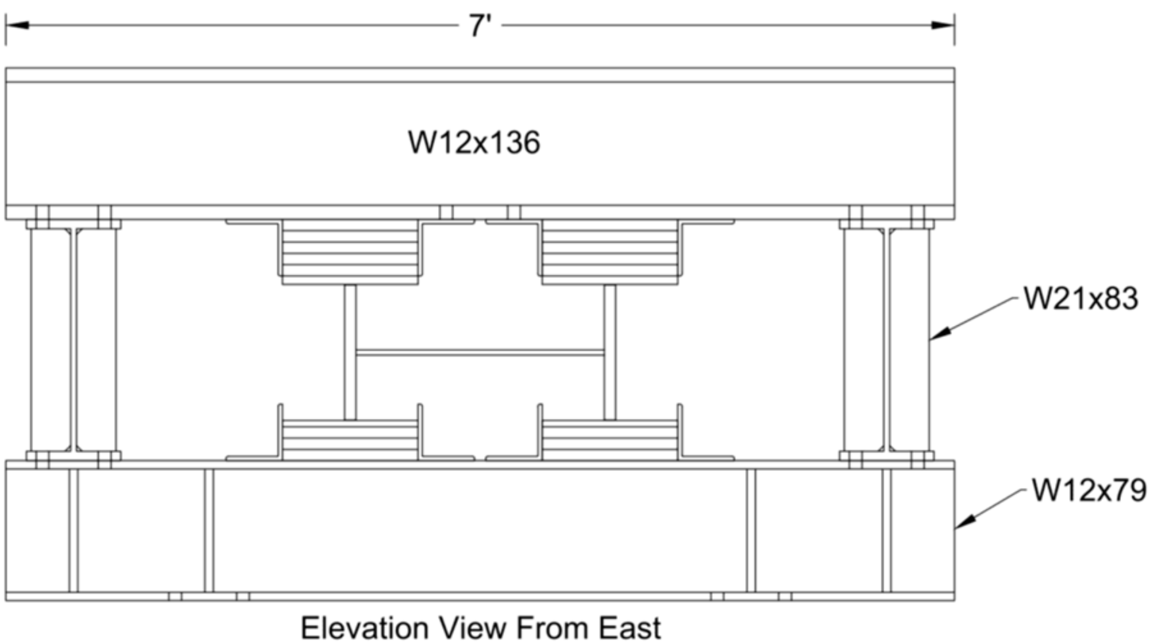


Figure 5-26 Elevation View of Lateral Bracing at Connection End

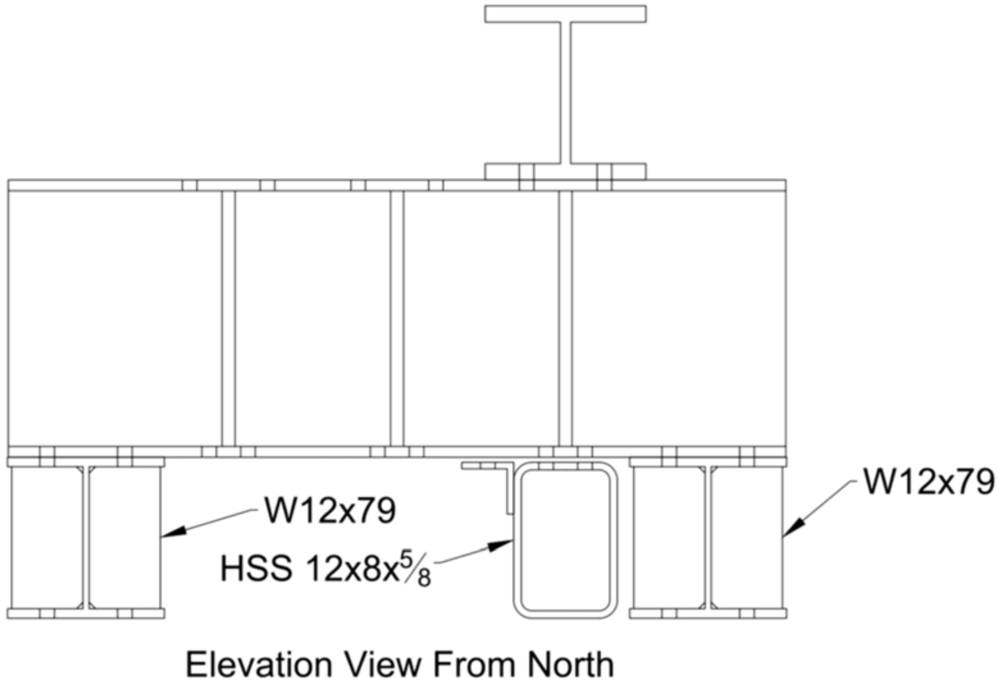


Figure 5-27 Side Elevation View of Lateral Bracing at Connection End

5.2.3. Bolt Pretensioning Process for End-Plate Moment Connection

The bolts in the end-plate moment connection for each specimen were pretensioned to at least the minimum level as prescribed in Table J3.1 of AISC 360-10 (AISC 2010b). The turn-of-nut method as detailed in the Specification for Structural Joints Using High-Strength Bolts was used as a reference for a method of properly pretensioning a bolt (RCSC 2014). However, the method used for each specimen to pretension the bolts was slightly modified for the purposes of these tests. No washers were used for any of the bolts in the end-plate moment connection for any specimen.

The process for pretensioning the bolts in the end-plate moment connection for each specimen was different. The first step for each specimen was the same, which was all of the bolts were made snug-tight with a spud wrench. A bolt was deemed snug-tight when it could not be turned any further by a spud wrench. To get all of the bolts snug-tight, it required multiple passes over each bolt with a spud wrench. The remaining steps for pretensioning of the bolts for each specimen is as follows below.

5.2.3.1. Specimen 12ES-0.75-1.00-44

After all of the bolts were snug-tight, the bolts were pretensioned by a calibrated turn-of-nut method in a specific sequence. This sequence was determined before the first specimen for consistency. This sequence is shown in Figure 5-28 below by the numbering next to each bolt.

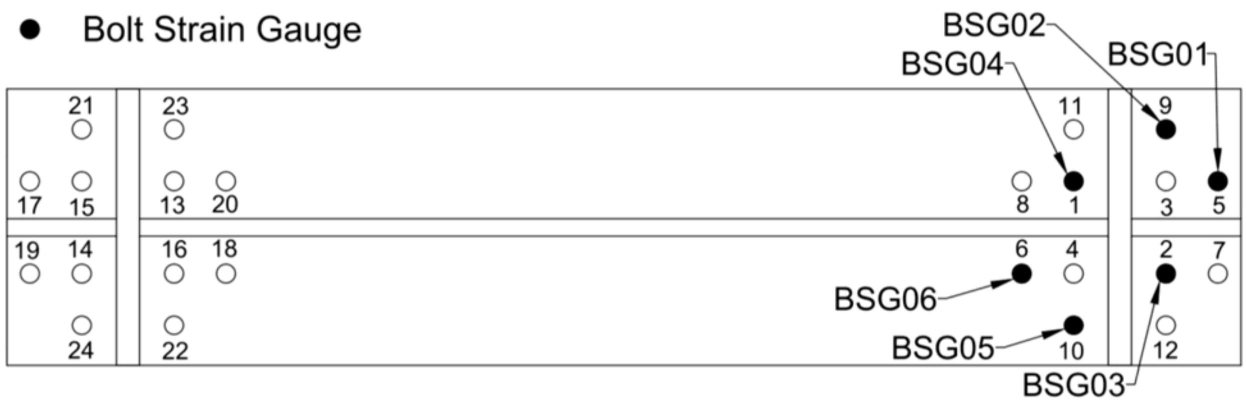


Figure 5-28 Bolt Pretensioning Sequence (View from Beam Side)

The bolts that were instrumented with bolt strain gauges were calibrated to read out the force in the bolt. These bolts allowed for a specific number of turns to be determined, which pretensioned a bolt to at least the minimum prescribed level. By starting with two bolts that were instrumented with bolt strain gauges, the required number of turns could be determined more consistently and more accurately. For Specimen 12ES-0.75-1.00-44, the bolts required about 1/4 of a turn of the nut based on the strain gauged bolts, which was a different amount of turn than prescribed by turn-of-nut method in the specification. An air impact wrench, which is shown in Figure 5-29 below, was used to obtain that 1/4 of a turn. Since nearby bolts loosen as a bolt is tightened, each bolt was checked to be snug-tight before the 1/4 of a turn was applied. Bolt 1 through Bolt 12 were pretensioned by 1/4 of a turn. Also due to that loosen effect of bolts while tightening, all of the bolts dropped below the minimum pretension level after bolt 12 was tightened by the 1/4 of a turn. The nut of Bolt 1 was turned again until the minimum pretension level was reached. The nut was turned by an indistinguishable amount, which may have been about 5 degrees of rotation. Bolt 2 reached the minimum pretension with the same amount of extra turn. For Bolt 1 through Bolt 12, the nut was turned by the approximate 5^{o} to obtain the minimum prescribed pretension level. Then, this same process was applied to Bolt 13 through Bolt 24: 1/4 of a turn and then 5° of turn.

Figure 5-29 Air Impact Wrench

5.2.3.2. Specimen 12ES-1.25-1.50-44

After all of the bolts were snug-tight, the bolts were pretensioned by a calibrated turn-of-nut method in a specific sequence. This sequence is the same as the one used for Specimen 12ES-0.75-1.00-44, which is shown above in Figure 5-28 by the numbering next to each bolt. These bolts were tightened beyond snug-tight by using the Hydratight hydraulic torque wrench, which is shown above in Figure 5-19. Bolt 1 required 1/2 turn to reach 102 kips of pretension, whereas Bolt 2 needed 3/4 turn to reach 87 kips of pretension. After Bolt 2 was tightened, Bolt 1 dropped to 80 kips. The other instrumented bolts also showed inconsistency with the amount of turn required to reach the minimum level of pretension. Due to this inconsistency, 3/8 turn was applied to the non-instrumented bolts for the first round of tightening. For the second round of tightening, the strain gauged bolts all required at least an additional 1/4 of a turn. The non-instrumented bolts received an additional amount of turn until the total turn for each bolt met the amount of turn prescribed by the turn-of-nut method as detailed in the Specification for Structural Joints Using High-Strength Bolts (RCSC 2014). After the second round, all of the bolts were deemed to be pretensioned to at least the minimum level.

5.2.3.3. Specimen 12ES-0.875-0.75-24

After all of the bolts were snug-tight, the bolts were pretensioned by a modified turn-of-nut method in a specific sequence. This sequence is the same as the one used for Specimen 12ES-0.75-1.00-44, which is shown above in Figure 5-28 by the numbering next to each bolt. The bolts were tightened by an air impact wrench, which is shown above in Figure 5-29. Before using the impact wrench, each bolt was checked to be snug-tight. Since other bolts loosen as a bolt is tightened, multiple rounds of using the air impact wrench were applied until each bolt could not be turned further. After these rounds, the instrumented bolts varied from 20 kips of pretension to 44 kips of pretension. To get the instrumented bolts to at least the minimum level of pretension, a Hydratight hydraulic torque wrench, which is shown above in Figure 5-19, was used. While these bolts were tightened further by the hydraulic torque wrench, the pretension force in neighboring bolts did not drop. After these bolts were fully pretensioned, the number of turn applied to these bolts varied from 1/3 of a turn to 2/3 of a turn. Due to this inconsistency, the other bolts which were non-instrumented were turned by the hydraulic torque wrench until they met the amount of turn prescribed by the turn-of-nut method as detailed in the Specification for Structural Joints Using High-Strength Bolts (RCSC 2014).

5.2.3.4. Specimen 12ES-1.125-1.25-24

The same process that was used for Specimen 12ES-0.875-0.75-24 for pretensioning the bolts in the end-plate moment connection was also applied to this specimen. This process seemed to be the quickest and most consistent method. After all of the bolts were snig-tight, the bolts were tightened by an air impact wrench, which is shown above in Figure 5-29, in the sequence shown above in Figure 5-28. After multiple rounds of tightening, the bolts could not be tightened any further by the air impact wrench. After these rounds, most bolts were turned about 1/4 of a turn after snug tight. A few bolts were turned 1/8, 1/3, or 1/2 of a turn. The strain gauged bolts were all turned by 1/4 of a turn, but varied from 32 kips to 52 kips of pretension. The Hydratight hydraulic torque wrench, which is shown above in Figure 5-19, was used to tighten the bolts further. The strain gauged bolts were first tightened to the minimum prescribed level. The total amount of turn required for each of these bolts varied significantly. For that reason, the non-instrumented bolts were turned by the hydraulic torque wrench until they met the amount of turn prescribed by the turn-of-nut method as detailed in the Specification for Structural Joints Using High-Strength Bolts (RCSC 2014).

5.3. Instrumentation

A variety of instrumentation was used during the tests. The instrumentation that was used for each test included 14 string potentiometers, 1 linear potentiometer, 6 outside spring calipers, 24 strain gauges, and 6 bolt strain gauges. This total is based on instrumentation that was placed on the test frame and instrumentation that was placed on the specimen. A diagram of the instrumentation locations is shown below in Figure 5-30. The full instrumentation plan is placed in Appendix J. The data acquisition system that was used included: one National Instruments SCXI-1001 chassis, two National Instruments SCXI-1520 universal strain gauge input modules, two National Instruments SCXI-1314 front-mounting terminal blocks, and two National Instruments SCXI-1317 front-mounting terminal blocks.

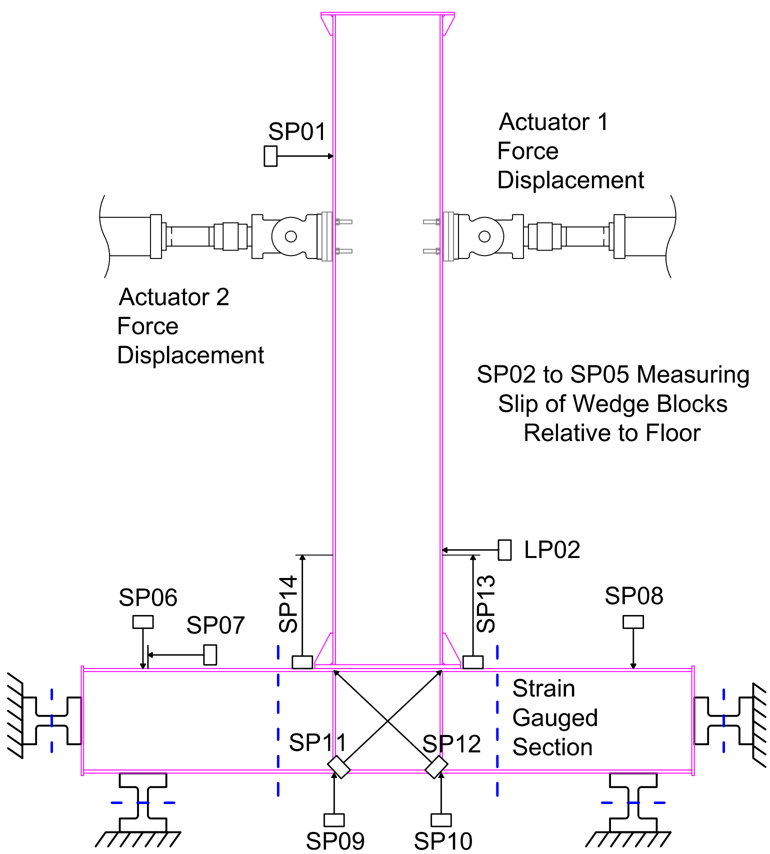


Figure 5-30 Instrumentation Plan

The strain gauges that were used were TML WFLA-6-11. On each of the four W14x873 stub sections which framed the column specimen, four strain gauges were placed on their web. These strain gauges were used to measure the axial force in the W14x873. Two of the stub sections were designed to take the axial force in the column and the other two sections were designed to take the shear in the column. On each side of the beam on the column, two strain gauges were placed on the outside of each flange. This totaled to 8 strain gauges on the column. These strain gauges were used as another way to measure the shear in the column. The procedure that was used to calibrate the data acquisition system for strain measurements can be found in Appendix K. Bolt strain gauges were also used. The bolt strain gauges that were used were TML BTM-6C. Each specimen had six bolts that were each instrumented with a bolt strain gauge. The bolt strain gauges were embedded with an adhesive in the shank of the bolt by inserting them in a hole drilled in the center of the bolt. An instrumented bolt is shown below in Figure 5-31.

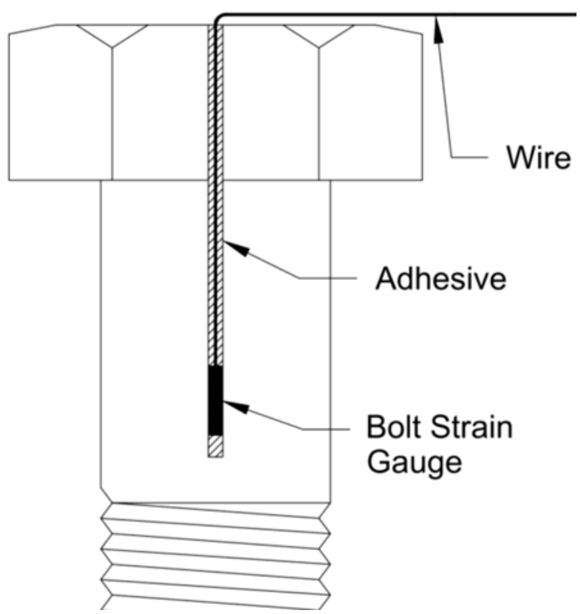


Figure 5-31 Cut View of Bolt Instrumented With Bolt Strain Gauge

Outside spring calipers were used to measure end-plate separation. Six calipers were used for each specimen, and the calipers were placed at different locations. These locations were relatively consistent between specimens. There were 4 large calipers and 2 larger calipers. The number of calipers used and the placement of these calipers was controlled by the column flange width and the reach of the calipers. The column flange width was greater than any of the end-plate widths. The reach of the larger calipers was about 5 inches, and the reach of the small caliper was about 2-1/2 inches. An example of a large caliper and a small caliper is shown in Figure 5-32. A diagram of the bolt strain gauge locations and the caliper location is shown in Figure 5-33.

Any potential movement of the test frame and specimen was captured by string potentiometers. The string potentiometers that were used were Celesco PT101. Two string potentiometers were place on each wedge reaction block to measure any potential slip. On each block, one string potentiometer measured in the north-south direction and the other one measured in the east-west direction. One string potentiometer was placed in-line with each of the W14x873 stub sections, which attached to the column. These string potentiometers measured separation of the column from the stub sections. Another string potentiometer was placed near the end of the column to measure movement of the column in the north-south direction. Four string potentiometers were placed in the panel zone to measure the panel zone deformation. A string potentiometer was place on the outside of each flange of the beam to measure the extension in the plastic hinge region. The rotation of the plastic hinge region was captured by a linear potentiometer, which measured in the north-south direction, perpendicular to the other two string potentiometers. The linear potentiometer that was used was a Celesco CLP150. The last string potentiometer was placed at the end of the beam near the actuators to measure the total deflection of the specimen. Each actuator includes an LVDT, which validates the value of the string potentiometer.

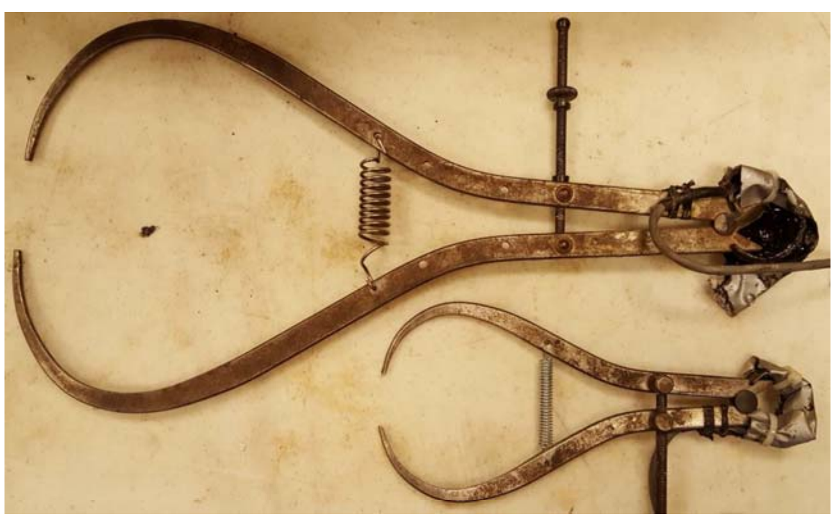


Figure 5-32 Large and Small Outside Spring Calipers

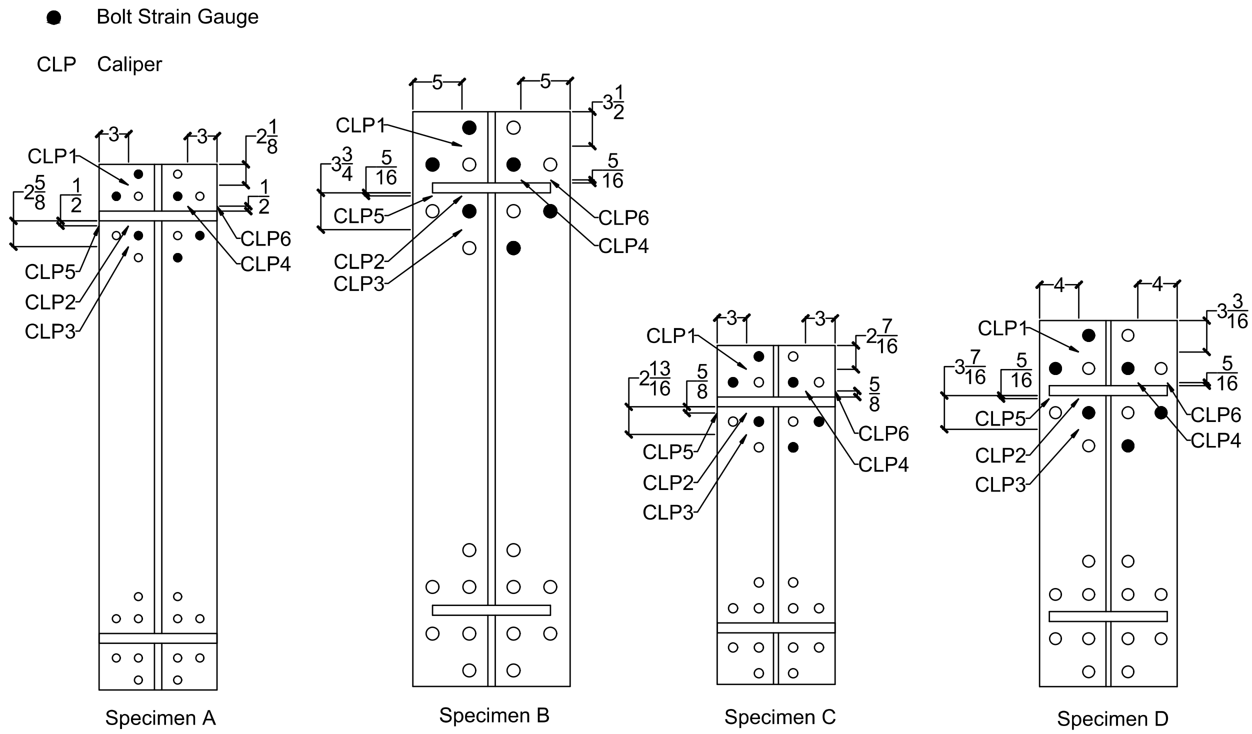


Figure 5-33 Bolt Strain Gauge and Caliper Locations

5.3.1. Calibration of Outside Spring Calipers

There were a total of six outside spring calipers: four large and two small. Each caliper was instrumented with a strain gauge, which allowed for the caliper to be digitally calibrated for separation of the tips. The range of separation of the caliper tips was calculated. The minimum separation was based on the minimum sum of the end-plate thickness and the column flange thickness between all specimens. The maximum separation was based on the maximum sum of the end-plate thickness, the column flange thickness and an end-plate separation of 3/8 inch across all specimens. This led to a range of separation of 2 inches to 3-1/8 inches. For each caliper, strain readings were recorded at 1/8 inch increments across the range using machined steel parallels. A strain reading was recording at each increment going from minimum to maximum separation. The process was repeated going from maximum to minimum separation. The plot of separation versus strain was linear over this range for each caliper.

5.3.2. Calibration of Bolt Strain Gauges

Several bolts per specimen were fitted with bolt strain gauges. The strain readings for each bolt were calibrated for force so that bolt force could be recorded during the testing. The bolt strain gauges that were used were TML BTM-6C. The procedure for inserting the bolt strain gauges with TML A-2 adhesive for BTM strain gauges was done according to the documentation provided in the packaging of the bolt strain gauges. After all of the strain gauges were inserted in the bolts and the adhesive was cured, each bolt was loaded to 50% of R_n of the bolt. This was considered a significant range, but low enough that the bolt remained elastic. The bolts were loaded using a SATEC 300 kip Universal Testing Machine. During loading, the strain and corresponding force was recorded. This load versus stain data was plotted, which was linear for each strain gauged bolt. A trendline was fitted to the data for each strain gauge. The slope of the trendline was used to convert the strain to force. This allowed for the forces in the bolts to be recorded during testing.

5.3.3. Story Drift Angle Decomposition

In order to prequalify a beam-to-column connection, the connection must be able to sustain at least one full cycle of loading at a certain story drift angle after a prescribed loading protocol. This loading protocol and the requirements for prequalification are detailed in the AISC *Seismic Provisions for Structural Steel Buildings* and is discussed in the next section of this document. For special moment frames (SMF), this required story drift angle is 0.04 radians. For intermediate moment frames (IMF), this required story drift angle is 0.02 radians. It is important that the sources of the story drift angle are determined and the percentage of the total story drift provided by each source. The amount of inelastic rotation provided by the connection must be within 25% of the expected amount of inelastic rotation. This means that for special moment frames, the connection must account for at least 0.03 radians of the total story drift angle. For intermediate moment frames, the connection must account for at least 0.015 radians of the total story drift angle. The components of these tests that contribute to story drift include:

- Movement of the reaction frame: $\alpha_{col,NS}$ & $\alpha_{col,EW}$
- Rigid body movement of the column within the reaction frame: $\alpha_{col,NS}$ & $\alpha_{col,EW}$
- Shear deformation of the panel zone: α_{PZ}
- Flexural deformation of the column and panel zone: α_{CF}
- Separation of end-plate from column flange: α_{SEP}
- Elastic flexural and shear deformation of the beam outside the plastic hinge region: α_{el}

- Deformation of the plastic hinge region of the beam: α_{PH}

The following sections describe how these sources listed above were accounted for in the tests via instrumentation. The data recorded from the instrumentation was used by way of the equations listed below. Most of these equations were based on a small angle assumption, which means that the angles were small enough that the ratios used were essentially equal to the appropriate trigonometric function. The positive sign convention for the story drift angle is shown below in Figure 5-34, where α is the angle of rotation in radians. The equations below describing each source of story drift are expressed according to this positive sign convention.

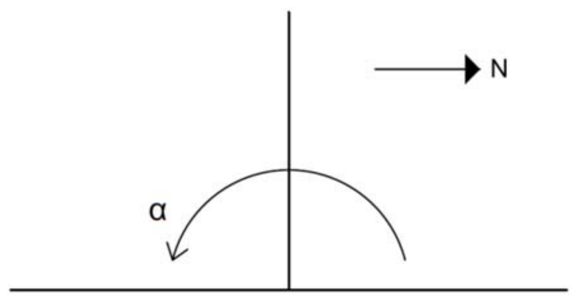


Figure 5-34 Story Drift Sign Convention

i) Movement of reaction frame and rigid body movement of the column within the reaction frame

Although all of the bolted connections were pretensioned, there was still a possibility that there could be movement in the connections during testing; whether it was slip or separation. String potentiometers were attached to the big wedge reaction blocks to capture the slip of these blocks. By capturing the movement of these reaction blocks, it was assumed that other movement of the reaction frame as a unit would also be captured. String potentiometers were also attached to the column to record the movement of the column as a rigid body at the location of the connections to the reaction frame. In terms of story drift decomposition, the measurements from these string potentiometers included both the movements of the column within the reaction frame and the movement of the reaction frame as a unit. Because of this, the measurements from the string potentiometers attached to the wedge reaction blocks were ignored for story drift decomposition. The movement of the wedge reaction blocks was still monitored though. The calculation of the story drift angle due to the movement of the column in the north-south direction is expressed below by Eq. 5.1. The calculation of the story drift angle due to the movement of the column in the east-west direction is expressed below by Eq. 5.2. Each variable in these equations is depicted below in Figure 5-35.

$$\alpha_{col,NS} = \frac{-\Delta SP_7}{L_{CL}} \qquad \text{(Eq. 5.1)}$$

L_{CL} is the distance from the string potentiometer at the end of the beam to the centerline of the column.

$$\alpha_{col,EW} = \frac{\Delta SP_6 - \Delta SP_8}{L_{bC}} \qquad \text{(Eq. 5.2)}$$

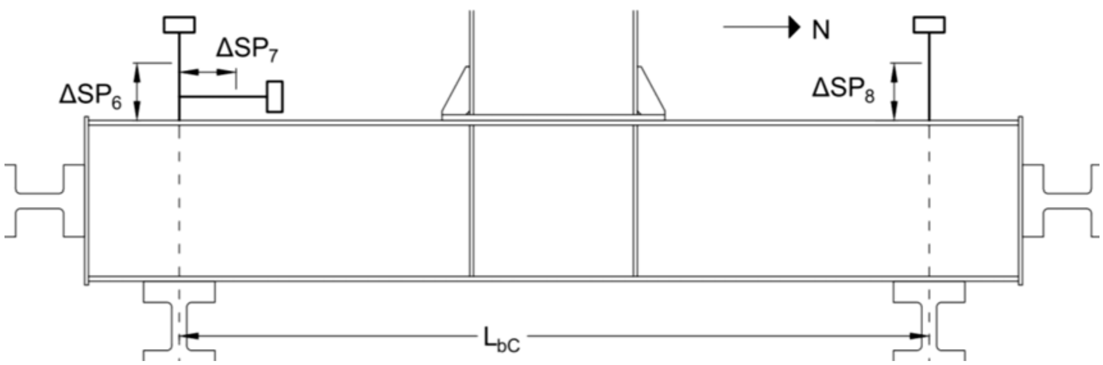


Figure 5-35 Sign Convention for Separation of Column from Reaction Frame

ii) Shear deformation of the panel zone

The panel zone of the column is instrumented with two string potentiometers, which were placed on the opposite side of the column from the beam. The string of each string potentiometer attached to the column flange at the opposite corner of the panel zone, which forms an “X”. This configuration can be seen below in Figure 5-36, in which the panel zone is framed by the dashed lines. This same rectangle with dashed lines is shown in Figure 5-37 below as the undeformed shape of the panel zone. Next to it is the assumed deformed shape of the panel zone, which gives a positive story drift angle. The aligned dimensions of each side of the deformed shape were assumed to be the same as the dimensions of the undeformed panel zone.

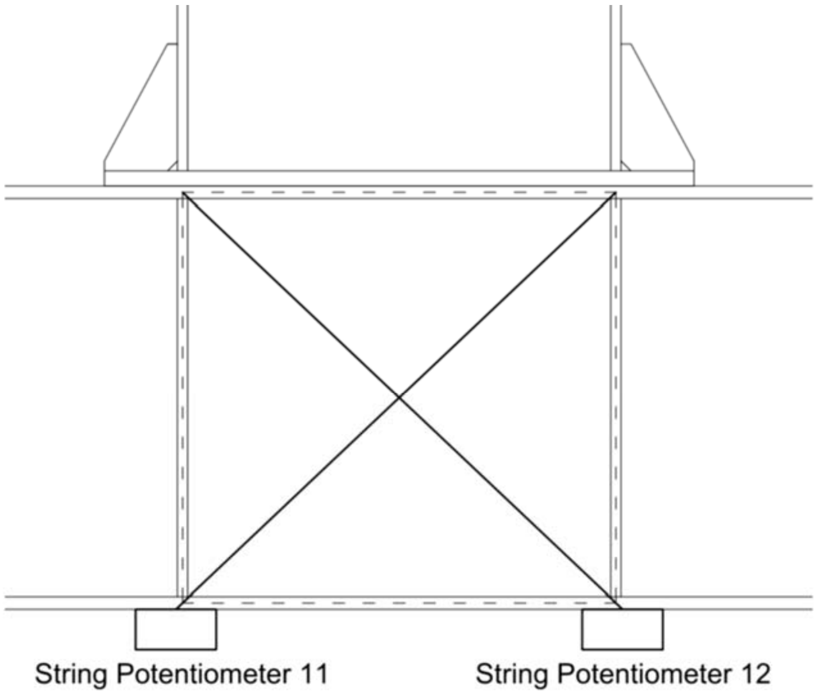


Figure 5-36 Instrumented Panel Zone

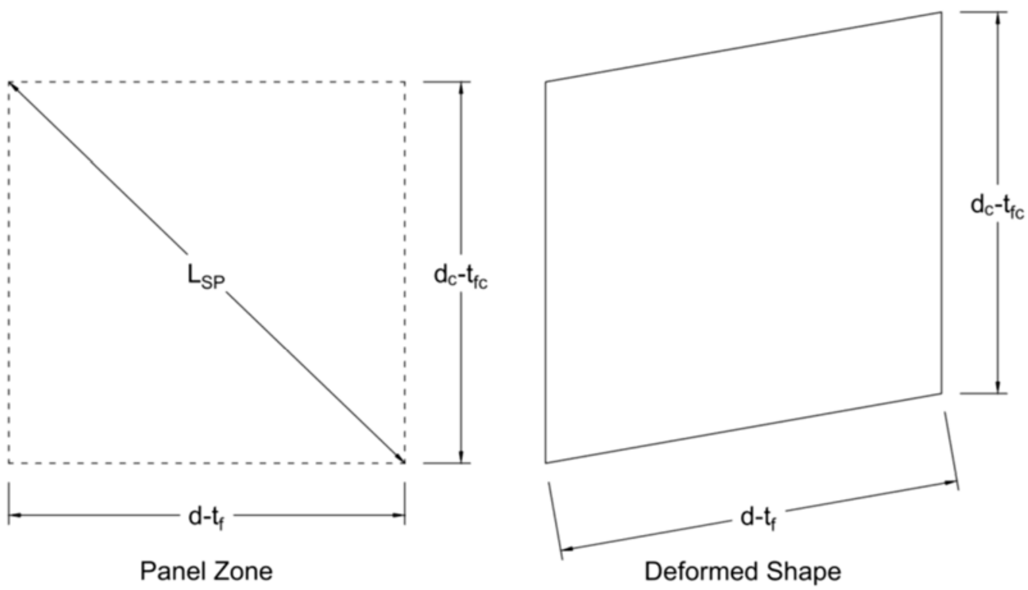


Figure 5-37 Assumed Panel Zone Geometry

Each string divides the panel zone into two separate triangles. Given that two sides of each of these triangles are known since they are assumed to remain constant, and if the length of the diagonal is known, then angle interior angle of each triangle can be calculated via the law of cosines. Figure 5-38 below depicts one of the triangles formed by each string potentiometer in the deformed shape of the panel zone. These triangles represent the case of positive story drift. The dashed lines in Figure 5-38 correspond to the undeformed shape of the panel zone. The length of the undeformed diagonals, L_{SP}, can be calculated by using Pythagoras' theorem, which is shown below in Eq. 5.3.

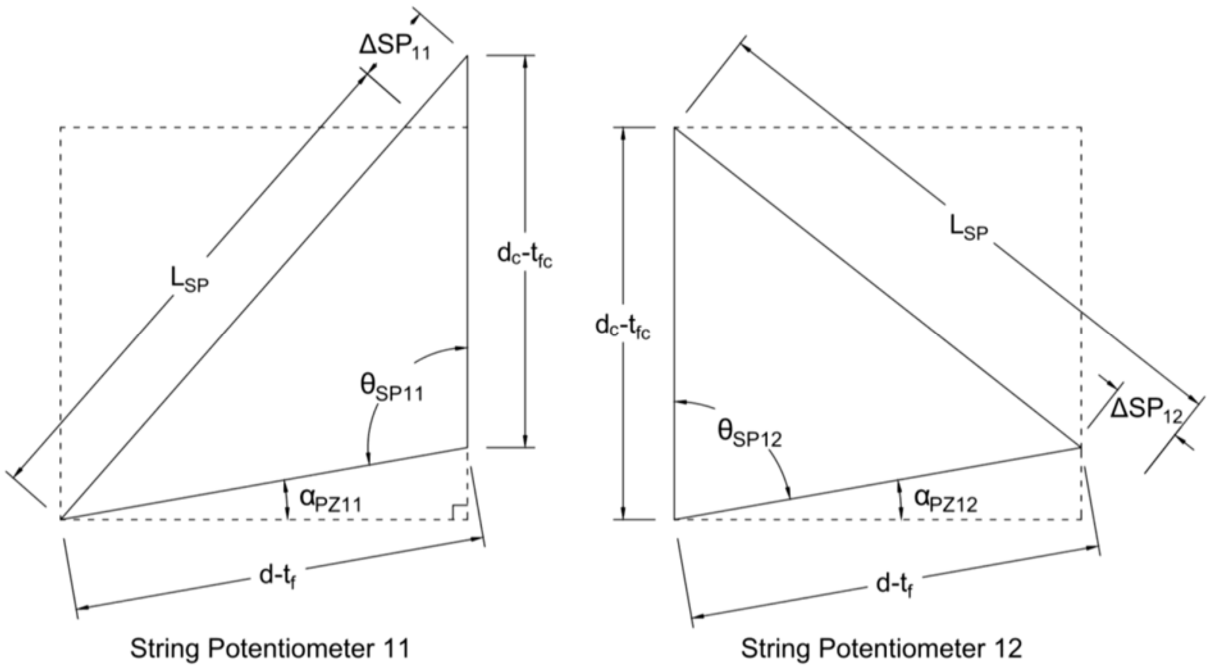


Figure 5-38 Measuring Panel Zone Story Drift

$$L_{SP} = \sqrt{\left(d_c - t_{fc}\right)^2 + \left(d - t_f\right)^2} \qquad \text{(Eq. 5.3)}$$

The change in length of the diagonals was measured by the string potentiometers. ΔSP_{12} is the change in length of string potentiometer 12 and ΔSP_{11} is the change in length of string potentiometer 11. The new length of each diagonal is the sum of the original length and the change in length. A negative change in length corresponds to shortening of the string potentiometer string. In the case shown, string potentiometer 12 is elongated and string potentiometer 11 is shortened. The law of cosines is applied to calculate an interior angle for string potentiometer 12 in Eq 5.4 below and for string potentiometer 11 in Eq. 5.6 below. These interior angles are used to calculate the story drift angle based on each string potentiometer in Eq

5.5 and Eq. 5.7 respectively. Theoretically, α_{PZ12} and α_{PZ11} should be equal, thus they can be used to as comparison against each other. Due to error that inherently exists, these two values were expected to differ, and consequently were averaged to calculate the component of the story drift angle due to panel zone shear.

$$\theta_{SP11} = \text{acos}\left(\frac{(d_c-t_{fc})^2+(d-t_f)^2-(L_{SP}+\Delta SP_{11})^2}{2(d_c-t_{fc})(d-t_f)}\right) \qquad \text{(Eq. 5.4)}$$

$$\alpha_{PZ11} = \theta_{SP11} - \frac{\pi}{2} \qquad \text{(Eq. 5.5)}$$

$$\theta_{SP12} = \text{acos}\left(\frac{(d_c-t_{fc})^2+(d-t_f)^2-(L_{SP}+\Delta SP_{12})^2}{2(d_c-t_{fc})(d-t_f)}\right) \qquad \text{(Eq. 5.6)}$$

$$\alpha_{PZ12} = \frac{\pi}{2} - \theta_{SP12} \qquad \text{(Eq. 5.7)}$$

$$\alpha_{PZ} = \frac{\alpha_{PZ11}+\alpha_{PZ12}}{2} \qquad \text{(Eq. 5.8)}$$

iii) Flexural deformation of the column and panel zone

Movement of the column as a rigid body, and deformation of the column due to shear has been accounted for. Since the column is resisting the moment at the end of the beam, the column also deforms flexurally. This deformation was accounted for by two string potentiometers, which were placed on the outside of the column flanges opposite from the beams. Each string was attached to the column flange at the centerline of each continuity plate. The placement of the string potentiometers is shown below in Figure 5-39. The component of the story drift angle due to flexure in the column can be calculated as the difference between the deflections measured by each string potentiometer divided by the distance between the string potentiometers. The measurements from these string potentiometers also include the rigid body motion of the column and the shear deformation in the panel zone. As a result, these components of the story drift angle must each be subtracted out as shown below in Eq. 5.9, which gives the portion of the story drift angle due to the flexural deformation of the column.

$$\alpha_{CF} = \frac{\Delta SP_{10} - \Delta SP_9}{d - t_f} - \alpha_{col,EW} - \alpha_{PZ} \qquad \text{(Eq. 5.9)}$$

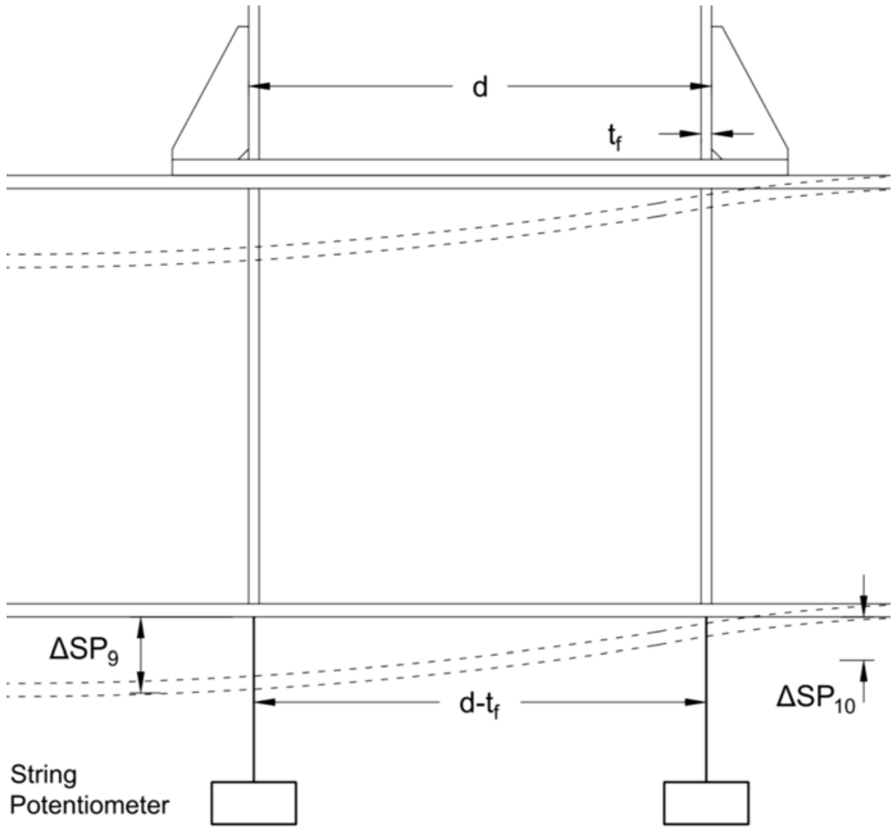


Figure 5-39 Contribution of Column Flexure to Story Drift

iv) Separation of end-plate from column flange

The bolts connecting the end-plate to the column were pretensioned to at least the minimum level as specified by the AISC *Specification for Structural Steel Buildings*. At low levels of moment, the pretension in the bolts would be enough to prevent any separation of the end-plate from the column. Eventually, the moment would increase to a level where the force in the bolts exceeds the pretension, leading to end-plate separation. This separation is a component that contributes to the total story drift angle. On each specimen, at least one outside spring caliper was place at the inside face of the beam flange. With the measurement from the caliper, a story drift angle can be computed as shown below in Eq. 5.10, which applies the small angle assumption. The location of the caliper, and a depiction of the end-plate separating from the column flange are shown below in Figure 5-40. The horizontal distance, $d\text{-}3t_f/2$, is based on the assumption that the end-plate is rotating about the centerline of the opposite flange from the caliper. This same assumption is also applied to the yield line derivation of the end-plate, as shown in Appendix N.

$$\alpha_{SEP} = \frac{-\Delta CLP}{d - \frac{3t_f}{2}} \qquad \text{(Eq. 5.10)}$$

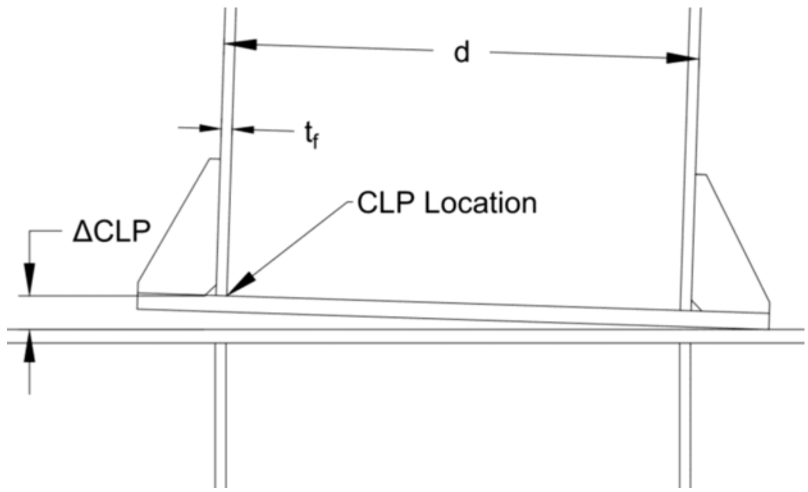


Figure 5-40 Separation of End-Plate from Column Flange

v) Elastic deformation of the beam outside the plastic hinge region

At any level of load, the beam is expected to have some elastic portion of the deformation. After a certain point, the beam would also experience plastic deformations near the connection. The region of the beam outside the plastic hinge region at the free end of the cantilever would be undergoing only elastic deformations, which is depicted below in Figure 5-41. The flexural deformation of the beam over the elastic region can be calculated using the load as shown below in the first term of Eq. 5.12. The deformation due to shear as calculated by Timshenko is the second term of Eq. 5.12. In this equation, α is a shape factor, which is calculated by using Eq. 5.11 below. This deflection can be used to calculate the component of the story drift angle due to elastic deformations of the beam outside the plastic hinge region as shown below in Eq. 5.13.

$$\alpha = \frac{A}{8I_x t_w}\left(b_f d^2 - b_f h_w^2 + t_w h_w^2\right) \qquad \text{(Eq. 5.11)}$$

"A" is the cross-sectional area of the beam. "I_x" is the moment of inertia of the beam about the x-axis. "t_w" is the thickness of the beam web. "b_f" is the beam flange width. "d" is the total beam depth. "h_w" is the clear distance between the beam flanges.

$$\delta_{el} = \frac{PL_{el}^3}{3EI} + \frac{P(L_{el})\alpha}{AG} \quad \text{(Eq. 5.12)}$$

$$\alpha_{el} = \frac{\delta_{el}}{L_{CL} - L_{PSP}} \quad \text{(Eq. 5.13)}$$

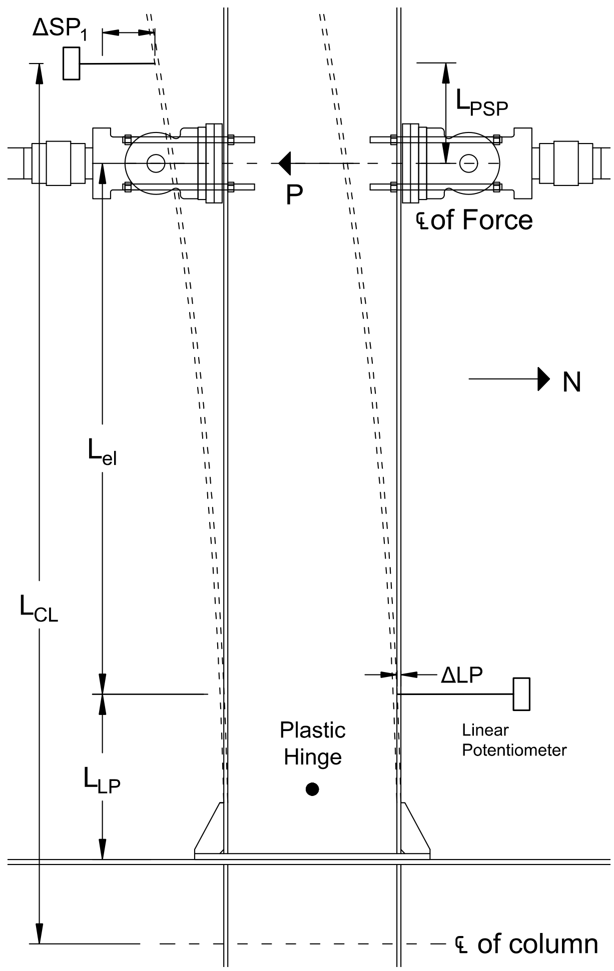


Figure 5-41 Contribution of Elastic Beam Deformation to Story Drift

vi) Plastic deformation of the beam

The elastic portion of the beam deformation has been accounted for only in the region outside of the plastic hinge. Some of the specimens were expected to fail due to plastic hinging of the beam. For extended stiffened end-plate moment connections, the plastic hinge is expected to form at the end of the stiffeners. To capture this deformation, two string potentiometers were positioned on the column flanges and extended to attach to the beam at a distance of 'd' from the outside of the column flange, as shown below in Figure 5-42. As one spring potentiometer extends, the other one should retract. This component of story drift angle can be calculated by the difference of the change in lengths of the string potentiometers divided by the distance between the string potentiometers. This is shown below in Eq. 5.14. In this region of the beam from the column face to where the string potentiometers attach to the beam, these string potentiometers are actually measuring the plastic deformation of the beam, the elastic deformation of the beam and the end-plate separation from the column. The end-plate separation is factored into Eq. 5.14 below. These string potentiometers attached to the beam at the same location where the linear potentiometer attached to the beam for the measurement of the elastic deformation of the beam outside the plastic hinge region.

$$\alpha_{PH} = \frac{\Delta SP_{13} - \Delta SP_{14}}{d_{SP}} - \alpha_{SEP} \qquad \text{(Eq. 5.14)}$$

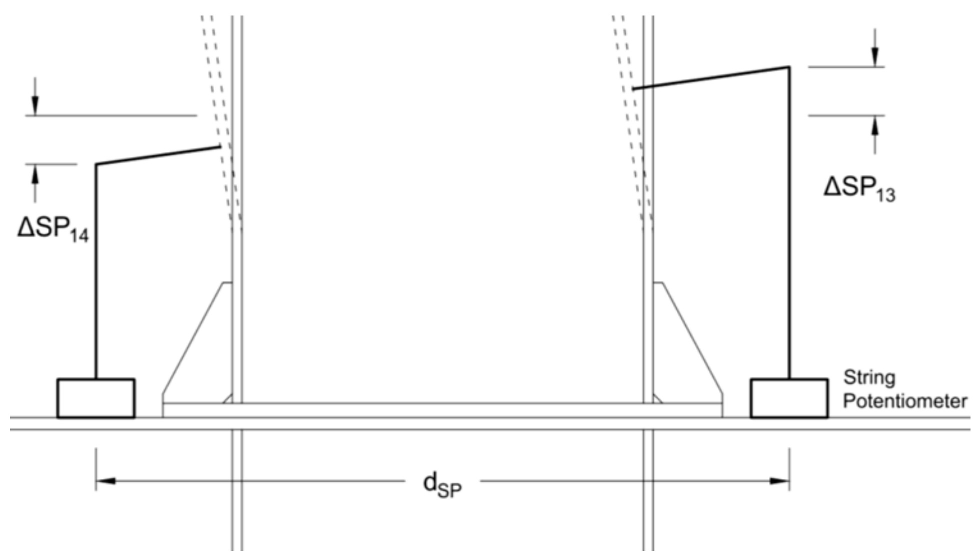


Figure 5-42 Contribution of Plastic Deformation of Beam to Story Drift

vii) Total story drift angle

Each of the terms detailed above are summed together to equal the total story drift angle, which is expressed below in Eq. 5.15. The total story drift can also be calculated by the ratio of the deflection measured by the string potentiometer at the end of the beam to the distance from that string potentiometer to the centerline of the column. This ratio is expressed below by Eq. 5.16, which could be used for comparison against Eq. 5.15.

$$\alpha_{total} = \alpha_{col,NS} + \alpha_{col,EW} + \alpha_{PZ} + \alpha_{CF} + \alpha_{SEP} + \alpha_{el} + \alpha_{PH} \qquad \text{(Eq. 5.15)}$$

$$\alpha_{total,actual} = \frac{-\Delta \mathrm{SP}_1 - \Delta SP_7}{L_{CL}} \qquad \text{(Eq. 5.16)}$$

ΔSP_1 is the maximum deflection recorded by the string potentiometer at the free end of the beam.

viii) Story drift checks

The calculated total story drift angle is compared to the actual total story drift angle. This is important, however, there are a lot of sources that contribute to the total story drift angle. Checking the accuracy of these components is important. Ensuring the accuracy of the components improves the accuracy of the calculated story drift angle. This first check can be used for checking the elastic deformations of the column. Since the load is at a low level, the rigid body motion of the column is assumed to be zero. Given the load, P, the following variables can be calculated: the moment at the centerline of the column at the intersection with the beam, M_{CL}, the rotation of the column due to this moment at the same location, θ, and the moments, M_1 and M_2. These moments are located at the strain gauges placed on the outside of the column flanges at a distance of ‘d’ from the centerline of the beam. These variables are depicted below in Figure 5-43. In this figure, the dashed lines are the deflected shape of the beam and column for a positive story drift angle. The locations of the strain gauges are shown in the instrumentation plan in Appendix J. For the cycles of small applied beam displacements, these values can then be used to check ΔSP_9, ΔSP_{10}, ΔSP_1, and the strains at the locations of M_1 and M_2, $\varepsilon_{M1,check}$ and $\varepsilon_{M2,check}$ respectively. The equations for checking these instrumentation readings are listed below as Eq. 5.17 through Eq. 5.23. Due to symmetry, the magnitudes of M_1 and M_2 are equal. Also due to symmetry of the placement of the strain gauges on the outside of the column flanges, magnitudes can be used to check the strains, $\varepsilon_{M1,check}$ and $\varepsilon_{M2,check}$. The small angle assumption was applied in Eq. 5.19, Eq. 5.20 and Eq. 5.21.

$$M_{CL} = P(L_{CL} - L_{PSP}) \quad \text{(Eq. 5.17)}$$

$$\theta = \frac{M_{CL} L_{bC}}{12 E I_{xc}} \quad \text{(Eq. 5.18)}$$

I_{xc} is the moment of inertia of the column.

$$\Delta SP_{9,check} = -\theta \left(\frac{d - t_f}{2}\right) \quad \text{(Eq. 5.19)}$$

$$\Delta SP_{10,check} = \theta \left(\frac{d - t_f}{2}\right) \quad \text{(Eq. 5.20)}$$

$$\Delta SP_{1,check} = \frac{P(L_{CL} - L_{PSP})^2}{6 E I_x}(2L_{CL} + L_{PSP}) + L_{CL}\theta \quad \text{(Eq. 5.21)}$$

I_x is the moment of inertia of the beam.

$$\left|M_{1,check}\right| = \left|M_{2,check}\right| = \frac{|M_{CL}|\left(\frac{L_{bC}}{2} - d\right)}{L_{bC}} \quad \text{(Eq. 5.22)}$$

$$\left|\varepsilon_{M1,check}\right| = \left|\varepsilon_{M2,check}\right| = \frac{\left|M_{1,check}\right| d_c}{2 E I_{xc}} \quad \text{(Eq. 5.23)}$$

d_c is the depth of the column.

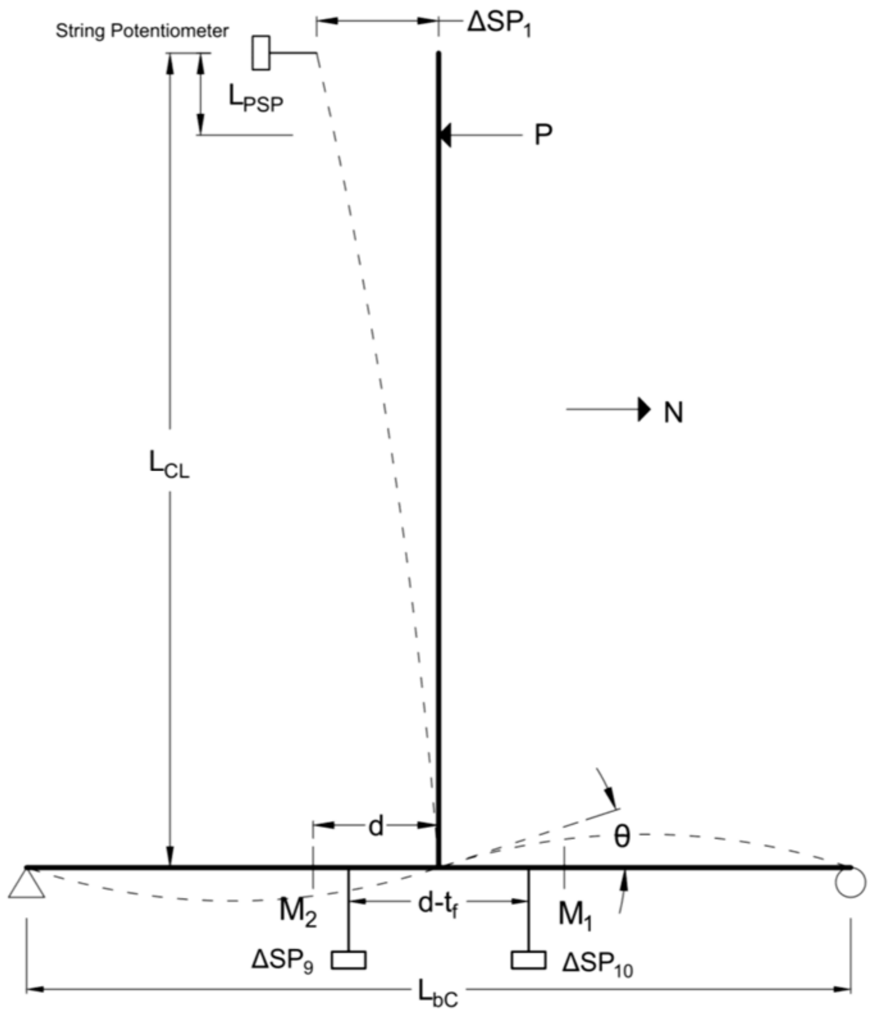


Figure 5-43 Small Deformations Check

Another check of the components of story drift angle can be used to check the sum of all components except the portion due to the elastic deformations of the beam outside of the plastic hinge region. This check used the displacements from the linear potentiometer attached to the beam just outside the plastic hinge region, and a string potentiometer attached to the column, which captures the rigid body motion of the column in the north-south direction. This layout is depicted below in Figure 5-44. In this figure, the dashed lines are the deflected shape of the beam and column for a positive story drift angle. The sum of these components of story drift angle can be compared to the calculation shown below in Eq. 5.24. The terms in the numerator of this equation are summed together because, for the case of a positive story drift angle, the linear potentiometer extends, making ΔLP positive. If the column were to move, it would be in the same direction as the applied force, which is to the south in this case. This means that the string potentiometer would retract, making ΔSP_7 negative. Therefore, the signs of the instrumentation were taken into account. Also, the small angle assumption was applied here.

$$\alpha_{check} = \frac{\Delta LP - \Delta SP_7}{L_{LP} + \frac{d_c}{2}} \qquad \text{(Eq. 5.24)}$$

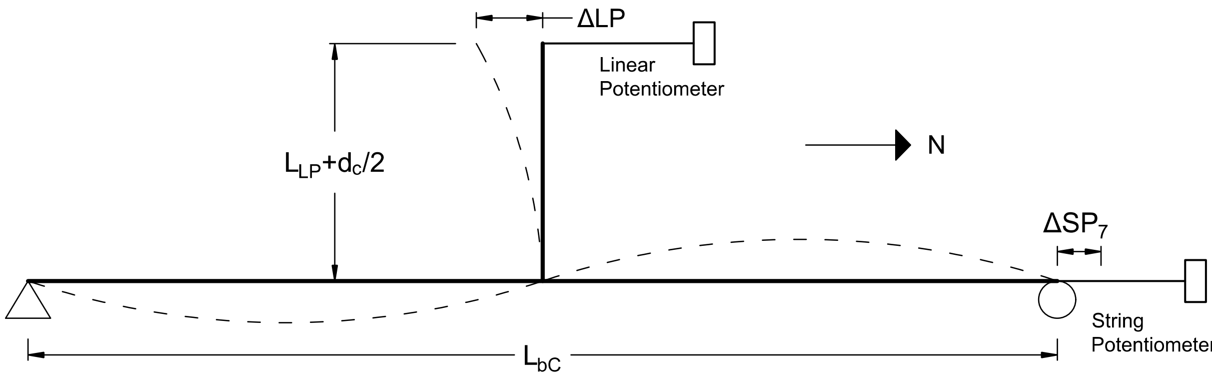


Figure 5-44 Large Deformations Check

5.4. Testing Procedure

In order to prequalify a beam-to-column moment connection, a certain loading sequence must be followed, which is detailed in the AISC *Seismic Provisions for Structural Steel Buildings*. This loading sequence is displacement controlled and is shown below in Table 5-19. The story drift angle is the ratio of the deflection of the beam to the distance to the centerline of the column. The deflection of the beam was measured by a string potentiometer near the actuator. Displacement can be measured by the actuators with its internal LVDT. However, these displacement readings would not be as accurate as the readings from the string potentiometer because the actuator was fixed to the reaction frame, which can move. The string potentiometer was stationary because it was fixed to the ground. The applied displacement sequence is shown in Figure 5-45 below.

Table 5-19 Loading Sequence for Beam-to-Column Moment Connections

Number of Cycles	Story Drift Angle, θ (rad)	Displacement (in)
6	0.00375	0.728
6	0.005	0.971
6	0.0075	1.457
4	0.01	1.943
2	0.015	2.914
2	0.02	3.885
2	0.03	5.828
2	0.04	7.770
Continue loading at increments of θ = 0.01 rad, with two cycles of loading at each step.		

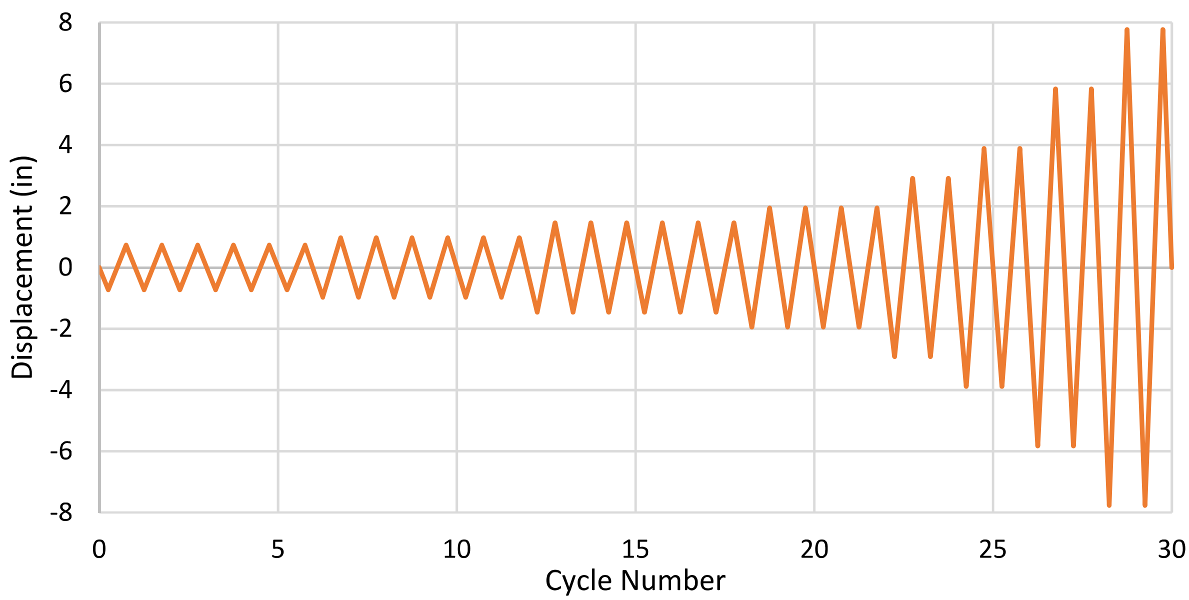


Figure 5-45 Applied Displacement Sequence

The specimens were tested to failure. Two of the specimens were expected to experience bolt failures, and the remaining specimens were expected to form a plastic hinge. The loading was applied by hydraulic actuators, which were at a distance of 16’-2-1/4” from the center line of the column. All of the specimens required only one hydraulic actuator, except for Specimen 12ES-1.25-1.50-44, which required two hydraulic actuators. Both of these actuators were placed in the same line. The story drift angle that could be reached was limited by the stroke of the actuator, which is 30 inches. This means that the maximum story drift angle that could be reached during the tests was slightly more than 0.07 rad. The full loading protocol can be found in Appendix Q.

6. EXPERIMENTAL RESULTS

Between the actuators, the string potentiometers, the linear potentiometer, the strain gauges, the bolt strain gauges, and the calipers, there were a total of 55 channels of data for each test. The data was sampled at a rate of 5 Hz. All data points were plotted, unless stated otherwise. For each specimen, the figures that are not included in this chapter are shown in the appendices. The additional figures for Specimen 12ES-0.75-1.00-44 are shown in Appendix S. The additional figures for Specimen 12ES-1.25-1.50-44 are included in Appendix T. The additional figures for Specimen 12ES-0.875-0.75-24 are shown in Appendix U. The additional figures for Specimen 12ES-1.125-1.25-24 are included in Appendix V. The results of each test are discussed in the following sections.

6.1. Testing of Specimen 12ES-0.75-1.00-44

Specimen 12ES-0.75-1.00-44 was a 44 inch deep beam with an end-plate that was designed for thick end-plate behavior. This specimen was connected to the column specimen with 3/4" A490 bolts. The end-plate was 1 inch thick.

6.1.1. Limit States – Progression and Predictions

This specimen was designed to fail due to bolt tension rupture without prying action. The bolts fractured as the specimen was approaching the first peak in the 1% story drift cycles. The applied displacement at fracture was 1.42 inches. All twelve bolts at the southern flange failed simultaneously. This follows the assumption, in the bolt force model discussed in Chapter 4, that all bolts reach their full capacity at ultimate. The predicted capacity was conservative by just under 7%. The predicted and experimental capacities are outlined below in Table 6-1. The variation of the ratio of the applied moment to the predicted moment capacity associated with bolt tension rupture without prying action compared to the total story drift is shown below in Figure 6-1.

Table 6-1 Predicted & Experimental Moment Capacities for Specimen 12ES-0.75-1.00-44

Predicted (kip-ft)	Experimental (kip-ft)	Ratio
M_{np}^{TT}	$M_{u,cf}$	$M_{u,cf}/M_{np}^{TT}$
2338	2494	1.067

M_{np}^{TT} – Moment capacity associated with bolt tension rupture without prying action calculated based on material properties from the tensile testing

$M_{u,cf}$ – Maximum moment at the column face during the test

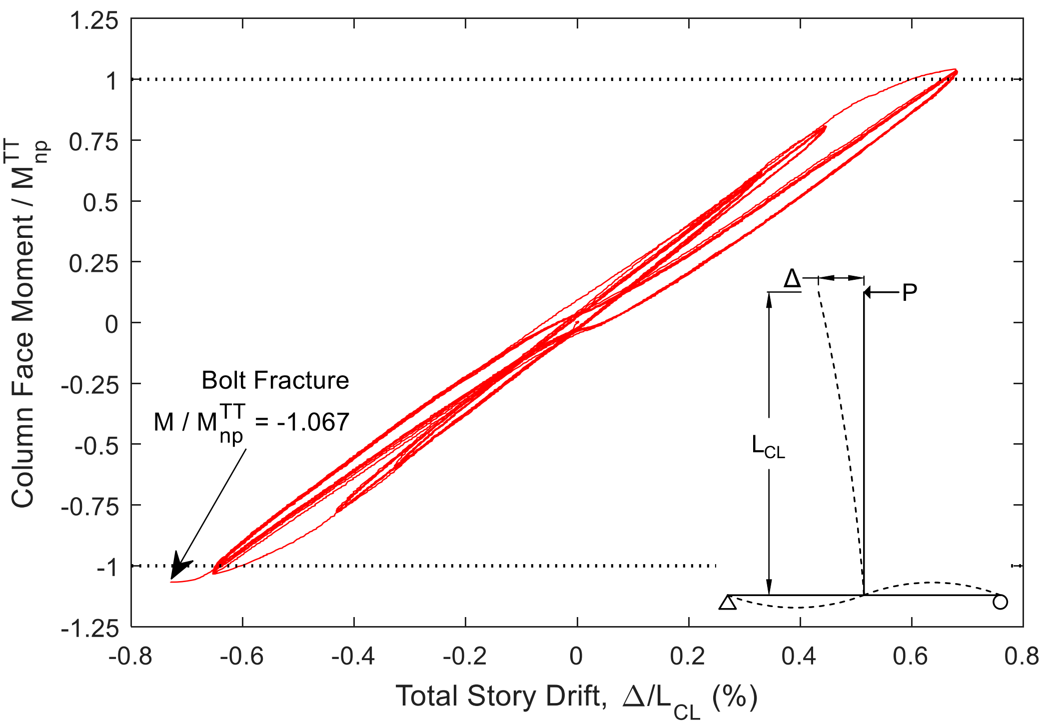


Figure 6-1 Moment Ratio vs. Total Story Drift for Specimen 12ES-0.75-1.00-44

6.1.2. Experimental Results – Yield and Ultimate Moments

The specimen failed as designed: bolt rupture before yielding of anything else. The ultimate moment in the beam was 2494 kip-ft at the face of the column, which was significantly less than the predicted beam yield moment of 3533 kip-ft, or end-plate yield moment of 3126 kip-ft. The calculated beam yield moment was based on the material properties from the tensile testing. The predicted moment capacity associated end-plate yielding was 3126 kip-ft. No white wash flaked off during the test. The beam, the end-plate and the column remained elastic throughout the test based on the predicted capacities, and the white wash that was applied to the specimen.

6.1.3. Experimental Results – End-Plate Separation

The end-plate separation remained relatively small throughout this test. During the 0.75% cycles, the end-plate was consistently just over 0.05 inches at the inside of the beam flange. The end-plate separation was generally linear with respect to applied load. However, over the brief instances before bolt fracture, the end-plate separation rapidly increased from about 0.05 inches to about 0.075 inches. How the end-plate separation progressed with respect to applied load in the cycle leading up to failure can be seen below in Figure 6-2.

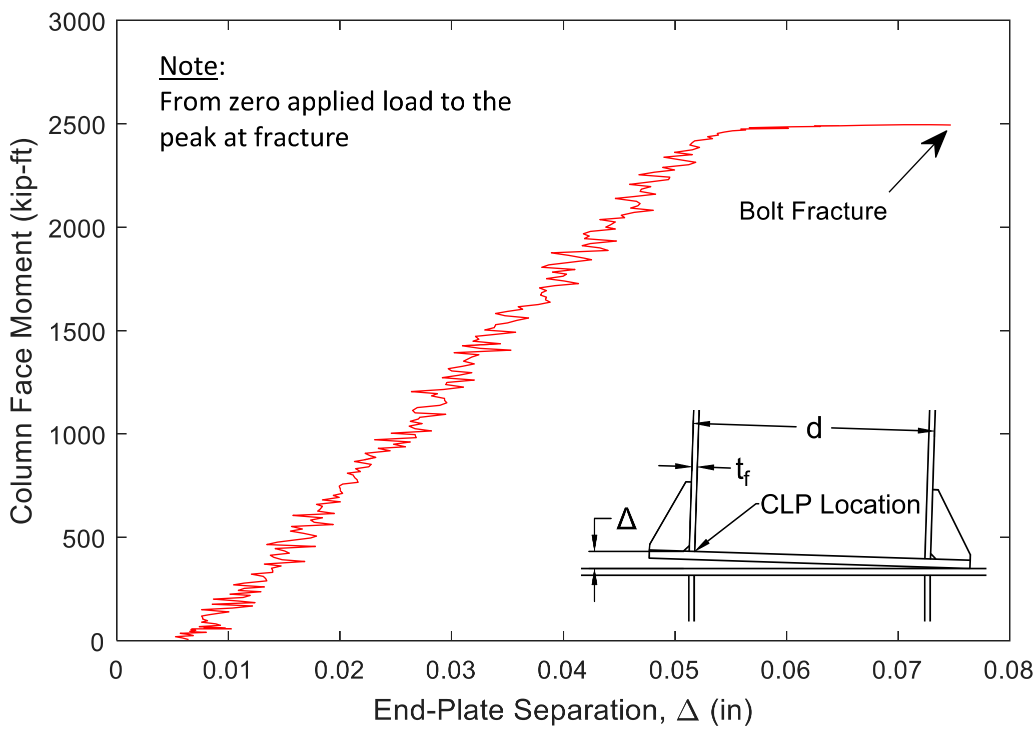


Figure 6-2 End-Plate Separation in Failure Cycle for Specimen 12ES-0.75-1.00-44

6.1.4. Experimental Results – Bolt Forces

The bolts were pretensioned to at least the minimum of 35 kips as prescribed in Table J3.1 of AISC 360-10 (AISC 2010b). The force in each bolt varied by bolt hole location. However, the following discussion on bolt forces is based on one bolt hole location. For the

0.375% story drift cycles, the force in the bolts varied by about 2 kips above and below the initial pretension level. For the 0.5% cycles, the force in the bolts varied by about 5 kips above and below the initial pretension level. After the first the first peak in the 0.75% cycle, the bolts lost their pretension, meaning that when the flange was in compression, the force in the bolts was essentially zero. The relationship of bolt force and applied moment is shown below in Figure 6-3. The capacity of the bolts, T_{ult}^{TT}, was 54.6 kips, which was based on the results from the bolt tensile testing. After the 0.5% cycles, the bolt force exceeded T_{ult}^{TT}. These recorded forces are not accurate because the bolt strain gauges were calibrated in the linear-elastic range. At this point during the test, the bolts became nonlinear. This means that the recorded bolt force values are greater than the actual force in the bolts. The force in the bolts at fracture would be significantly closer to 54.6 kips.

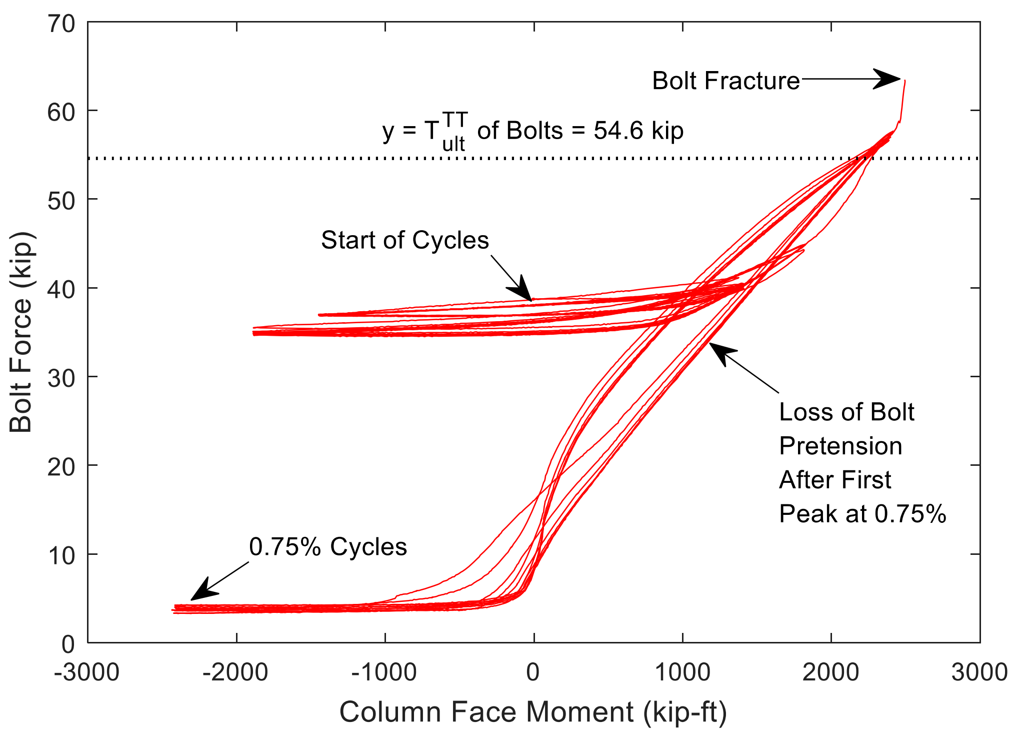


Figure 6-3 Bolt Force vs. Column Face Moment for Specimen 12ES-0.75-1.00-44

6.1.5. Experimental Results – Pictures of Specimen 12ES-0.75-1.00-44

Below, Figure 6-4 and Figure 6-5 show two different views of Specimen 12ES-0.75-1.00-44 at the instant before the bolts fractured. These figures show the end-plate separation, and the state of the white wash, which may indicate the level of inelasticity in the beam and in the end-plate. No white wash flaked off on either the beam or the end-plate. Figure 6-6 shows the specimen after the bolts fractured. An elevation view and a plan view of a typical fracture surface can be seen below in Figure 6-7.

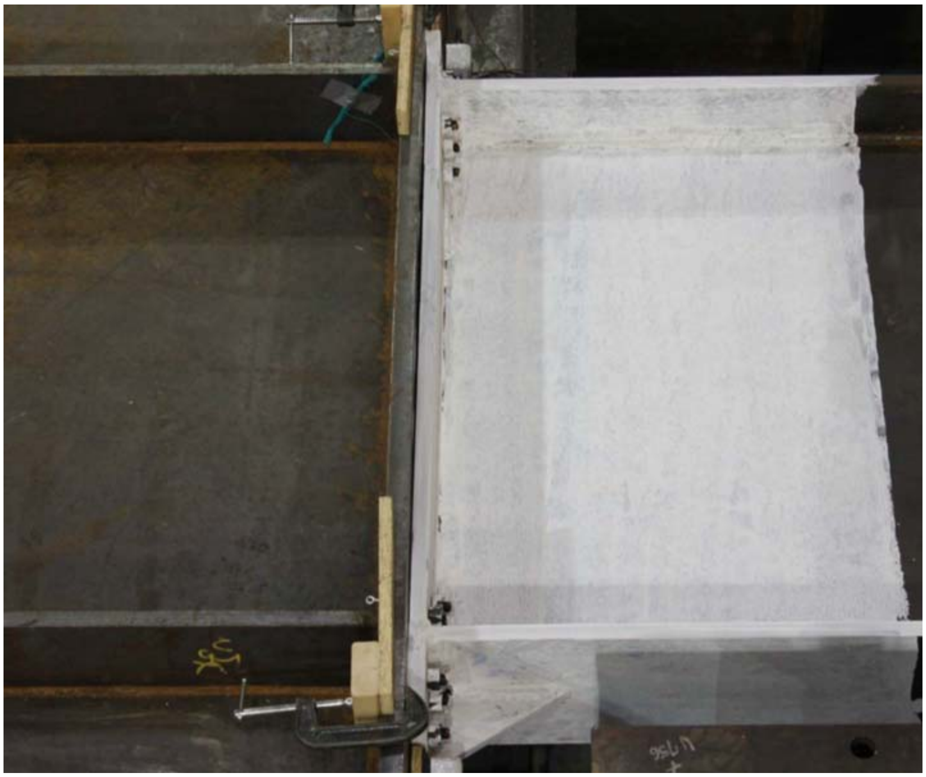


Figure 6-4 View of End-Plate Separation Before Fracture for Specimen 12ES-0.75-1.00-44

Figure 6-5 View of End-Plate White Wash Before Fracture for Specimen 12ES-0.75-1.00-44

Figure 6-6 Specimen 12ES-0.75-1.00-44 After Bolt Rupture

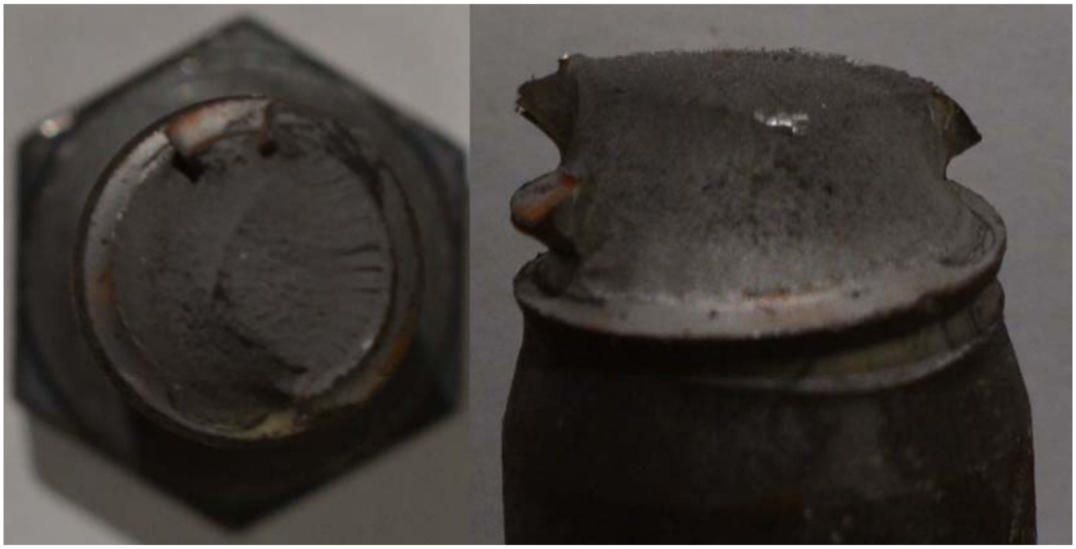


Figure 6-7 Fracture Surface of Typical Fractured Bolt for Specimen 12ES-0.75-1.00-44

6.2. Testing of Specimen 12ES-1.25-1.50-44

Specimen 12ES-1.25-1.50-44 was a 44 inch deep beam. This specimen was connected to the column specimen with 1-1/4” A490 bolts. The end-plate was 1-1/2 inch thick.

6.2.1. Limit States – Progression and Predictions

This specimen was designed for plastic hinging of the beam. The goal was to cycle the beam through the 4% story drift cycles, which is a requirement in the seismic provisions for qualification for SMRF (AISC 2010a). However, the beam fractured at a point significantly before reaching the 4% cycles. The beam flange and web fractured suddenly at the same time. The fracture went through the entire flange, and halfway down the depth of the web.

The beam remained elastic through all of the 1% story drift cycles. Yielding may have initiated in the beam flanges, in the beam web, and in the stiffeners at the first peak of the 1.5% story drift cycles because going into the first peak of the 1.5% story drift cycles is when white wash first started to flake off of the flanges and the stiffeners. The beam continued through all of the 1.5% story drift cycles and through the first peak of the first 2% cycle. At the first peak of the first 2% cycle, 347.5 kips were applied at the end of the beam, producing a moment of 4554 kip-ft at the end of the beam stiffener. The beam flange and web fractured as the specimen was approaching the second peak in the 2% story drift cycles. The applied load at fracture was 302.5 kips, producing a moment of 3964 kip-ft at the end of the beam stiffener. The applied displacement at fracture was 2.24 inches, whereas at the previous peak, the beam reach an applied displacement of 3.70 inches. Leading up to fracture, there was significant yielding in the beam flanges. The stiffeners and beam web showed signs of some yielding.

The beam failed at a moment which was 94.8% of the plastic section moment capacity determined based on the material properties from the tensile testing. This moment ratio versus the total story drift is shown below in Figure 6-8. The predicted versus the experimental moment capacities for Specimen 12ES-1.25-1.50-44 are shown below in Table 6-2. The predicted plastic section moment capacity based on the material properties from the tensile testing only differed from the experimentally determined value by about 2.3%. The method for determining the experimental value is shown below in Figure 6-9.

Table 6-2 Predicted & Experimental Moment Capacities for Specimen 12ES-1.25-1.50-44

Predicted (kip-ft)	Experimental (kip-ft)		Ratio	
M_p^{TT}	M_p^{exp}	$M_{u,st}$	$M_{u,st}/M_p^{TT}$	M_p^{exp}/M_p^{TT}
4182	4278	4554	1.089	1.023

M_p^{TT} – Plastic section moment capacity of beam calculated based on material properties from the tensile testing

M_p^{exp} – Plastic section moment capacity of the beam determined by the intersection of the elastic and post-yield stiffness from the moment-story drift curve for the test

$M_{u,st}$ – Maximum moment experienced by the beam at the end of the stiffener during the test

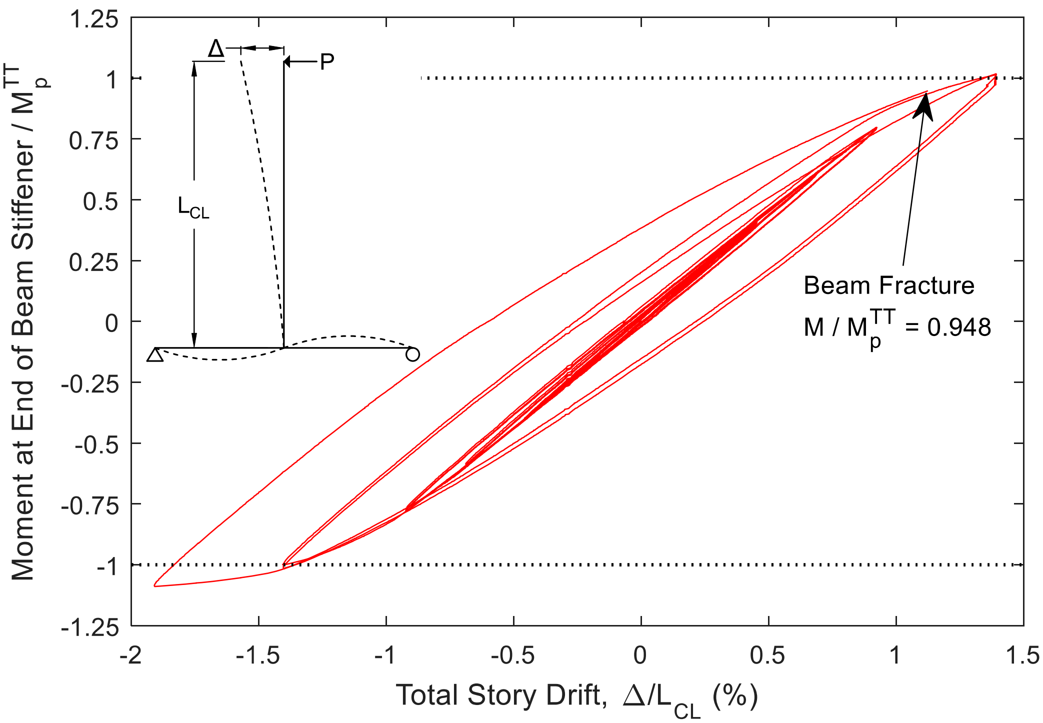


Figure 6-8 Moment Ratio vs. Total Story Drift for Specimen 12ES-1.25-1.50-44

6.2.2. Experimental Results – Yield and Ultimate Moments

The beam yielded as it was approaching the first peak in the first 1.5% story drift cycle. It also reached its plastic section moment capacity in the first peak of the 2% cycles at an applied moment of 4278 kip-ft at the end of the beam stiffener. The ultimate applied moment was reached at the peak before failure, which was 4554 kip-ft at the end of the beam stiffener. The beam failed at a moment of lesser magnitude than the ultimate applied moment.

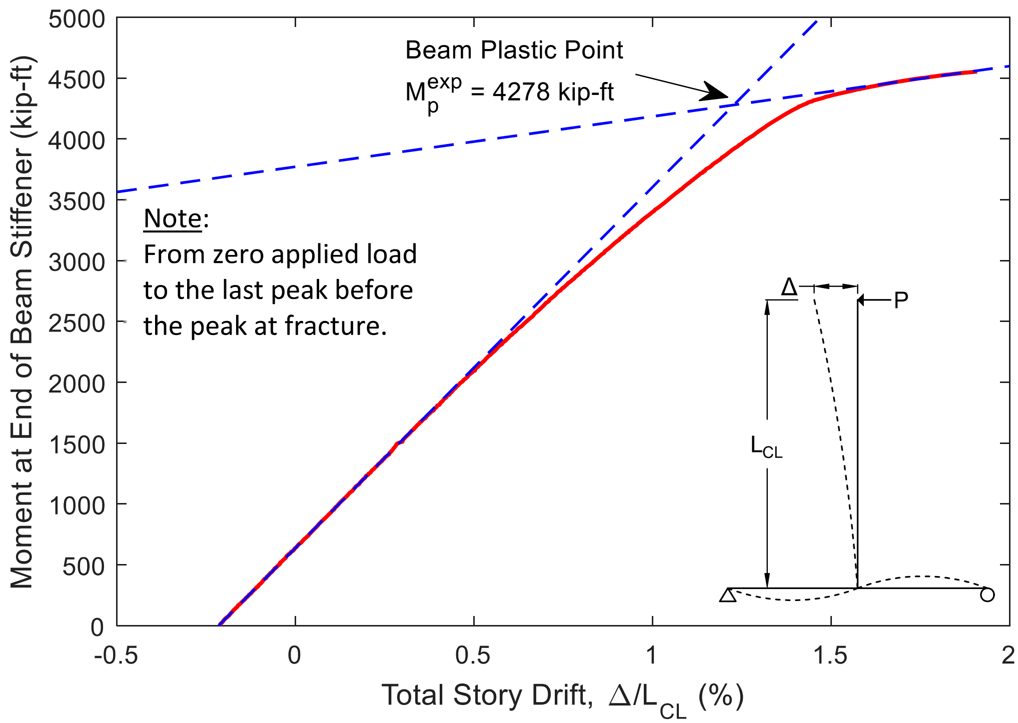


Figure 6-9 Determination of Experimental Plastic Moment Capacity for Specimen 12ES-1.25-1.50-44

6.2.3. Experimental Results – End-Plate Separation

The separation of the end-plate from the column face remained relatively low through the 1% story drift cycles. The maximum end-plate separation at the inside face of the beam flange through the 1% cycle was just under 0.05 inches. During the 1.5% cycles, the separation increased to about 0.11 inches. At the instant of fracture, the end-plate separation was about 0.13 inches. The relationship of end-plate separation to applied moment was relatively linear through the 1% cycles. After those cycles, that relationship became nonlinear. The relationship of applied moment versus rotation due to end-plate separation can be seen below in Figure 6-10.

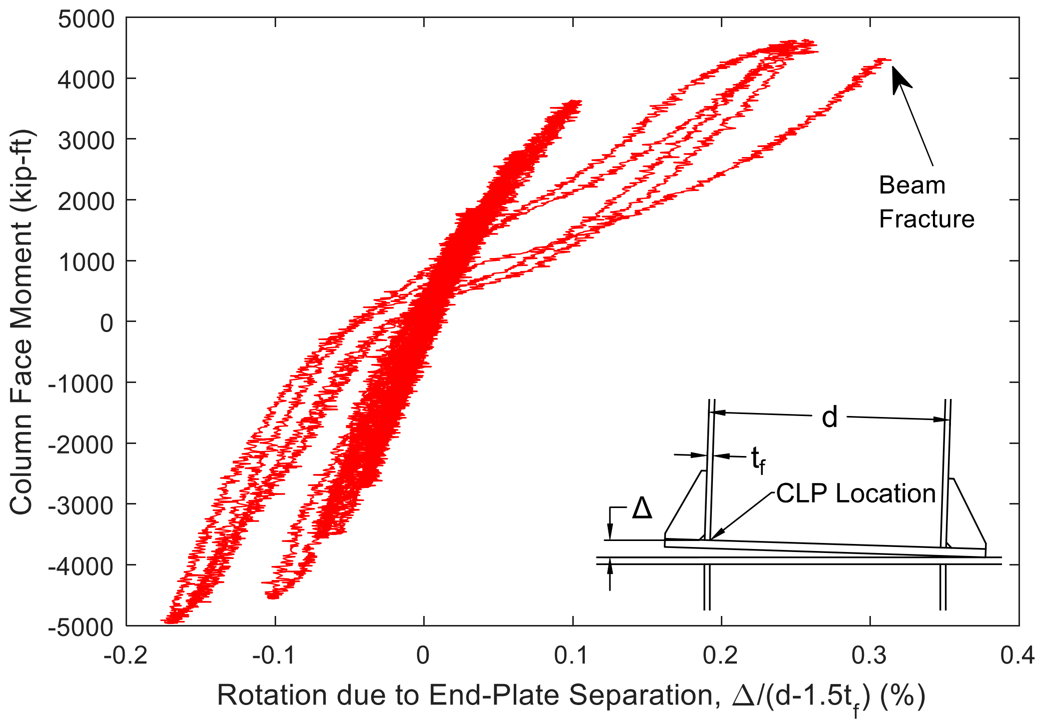


Figure 6-10 Rotation due to End-Plate Separation for Specimen 12ES-1.25-1.50-44

6.2.4. Experimental Results – Bolt Forces

The bolts were 1-1/4" A490, which had a tensile capacity, T_{ult}^{TT}, of 160.6 kip based on the results from the bolt tensile testing. They were pretensioned to at least the minimum of 102 kips as prescribed in Table J3.1 of AISC 360-10 (AISC 2010b). The force in each bolt varied by bolt hole location. However, the following discussion on bolt forces is based on one of the bolt hole locations. During the 0.375% story drift cycles, the force in the bolts drop to about 94 kips when the flange was under compression, and increased to about 110 kips when the flange was under tension. During the 0.5% cycles, the force in the bolts drop to about 89 kips when the flange was under compression, and increased to about 115 kips when the flange was under tension. This trend continued with each new level of cycles, which can be seen below in Figure 6-11. The maximum recorded bolt force for this bolt location was 141.7 kips at the last peak before fracture, which is notably less than T_{ult}^{TT}. Since the bolt forces were calculated based on a linear calibration of the strain, the bolt forces would become inaccurate when the actual bolt force approaches T_{ult}^{TT} because the behavior of the bolts becomes nonlinear as the actual bolt force approaches T_{ult}^{TT}. Since the calculated bolt forces shown below in Figure 6-11 are significantly below T_{ult}^{TT}, the calculated bolt forces should be reasonably accurate.

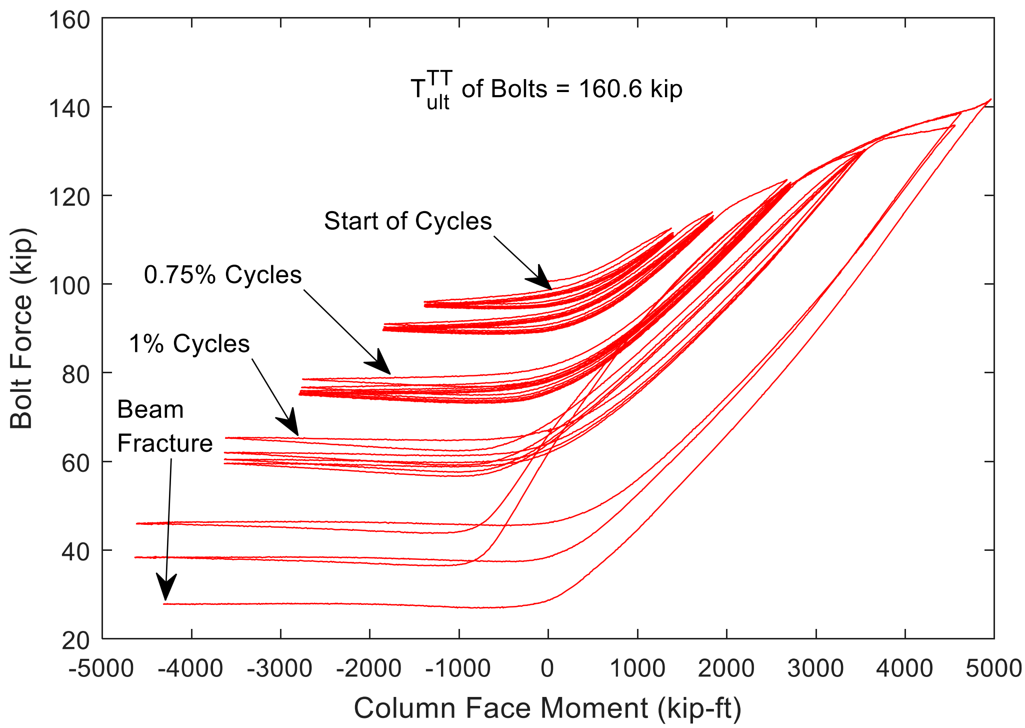


Figure 6-11 Bolt Force vs. Column Face Moment for Specimen 12ES-1.25-1.50-44

6.2.5. Experimental Results – Pictures of Specimen 12ES-1.25-1.50-44

The beam fractured completely through one of the flanges, and halfway down the depth of the web. This fracture can be seen below in Figure 6-12. The fracture initiated at the outside face of the flange at the toe of the stiffener. A close up of this location can be seen below in Figure 6-13. The fracture surface can be seen below in Figure 6-16, which indicates the exact location of fracture initiation based on the chevron markings. After cleaning off some of the white wash and inspecting the fractured beam further, two more fracture locations were discovered. The second fracture was forming in the weld at the toe of the stiffener on the other flange. This fracture can be seen below in Figure 6-14. The third fracture was forming in the flange-to-end-plate weld in the flange opposite the main fracture. This fracture was initiating at the edge of the flange, which can be seen below in Figure 6-15.

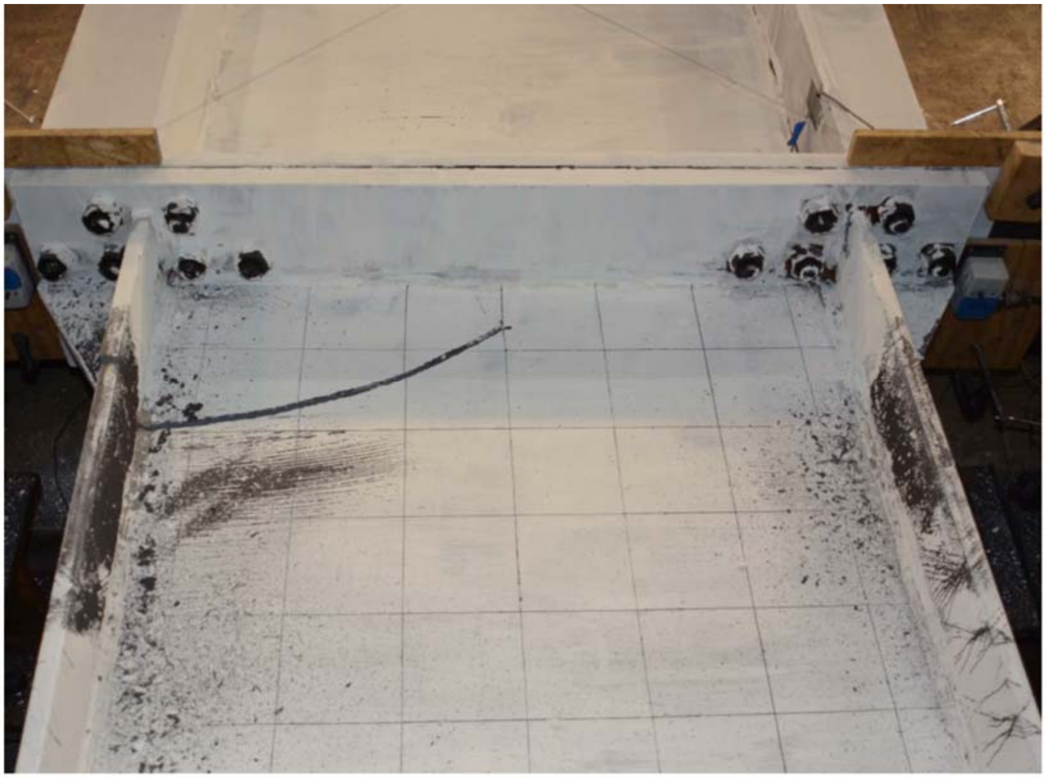


Figure 6-12 Fractured Specimen 12ES-1.25-1.50-44

Figure 6-13 Specimen 12ES-1.25-1.50-44 Fractured Flange

Figure 6-14 Specimen 12ES-1.25-1.50-44 Second Fracture - Other Flange

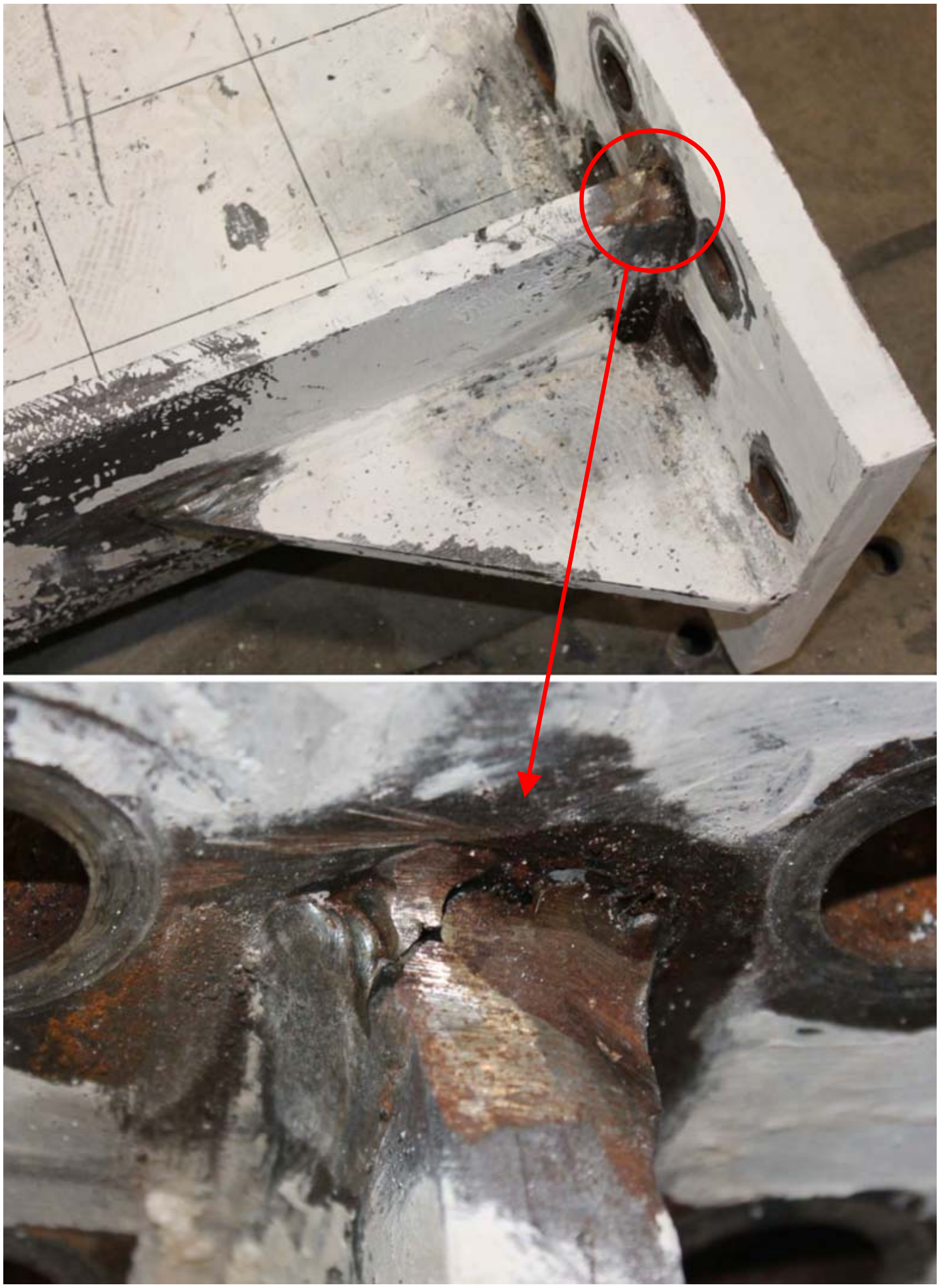


Figure 6-15 Specimen 12ES-1.25-1.50-44 Third Fracture - Other Flange

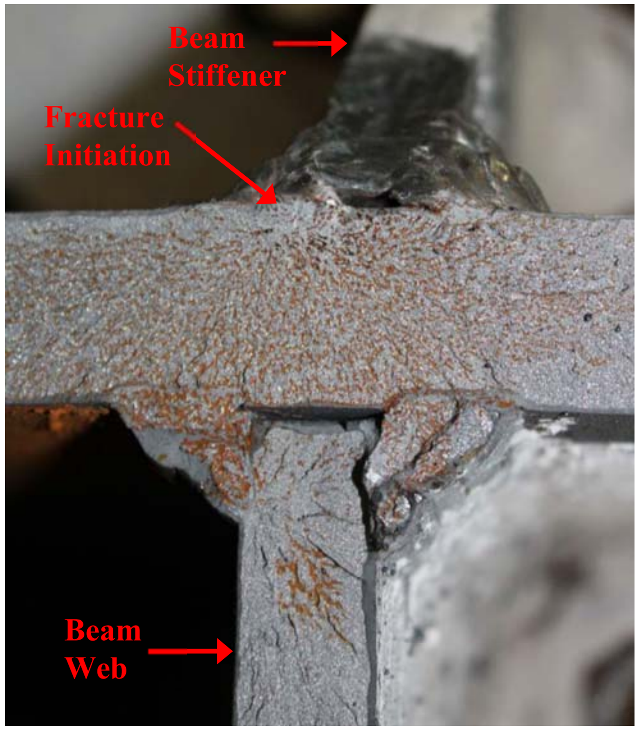


Figure 6-16 Fracture Surface of Specimen 12ES-1.25-1.50-44

6.3. Testing of Specimen 12ES-0.875-0.75-24

Specimen 12ES-0.875-0.75-24 was a 24 inch deep beam with an end-plate that was designed for thin end-plate behavior. This specimen was connected to the column specimen with 7/8" A490 bolts. The end-plate was 3/4 inch thick.

6.3.1. Limit States – Progression and Predictions

This specimen was designed to fail due to bolt tension rupture with prying action. The bolts fractured as the specimen was approaching the first peak in the first 3% story drift cycles. Eleven of the twelve bolts at the southern flange fractured at the same time. The North end on the test setup is indicated in Figure 5-17 above. The locations of these fractured bolt can be seen below in Figure 6-17. After these bolts fractured, the applied load dropped from a peak of 111.7 kips to about 16 kips, and then the loading protocol was paused. After the specimen was inspected, the loading resumed until the last bolt at that flange failed.

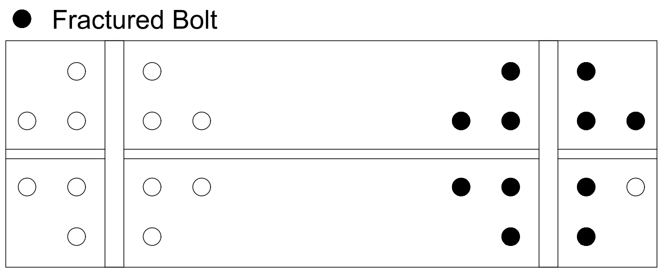


Figure 6-17 First Set of Fractured Bolts for Specimen 12ES-0.875-0.75-24

End-plate yielding started in the 1% story drift cycles. The yielding in the end-plate developed further with each new level of cycles. The beam flanges began to yield in the 2% story drift cycles, whereas at this point, there was already significant yielding in the end-plate. The prediction for the moment capacity associated with end-plate yielding calculated based on material properties from the tensile testing is shown below in Table 6-3. The predicted capacity was about 9% conservative compared to the experimentally determined value. The method for determining the experimental value is shown below in Figure 6-19. The prediction for the moment capacity associated with bolt tension rupture with prying action calculated based on material properties from the tensile testing was just under 19% conservative compared to the ultimate moment. This moment ratio versus the total story drift is shown below in Figure 6-18.

Table 6-3 Predicted & Experimental Moment Capacities for Specimen 12ES-0.875-0.75-24

Predicted (kip-ft)		Experimental (kip-ft)		Ratio		
M_{pl}^{TT}	M_q^{TT}	M_{pl}^{exp}	$M_{u,cf}$	$M_{u,cf}/M_{pl}^{TT}$	$M_{u,cf}/M_q^{TT}$	M_{pl}^{exp}/M_{pl}^{TT}
1034	1367	1125	1620	1.567	1.185	1.088

M_{pl}^{TT} – Moment capacity associated with end-plate yielding calculated based on material properties from the tensile testing

M_q^{TT} – Moment capacity associated with bolt tension rupture with prying action calculated based on material properties from the tensile testing

M_{pl}^{exp} – Moment associated with yielding of the end-plate based on experimental results

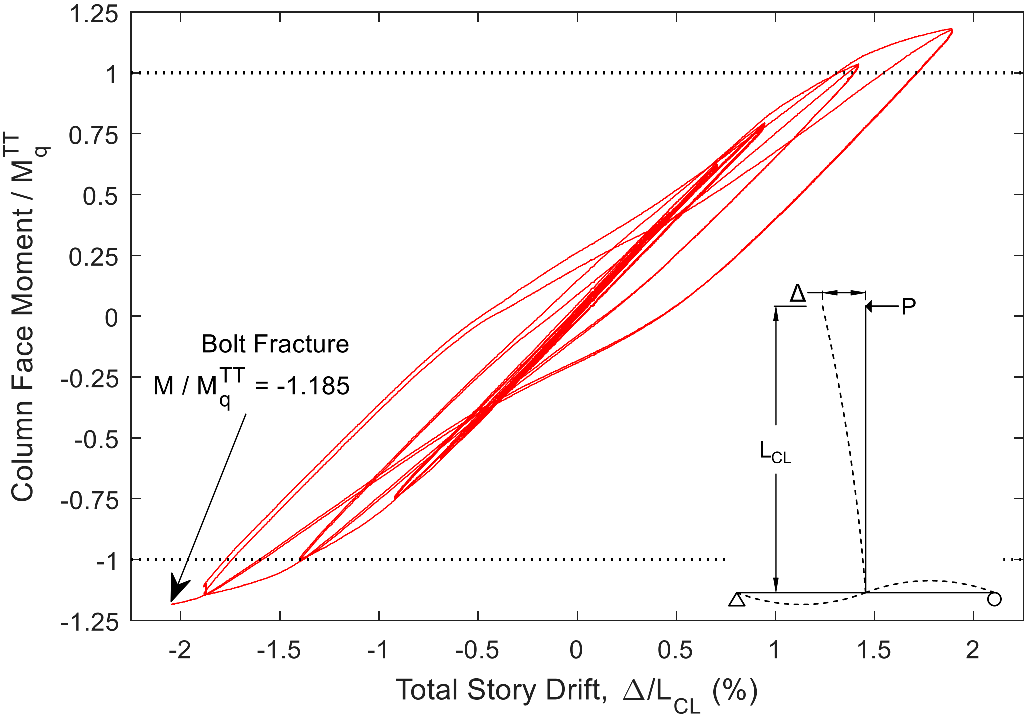


Figure 6-18 Moment Ratio vs. Total Story Drift for Specimen 12ES-0.875-0.75-24

6.3.2. Experimental Results – Yield and Ultimate Moments

The end-plate yielded before the beam flange yielded. The end-plate yielded at a moment of 1350 kip-ft. This point is shown below in Figure 6-19. The bolts fractured at an applied force of 111.7 kips, producing an applied moment of 1620 kip-ft at the face of the column.

6.3.3. Experimental Results – End-Plate Separation

The end-plate separation remained relatively linear with respect to applied load through the 0.75% story drift cycles. However, the magnitude of the end-plate separation was no greater than 0.03 inches at the inside face of the beam flange. The end-plate separation did not become noticeable to the naked eye until the 2% story drift cycles. The separation at the inside face of the flange was about 0.11 inches. The shape of this end-plate separation was bell-curve with the peak separation at the beam flange. The separation returned to zero a few inches inside and outside the beam flange. This shape is a result of the thin end-plate behavior. The yielding in the end-plate led to greater localized deformations in the end-plate. The relationship of applied moment and the rotation due to end-plate separation can be seen below in Figure 6-19. After the end-plate yielded, the behavior of the separation became nonlinear.

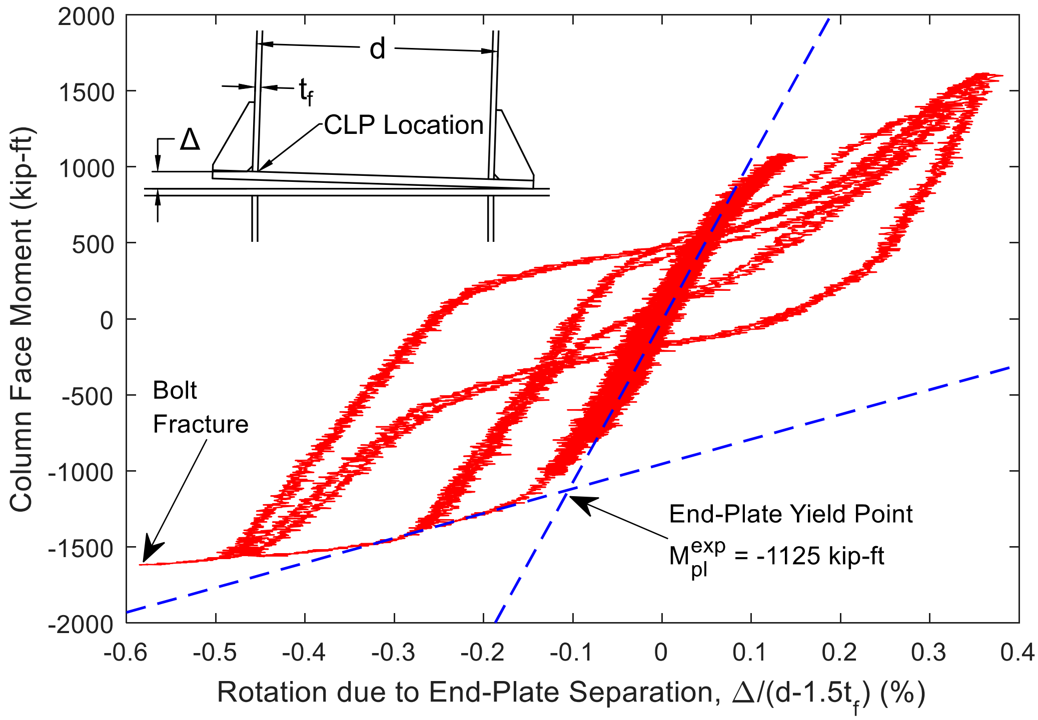


Figure 6-19 Rotation due to End-Plate Separation for Specimen 12ES-0.875-0.75-24

6.3.4. Experimental Results – Bolt Forces

The bolts were 7/8" A490, which had a tensile capacity, T_{ult}^{TT}, of 76.4 kip based on the results from the bolt tensile testing. They were pretensioned to at least the minimum of 49 kips as prescribed in Table J3.1 of AISC 360-10 (AISC 2010b). The force in each bolt varied by bolt hole location. However, the following discussion on bolt forces is based on one of the bolt hole locations. During the 0.375% story drift cycles, the force in the bolts drop to about 48 kips when the flange was under compression, and increased to about 50 kips when the flange was under tension. During the 0.5% cycles, the force in the bolts drop to about 47 kips when the flange was under compression, and increased to about 49 kips when the flange was under tension. This trend continued with each new level of cycles, which can be seen below in Figure 6-20. However, the change in bolt force increased significantly at the greater levels of applied displacement. The maximum recorded bolt force for this bolt location was about 60 kips at the first peak in the first

2% story drift cycle, which is notably less than T_{ult}^{TT}. Since the bolt forces were calculated based on a linear calibration of the strain, the bolt forces would become inaccurate when the actual bolt force approaches T_{ult}^{TT} because the behavior of the bolts becomes nonlinear as the actual bolt force approaches T_{ult}^{TT}. Since the calculated bolt forces shown below in Figure 6-20 are significantly below T_{ult}^{TT}, the calculated bolt forces should be reasonably accurate. The bolt strain gauges were inserted in a hole at the center of the bolts in order to measure the axial strain in the bolts. However, there was prying action on the bolts after the end-plate yielded. This means that the recorded strains, which were calibrated for force, are not indicative of the peak strain, or peak stress in the bolts. The prying induces a moment in the bolt, causing maximum strains to be at the extreme fiber of the bolt.

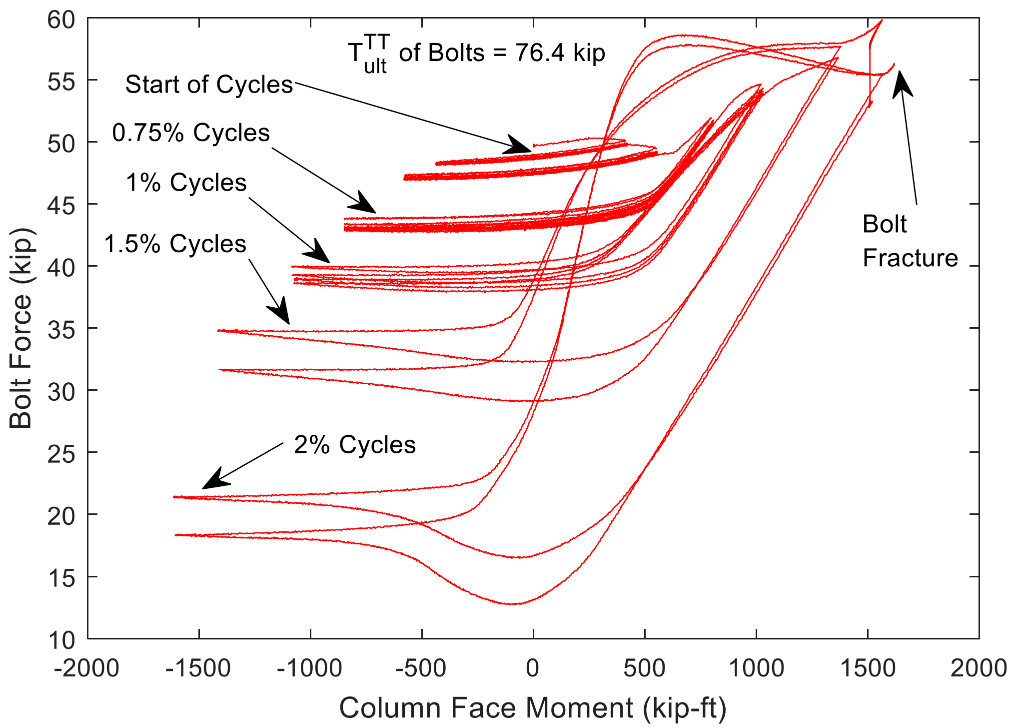


Figure 6-20 Bolt Force vs. Column Face Moment for Specimen 12ES-0.875-0.75-24

6.3.5. Experimental Results – Pictures of Specimen 12ES-0.875-0.75-24

The pattern of yielding in the end-plate was indicated by the flaking off of the white wash. The yielding pattern in the end-plate at the instant just before the bolts fractured can be seen below in Figure 6-21. The red dashed lines indicate where the yield lines formed. Although this pattern does not match the one that was chosen previously in Chapter 3, the yield line mechanisms in Chapter 3 are just numerical models that are used to help predict the moment capacity associated with yielding of the end-plate.

Also at the instant just before the bolts fractured, the separation of the end-plate from the column face can be seen below in Figure 6-22. The bell-curve shape previously mentioned is located where the flange in the upper part of the picture meets the column flange. The separation is small, but still noticeable with the naked eye. An elevation view and a plan view of a typical fracture surface can be seen below in Figure 6-23.

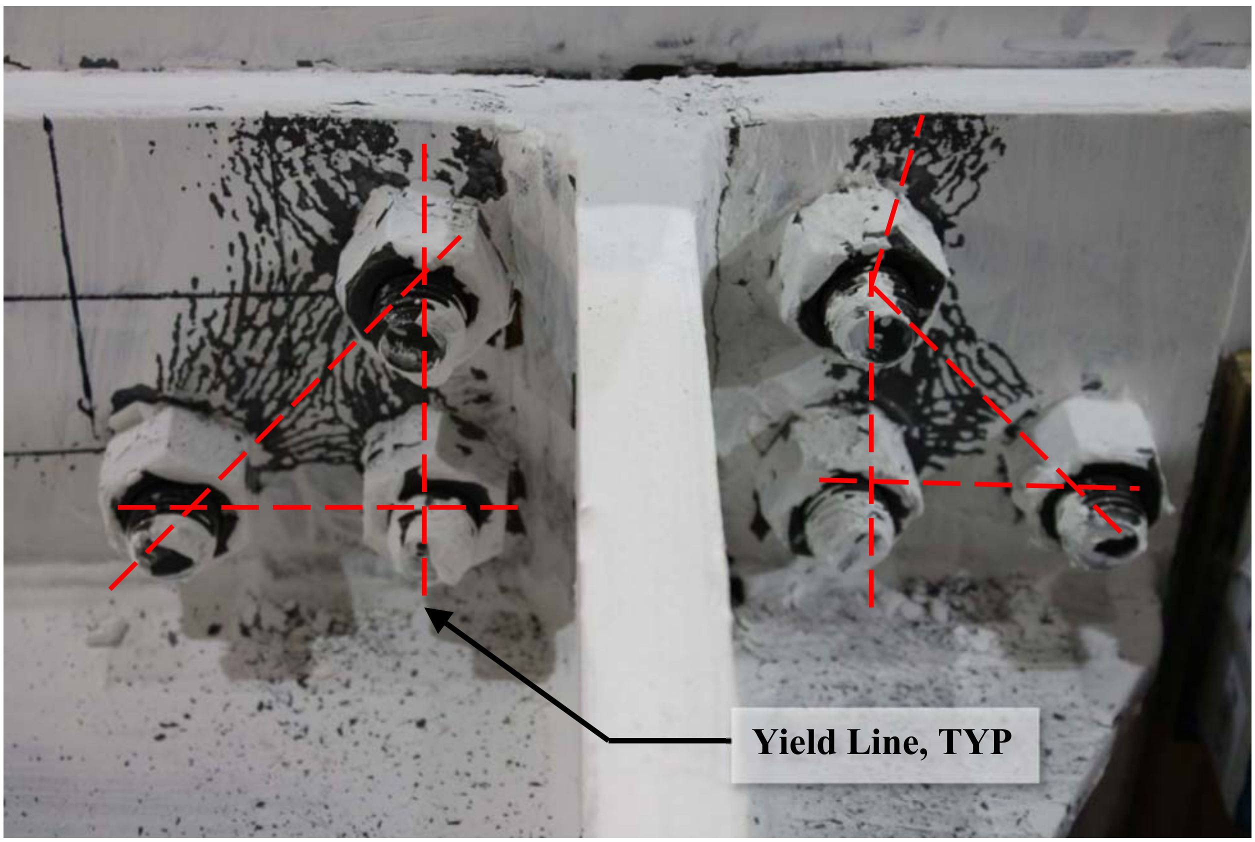


Figure 6-21 View of End-Plate Yielding for Specimen 12ES-0.875-0.75-24

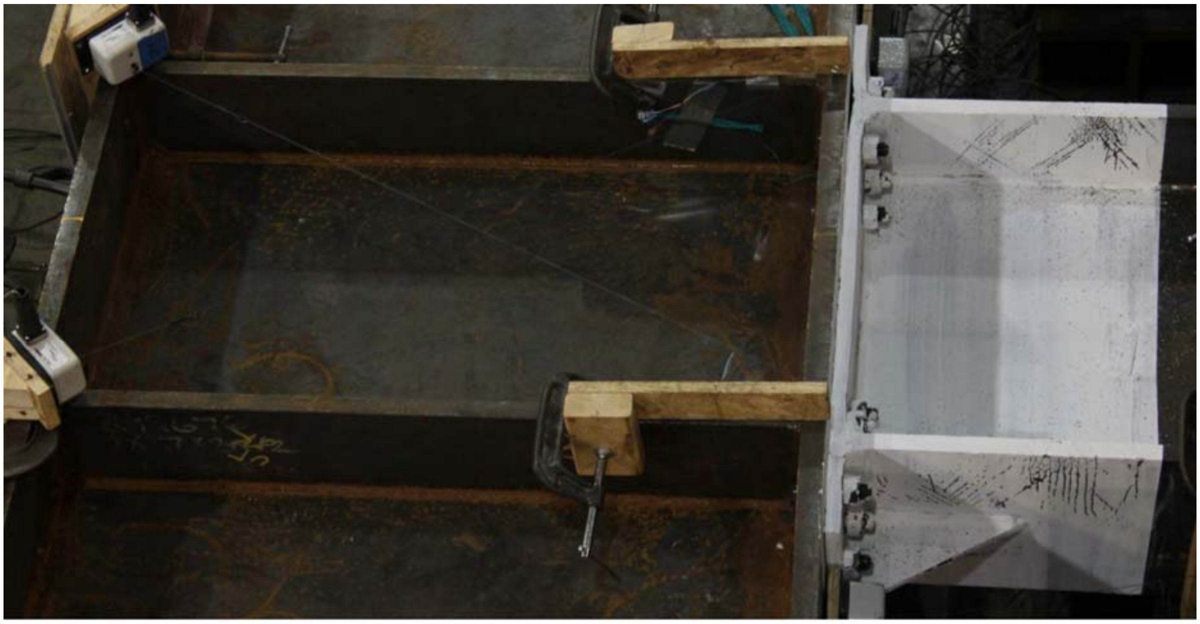


Figure 6-22 View of End-Plate Separation for Specimen 12ES-0.875-0.75-24

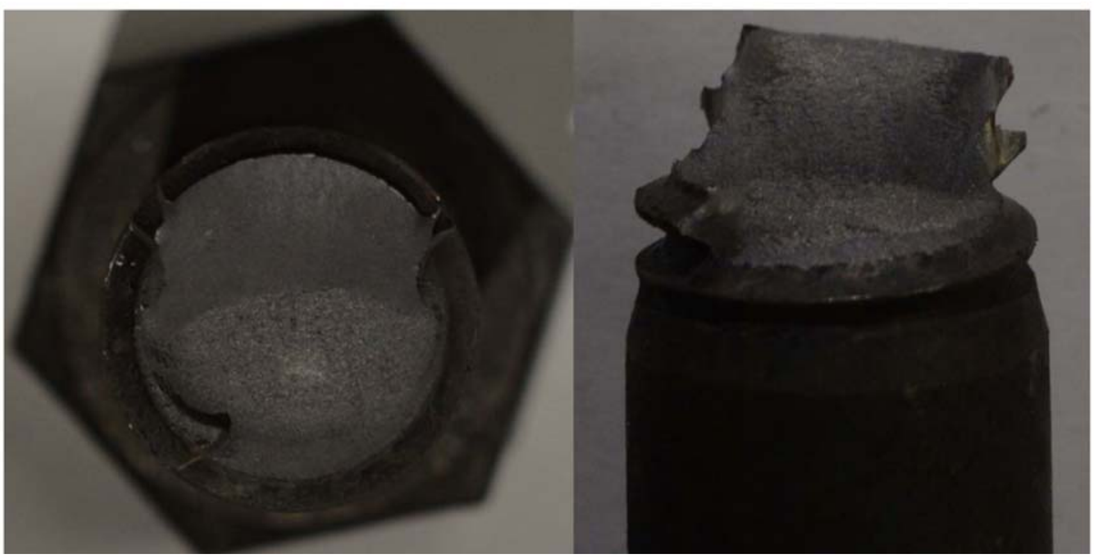


Figure 6-23 Fracture Surface of Typical Fractured Bolt for Specimen 12ES-0.875-0.75-24

6.4. Testing of Specimen 12ES-1.125-1.25-24

Specimen 12ES-1.125-1.25-24 was a 24 inch deep beam. This specimen was connected to the column specimen with 1-1/8” A490 bolts. The end-plate was 1-1/4 inch thick.

6.4.1. Limit States – Progression and Predictions

This specimen was designed for plastic hinging of the beam. The goal was to cycle the beam through the 4% story drift cycles, which is a requirement in the seismic provisions for qualification for SMRF (AISC 2010a). However, the beam fractured at a point significantly before reaching the 4% cycles. The beam flange fractured suddenly, and the fracture went through the entire flange.

The beam remained elastic through all of the 1% story drift cycles. Yielding may have initiated in the beam flanges at the first peak of the 1.5% story drift cycles because going into the first peak of the 1.5% story drift cycles is when white wash first started to flake off of the flanges. The beam continued through all of the 1.5% story drift cycles, and through all of the 2% cycles. Yielding in the stiffeners started in the first peak of the 2% story drift cycles, which is based on the same reasoning as initiation of yielding in the beam flanges. At the second peak of the first 3% cycle, 140.7 kips were applied at the end of the beam, producing a moment of 1906 kip-ft at the end of the beam stiffener. The beam flange fractured as the specimen was approaching the first peak in the second 3% story drift cycles. The applied load at fracture was 127.6 kips, producing a moment of 1726 kip-ft at the end of the beam stiffener. The applied displacement at fracture was 3.81 inches, whereas at the previous peak, the beam reach an applied displacement of 5.59 inches. In the cycle leading up to fracture, there was significant yielding in the beam flanges, some yielding at the extreme fibers of the web, and slightly more yielding in the stiffeners than when yielding initiated in the stiffeners.

The beam failed at a moment which was about 101% of the plastic section moment capacity determined based on the material properties from the tensile testing. This moment ratio versus the total story drift is shown below in Figure 6-24. The predicted and the experimental moment capacities for Specimen 12ES-1.125-1.25-24 are shown below in Table 6-4. The predicted plastic section moment capacity based on the material properties from the tensile testing only differed from the experimentally determined value by about 1%. The method for determining the experimental value is shown below in Figure 6-25.

Table 6-4 Predicted & Experimental Moment Capacities for Specimen 12ES-1.125-1.25-24

Predicted (kip-ft)	Experimental (kip-ft)		Ratio	
M_p^{TT}	M_p^{exp}	$M_{u,st}$	$M_{u,st}/M_p^{TT}$	M_p^{exp}/M_p^{TT}
1715	1731	1906	1.111	1.009

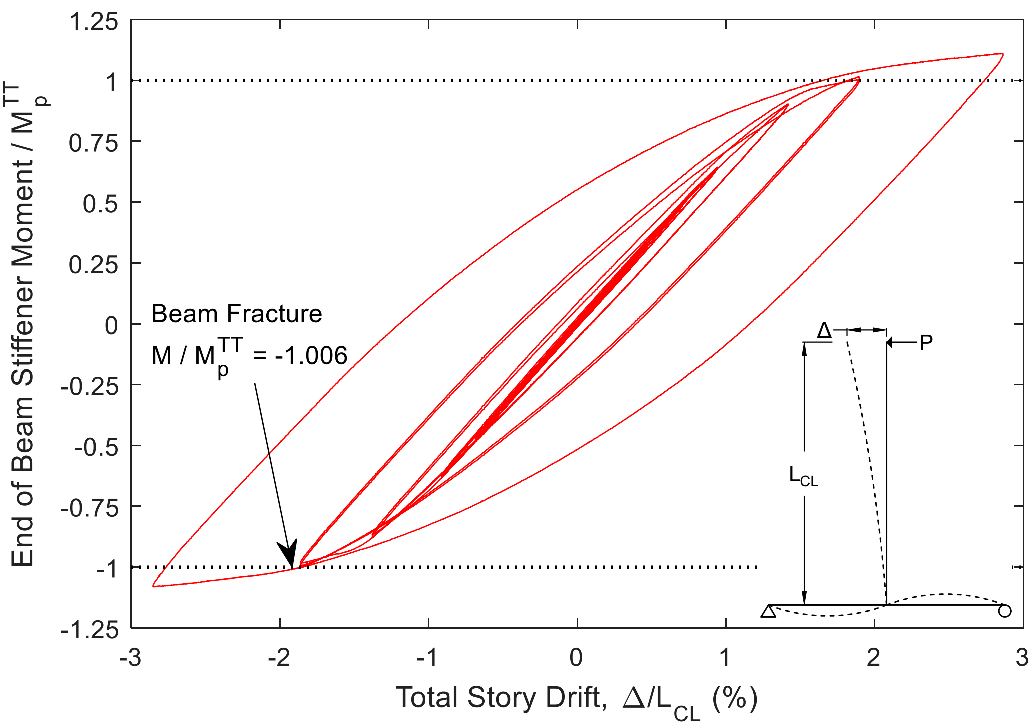


Figure 6-24 Moment Ratio vs. Total Story Drift for Specimen 12ES-1.125-1.25-24

6.4.2. Experimental Results – Yield and Ultimate Moments

The beam yielded as it was approaching the first peak in the first 1.5% story drift cycle. It also reached its plastic section moment capacity in the 2% cycles at an applied moment of 1731 kip-ft at the end of the beam stiffener. The ultimate applied moment was reached at the peak before failure, which was 1906 kip-ft at the end of the beam stiffener. The beam failed at a moment of lesser magnitude than ultimate applied moment.

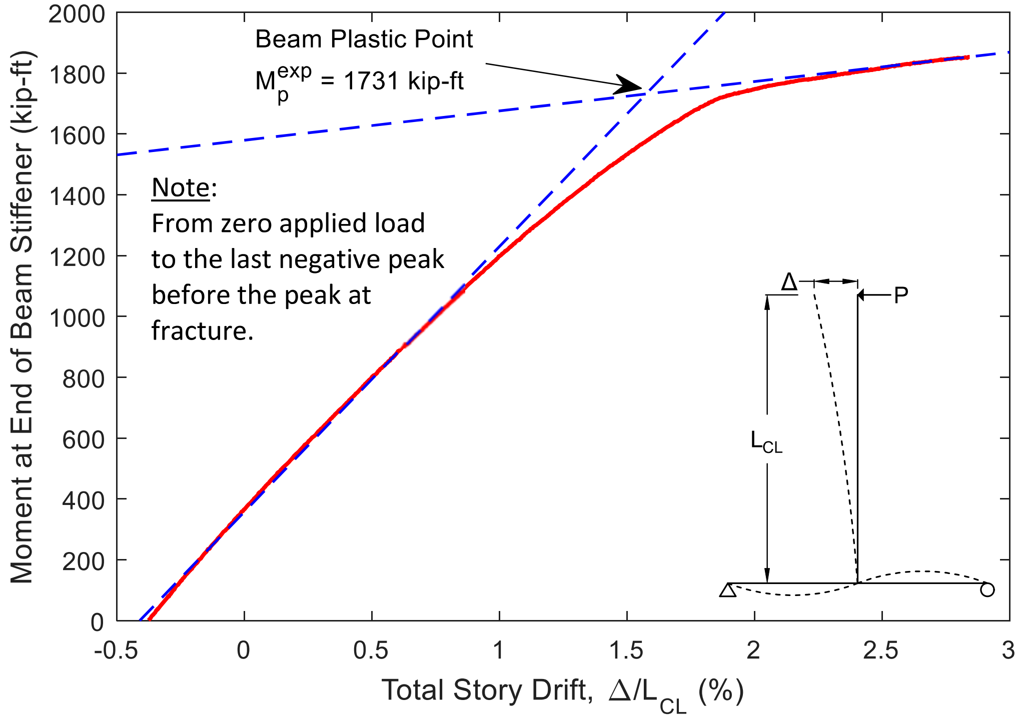


Figure 6-25 Determination of Experimental Plastic Moment Capacity for Specimen 12ES-1.125-1.25-24

6.4.3. Experimental Results – End-Plate Separation

The separation of the end-plate from the column face remained relatively low through the entire test. The maximum end-plate separation at the inside face of the beam flange through the test was just under 0.07 inches. Up to and through the 1% cycles, the end-plate separation at the inside face of the beam flange did not exceed much more than 0.02 inches, which is negligible. The relationship of end-plate separation to applied moment was relatively linear through the 1% cycles. After those cycles, that relationship became nonlinear. The relationship of applied moment versus rotation due to end-plate separation can be seen below in Figure 6-26.

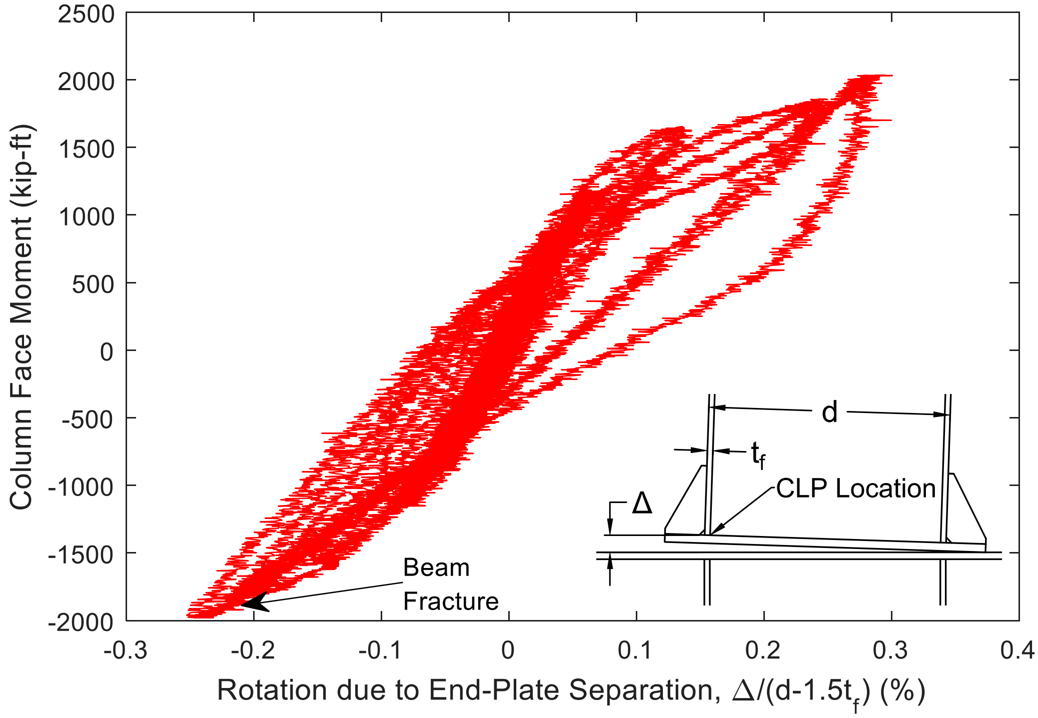


Figure 6-26 Rotation due to End-Plate Separation for Specimen 12ES-1.125-1.25-24

6.4.4. Experimental Results – Bolt Forces

The bolts were 1-1/8" A490, which had a tensile capacity, T_{ult}^{TT}, of 121.2 kip based on the material properties from the bolt tensile testing. They were pretensioned to at least the minimum of 80 kips as prescribed in Table J3.1 of AISC 360-10 (AISC 2010b). The force in each bolt varied by bolt hole location. However, the following is based on one of the bolt hole locations. During the 0.375% story drift cycles, the force in the bolts drop to about 66.5 kips when the flange was under compression, and increased to about 83 kips when the flange was under tension. During the 0.5% cycles, the force in the bolts drop to about 62 kips when the flange was under compression, and increased to about 85 kips when the flange was under tension. This trend continued with each new level of cycles, which can be seen below in Figure 6-27. The maximum recorded bolt force for this bolt location was 109.7 kips at the last peak before fracture, which is notably less than T_{ult}^{TT}. Since the bolt forces were calculated based on a linear calibration of the strain, the bolt forces would become inaccurate when the actual bolt force approaches T_{ult}^{TT} because the behavior of the bolts becomes nonlinear as the actual bolt force approaches T_{ult}^{TT}.

Since the calculated bolt forces shown below in Figure 6-27 are significantly below T_{ult}^{TT}, the calculated bolt forces should be reasonably accurate.

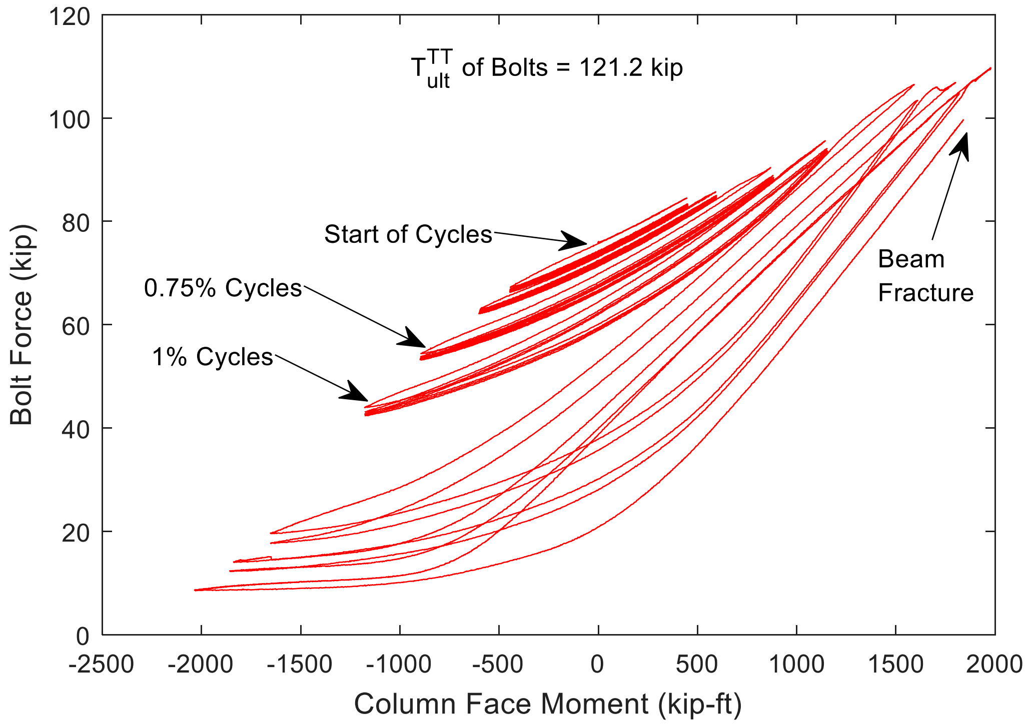


Figure 6-27 Bolt Force vs. Column Face Moment for Specimen 12ES-1.125-1.25-24

6.4.5. Experimental Results – Pictures of Specimen 12ES-1.125-1.25-24

The figures below show the fracture in the beam flange from different views. The fracture going through the entire beam flange can be seen below in Figure 6-28. The fracture initiated at the toe of the stiffener. A close up of the toe of the stiffener after the flange fractured can be seen below in Figure 6-29. The fracture surface can be seen below in Figure 6-31, which indicates the exact location of fracture initiation based on the chevron markings. A view of the fracture from the inside face of the flange can be seen below in Figure 6-30, which shows that the fracture went through the flange, and the fillet welds that connect the web to the flange.

Figure 6-28 View of Fracture from Outside of Flange

Figure 6-29 Close-Up View of Fracture at Stiffener of Specimen 12ES-1.125-1.25-24

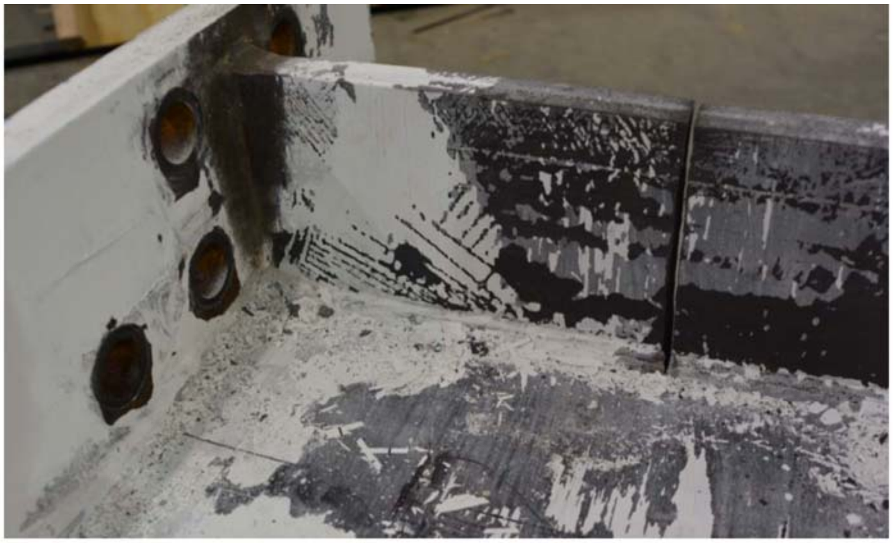


Figure 6-30 View of Fracture from Inside of Flange

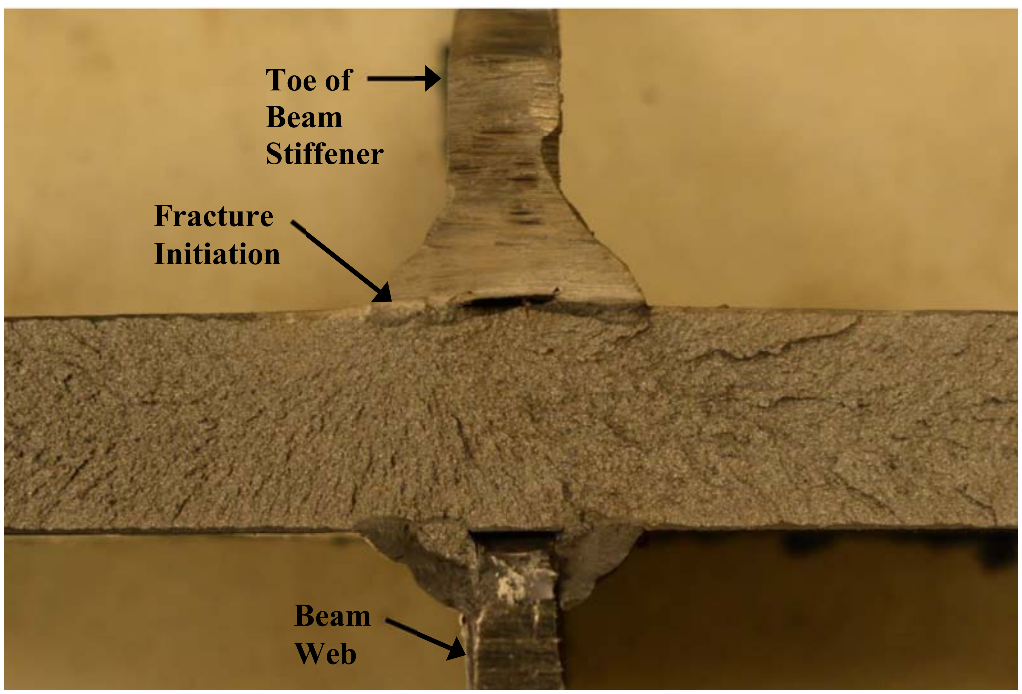


Figure 6-31 Fracture Surface of Specimen 12ES-1.125-1.25-24

7. INVESTIGATION OF BEAM FRACTURES

The fractures that occurred in the deep and shallow beam specimens were unexpected. The goal was to form a plastic hinge in the beam for Specimen 12ES-1.25-1.50-44 and for Specimen 12ES-1.125-1.25-24, and to cycle each beam through the 4% story drift cycles, which is a requirement in the seismic provisions for qualification for SMRF (AISC 2010a). There are many factors that could have contributed to the initiation of those fractures. In order to narrow down these factors into the most probable cause, various different aspects of the testing need to be examined. This chapter explores the potential issues that could have led to brittle fracture, and investigation measures that were conducted to determine the cause(s) of the fractures.

The investigation involved examining and comparing previous cyclic testing of end-plate moment connections, conducting tensile testing of the beam material, performing Charpy V-notch testing of the beam flange material, and doing an idealized cross-section analysis to explore yielding of the stiffener. After this series of investigation, it was determined that the most probable cause of the fractures was the ratio of the yield stress of the stiffener compared to the yield stress of the beam. This ratio was low (less than 1.0) for previous testing, however, it was high (greater than 1.0) for this testing. It is believed that yielding of the stiffeners reduces the concentration of plastic strain and stress triaxiality at the toe of the stiffener. This was supported by finite element models, which are not shown here.

7.1. Summary of Previous Testing

There has been a lot of previous testing on end-plate moment connections of various configurations, which have been tested under monotonic loading, and cyclic loading. The number of bolts at the tension flange varies from four bolts to twelve bolts. The connections can either be flush or extended, and the extended end-plate moment connections can either be stiffened or unstiffened. Examining this previous testing can be used to compare the various aspects of end-plate moment connections and to determine which of the several factors that are most likely to contribute to brittle fracture of the beam. However, not all previous testing is relevant to the specimens that failed in a brittle manner. The previous testing of interest is of extended, stiffened end-plate moment connections that underwent cyclic loading. A summary of the relevant previous cyclic testing of 4-bolt extended stiffened end-plate moment connections is shown below in Table 7-1. A summary of the relevant previous cyclic testing of extended stiffened end-plate moment connections with eight bolts at each flange is shown below in Table 7-2. In both of these tables, the specimens that observed fracture initiation in the flange at the toe of the stiffener are circled in red. Also in both of these tables, the cells highlighted in yellow indicate specimens that underwent cycles at high levels of story drift (at least 4 percent), and have equal grade steel for the beam and the stiffener.

Many of the specimens from the previous testing had stiffeners which were a grade of steel less than 50 ksi. Some of these specimens had a lower grade steel for the stiffeners than for the beam. Using a lower grade steel for the stiffeners would lead to more yielding in the stiffeners. The three specimens that are circled in red used equal grades of steel for the stiffeners and for the beam. Using equal grades of steel would result in less yielding of the stiffener compared to using a lesser grade of steel for the stiffeners than for the beam. Since the specimens circled in red had fracture initiation in the flange at the toe of the stiffener, it could be concluded that the grade of steel used for the stiffeners has an effect on the ductility of the beam at the toe of the stiffener.

Table 7-1 Summary of Previous 4-Bolt Cyclic Testing

Reference	Specimen	Bolt Layout	Bolts	Beam	Beam Material	Stiffener Material	Failure	Max Story Drift (%)	Notes
Blumenbaum 2004	4ES-1.0-1.0-24*	4ES	1" A490	Built-Up 5/8"x7" Flange 3/8" Web	A572 Gr. 50	A572 Gr. 50	Plastic hinging of the beam. Stopped during first 5% cycle due to LTB	4	Stiffeners similar to AISC 358 detail
Meng 1996	3/95	4ES	1" A325	W18x35	A36	A36	Local flange buckling	2.4	Shimmed end-plate
	7/95	4ES	1-1/4" A325	W24x62	A36	A36	Local flange buckling. Yielding of end-plate	4.0	Triangular stiffeners
Ryan 1999	ES-1-1/2-24a*	4ES	1" A325	Built-Up 3/8"x8" Flange 1/4" Web	A572 Gr. 50	A572 Gr. 50	Weld fractures	2.2	Rectangular stiffeners
	ES-1-1/2-24a*	4ES	1" A325	Built-Up 3/8"x8" Flange 1/4" Web	A572 Gr. 50	A572 Gr. 50	Local buckling of bottom flange. Flange-to-End-Plate weld fracture along entire length of bottom flange. Stiffener-to-End-	3.4	
	ES-1-1/2-24b*	4ES	1" A325	Built-Up 3/8"x8" Flange 1/4" Web	A572 Gr. 50	A572 Gr. 50	Fracture of end-plate at the stiffener-to-end-plate interface, both flanges.	2.2	
Ghobarah et al. 1992	CC-3	4ES	1" A490	W16x40	A36	A36	Local flange buckling	6.1	Triangular stiffeners
Korol et al. 1990	A-3	4ES	1" A490	W14x30	300 MPa	300 Mpa	Beam local buckling	$\theta_p = 6$	Triangular stiffeners
	A-5	4ES	1" A490	W14x30	300 MPa	300 Mpa	Crack at the toe of the end-plate stiffener	$\theta_p = 6$	
	B-2	4ES	1" A490	W14x38	300 MPa	300 Mpa	Damage to column flange and beam buckling	$\theta_p = 6$	
Shi Shi & Wang 2007	JD2	4ES	3/4" GR 10.9	Built-Up 0.5"x7.875" Flange 5/16" Web	Q345	N/P	Bolt rupture	$\phi = 4.5$	11.8" Beam depth. Triangular stiffeners.
	JD4	4ES			Q345	N/P	Buckling of the column web panel in compression, and bolt rupture	$\phi = 4$	
	JD5	4ES			Q345	N/P	Bolt rupture	$\phi = 3.8$	
	JD8	4ES			Q345	N/P	End-plate stiffener fracture, end-plate fracture, bolt rupture	$\phi = 4.5$	
	JD6	4ES	1" GR 10.9		Q345	N/P	End-plate stiffener fracture, shearing buckling of the panel zone, local buckling of the column flange, weld between the beam flange, and end-plate crack	$\phi = 7.9$	
	JD7	4ES			Q345	N/P	End-plate stiffener fracture, weld between the beam, and end-plate crack	$\phi = 4.1$	
Tsai & Popov 1990	10R	4ES	1" A354BD	W18x40	A36	A36	Flange and web buckling	5	Stiffeners similar to AISC 358 detail

*Test Identification: "Connection Type - Bolt Size - End-Plate Thickness - Beam Depth"
4ES = 4-Bolt Extended Stiffened
N/P = Not Provided
θ_p = Beam inelastic rotation
ϕ = Rotation capacity of the specimen connection

Table 7-2 Summary of Previous 8-Bolt Cyclic Testing

Reference	Specimen	Bolt Layout	Bolts	Beam	Beam Material	Stiffener Material	Failure	Max Story Drift (%)	Notes
Adey et al. 2000	M-5	8ES	1-1/8" A325	W18x65	CAN/CSA-G40.21-92 GR300W (43.5 ksi)	CAN/CSA-G40.21-92 GR300W (43.5 ksi)	Lateral buckling of beam at plastic moment	5.2	Triangular stiffeners. Developed full flexural strength of beam prior to failure
	M-7	8ES	1-1/8" A325	W18x65			Rupture of end-plate around beam flange pullout of stiffener from beam flange	4.9	
	L-5	8ES	1-1/8" A325	W24x84			Excessive deflection of test frame	N/A	
Toellner 2013	10	8ES	1-3/8" A490	W36x150	A992	Gr. 50	Completed 5 cycles at 4.7% story drift. Tearing in flange at stiffener toe	4.7	No PAFs. Stiffeners according to AISC 358
	11	8ES	1-3/8" A490	W36x150	A992	Gr. 50	Completed 5 cycles at 4.7% story drift. Tearing in flange at stiffener toe	4.7	PAFs at 12 inches on flange
Sumner & Murray 2002	8ES-1.25-1.75-30*	8ES	1-1/4" A490	W30x99	A572 Gr. 50	A36	Plastic hinging of the beam	5	Column with cantilever beam
	8ES-1.25-2.5-36*	8ES	1-1/4" A490	W36x150	A572 Gr. 50	A36	Plastic hinging of the beam	5	
Seek & Murray 2008	1	8ES	1-1/8" A325	W27x84	A992	A36	Test stopped short of 4% story drift due to test frame.	3	Two beams with concrete slab.
Meng 1996	12/95	8B-4WS	1-1/4" A325	W36x135	A36	A36	Local flange buckling	3.2	Weld access holes

*Test Identification: "Connection Type - Bolt Size - End-Plate Thickness - Beam Depth"
8ES = 8-Bolt Extended Stiffened
8B-4WS = Stiffened 8-Bolt - 4-Bolts Wide

Out of all of the testing, the tests conducted by Toellner stands out the most because there were similarities in connection geometry as the testing presented here, but the beam did not fracture in a brittle manner. However, there was fracture initiation in the beam flange at the toe of the stiffener, which means that there is some overlap between the specimens that fractured in a brittle manner and the specimens tested by Toellner. The specimens by Toellner were about 36 inches deep, the stiffener geometry met the geometry prescribed by AISC 358, the bolts were A490, and the specimens completed cycles at high levels of story drift (Toellner 2013). The two important differences between these specimens and the shallow specimen that fractured were the material, and the beam sections used. The beams in Toellner's tests were hot rolled W-sections made of A992 steel, whereas the beams tested here were built-up sections made from a combination of plate material, either A529 or A572 steel. Although fractures started to initiate at the toe of the stiffeners in those two specimens tested by Toellner, they did not fracture in a brittle manner at relatively low levels of story drift.

The test by Blumenbaum was also an important test because it was the only built-up beam specimen that was tested through high levels of story drift. The beam was 24 inches deep. However, the section was not as heavy as the 24 inch deep specimen tested here. Although the bolts were also A490, there were only four bolts at the tension flange. From this previous testing, there appears to be no effect of the number of bolts on the plastic hinging capabilities. However, for the same level of story drift, more bolts could push more strain demand into the beam. This means that there could be an upper limit on the number of bolts at the tension flange. However, for the same size beam, assuming that the bolts are placed symmetrically about the flange, which is the case for the 4ES, 8ES, 12ES, and 8B-4WS, there is a minimum total bolt area that is required to achieve enough capacity for thick end-plate behavior in order to form a plastic hinge in the beam. If each of these configurations were designed for the same size beam, this means that if the minimum bolt size is selected, the total area of bolts at the tension flange will roughly be equivalent. Assuming that the bolt lengths are the same, the connections will have the same stiffness as a result of the bolts because stiffness depends on the modulus of elasticity, the area, and the length. For the end-plate moment connection configurations listed above and based on the assumptions mentioned, all three of those variables would be equal, and thus the connection stiffness would be equal. This means that having more bolts at the tension flange should not have an impact on the strain demands in the beam.

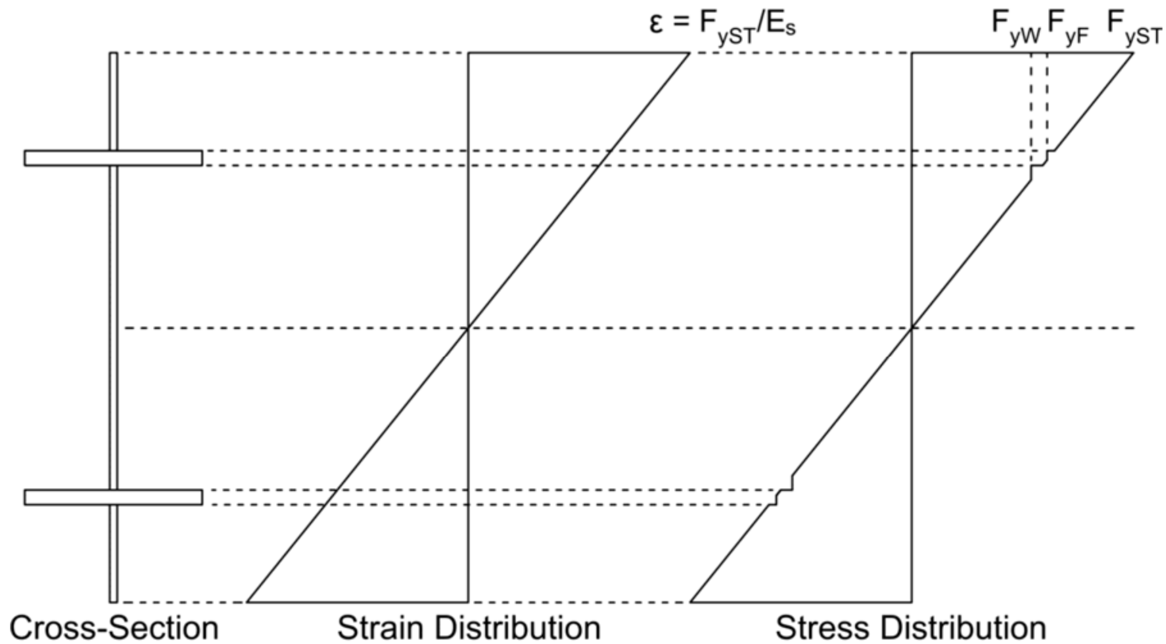
Between all of the testing listed above in Table 7-1, there were three different stiffener geometries used: triangular, rectangular, and the geometry prescribed by AISC 358. Based on this testing, stiffener geometry does not have a significant impact on the performance of the beam. This means that it is unlikely that stiffener geometry played a role in the brittle fracture of the beams.

7.2. Stiffener Yielding

The stiffeners yielded for Specimen 12ES-1.25-1.50-44 and for Specimen 12ES-1.125-1.25-24, both of which were the specimens that were designed to form a plastic hinge. After studying the previous testing, several examples were found of stiffener yielding. Examples of stiffener yielding can be found in Figure 7 of Adey et al. 2000, in Figure 51 from Toellner 2013, in Figure 6 from Seek and Murray 2008, and in Figure 7 from Sumner and Murray 2002. These examples show that the yielding of the stiffeners is common for extended stiffened end-plate moment connection. More yielding in the stiffener reduces the potential for fracture at the toe of the stiffener.

More investigation was done through simple cross-sectional analysis. An assumed strain distribution was applied to the beam cross-section, which was based on yielding initiating at the extreme fiber of the beam. A stress distribution was calculated based on the assumed strain distribution. In the unstiffened region of the beam, the extreme fiber was the flanges, whereas in the stiffened region of the beam, the extreme fiber was the stiffeners. For the analysis of the stiffened region, the assumed strain distribution and the resulting stress distribution that were used are shown below in Figure 7-1. Since the yield strength of the stiffener, the flanges, and the web could all be significantly different from each other, partially plastic sections of the flanges and web were taken into account. Since the yield strength of the stiffener for Specimen 12ES-1.125-1.25-24 was much greater than the yield strength of the flanges, partially plastic flanges were a likely possibility.

For each beam specimen that was analyzed, hundreds of cross-section cuts were taken in the stiffened region of the beam to take into account the change in stiffener height across the length of the beam. At each cross-section cut, the stress distribution was used to calculate a moment capacity, which is later referred to as the yield moment capacity, M_y. A similar analysis was applied to determine the yield moment capacity of the beam outside the stiffened region.



Note: F_{yST} – Yield strength of the stiffeners
F_{yW} – Yield strength of the web
F_{yF} – Yield strength of the flanges

Figure 7-1 Cross-Section Analysis Stress & Strain Distribution

Various sections were analyzed, including the two plastic hinging specimens from this testing, five specimens from previous testing, and varying beam geometry. The dimensions that were varied were stiffener thickness, flange thickness, flange width, and stiffener length. While those dimensions varied in separate cases, all other dimensions were those of Specimen 12ES-1.125-1.25-24. The figures below in this section show a moment ratio of M to M_y, where M is calculated as the demand when M = M_p at the plastic hinge, and M_y is the yield moment capacity at each section. The moment demand, M, was calculated based on the moment diagram corresponding to the loading shown in Figure 7-2 below. For each section analyzed, M_p was calculated based on the same material properties that were assumed for M_y. t_p is the thickness of the end-plate. Yielding is indicated when this ratio, M / M_y, is greater than 1.0. For the figures below, where specimens with different length stiffeners are compared, the x-axis was normalized by the length of the stiffener, L_{st}, for each specimen.

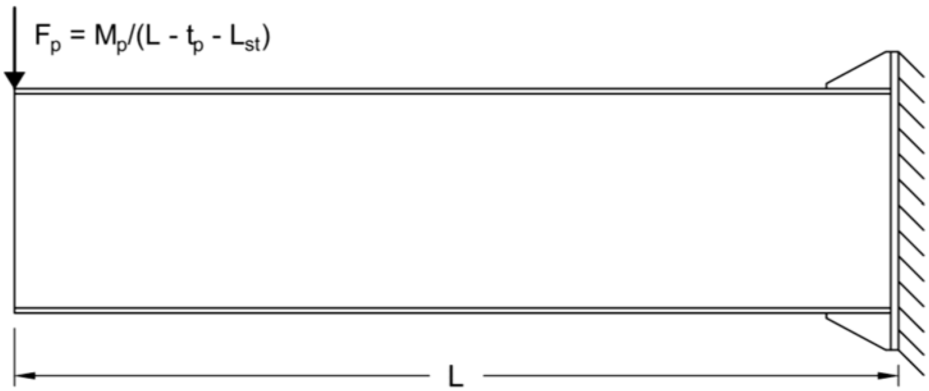


Figure 7-2 Cross-Section Analysis Moment Demand

7.2.1. Plastic Hinging Specimens

The analysis of Specimen 12ES-1.25-1.50-44 and Specimen 12ES-1.125-1.25-24 was based on F_y^{TT}, which is the yield strength of the material based on tensile testing. Since the beams for these two specimens were built-up sections, the effect of the different yield strengths of the stiffeners, the web, and the flanges were taken into account. Figure 7-3 below shows the analysis of Specimen 12ES-1.25-1.50-44 and Specimen 12ES-1.125-1.25-24, as well as the analysis of each specimen based on F_y^{Nom}, which is the nominal yield strength of the material. For these specimens, the nominal yield strength of all of the material was 55 ksi. The M_p that was used to determine the moment demand for each specimen was calculated based on the appropriate yield strength values, whether nominal or from tensile testing, as indicated for each specimen in Figure 7-3 below.

The specimens based on nominal yield strength indicate more yielding in the stiffener because for each case based on material properties from tensile testing, the yield strength of the stiffener was greater than the yield strength of the beam flange and beam web. The difference in the amount of yielding between using nominal and using actual yield strengths was greater for the 24 inch deep specimen because the difference in yield strength of the stiffener and the yield strength of the beam flange was greater for the 24 inch deep specimen than for the 44 inch deep specimen.

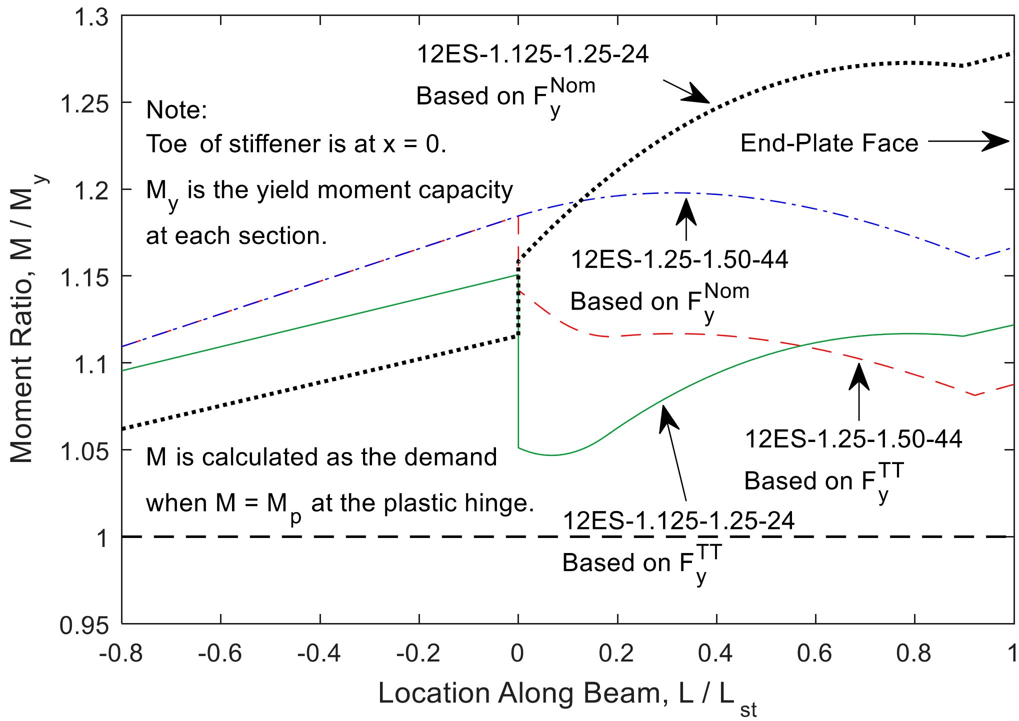


Figure 7-3 Yielding of Extreme Fiber for Plastic Hinging Specimens

7.2.2. Specimens from Previous Testing

Below, Figure 7-4 shows the analysis of specimens from previous testing, which were included in Table 7-1 and Table 7-2 above. M_y^{TT} is the yield moment capacity based on material properties from tensile testing. $M_p{}^{TT}$ is the plastic section moment capacity of beam calculated based on material properties from tensile testing. The analysis of the specimens from previous testing used the appropriate yield strength values that were reported in their respective papers. For each specimen, the authors and specimen number are referenced below. The specimens M-5 by Adey et al. and 10 by Toellner were chosen because it was determined from pictures in their respective papers that the stiffeners yielded during testing (Adey et al. 2000; Toellner 2013).

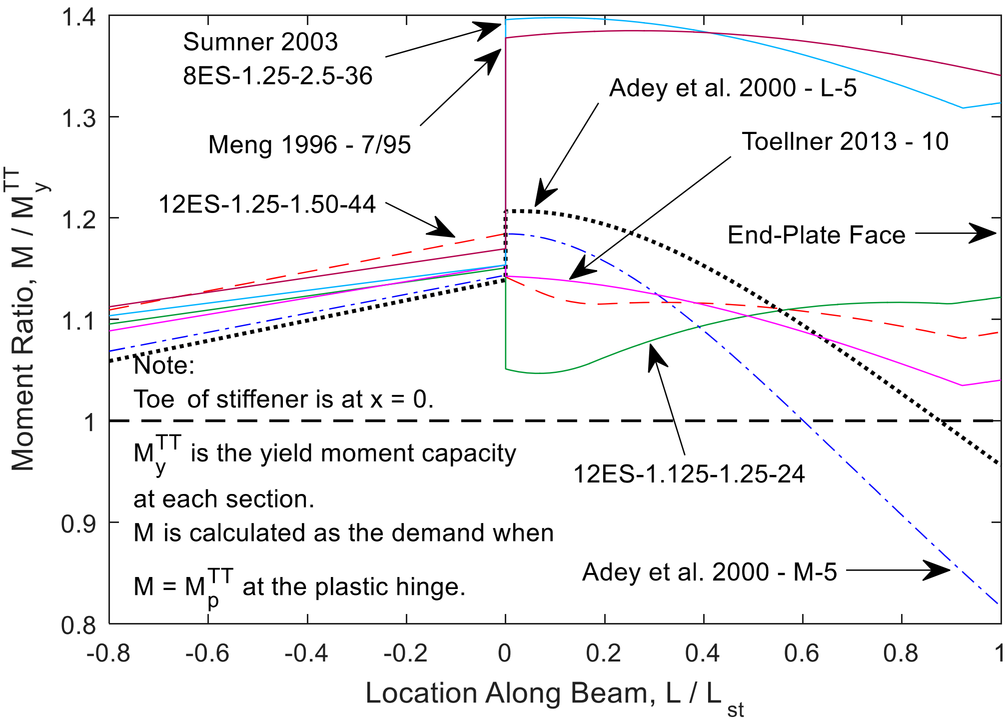


Figure 7-4 Yielding of Extreme Fiber for Specimens from Previous Testing

All of the specimens from previous testing included in Figure 7-4 above were able to undergo cyclic displacement based loading and achieve high levels of story drift (at least 4 percent). The main specimen that draws for comparison here is the one tested by Toellner because fracture initiated in the beam flange at the toe of the stiffener. The specimen tested by Toellner did not fracture in a brittle manner though. After examining Figure 7-4, the three specimens that had fracture initiation in the flange at the toe of the stiffener have a lower ratio of M to M_y than the rest of the specimens in the region of the toe of the stiffener. The two specimens that fractured in a brittle manner have the lowest ratio near the toe of the stiffener. The specimen tested by Toellner draws the separation between the specimens that fractured in a brittle manner and the ones that exhibited ductile behavior. Since the ratio of M to M_y generally indicates the level of yielding in the stiffener and a greater ratio indicates a greater level of yielding in the stiffeners, it can be concluded from Figure 7-4 that a greater level of yielding in the stiffeners reduces the potential for fracture.

7.2.3. Varying Beam Geometry

Below, Figure 7-5, Figure 7-6, Figure 7-7, and Figure 7-8 show the influence of t_s, t_f, b_f, and L_{st} on the yielding of the stiffener, respectively. t_s is the thickness of the stiffener. t_f is the thickness of the beam flange. b_f is the width of the beam flange. L_{st} is the stiffener length. For each case, the beam and stiffener geometry was the same as Specimen 12ES-1.125-1.25-24 other than the dimension that was varied. The yield strength of the stiffener, the flanges, and the web was equal to the nominal yield strength of 55 ksi. It can be seen below that increasing t_s reduces stiffener yielding, whereas increasing t_f or b_f increases the yielding of the stiffener. Increasing the stiffener length increases the yielding in the stiffener. However, the increase in stiffener yielding is the greatest at the end-plate end. There is no increase in stiffener yielding at the toe of the stiffener for a longer stiffener.

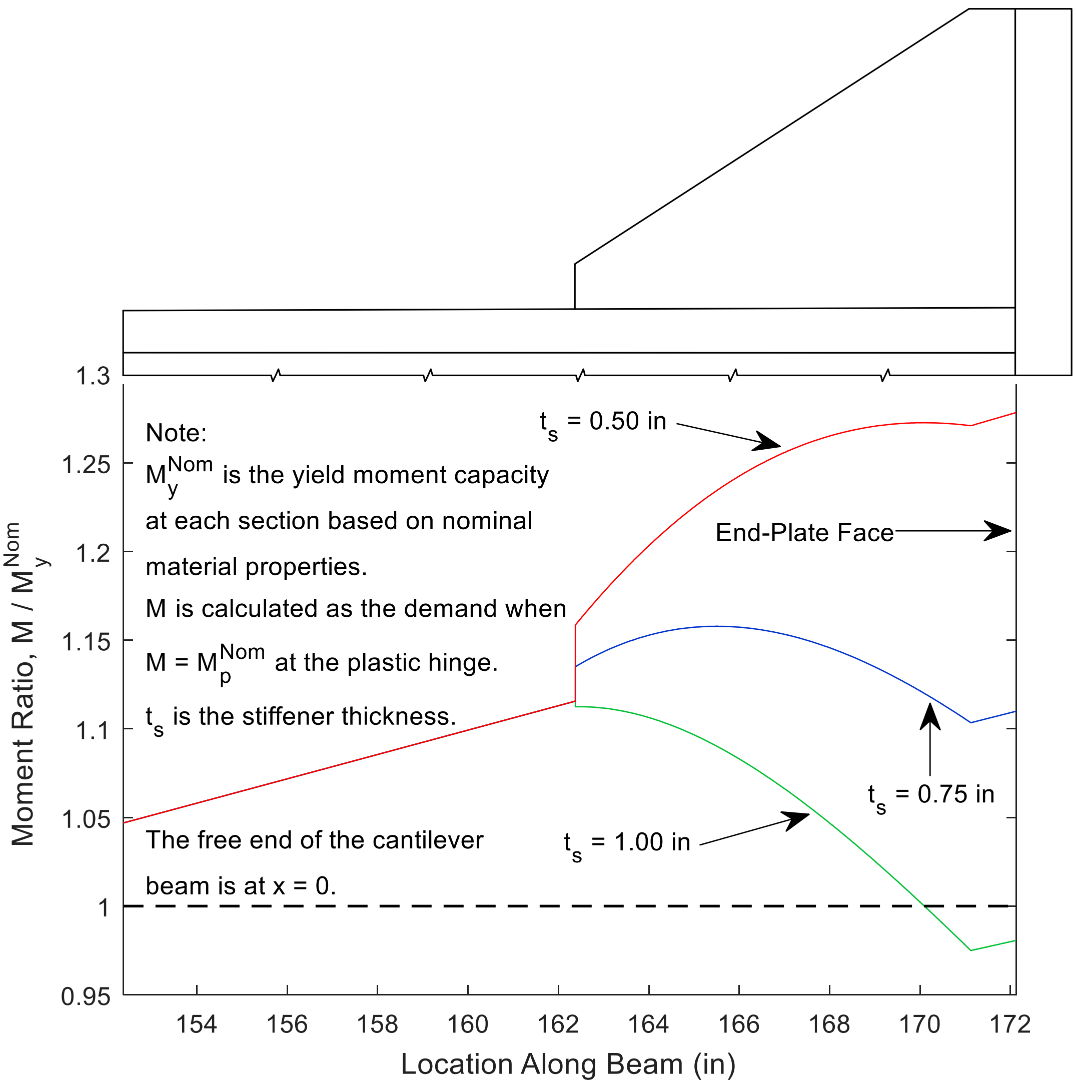


Figure 7-5 Yielding of Extreme Fiber for Varying Stiffener Thickness, t_s

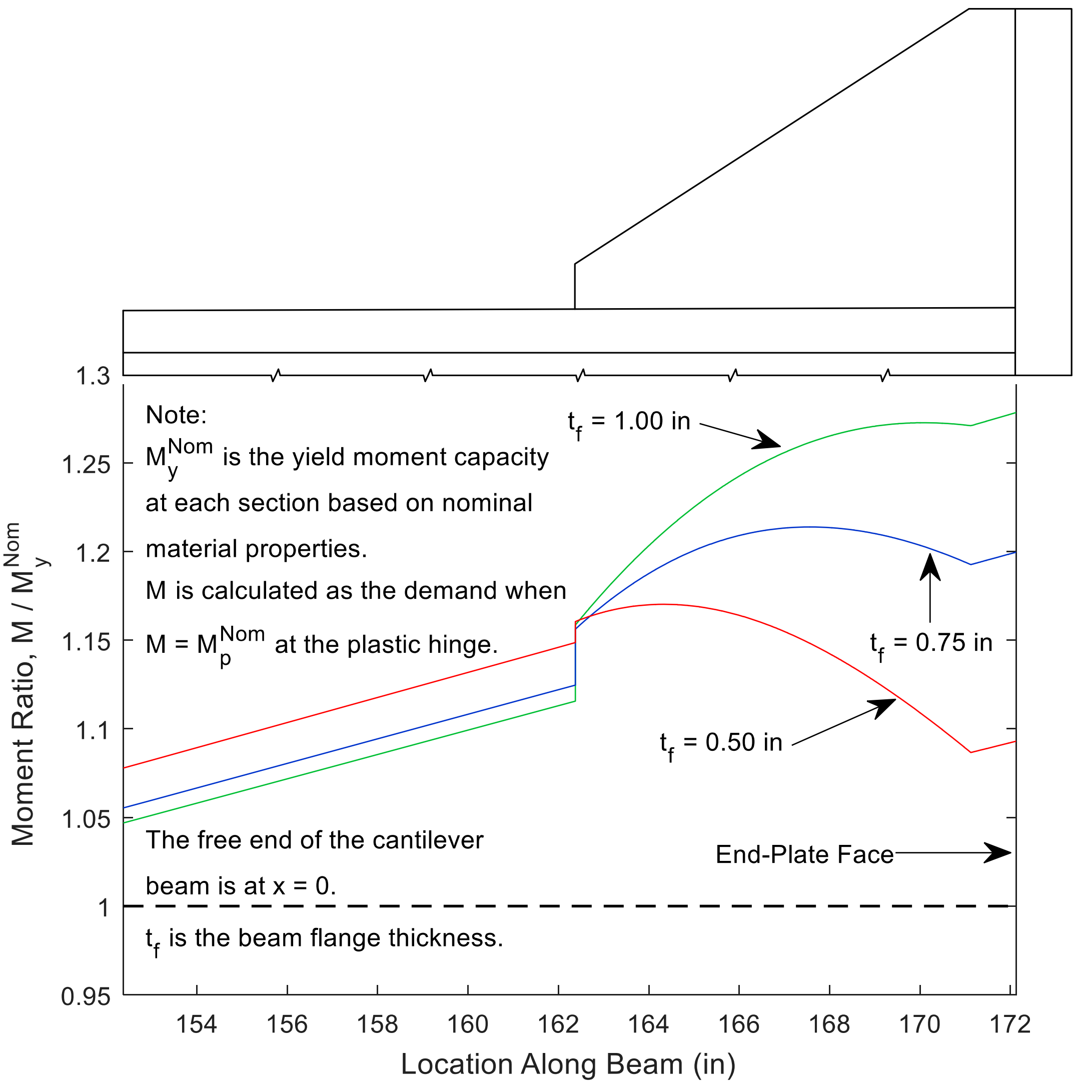


Figure 7-6 Yielding of Extreme Fiber for Varying Beam Flange Thickness, t_f

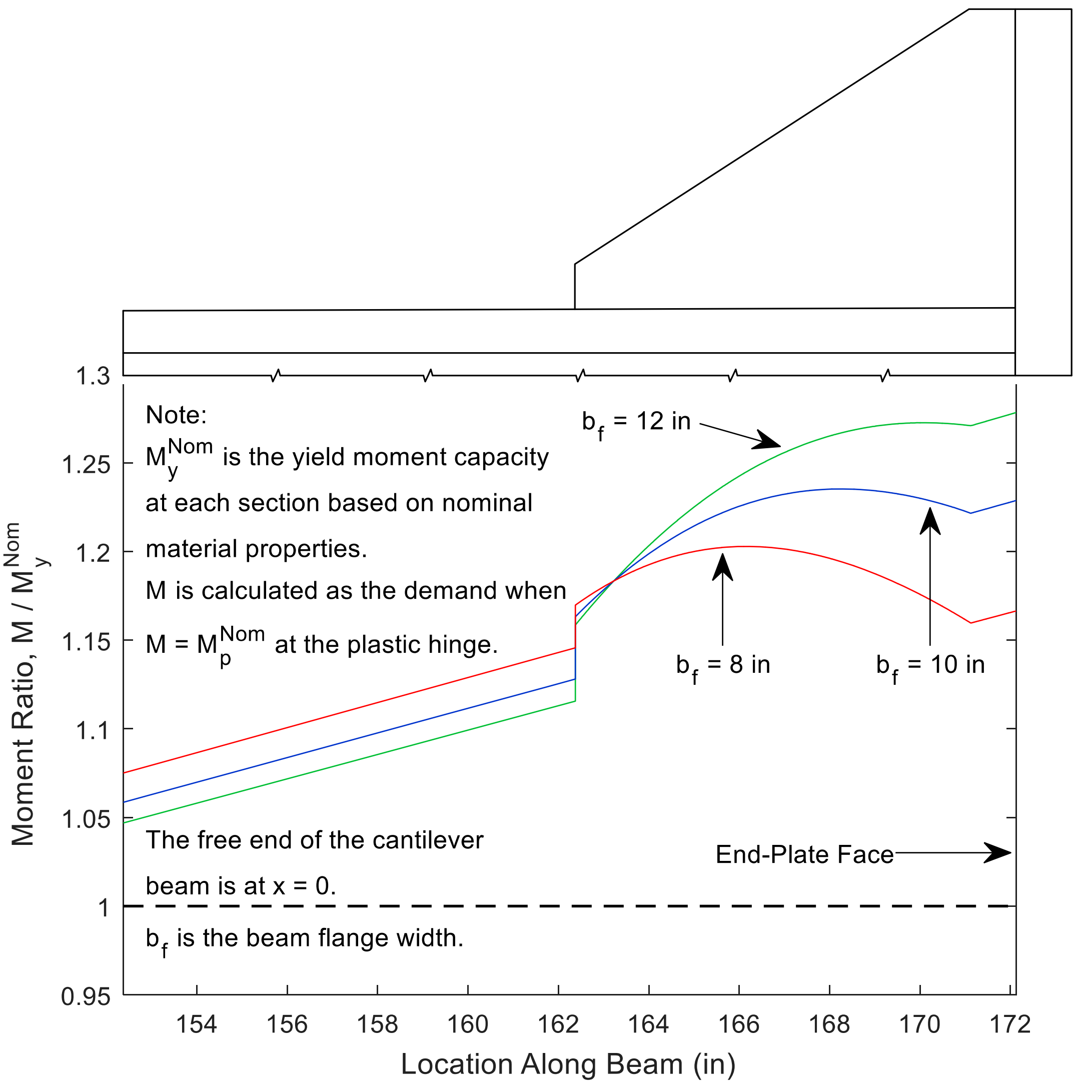


Figure 7-7 Yielding of Extreme Fiber for Varying Beam Flange Width, b_f

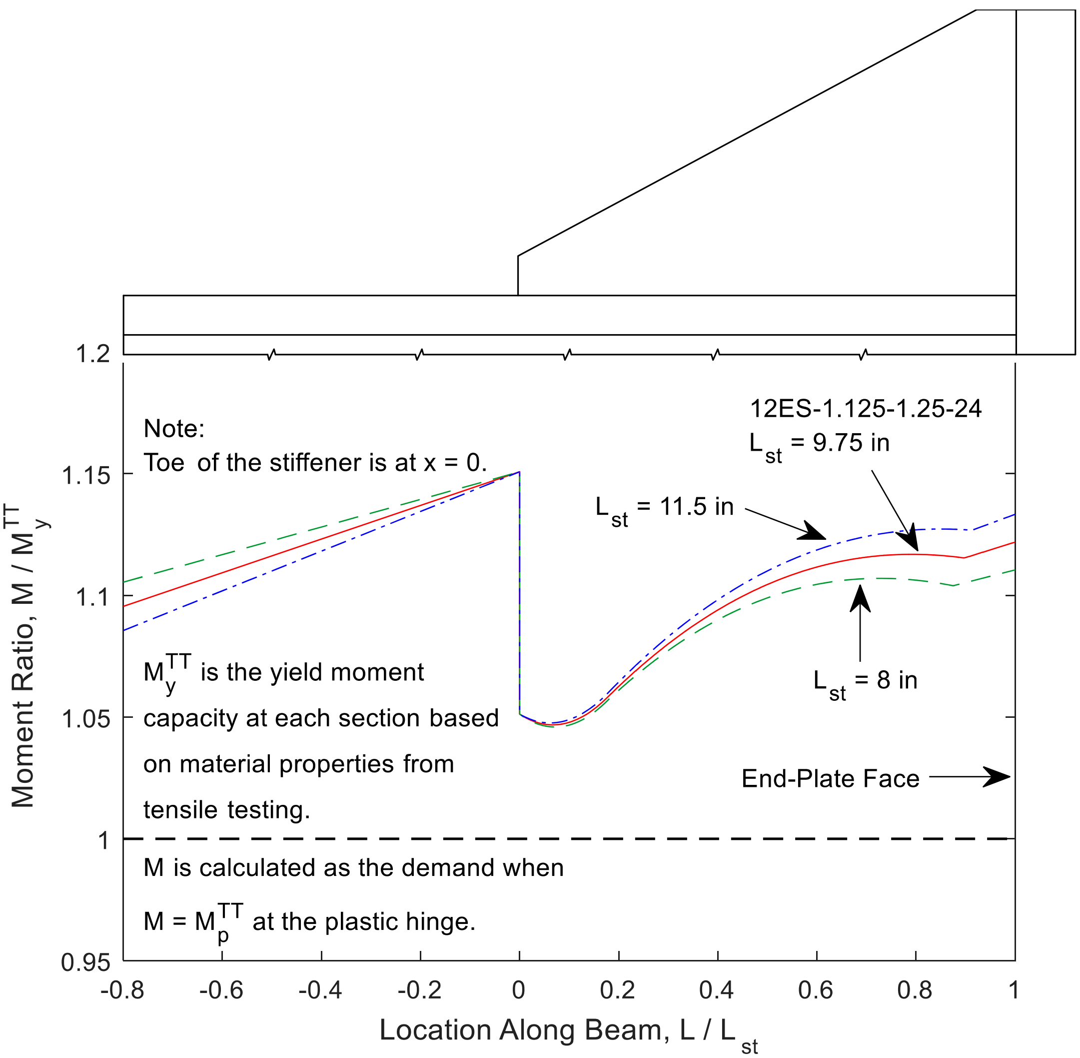


Figure 7-8 Yielding of Extreme Fiber for Varying Stiffener Length, L_{st}

7.3. Material Testing

Testing of the beam material was conducted. This included tensile testing of the flange material, the web material, and the end-plate material, as well as Charpy V-notch testing of the beam flange material. The results of this testing can be found above in Chapter 5 under Section 5.1.2 and Section 5.1.3. The data for the Charpy V-notch testing of the beam flange material was inconsistent. Variable data for Charpy V-notch testing generally indicates that the material is in the transition region. The Charp V-notch specimens were tested at room temperature like the beam specimens. If the fracture toughness curve is in the transition region at room temperature, then there is a potential for low toughness material. The presence of low toughness material may have contributed to the brittle fractures that were observed.

7.4. Summary of Investigation

This investigation examined the potential causes of the beam fractures by exploring previous cyclic testing of end-plate moment connections, the effect of stiffener, and the results of the material testing. Four possible ways were determined to improve the fracture resistance, which include: reduce the stiffener yield stress, reduce the stiffener thickness, adjust the geometry of the stiffener, and weaken the stiffener with hole or cutouts. Each of these approaches would increase the level of yielding in the stiffener, which would reduce the concentration of plastic strain and stress triaxiality at the toe of the stiffener. In addition, the fracture resistance may be increased through detailing, such as wrapping the welds around the toe of the stiffener, and grinding the weld with burr grinder, or peening the surface of the welds around the toe of the stiffener.

8. SUMMARY AND CONCLUSIONS

8.1. Summary

For this research, the application of interest for end-plate moment connections is in special moment resisting frames (SMRF). To ensure that these connections are able to undergo the large demands of an earthquake, they must be tested at full-scale and meet certain criteria. This is detailed by AISC's *Seismic Provisions for Structural Steel Buildings* (AISC 2010a). Connections for steel moment resisting frames that have gone through the full-scale testing and met the demand criteria are considered prequalified. These connections are detailed in AISC's *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC 2011). Currently, three different end-plate moment connection configurations are prequalified, which include: the 4-bolt extended unstiffened, the 4-bolt extended stiffened, the 8-bolt extended stiffened. These current options of prequalified end-plate moment connections are limiting. In order to design buildings with longer spans, connections require a greater moment capacity. The moment capacity of end-plate moment connections is limited by the number of bolts are each flange, since the maximum size bolt is 1-1/2 inches. A configuration with more bolts at each flange is needed to meet this demand for longer spans, and thus connections with greater moment capacities. To satisfy this demand, an end-plate moment connection with twelve bolts at each flange has been developed.

The objective of this research was to propose and verify a design procedure for a new configuration of end-plate moment connection with twelve bolts at the tension flange: twelve-bolt extended stiffened (12ES). An expected outcome would be to have this connection prequalified for use in SMRF.

A design procedure was proposed for this twelve-bolt end-plate moment connection. The design procedure for thick-plate behavior was based on previous work, where each of the bolts at the tension flange are assumed to reach full strength simultaneously. For thin end-plate behavior, a yield line analysis was conducted for several yield line patterns following assumptions, which were states previously in Chapter 3. Since the kinematic approach was used, which is an upper bound method, an analysis of the different yield line patterns was performed in order to determine which case produced the minimum moment capacity associated with end-plate yielding. After an end-plate yields, prying action occurs in the bolts. The moment capacity associated with bolt tension rupture with prying action was determined by using an effective tee stub model.

A total of four specimens were tested, which were a new twelve bolt end-plate moment connection configuration. These specimens were tested with the intention of qualifying the connection, and validating the proposed design procedure. Two specimens were a deep beam, which was 44 inches deep. The other two specimens were a shallow beam, which was 24 inches deep. Of the deep beam specimens, one was designed to fail due to bolt tension rupture without prying action, and the other was designed to fail due to plastic hinging of the beam. One shallow beam specimen was designed for thin end-plate behavior, and thus designed to fail due to bolt tension rupture with prying action. The other shallow beam specimen was designed to fail due to plastic hinging of the beam. All specimens were connected to the same size columns, which were 42 inches deep. The columns were designed based on the strong column – weak beam criterion, and were designed to have minimal panel zone deformation.

The testing of the deep specimen that was designed to fail due to bolt tension rupture without prying action exhibited the limit states and strength as expected. All twelve bolts at the tension flange failed simultaneously, which agrees with the thick end-plate bolt force model. The beam and end-plate remained elastic. The predicted capacity was conservative by just under 7%. The testing of the shallow specimen that was designed to fail due to bolt tension rupture with prying action also went as expected. The predicted moment capacity associated with end-plate yielding was about 9% conservative compared to the experimentally determined value. The end-plate yielded first, and then the beam yielded at a higher level of applied displacement. The prediction for the moment capacity associated with bolt tension rupture with prying action was just under 19% conservative compared to the ultimate moment.

During the testing of the beam specimens that were designed to form a plastic hinge, rather than progressing through the 4% story drift cycles, both beam specimens fractured significantly before reaching the 4% cycles. The flange and half the depth of the web of the deep beam fractured in the first 2% cycle. The flange of the shallow beam completely fractured in the second 3% story drift cycle. The fractures for both of these beam specimens occurred at the toe of the beam stiffener, and initiated at the outside face of the flange where the fillet weld from the stiffener meets the flange. The applied moment at the end of the stiffener for both of these specimens exceeded the expected plastic moment capacities. For the deep beam specimen, the experimentally determined plastic moment capacity was just over 2% of the predicted capacity. For the shallow beam specimen, the experimentally determined plastic moment capacity was within 1% of the predicted capacity.

8.2. Conclusions

The primary conclusions that can be drawn from this testing of this new twelve-bolt end-plate moment connection are as follows:

- The thick end-plate bolt force model conservatively predicts M_{np}.
- The yield line model conservatively predicts M_{pl}.
- The effective tee stub model conservatively predicts M_q.
- The partial joint penetration welds from the beam flange to end-plate exhibited sufficient capacity throughout the testing.
- Previous cyclic testing of 4ES and 8ES end-plate moment connections indicates that brittle fracture of the beam is not common. However, a few previous specimens did have fracture initiation in the beam flange at the toe of the stiffener. The fracture initiation was after achieving high levels of story drift though.

The causes of the fracture in the specimens that were designed for plastic hinging of the beam are as follows:

- High yield stress stiffeners, resulting in a low level of stiffener yielding
- Increased strain demands in deeper beams
- Low fracture toughness beam flange material
- High concentration of plastic strain and stress triaxiality at the toe of the stiffener

8.3. Potential Future Research

The investigation of the beam fractures indicates that the yield stress of the stiffener is the main cause of the fractures. However, more research is required to further prove that stiffener yield stress is the culprit. Then, additional research is required to evaluate potential solutions. The following is a suggested research program to further prove that stiffener yield stress is the issue, as well as to evaluate potential solutions. This program can be conducted either experimentally or computationally; however, computationally is recommended.

The details of the specimens to further prove the issue with the stiffeners are listed below. This series of specimens can be grouped into two set, a) and b), as shown below.

- Beam geometry: $d = 24"$ $b_f = 12"$ $t_f = 1"$ $t_w = 0.5"$
- Beam material: grade 55 steel
- Use AISC 358 detail for stiffener geometry, and use the same length stiffener for all specimens

a) Same stiffener thickness, but vary stiffener grade for each specimen:
   1) Grade < 55
   2) Grade 55
   3) Grade > 55

b) Grade 55 stiffener, but vary stiffener thickness:
   1) Minimum thickness allowed by AISC 358 based on stiffener local buckling check
   2) ½*Minimum thickness
   3) 2*Minimum thickness

The details of the specimens to evaluate potential solutions are listed below.

- Beam geometry: $d = 24"$ $b_f = 12"$ $t_f = 1"$ $t_w = 0.5"$
- Beam and stiffener material: grade 55 steel

a) Vary the stiffener geometry (including thickness and length) to produce the following:
   1) Minimal stiffener yielding across length of stiffener
   2) High yielding in stiffener at toe, and low yielding in stiffener near end-plate
   3) Low yielding in stiffener at toe, and high yielding in stiffener near end-plate
   4) High stiffener yielding across length of stiffener

After a potential solution has been determined, in order to prove that this solution will work, experimental testing must be conducted. This experimental testing should include specimens with stiffeners that are designed to allow for the beam to exhibit ductile behavior, as well as specimens with stiffeners that are designed to inhibit the ductility of the beam.

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APPENDIX A MILL TEST REPORTS FOR BEAM MATERIAL

DATE
06/27/2016

Department / Resource Worklist Report
For **FRAMES** batch **1045092**
ENGINEERED PARTS

RAI

Orders: 1601181101

Department: 753B
Dept Desc: FRAMES - COMPONENTS FABRICATION
Resource: 003
Resource Desc: PLATE LINE
Raw Material: 0093888
RM Description: 0.7500" X 12.00" X 480" 55K

Ship Date: 15-JUN-16

Batch: 1045092
Sort: 16596754

16596754

0093888

Total Qty: 0.1 EA

Item/ Part Mark/Job	Description/Next Step	Stk/Seq	Qty	Dimensions	
003SP120062104*30305871 SBX002 1601181101:26008489:VP - HOUSE/D	FG-PLATE-34.500 752 - 22222 - CONRAC AUTOWELD - MACHINE TIME	1-2	1	L:	34.500
				T:	0.75
				W:	12.000

CAMGen File(s):01-1045092.FMI (20.PRT), 09-1045092.PCH (20.PRT), 20.txt)
Row 1 at 1.625, 3.75(.9375), 7.75(.9375), 26.75(.9375), 30.75(.9375)
Row 2 at 4.25, 1.125(.9375), 3.75(.9375), 7.75(.9375), 10.375(.9375), 24.125(.9375), 26.75(.9375), 30.75(.9375), 33.375(.9375)
Row 3 at 7.75, 1.125(.9375), 3.75(.9375), 7.75(.9375), 10.375(.9375), 24.125(.9375), 26.75(.9375), 30.75(.9375), 33.375(.9375)
Row 4 at 10.375, 3.75(.9375), 7.75(.9375), 26.75(.9375), 30.75(.9375)
Weld Mark: 5.250

Sort Length: 2.9'

Heat #
~~3949727~~
5504486?

GERDAU

Page 1/1

US-ML-CARTERSVILLE
384 OLD GRASSDALE ROAD NE
CARTERSVILLE, GA 30121
USA

CUSTOMER SHIP TO	CUSTOMER BILL TO	GRADE	SHAPE / SIZE	DOCUMENT ID:
BLUESCOPE BUILDINGS NORTH AMER 432 DILBECK RD W RAINSVILLE,AL 35986-4310 USA	BLUESCOPE BUILDINGS N AMERICA KANSAS CITY,MO 64141-6917 USA	A529-55M	Flat Bar / 3/4 X 12	0000025337

LENGTH	WEIGHT	HEAT / BATCH
40'00"	42,840 LB	55044867/03

SALES ORDER	CUSTOMER MATERIAL N°	SPECIFICATION / DATE or REVISION
3804452/000040	093888	ASTM A529-14 ASTM A6-14

CUSTOMER PURCHASE ORDER NUMBER	BILL OF LADING	DATE
298154	1323-0000071825	06/15/2016

CHEMICAL COMPOSITION

C %	Mn %	P %	S %	Si %	Cu %	Ni %	Cr %	Mo %	V %	Nb %	N %	Pb %
0.19	0.98	0.018	0.032	0.17	0.34	0.13	0.12	0.031	0.031	0.000	0.0100	0.0000

CHEMICAL COMPOSITION

Sn %
0.015

MECHANICAL PROPERTIES

Elong. %	G/L Inch	UTS PSI	UTS MPa	YS 0.2% PSI	YS MPa
18.50	8.000	82000	565	61800	426
18.90	8.000	83200	574	60900	420

COMMENTS / NOTES

MB 6/17/16

Ronnie Walton

6-15-16

The above figures are certified chemical and physical test records as contained in the permanent records of company. We certify that these data are correct and in compliance with specified requirements. This material, including the billets, was melted and manufactured in the USA. CMTR complies with EN 10204 3.1.

Bhaskar
BHASKAR YALAMANCHILI
QUALITY DIRECTOR

YAN WANG
QUALITY ASSURANCE MGR.

GERDAU

Straight Bill of Lading

Shippers No. 1323-0000071825

Shipment Doc. No. 5162615

OLD GRASSDALE ROAD NE CARTERSVILLE, GA 30121 US

RECEIVED, subject to the classification and tariffs in effect on the date of the issue of this Bill of Lading.

The property described below, in apparent good order, except as noted (contents and condition of contents of package unknown), marked, consigned, and destined as indicated below, which said carrier (the word carrier being understood throughout this contract as meaning any person or corporation in possession of the property under the contract) agrees to carry to its usual place of delivery as said description, if on its route, otherwise to deliver to another carrier on the route to said destination. It is mutually agreed, as to each carrier of all or any of said property over all or any portion of said route to destination, and as to each party at any time interested in all or any of said property, that every service to be performed hereunder shall be subject to all the terms and conditions of the Uniform Domestic Straight Bill of Lading set forth (1) in Uniform Freight Classification, in effect on the date hereof, if this is a rail or rail-water shipment, or (2) in the applicable motor carrier classification or tariff if this is a motor carrier shipment. Shipper hereby certifies that he is familiar with all the terms and conditions of the said bill of lading, including those on the back thereof, set forth in the classification or tariff which governs the transportation of this shipment, and the said terms and conditions arehereby agreed to by the shipper and accepted for himself and his assigns.

CONSIGNED TO
BLUESCOPE BUILDINGS NORTH AMERICA INC
432 DILBECK RD W
RAINSVILLE, AL, 35986-4310

INVOICE TO
BLUESCOPE BUILDINGS N AMERICA INC
KANSAS CITY, MO, 64141-6917

SHIP DATE 15-JUN-16

CUST.ACCOUNT NO 100059830

APPROVAL CODE OPERACAO

CARRIER: 100067879 WARNER TRUCKING INC

DESTINATION: SEE CONSIGNED TO

ROUTE: T-1323-AL-RAINSVILLE

CAR / VEHICLE INITIALS: w11/s8

DELIVERY ADDRESS: SEE CONSIGNED TO

DELIVERY CARRIER: 100067879 WARNER TRUCKING INC

SPECIAL INSTRUCTIONS: TARPING : FULL TARP REQUIRED ***CANNOT RECEIVE MORE THAN 3 LOADS PER DAY. "CARRIER/DRIVER MUST WEAR

DESCRIPTION OF ARTICLES: 5 BUNDLES, IRON OR STEEL RODS OR BARS, DESCRIBED BELOW

Freight Charges are to be PREPAID, unless marked Collect. Check Box if Collect. [ ]

SHAPE AND SIZE PART #	DRAW #	GRADE BUNDLE ID	MILL	LENGTH	CUST PO # HEAT / BATCH	SALES ORDER # WEIGHT (LB)	ITEM # WEIGHT (KG)	DELIVERY # PCS	ITEM #	CUSTOMER MATERIAL NUMBER
Flat Bar 3/4 X 12		A529-55M		40'00"	298154	3804452	40	8103971690	10	093888
		11323250000013949726	1323		55044867/03	8,568 LB	3,886 KG	7		
		11323250000013949727	1323		55044867/03	8,568 LB	3,886 KG	7		
		11323250000013949728	1323		55044867/03	8,568 LB	3,886 KG	7		
		11323250000013949729	1323		55044867/03	8,568 LB	3,886 KG	7		
		11323250000013949732	1323		55044867/03	8,568 LB	3,886 KG	7		
		TOTAL		5 BUNDLES		42,840 LB	19,430 KG	35		

R# 26047

Note – Where the rate is dependent on value, shippers are required to state specifically in writing the agreed or declared value of the property. The agreed or declared value of the property is hereby specifically stated by the shipper to be not exceeding
per

Subject to Section 7 of the conditions, if this shipment is to be delivered to the consignee without recourse on the consigner, the consignor shall sign the following statement. The carrier shall not make delivery of this shipment without payment of freight and all other lawful charges.

GERDAU
(SIGNATURE OF CONSIGNOR)

Shippers Total Net Weight (LB)	42,840	AGENT	
Shippers Total Net Weight (KG)	19,430	DATE	

Gerdau Corporation Shipper, per ....

Permanent post office address of shipper OLD GRASSDALE ROAD NE CARTERSVILLE GA 30121 US

Time of Day 06/15/2016 02:50:12

Sonnie Walston 6-15-16

Department / Resource Worklist Report

For **FRAMES** batch **1045092**
ENGINEERED PARTS

Orders: 1601181101

Department: 753B
Dept Desc: FRAMES - COMPONENTS FABRICATION
Resource: 002
Resource Desc: SHEAR STRUCTURAL
Raw Material: 150073
RM Description: 1.500" X 96" X 240" 55K

Ship Date: 15-JUN-16

Batch: 1045092
Sort: 16596752

Total Qty: 0.0 EA

16596752   *150073*

Item/ Part Mark/Job	Description/Next Step	Stk/Seq	Qty	Dimensions	
002SP160144104*30305849	FG-PLATE-58.500	1-1	1	L:	58.500
SBX001	752 - 22222 - CONRAC AUTOWELD -			T:	
1601181101:26008482:VP - HOUSE/D	MACHINE TIME			W:	16.000

CAMGen File(s):01-1045092.FMI (9.PRT), 09-1045092.PCH (9.PRT), 9.txt)
Row 1 at 2, 5.375(1.3125), 10.125(1.3125), 48.375(1.3125), 53.125(1.3125)
Row 2 at 5.75, 1.625(1.3125), 5.375(1.3125), 10.125(1.3125), 13.875(1.3125), 44.625(1.3125), 48.375(1.3125), 53.125(1.3125), 56.875(1.3125)
Row 3 at 10.25, 1.625(1.3125), 5.375(1.3125), 10.125(1.3125), 13.875(1.3125), 44.625(1.3125), 48.375(1.3125), 53.125(1.3125), 56.875(1.3125)
Row 4 at 14, 5.375(1.3125), 10.125(1.3125), 48.375(1.3125), 53.125(1.3125)
Weld Mark: 7.250

Sort Length: 4.9'

5506374

or

~~0062916~~

Leeco Steel
1011 Warrenville Rd STE 500
Lisle, IL 60532
Phone: (800) 621-4366

DATE: 5/27/2016

STRAIGHT BILL OF LADING

BOL No. 4920113

SHIP FROM:	SOLD/CONSIGNED TO:
LEECO CHATTANOOGA 2605 East 39th Street Chattanooga , TN 37407 P: 423-558-0470 F: 423-475-5765	BLUESCOPE BUILDINGS NORTH AMERICA, INC. 432 Dilbeck Road West RAINSVILLE , AL 35986 P: 256-638-6800 F: - -

CARRIER: **WATKINS TRUCKING CO. INC.** FREIGHT TERMS: **PREPAID** LEECO PO: **PO00121802**

QTY	ITEM DESC.	WEIGHT	HEAT	CUSTOMER PO	LEECO ORDER
4	1-1/2 X 96 X 240 A572 GR 55 DOMESTIC	39,204.86	5506374	298059	O323414

TOTAL QTY: 4 **TOTAL WEIGHT: 39,204.86**

R#25840

5-27-16

OK
JW
6/2/16

LOADS MUST BE TARPED BEFORE LEAVING PREMISES ☑ Yes ☐ No

The property described below, in apparant good order, except as noted (contents of packages unknown), marked, consigned and destined as indicated below, which said carrier (the word carrier being understood throughout this contract as meaning any person or corporation in possession of the property under the contract) agrees to carry to its usual place of delivery at said destination, if on its route, otherwise deliver to another carrier on the route to said destination. It is mutually agresd, as to each carrier of all or any of said property over all or any portion of said route to destination, and as to each party at any time interedted in all or any of said property, that every service to be performed hereunder shall be subject to the terms and condition sof the Uniform Domestic Straight Bill of Lading set forth (1) in Official, Southern, Western and Illinois Freight Classifications in effect on the ate hereof, if this is a rail or railwater shipment, or (2)in the applicable motor carrier classification or tariff if this is a motor carrier shipment.

Shipper hereby certifies that he is familiar with all terms and conditions of the said bill of lading, including those on the back thereof, set forth in the classification or tariff which governs the transportation of this shipment, and the said terms and conditions are hereby agreed to by the shipper and accepted for himself and his assigns.

Subject to Section 7 of Conditions of applicable bill of lading, if this shipment is to be delivered to the consignee without recourse on the cosignor, the consinor shall sign the following statement. The carrier shall not make delivery of this shipment without payment of freight and all other lawful charges.

SPECIAL COMMENTS:

SHIPPER [illegible] DATE 5

CARRIER [illegible] DATE 5-27-16

Doc 23.0 Rev 3

Department / Resource Worklist Report

For **FRAMES** batch **1045092**
ENGINEERED PARTS

Orders: 1601181101

Department: 753B
Dept Desc: FRAMES - COMPONENTS FABRICATION
Resource: 002
Resource Desc: SHEAR STRUCTURAL
Raw Material: 0093581
RM Description: 1.25" X 96" X 240" 55K

Ship Date: 15-JUN-16

Batch: 1045092
Sort: 16596751

Total Qty: 0.0 EA

16596751

0093581

Item/ Part Mark/Job	Description/Next Step	Stk/Seq	Qty	Dimensions	
004SP140123012*30305869	FG-PLATE-37.250	1-2	1	L:	37.250
SBX002	752 - 22222 - CONRAC AUTOWELD -			T:	1.25
1601181101:26008514:VP - HOUSE/D	MACHINE TIME			W:	14.000

CAMGen File(s):01-1045092.FMI (10.PRT), 09-1045092.PCH (10.PRT), 10.txt)
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Row 2 at 5, 1.5(1.1875), 4.875(1.1875), 9.375(1.1875), 12.75(1.1875), 24.5(1.1875), 27.875(1.1875), 32.375(1.1875), 35.75(1.1875)
Row 3 at 9, 1.5(1.1875), 4.875(1.1875), 9.375(1.1875), 12.75(1.1875), 24.5(1.1875), 27.875(1.1875), 32.375(1.1875), 35.75(1.1875)
Row 4 at 12.375, 4.875(1.1875), 9.375(1.1875), 27.875(1.1875), 32.375(1.1875)
Weld Mark: 6.625

Sort Length: 3.1'

Heat # B6Q3803

NUCOR
PLATE MILL

P.O.Box 279
Winton, NC 27986
(252) 356-3700

Mill Test Report

Page 1

NUCOR

Issuing Date :	10/25/2015	**B/L No. :** 428552	**Load No. :** 435526	**Our Order No. :** 131841/5	**Cust. Order No. :** V06296
Vehicle No:	NOKL 725220		**Sold To:**	LEECO STEEL PRODUCTS 1011 WARRENVILLE RD STE 500 LISLE,IL 60532	**Ship To:** LEECO STEEL PRODUCTS-CHATTANOOGA 2605 E 39TH ST CHATTANOOGA,TN 37405
Specification:	1.5000" x 96.000" x 240.000" ASTM A572 Grade55-15				
Marking :	V06296				

Heat No	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Al(tot)	V	Nb	Ti	N	Ca	B	Sn	Ceq	Pcm
5506374	0.19	1.30	0.015	0.002	0.25	0.27	0.11	0.13	0.02	0.033	0.065	0.002	0.003		0.0013	0.0003	0.009	0.48	0.30

Tensile Test								Charpy Impacts												
Plate Serial No	Pieces	Tons	Dir.	(psi) Yield	(psi) Tensile	Elongation % in 2"	Elongation % in 8"	Dir.	1	(%) shear	2	(%) shear	3	(%) shear	Ave.	(%) shear	Size	Temp	Min Ave.
5506374-06	2	9.80	T	66,500	91,300		22.1												
			T	57,100	83,900		21.9												

MB
6/2/16

Sonnie Walston

5-27-16

Manufactured to fully killed fine grain practice by Electric Arc Furnace. Welding or weld repair was not performed on this material. Mercury has not been used in the direct manufacturing of this material. Produced as continuous cast discrete plate as-rolled, unless otherwise noted in Specification. For Mexico shipments: nhc-SalesMX@Nucor.com
Yield by 0.5EUL method unless otherwise specified. Ceq = C+(Mn/6)+((Cr+Mo+V)/5)+((Cu+Ni)/15)
Pcm = C+(Si/30)+(Mn/20)+(Cu/20)+(Ni/60)+(Cr/20)+(Mo/15)+(V/10)+5B
Melted and Manufactured in the USA. ISO 9001:2008 certified (#010940) by SRI Quality System Registrar (#0985-09). PED 97/23/EC 7/2 Annex 1, Para. 4.3 Compliant. DIN 50049 3.1.B/EN 10204 3.1B(2004), DIN EN 10204 3.1(2005) compliant. For ABS grades only, Quality Assurance certificate 14-MMPQA-723

We hereby certify that the contents of this report are accurate and correct. All test results and operations performed by the material manufacturer are in compliance with the applicable specifications, including customer specifications.

T. A. Depretis

T. A. Depretis, Metallurgist

10/26/2015 1:57:20 PM

Leeco STEEL

Leeco Steel
1011 Warrenville Rd STE 500
Lisle, IL 60532
Phone (800) 621-4366

DATE: 5/18/2016

STRAIGHT BILL OF LADING

BOL No. 4920055

SHIP FROM:	SOLD/CONSIGNED TO:
LEECO CHATTANOOGA 2605 East 39th Street Chattanooga , TN 37407	BLUESCOPE BUILDINGS NORTH AMERICA, INC. 432 Dilbeck Road West RAINSVILLE , AL 35986
P: 423-558-0470 F: 423-475-5765	P: 256-638-6800 F: - -

OK
Sew
5/20/16

CARRIER: **WATKINS TRUCKING CO. INC.** FREIGHT TERMS: **PREPAID** LEECO PO: **PO00121397**

QTY	ITEM DESC.	WEIGHT	HEAT	CUSTOMER PO	LEECO ORDER
6	1-1/4 X 96 X 240 A572 GR 55 DOMESTIC	49,006.08	B6Q3803	297068	0322767

TOTAL QTY: 6 **TOTAL WEIGHT: 49,006.08**

RH25730

5-18-16

LOADS MUST BE TARPED BEFORE LEAVING PREMISES ☑ **Yes** ☐ **No**

The property described below, in apparent good order, except as noted (contents of packages unknown), marked, consigned and destined as indicated below, which said carrier (the word carrier being understood throughout this contract as meaning any person or corporation in possession of the property under the contract) agrees to carry to its usual place of delivery at said destination, if on its route, otherwise deliver to another carrier on the route to said destination. It is mutually agreed, as to each carrier of all or any of said property over all or any portion of said route to destination, and as to each party at any time interested in all or any of said property, that every service to be performed hereunder shall be subject to the terms and condition sof the Uniform Domestic Straight Bill of Lading set forth (1) in Official, Southern, Western and Illinois Freight Classifications in effect on the ate hereof, if this is a rail or railwater shipment or (2) in the applicable motor carrier classification or tariff if this is a motor carrier shipment.

Shipper hereby certifies that he is familiar with all terms and conditions of the said bill of lading, including those on the back thereof, set forth in the classification or tariff which governs the transportation of this shipment, and the said terms and conditions are hereby agreed to by the shipper and accepted for himself and his assigns.

Subject to Section 7 of Conditions of applicable bill of lading, if this shipment is to be delivered to the consignee without recourse on the cosignor, the consinor shall sign the following statement. The carrier shall not make delivery of this shipment without payment of freight and all other lawful charges.

SPECIAL COMMENTS:

SHIPPER [signature] DATE 51816

CARRIER Mike Dreuer DATE 51816

Doc 23.0 Rev 3

5-18-16

NUCOR
NUCOR STEEL TUSCALOOSA, INC.

MILL TEST CERTIFICATE

1700 HOLT RD N.E.
Tuscaloosa, AL 35404-1000
800 800-8204
customerservice@nucortusk.com

Load Number	Tally	Mill Order Number	PO NO / Line NO	Part Number	Certificate Number	Prepared
T120923	00000000663039	N-145332-005	V07510FF04 5		D66303902-1	04/18/2016 08:26

Grade

Order Description:
Hot Roll Plate
A572 50, 1.2500 IN x 96.000 IN x 240.000 IN

Quality Plan Description:
A57255 1-1.50DP: ASTM A572-50/55-13 TYPE 2

Customer:
Sold TO:
LEECO STEEL LLC Chattanooga TN
Ship TO:
LEECO STEEL, LLC Chattanooga TN
Sent TO:

Shipped Item	Heat/Slab Number	Certified By	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Cb	V	Al	Ti	N2	B	Ca	Sn	CEV	ACI
6D0445CA	B6Q3803-03 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0445DA	B6Q3803-03 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0445DB	B6Q3803-03 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0446AA	B6Q3803-04 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0446DB	B6Q3803-04 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13

Shipped Item	Certified By	Heat Number	Yield ksi	Tensile ksi	Y/T %	Elongation % 2"	Elongation % 8"	Bend OK?	Hard HB	Charpy Impacts (ft-lbs) Size mm	1	2	3	Avg	Shear % 1	2	3	Avg	Test Temp
6D0445CA	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0445DA	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0445DB	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0446AA	S6D0446FTT	B6Q3803 ***	59.3	83.1	71.4		26.8												
6D0446DB	S6D0446FTT	B6Q3803 ***	59.3	83.1	71.4		26.8												

Items: 5 PCS: 5 Weight: 40838.5 LBS

Sammie Walston
5-18-16

Mercury has not come in contact with this product during the manufacturing process nor has any mercury been used by the manufacturing process. Certified in accordance with EN 10204 3.1. No weld repair has been performed on this material.
Manufactured to a fully killed fine grain practice.
ISO 9001:2008 Registered, PED Certified

We hereby certify that the product described above passed all of the tests required by the specifications.

Quilin Yu
Dr. Quilin Yu - Metallurgist

"***" indicates Heats melted and Manufactured in the U.S.A.

NUCOR
NUCOR STEEL TUSCALOOSA, INC.

MILL TEST CERTIFICATE

1700 HOLT RD N.E.
Tuscaloosa, AL 35404-1000
800 800-8204
customerservice@nucortusk.com

Load Number	Tally	Mill Order Number	PO NO | Line NO	Part Number	Certificate Number	Prepared
T120923	00000000663039	N-145332-005	V07510FF04 5		D66303902-1	04/18/2016 08:26

Grade

Order Description:
Hot Roll Plate
A572 50, 1.2500 IN x 96.000 IN x 240.000 IN

Quality Plan Description:
A57255 1-1.50DP: ASTM ~~A572-50~~/55-13 TYPE 2

Customer:
Sold TO:
LEECO STEEL LLC Chattanooga TN
Ship TO:
LEECO STEEL, LLC Chattanooga TN
Sent TO:

Shipped Item	Heat/Slab Number	Certified By	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Cb	V	Al	Ti	N2	B	Ca	Sn	CEV	ACI
6D0445CA	B6Q3803-03 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0445DA	B6Q3803-03 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0445DB	B6Q3803-03 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0446AA	B6Q3803-04 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0446DB	B6Q3803-04 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13

Shipped Item	Certified By	Heat Number	Yield ksi	Tensile ksi	Y/T %	Elongation % 2"	Elongation % 8"	Bend OK?	Hard HB	Charpy Impacts (ft-lbs) Size mm	1	2	3	Avg	Shear % 1	2	3	Avg	Test Temp
6D0445CA	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0445DA	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0445DB	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0446AA	S6D0446FTT	B6Q3803 ***	59.3	83.1	71.4		26.8												
6D0446DB	S6D0446FTT	B6Q3803 ***	59.3	83.1	71.4		26.8												

Items: 5 PCS: 5 **Weight:** 40838.5 LBS

5/23/16

5-18-16

Mercury has not come in contact with this product during the manufacturing process nor has any mercury been used by the manufacturing process. Certified in accordance with EN 10204 3.1. No weld repair has been performed on this material.
Manufactured to a fully killed fine grain practice.
ISO 9001:2008 Registered, PED Certified

We hereby certify that the product described above passed all of the tests required by the specifications.

Dr. Quilin Yu - Metallurgist

"***" Indicates Heats melted and Manufactured in the U.S.A.

NUCOR STEEL TUSCALOOSA, INC.

MILL TEST CERTIFICATE

1700 HOLT RD N.E.
Tuscaloosa, AL 35404-1000
800 800-8204
customerservice@nucortusk.com

Load Number	Tally	Mill Order Number	PO NO | Line NO	Part Number	Certificate Number	Prepared
T120939	00000000663084	N-145444-006	V07612FF04 06		D66308402-1	05/17/2016 09:03

Grade

Order Description:
Hot Roll Plate
A572 50, 1.2500 IN x 96.000 IN x 240.000 IN

Quality Plan Description:
A57250/55 DUAL: ASTM A572-50 & 55-13/M270-50/A709-50

Customer:
Sold TO:
LEECO STEEL LLC Chattanooga TN
Ship TO:
LEECO STEEL, LLC Chattanooga TN
Sent TO:

Shipped Item	Heat/Slab Number	Certified By	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Cb	V	Al	Ti	N2	B	Ca	Sn	CEV	ACI
6D0443CA	B6Q3803-01 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0444AA	B6Q3803-02 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0444AB	B6Q3803-02 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0444BA	B6Q3803-02 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13
6D0444BB	B6Q3803-02 ***	B6Q3803	0.20	1.15	0.010	0.002	0.20	0.11	0.05	0.06	0.021	0.002	0.076	0.032	0.002	0.006	0.0001	0.0010	0.005	0.43	3.13

Shipped Item	Certified By	Heat Number	Yield ksi	Tensile ksi	Y/T %	ELONGATION % 2"	ELONGATION % 8"	Bend OK?	Hard HB	Charpy Impacts (ft-lbs) Size mm	1	2	3	Avg	Shear % 1	2	3	Avg	Test Temp
6D0443CA	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0443CA	S6D0446FTT	B6Q3803 ***	59.3	83.1	71.4		26.8												
6D0444AA	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0444AA	S6D0446FTT	B6Q3803 ***	59.3	83.1	71.4		26.8												
6D0444AB	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0444AB	S6D0446FTT	B6Q3803 ***	59.3	83.1	71.4		26.8												
6D0444BA	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0444BA	S6D0446FTT	B6Q3803 ***	59.3	83.1	71.4		26.8												
6D0444BB	S6D0445FTT	B6Q3803 ***	58.4	83.2	70.2		25.1												
6D0444BB	S6D0446FTT	B6Q3803 ***	59.3	83.1	71.4		26.8												

Items: 5 PCS: 5 Weight: 40838.5 LBS

MB 5/23/16 5-18-16

Mercury has not come in contact with this product during the manufacturing process nor has any mercury been used by the manufacturing process. Certified in accordance with EN 10204 3.1. No weld repair has been performed on this material.
Manufactured to a fully killed fine grain practice.
ISO 9001:2008 Registered, PED Certified

We hereby certify that the product described above passed all of the tests required by the specifications.

Dr. Quilin Yu - Metallurgist

"***" indicates Heats melted and Manufactured in the U.S.A.

DATE
06/27/2016

Department / Resource Worklist Report

For **FRAMES** batch **1045092**
ENGINEERED PARTS

Orders: 1601181101

Ship Date: 15-JUN-16

Department: 753B
Dept Desc: FRAMES - COMPONENTS FABRICATION
Resource: 002
Resource Desc: SHEAR STRUCTURAL
Raw Material: 0092356
RM Description: .750" X 96" X 240"

Batch: 1045092
Sort: 16596749

Total Qty: 0.0 EA

16596749

0092356

Item/ Part Mark/Job	Description/Next Step	Stk/Seq	Qty	Dimensions
BRG070412121001001110*30305843 SBX001 1601181101:26009018:VP - HOUSE/D	FG-PLATE-12.625 752 - 22222 - CONRAC AUTOWELD - MACHINE TIME	1-1	1	L: 12.625 T: 0.75 W: 7.250
CAMGen File(s):01-1045092.FMI (11.PRT), 09-1045092.PCH (11.PRT), 11.txt)				
BRG070412121001001110*30305845 SBX001 1601181101:26009019:VP - HOUSE/D	FG-PLATE-12.625 752 - 22222 - CONRAC AUTOWELD - MACHINE TIME	1-1	1	L: 12.625 T: 0.75 W: 7.250
CAMGen File(s):01-1045092.FMI (12.PRT), 09-1045092.PCH (12.PRT), 12.txt)				
BRG041212080401000704*30305839 SBX001 1601181101:26009012:VP - HOUSE/D	FG-PLATE-8.250 752 - 22222 - CONRAC AUTOWELD - MACHINE TIME	1-1	1	L: 8.250 T: 0.75 W: 4.750
CAMGen File(s):01-1045092.FMI (13.PRT), 09-1045092.PCH (13.PRT), 13.txt)				
BRG041212080401000704*30305841 SBX001 1601181101:26009013:VP - HOUSE/D	FG-PLATE-8.250 752 - 22222 - CONRAC AUTOWELD - MACHINE TIME	1-1	1	L: 8.250 T: 0.75 W: 4.750
CAMGen File(s):01-1045092.FMI (14.PRT), 09-1045092.PCH (14.PRT), 14.txt)				

Sort Length: 3.5'

Heat #

~~B6Q4402001~~

~~B6Q036~~

B6Q3964

STRAIGHT BILL OF LADING – SHORT FORM – ORIGINAL – Not Negotiable
RECEIVED, subject to the classifications and tariffs in effect on the date of the issue of this Bill of Lading

CHAPEL STEEL
A Reliance Steel Company
2000 Avenue C
Birmingham, AL 35218
205-781-0317

```
                                                   No:    BRM  50444
Ship Date  20Jun16 at 12:26 From BHM
Probill
Via   ALABAMA CARRIER
FOB   CHAPEL STEEL
Frt   Freight Allowed
Route   0-   0 Manifest
Vhcle                 Trailer
Slp   Karl Fridrich
Sold To:    ( 17230)
BLUESCOPE BUILDINGS NA, INC.
1540 GENESSEE STREET
KANSAS CITY, MO  65102

Ship To:         (  1)
BLUESCOPE BUILDINGS NA, INC.
1274 CHURCH AVENUE
RAINSVILLE, AL  35986
Tel: 256-638-2264 Fax:
```

OK
fcy
6/23/16

```
==============================================================================
                   B I L L   O F   L A D I N G
 1) Our Order BRM- 49424   1 Your PO # 300277
 (572)PLATE A572 GRADE 50 TYPE 2              9  Pcs             44107 LB.
 3/4" X 96.0000" X 240.0000"
 HT#B6Q3899-03 3PCS
    B6Q3964-03 9PCS
------------------------------------------------------------------------------
                                                                      LBS
                                     TOTAL:              9           44107

*************************************************
**MUST CALL GREG 256-638-2264 EXT 234 FOR APPT****
*************************************************

ATTENTION FREIGHT CARRIERS:
*****STEEL PLATE CLASS 50 CODE 106280*****
```

R# 26129

6-21-16

Page: 1 .... Last

These commodities are controlled for export by the United States government under the Export Administration Regulations. Diversion contrary to U.S. law prohibited. Purchaser is responsible to comply with these regulations if the items are to be exported from the United States or re-exported from a foreign country. ECCN EAR99 applicable.

The property described below, in apparent good order, except as noted (contents and condition of contents of packages unknown), marked, consigned, and destined as indicated below, which said carrier (the word carrier being understood throughout this contract as meaning any person or corporation in possession of the property under the contract) agrees to carry to its usual place of delivery at said destination, if on its route, otherwise to deliver to another carrier on the route to said destination. It is mutually agreed, as to each carrier of all or any of said property over all or any portion of said route to destination, and as to each party at any time interested in all or any of said property, that every service to be performed hereunder shall be subject to all the terms and conditions of the Uniform Domestic Straight Bill of Lading set forth (1) in Official, Southern, Western and Illinois Freight Classifications in effect on the date hereof, if this is a rail or rail-water shipment, or (2) in the applicable motor carrier classification or tariff if this is a motor carrier shipment.

Shipper hereby certifies that he is familiar with all the terms and conditions of the said bill of lading, including those on the back thereof, set forth in the classification or tariff which governs the transportation of this shipment, and the said terms and conditions are hereby agreed to by the shipper and accepted for himself and his assigns.

*If the shipment moves between two ports by a carrier by water, the law requires that the bill of lading shall state whether it is carrier's or shipper's weight.

NOTE Where the rate is dependent on value, shippers are required to state specifically in writing the agreed or declared value of the property.
The agreed or declared value of the property is hereby specifically stated by the shipper to be not exceeding ______ per ______

Subject to Section 7 of Conditions of applicable bill of lading, if this shipment is to be delivered to the consignee without recourse on the consignor, the consignor shall sign the following statement:
The carrier shall not make delivery of this shipment without payment of freight and all other lawful charges.

Signature of consignor

If charges are to be prepaid, write or stamp here, "To be Prepaid"

Must be tarped, chained and protected from weather.

Rejections will only be honored if returned in original form sold and within 15 days from date of delivery.

DELIVERY RECEIPT MUST BE SIGNED AND RETURNED.

CARRIER'S

PART NO.

PO/Rel

Certificate of Mill Test Results

BRM-000000-000

Pg 1/1

Attn:

NUCOR

NUCOR STEEL TUSCALOOSA, INC.

MILL TEST CERTIFICATE

1700 HOLT RD N.E.
Tuscaloosa, AL 35404-1000
800 800-8204
customerservice@nucortusk.com

Page: 1 of 1

Load Number	Tally	Mill Order Number	PO NO	Line NO	Part Number	Certificate Number	Prepared
T122489	00000000666460	N-146435-003	BRM-4675	3		S66646001-1	05/04/2016 04:36

Grade:

Order Description:
Hot Roll Plate From Coil
A572 50, 0.7500 IN x 96.000 IN x 480.000 IN

Quality Plan Description:
A57250/A70950: ASTM A572-50-15/A709-50-15/M270-50

Customer:

Sold TO:
CHAPEL STEEL Birmingham AL

Ship TO:
CHAPEL STEEL COMPANY Birmingham AL

Sent TO:

Shipped Item	Heat/Slab Number	Certified By	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Cb	V	Al	Ti	N2	B	Ca	Sn	CEV	ACI
6D1652B	B6Q3964-03 ***	B6Q3964	0.18	0.86	0.010	0.007	0.03	0.11	0.05	0.06	0.013	0.000	0.016	0.026	0.000	0.008	0.0000	0.0037	0.005	0.35	2.84
6D1652C	B6Q3964-03 ***	B6Q3964	0.18	0.86	0.010	0.007	0.03	0.11	0.05	0.06	0.013	0.000	0.016	0.026	0.000	0.008	0.0000	0.0037	0.005	0.35	2.84

Shipped Item	Certified By	Heat Number	Yield ksi	Tensile ksi	Y/T %	Elongation % 2"	Elongation % 8"	Bend OK?	Hard HB	Charpy Impacts (ft-lbs) Size mm	1	2	3	Avg	Shear % 1	2	3	Avg	Test Temp
6D1652B	S6D1652FTT	B6Q3964 ***	62.8	83.9	74.9	36.9													
6D1652B	S6D1652MTT	B6Q3964 ***	64.4	79.8	80.7	33.1													
6D1652C	S6D1652FTT	B6Q3964 ***	62.8	83.9	74.9	36.9													
6D1652C	S6D1652MTT	B6Q3964 ***	64.4	79.8	80.7	33.1													

Items: 2 PCS: 4 Weight: 39204.8 LBS

MB 6/24/16

6-27-16

Mercury has not come in contact with this product during the manufacturing process nor has any mercury been used by the manufacturing process. Certified in accordance with EN 10204 3.1 No weld repair has been performed on this material. Manufactured to a fully killed fine grain practice NUTEMPER TEMPER PASSED plate from coil
ISO 9001:2008 Registered, PED Certified

"***" indicates Heats melted and Manufactured in the U.S.A.

We hereby certify that the product described above passed all of the tests required by the specifications

Dr. Quilin Yu - Metallurgist

Doc No. 326200 Indexed 5May16 by 146mnear

Certificate of Mill Test Results

BRM-000000-000

PO/Rel

PART NO.

Attn:

NUCOR
NUCOR STEEL TUSCALOOSA, INC.

MILL TEST CERTIFICATE

1700 HOLT RD N.E.
Tuscaloosa, AL 35404-1000
800 800-8204
customerservice@nucortusk.com

Load Number	Tally	Mill Order Number	PO NO / Line NO	Part Number	Certificate Number	Prepared
T122488	00000000666461	N-146435-003	BRM-4675 3		S66646101-1	05/02/2016 10:57

Grade

Customer:

Order Description:
Hot Roll Plate From Coil
A572 50, 0.7500 IN x 96.000 IN x 480.000 IN

Quality Plan Description:
A57250/A70950: ASTM A572-50-15/A709-50-15/M270-50

Sold TO:
CHAPEL STEEL Birmingham AL

Ship TO:
CHAPEL STEEL COMPANY Birmingham AL

Sent TO:

Shipped Item	Heat/Slab Number	Certified By	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Cb	V	Al	Ti	N2	B	Ca	Sn	CEV	ACI
6D1214D	B6Q3899-03 ***	B6Q3899	0.06	1.09	0.009	0.004	0.03	0.11	0.05	0.21	0.017	0.038	0.005	0.026	0.011	0.008	0.0002	0.0014	0.011	0.30	0.00
6D1214E	B6Q3899-03 ***	B6Q3899	0.06	1.09	0.009	0.004	0.03	0.11	0.05	0.21	0.017	0.038	0.005	0.026	0.011	0.008	0.0002	0.0014	0.011	0.30	0.00
6D1214F	B6Q3899-03 ***	B6Q3899	0.06	1.09	0.009	0.004	0.03	0.11	0.05	0.21	0.017	0.038	0.005	0.026	0.011	0.008	0.0002	0.0014	0.011	0.30	0.00
6D1214G	B6Q3899-03 ***	B6Q3899	0.06	1.09	0.009	0.004	0.03	0.11	0.05	0.21	0.017	0.038	0.005	0.026	0.011	0.008	0.0002	0.0014	0.011	0.30	0.00

Shipped Item	Certified By	Heat Number	Yield ksi	Tensile ksi	Y/T %	Elongation % 2"	Elongation % 8"	Bend OK?	Hard HB	Charpy Impacts (ft-lbs) Size mm	1	2	3	Avg	Shear % 1	2	3	Avg	Test Temp
6D1214D	S6D1214FTT	B6Q3899 ***	55.2	67.9	81.3	44.1													
6D1214D	S6D1214MTT	B6Q3899 ***	61.1	68.4	89.3	39.9													
6D1214E	S6D1214FTT	B6Q3899 ***	55.2	67.9	81.3	44.1													
6D1214E	S6D1214MTT	B6Q3899 ***	61.1	68.4	89.3	39.9													
6D1214F	S6D1214FTT	B6Q3899 ***	55.2	67.9	81.3	44.1													
6D1214F	S6D1214MTT	B6Q3899 ***	61.1	68.4	89.3	39.9													
6D1214G	S6D1214FTT	B6Q3899 ***	55.2	67.9	81.3	44.1													
6D1214G	S6D1214MTT	B6Q3899 ***	61.1	68.4	89.3	39.9													

Items: 4 PCS: 4 Weight: 39204.8 LBS

ms 6/24/16

6-21-16

Mercury has not come in contact with this product during the manufacturing process nor has any mercury been used by the manufacturing process. Certified in accordance with EN 10204 3.1. No weld repair has been performed on this material. Manufactured to a fully killed fine grain practice. NUTEMPER TEMPER PASSED plate from coil
ISO 9001:2008 Registered PED Certified

*** indicates Heats melted and Manufactured in the U.S.A.

We hereby certify that the product described above passed all of the tests required by the specifications.

Dr. Quilin Yu - Metallurgist

Doc No. 326074 Indexed 2May16 by 146mpear

Department / Resource Worklist Report

For **FRAMES** batch **1045092**
ENGINEERED PARTS

Orders: 1601181101

Department: 753B
Dept Desc: FRAMES - COMPONENTS FABRICATION
Resource: 002
Resource Desc: SHEAR STRUCTURAL
Raw Material: 0093393
RM Description: .4850" MT (.5000" NOM) X 60.00" X 241" 55K

Ship Date: 15-JUN-16

Batch: 1045092
Sort: 16596750

Total Qty: 0.0 EA

16596750   *0093393*

Item/ Part Mark/Job	Description/Next Step	Stk/Seq	Qty	Dimensions
BRG061008110801001008*30305863 SBX002 1601181101:26009016:VP - HOUSE/D	FG-PLATE-11.500 752 - 22222 - CONRAC AUTOWELD - MACHINE TIME	1-2	1	L: 11.500 T: 0.50 W: 6.625
CAMGen File(s):01-1045092.FMI (15.PRT), 09-1045092.PCH (15.PRT), 15.txt)				
BRG061008110801001008*30305865 SBX002 1601181101:26009017:VP - HOUSE/D	FG-PLATE-11.500 752 - 22222 - CONRAC AUTOWELD - MACHINE TIME	1-2	1	L: 11.500 T: 0.50 W: 6.625
CAMGen File(s):01-1045092.FMI (16.PRT), 09-1045092.PCH (16.PRT), 16.txt)				
BRG050408090201000802*30305859 SBX002 1601181101:26009014:VP - HOUSE/D	FG-PLATE-9.125 752 - 22222 - CONRAC AUTOWELD - MACHINE TIME	1-2	1	L: 9.125 T: 0.50 W: 5.250
CAMGen File(s):01-1045092.FMI (17.PRT), 09-1045092.PCH (17.PRT), 17.txt)				
BRG050408090201000802*30305861 SBX002 1601181101:26009015:VP - HOUSE/D	FG-PLATE-9.125 752 - 22222 - CONRAC AUTOWELD - MACHINE TIME	1-2	1	L: 9.125 T: 0.50 W: 5.250
CAMGen File(s):01-1045092.FMI (18.PRT), 09-1045092.PCH (18.PRT), 18.txt)				

Sort Length: 3.4'

Heat #
A507756

HIS MEMORANDUM is an acknowledgment that a Bill of Lading has been issued and is not the Original Bill of Lading, nor a copy or duplicate, covering the property named herein, and is intended solely for filing or record.

FMTF (LCAC/FMTF) (Name of Carrier) BOL# SHIP REF NO 194435-1

IECEIVED, subject to the classifications and tariffs in effect on the date of the issue of this Bill of Lading,

F FERROUSOUTH (9714)(03554474) MASTER BOL NO
R 38 (32 370 Page 1 of 1 194435
M IUKA, MS 38852 Date 06/13/16 08:14

he property described below, in apparent good order, except as noted (contents and condition of contents of packages unknown), marked, consigned, and destined as indicated below, which said carrier (the word carrier being understood throughout his contract as meaning any person or corporation in possession of the property under the contract) agrees to carry to its usual place of delivery at said destination, if on its route, otherwise to deliver to another carrier on the route to said estination. It is mutually agreed, as to each carrier of all or any of said property over all or any portion of said route to destination, and as to each party at any time interested in all or any of said property, that every service to be performed ereunder shall be subject to all the terms and conditions of the Uniform Domestic Straight Bill of Lading set forth (1) in Official, Southern, Western and Illinois Freight Classifications in effect on the date hereof, if this is a rail or a rail-water hipment, or (2) in the applicable motor carrier classification or tariff if this is a motor carrier shipment.

Shipper hereby certifies that he is familiar with all the terms and conditions of the said bill of lading, including those on the back thereof, set forth in the classification or tariff which governs the transportation of this shipment, and the aid terms and conditions are hereby agreed to by the shipper and accepted for himself and his assigns.

CONSIGNEE (100002606) Ship Ord: 097886 WO
OLYMPIC STEEL PO#: 218601 097886
587 BARROW PARK DRIVE
WINDER, GA 30680 6/17/16

SHP DUNS: 100000004

CONSIGNOR (100000001)
OLYMPIC STEEL SHIP BLUESCOPE BUILDINGS
587 BARROW PARK DRIVE 1274 CHURCH AVE
WINDER, GA 30680 RAINSVILLE, AL

TARP

Mode: TRUCK Nbr of Pkgs: 6 Mstr Wgt: 44276 Shp: COMPLETE
BOL Comments: LOAD# 66891 R# 218601
Tally Counts

Material ID	Cust Matl ID	Lin Ft/Pcs		Actual Wt	Actual Size(G x W x L)
Desc: PO#298019					
Pckg: 10,000#MAX LIFT / 2X4 LENGTH RUNNERS 6" FROM EDGE/ BAND BUNDLE 1 EA END					
Ship: TRUC OR FORKLIFT SIDE / BANDS NO GREATER THAN 1' FROM EA END					
140384131	527857	4	0	8071	.4850 x 60.000 x 241.000
Cust ... 298019 STENCIL STRETCH					
140384132	527857	4	0	8055	.4850 x 60.000 x 241.000
Cust ... 298019 STENCIL STRETCH					
140384133	527857	4	0	8055	.4850 x 60.000 x 241.000
Cust ... 298019 STENCIL STRETCH					
140384134	527857	4	0	8038	.4850 x 60.000 x 241.000
Cust ... 298019 STENCIL STRETCH					
140384135	527857	4	0	8038	.4850 x 60.000 x 241.000
Cust ... 298019 STENCIL STRETCH					
140384136	527857	2	0	4019	.4850 x 30.000 x 241.000
Cust ... 298019 STENCIL STRETCH					

HI ... A507756 R#26028 Sammi Walston

ITEM TOTALS: 6 Items 44,276 6-14-16

In: | Scheduled: | | Out: | Door:

Subject to Section 7 of conditions of applicable bill of lading, if this shipment is to be delivered to the consignee without recourse on the consignor, the consignor shall sign the following statement: "The carrier shall not make delivery of this shipment without payment of freight and all other lawful charges."

Signature of Consignor ______

If charges are to be prepaid, write or stamp here, "To be Prepaid."

PREPAID

Received $ ______ To apply in prepayment of the charges on the property described hereon.

Agent or Cashier: ______

Per ______ (The signature here acknowledges only the amount prepaid.)

Charges Advanced $ ______

†The Fibre Boxes used for this shipment conform to the specifications set forth in the box maker's certificate thereon and all other requirements of Consolidated Freight Classification.

†This is to certify that the above named articles are properly classified, described, packaged, marked and labeled, and are in proper condition for transportation, according to the applicable regulations of the Department of Transportation.

"Shipper's imprint in lieu of stamp; not a part of bill of lading approved by the Department of Transportation."

If the shipment moves between two ports by a carrier by water, the law requires that the bill of lading shall state whether it is "carrier's or shipper's weight."
NOTE: Where the rate is dependent on value, shippers are required to state specifically in writing the agreed or declared value of the property.

The agreed or declared value of the property is hereby specifically stated by the shipper to be not exceeding ______ per ______

Inspector ______ DRIVER ______ DATE ______ Customer ______

Rev. 10/98

Department / Resource Worklist Report

For **FRAMES** batch **1045092**
ENGINEERED PARTS

Orders: 1601181101

Department: 753B
Dept Desc: FRAMES - COMPONENTS FABRICATION
Resource: 003
Resource Desc: PLATE LINE
Raw Material: 0093886
RM Description: 1.0000" X 12.00" X 480" 55K

Ship Date: 15-JUN-16

Batch: 1045092
Sort: 16596753

Total Qty: 0.1 EA

16596753

0093886

Item/ Part Mark/Job	Description/Next Step	Stk/Seq	Qty	Dimensions	
001SP120104054*30305851	FG-PLATE-53.500	1-1	1	L:	53.500
SBX001	752 - 22222 - CONRAC AUTOWELD -			T:	1.00
1601181101:26008476:VP -	MACHINE TIME			W:	12.000
HOUSE/D					

CAMGen File(s):01-1045092.FMI (19.PRT), 09-1045092.PCH (19.PRT), 19.txt)
Row 1 at 1.75, 3.25(.8125), 7.25(.8125), 46.25(.8125), 50.25(.8125)
Row 2 at 4, 1(.8125), 3.25(.8125), 7.25(.8125), 9.5(.8125), 44(.8125), 46.25(.8125), 50.25(.8125), 52.5(.8125)
Row 3 at 8, 1(.8125), 3.25(.8125), 7.25(.8125), 9.5(.8125), 44(.8125), 46.25(.8125), 50.25(.8125), 52.5(.8125)
Row 4 at 10.25, 3.25(.8125), 7.25(.8125), 46.25(.8125), 50.25(.8125)
Weld Mark: 4.750

Sort Length: 4.5'

Heat # 1042587

order 2980.

Steel Dynamics®
Flat Roll Group
Columbus Division

1945 Airport Road
Columbus, MS 39701
Phone: 662-245-4200
Fax: 662-245-4297

Metallurgical Certification

Order Number: 284516
Order Dimensions: 0.4850X60.0000 (in)
Ordered Product: A572 GRADE 55 TYPE 1
HOT ROLLED BLACK - PRIME
Part Number: HR .4850X60.000
Alt Part#:

Sold To: OLYMPIC STEEL INC
WINDER, GA

Ship To: OLYMPIC STEEL WINDER
WINDER, GA 30680

Customer PO #: 214480-80 Load #: S540642 Ship Date: 07/16/2015

Chemical Analysis:

Coil Number:	Heat:	C	Mn	P	S	Si	Cu	Sn	Ni	Cr	Mo	Al	N	V	Nb	Ti	B	Ca	C(eq)
15B518956	A507756	[illegible]	[illegible]	[illegible]	.001	.04	.11	.0040	.04	.05	.010	.023	[illegible]	.038	.035	.001	.0001	[illegible]	.282

Mechanical Properties:	English	Metric
Yield Strength	73.4 ksi	506 MPa
Tensile Strength	81.7 ksi	563 MPa
Elongation	28 %	28 %
N-Value	Not Reported	
Hardness - HRBW	Not Reported	
Direction	Transverse	
Linear Footage	454 ft	138 m
Actual Gauge	.4893 in	12 mm

JTB
6/17/16

Tommie Walton
6-14-16

We hereby certify the above is correct as contained in the records of the company.
All tests performed according to ASTM standard E8, A370, E18, E415, and E1019
(yield strength determined using 0.2% offset method) or JIS Z2241 or DIN EN10325.
All heats are Al-killed and Ca treated.
THIS PRODUCT WAS MELTED AND MANUFACTURED IN THE USA

Certified by: Keegan D Wright
Keegan Wright
Hot Mill Metallurgical Engineer

Certificate Date: 07/16/2015

CMC

STRAIGHT BILL OF LADING-SHORT FORM ORIGINAL-NON NEGOTIABLE #71675432

OK JW
6/23/16

MATERIAL DETAILS

Cust PO No.	CMC SO.PO	Del. No.	Material Description.	Customer Material No.	Heat No.	Tag No.	Pcs/Bundle	Net Weight	UOM
299746	2376963	81829725	FLAT 1x12 40'0" A529-55M1	93886	1042587	AL01201856	5	8160	LB
299746	2376963	81829725	FLAT 1x12 40'0" A529-55M1	93886	1042587	AL01201860	5	8160	LB
299746	2376963	81829725	FLAT 1x12 40'0" A529-55M1	93886	1042587	AL01201866	7	11424	LB
299746	2376963	81829725	FLAT 1x12 40'0" A529-55M1	93886	1042587	AL01201861	5	8160	LB
299746	2376963	81829725	FLAT 1x12 40'0" A529-55M1	93886	1042587	AL01201854	5	8160	LB
TOTAL							~~27~~ 20	44064	LB

Sent back because it bundle over weight Per Lorrie

RECEIVED, subject to the classifications in effect on the date of the issue of the Bill of Lading, the property described above, in apparent good order, except as noted (contents of packages unknown), marked, consigned, and destined as indicated below, which said carrier (the word carrier being understood throughout this contract as meaning any person or corporation in possession of the property under the contract) agrees to carry to its usual place of delivery at said destination, if on its route, otherwise to deliver to another carrier on the route to said destination. It is mutually agreed, as to each carrier of all or any said property over all or any portion of said route to destination, and as to each party at any time interested in all or any of said property, that every service to be performed hereunder shall be subject to all the terms and conditions of the Uniform Domestic Straight Bill of Lading set forth (1) in Official Southern, Western and Illinois Freight Classifications in effect on the date hereof, if this is a rail or a rail-water shipment, or (2) in the applicable motor carrier classification or tariff if this is a motor carrier shipment. Shipper hereby certifies that he is familiar with all the terms and conditions of the said bill of lading, including those on the back thereof, set forth in the classification or tariff which governs the transportation of this shipment and the said terms and conditions are hereby agreed to by the shipper and accepted for himself and his assigns. This is to certify that the above articles are properly describe by name and are packed and marked and are in proper condition for transportation according to regulations by the Interstate Commerce Commission. * If the shipment moves between two ports by a carrier by water, the law requires that the bill of lading shall state whether it is "carrier's or shipper's weight." * Shipper's imprints in lieu of stamp, not a part of Bill of Lading approved by the Interstate Commerce Commission. NOTE: Where the rate is dependent on value, shippers are required to state specifically in writing the agreed or declared value of property. The agreed or declared value of the property is hereby specifically state by the shipper to be not exceeding

DRIVER'S SIGNATURE/AGENT : ____________

NOTICE TO RECEIVERS:

Please check each item on this shipping bill carefully. CMC will not be responsible for any exceptions to goods unless notified within twenty four hours and noted on this document.

RECEIVED BY : DATE : TIME : Page 2 of 2

CMC

CMC STEEL ALABAMA
101 S 50TH STREET
BIRMINGHAM AL 35212-3525

CERTIFIED MILL TEST REPORT

For additional copies call
800-637-3227

We hereby certify that the test results presented here are accurate and conform to the reported grade specification

Marcus W. McCluney - CMC Steel AL
Quality Assurance Manager

HEAT NO.:1042587 SECTION: FLAT 1x12 40'0" A529-55M1 GRADE: ASTM A529-05 Grade 55 Mod 1 ROLL DATE: 05/17/2016 MELT DATE: 05/12/2016	S O L D T O	BlueScope Buildings NA 1540 Genessee St Kansas City MO US 64102-1069 8169683243 8166278936	S H I P T O	Bluescope Buildings NA 432 Dilbeck Road West Rainsville AL US 35986-0000 2566382264	Delivery#: 81829725 BOL#: 71675432 CUST PO#: 299746 CUST P/N: 93886 DLVRY LBS / HEAT: 44064.000 LB DLVRY PCS / HEAT: 27 EA

Characteristic	Value	Characteristic	Value	Characteristic	Value
C	0.24%	Elongation test 1	22%		
Mn	0.85%	Elongation Gage Lgth test 1	8IN		
P	0.014%	Yield to tensile ratio test1	0.74		
S	0.023%	Yield Strength test 2	59.5ksi		
Si	0.21%	Tensile Strength test 2	80.3ksi		
Cu	0.29%	Elongation test 2	22%		
Cr	0.16%	Elongation Gage Lgth test 2	8IN		
Ni	0.10%	Yield to tensile ratio test2	0.74		
Mo	0.026%				
V	0.026%				
Cb	0.002%				
Sn	0.010%				
B	0.0001%				
Ti	0.002%				
N	0.0100%				
Carbon Eq A6	0.45%				
Carbon Eq A529	0.49%				
Yield Strength test 1	60.0ksi				
Tensile Strength test 1	81.2ksi				

AB
6/24/16

Sonnie Walston
6-22-16

IIS MATERIAL IS FULLY KILLED, 100% MELTED AND MANUFACTURED IN THE USA, WITH NO WELD REPAIR OR MERCURY CONTAMINATION IN THE PROCESS.

EMARKS :

06/21/2016 14:24:57

Page 1 OF 1

CMC

STRAIGHT BILL OF LADING-SHORT FORM ORIGINAL-NON NEGOTIABLE #71675432

SHIPMENT NO.(BOL) : 71675432	CARRIER'S NAME: CMC Logistics East OO	TRUCK/UNIT # :ALDROPS5 TRAILER/RAILCAR # :248FS02.
DATE AND TIME : 06/21/2016, 14:24:51	CMC INCO TERMS : CPT RAINSVILLE	SEAL NUMBER :
SHIP FROM : T003 CMC Steel Alabama Truck 101 S. 50th Street Birmingham, AL 35212-3525 USA **Contact Phone No. :** 205-592-8981 **Fax No.** :	**SHIP TO :** 3001619 Bluescope Buildings NA 432 Dilbeck Road West Rainsville, AL 35986-0000 USA **Contact Phone No. :** 2566382264 **Fax No.** :	**SOLD TO :** 3062385 BlueScope Buildings NA 1540 Genessee St Kansas City, MO 64102-1069 USA **Contact Phone No. :** 8169683243 **Fax No.** :

Subject to Section 7: Subject to Section 7 of Conditions of applicable bill of lading, if this shipment is to be delivered to the consignee without recourse on the consignor, the consignor shall sign the following statement. The carrier shall not make delivery of this shipment without payment of freight and all other lawful charges.

Consignor's Signature : ____________________

BOL INSTRUCTIONS:

TARP LOAD
Call 24 hRS in advance to schedule Gate Time to be unloaded 256-638-6800 X239
Call between 7am and 3pm M-F
Block for OHC unloading
-CARRIER/DRIVER MUST WEAR ALL REQUIRED PERSONAL PROTECTIVE EQUIPMENT TO ENTER BLUESCOPE PLANT. THIS INCLUDES (BUT NOT LIMITED TO) HARD HAT, SAFETY GLASSES AND SAFETY VEST.
-LOADS MUST REMAIN SECURED (CHAINED AND/OR STRAPPED) ON TRAILER UNTIL TRAILER IS STAGED IN FINAL UNLOADING AREA.
Max lift: 9000 lbs
Single stack bundles for 7"widths and greater
Double stack bundles acceptable for 5" and 6" widths

6-22-16
RH26160
Sammie Walston

NOTES/SPECIAL INSTRUCTIONS

Additional Instructions:

MATERIAL DETAILS

Cust PO No.	CMC SO,PO	Del. No.	Material Description.	Customer Material No.	Heat No.	Tag No.	Pcs/Bundle	Net Weight	UOM

RECEIVED, subject to the classifications in effect on the date of the issue of the Bill of Lading, the property described above, in apparent good order, except as noted (contents of packages unknown), marked, consigned, and destined as indicated below, which said carrier (the word carrier being understood throughout this contract as meaning any person or corporation in possession of the property under the contract) agrees to carry to its usual place of delivery at said destination, if on its route, otherwise to deliver to another carrier on the route to said destination. It is mutually agreed, as to each carrier of all or any said property over all or any portion of said route to destination, and as to each party at any time interested in all or any of said property, that every service to be performed hereunder shall be subject to all the terms and conditions of the Uniform Domestic Straight Bill of Lading set forth (1) in Official Southern, Western and Illinois Freight Classifications in effect on the date hereof, if this is a rail or a rail-water shipment, or (2) in the applicable motor carrier classification or tariff if this is a motor carrier shipment. Shipper hereby certifies that he is familiar with all the terms and conditions of the said bill of lading, including those on the back thereof, set forth in the classification or tariff which governs the transportation of this shipment and the said terms and conditions are hereby agreed to by the shipper and accepted for himself and his assigns. This is to certify that the above articles are properly describe by name and are packed and marked and are in proper condition for transportation according to regulations by the Interstate Commerce Commission. * If the shipment moves between two ports by a carrier by water, the law requires that the bill of lading shall state whether it is "carrier's or shipper's weight." * Shipper's imprints in lieu of stamp; not a part of Bill of Lading approved by the Interstate Commerce Commission. NOTE: Where the rate is dependent on value, shippers are required to state specifically in writing the agreed or declared value of property. The agreed or declared value of the property is hereby specifically state by the shipper to be not exceeding.

DRIVER'S SIGNATURE/AGENT : ____________________

NOTICE TO RECEIVERS:

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RECEIVED BY : DATE : TIME : Page 1 of 2

APPENDIX B   MILL TEST REPORTS FOR COLUMN MATERIAL

NUCOR PLATE MILL

P.O.Box 279
Winton, NC 27986
(252) 356-3700

Mill Test Report

Page 1

1505 River Rd
Cofield, NC 27922
(252) 356-3700

NUCOR It's Our Nature.

Issuing Date : 06/01/2016 B/L No. : 445564 Load No. : 453113 Our Order No. : 137772/5 Cust. Order No. : VA-511250

Vehicle No: 9181

Specification: 1.2500" x 96.000" x 240.000"
AASHTO M270 GR50/345 T2&T1/ASTM A709-13a GR50/345 T2&T1/A572 50-15

Sold To: INFRAMETALS CO PETERSBURG
580 MIDDLETON BLVD STE D100
LANGHORNE,PA 19047

Ship To: INFRA - METALS
1900 BESSEMER RD
PH 8049575900
PETERSBURG,VA 23805

Marking :

Heat No	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Al(tot)	V	Nb	Ti	N	Ca	B	Sn	Ceq	Pcm
6503350	0.18	1.26	0.008	0.002	0.17	0.24	0.10	0.13	0.02	0.027	0.043	0.001	0.002		0.0020	0.0002	0.013	0.45	0.27

Tensile Test

Plate Serial No	Pieces	Tons	Dir.	(psi) Yield	(psi) Tensile	Elongation % in 2"	Elongation % in 8"
6503350-07	1	4.08	T	67,900	85,000		22.6
			T	55,500	79,200		20.2

Charpy Impacts

Dir.	(ft-lbs) 1	(%) shear	(ft-lbs) 2	(%) shear	(ft-lbs) 3	(%) shear	(ft-lbs) Ave.	(%) shear	Size	Temp (°F)	Min Ave
H-L	44.8		109.4		44.5		66.2		10mm	10	15
H-L	52.9		74.7		90.7		72.8		10mm	-4	15
H-L	98.5		102.7		103.4		101.5		10mm	10	15
H-L	114.5		99.3		117.3		110.4		10mm	10	15
H-L	20.3		28.2		21.1		23.2		10mm	-5	15

NFCM, T1 and T2, 15ft-lbs @ +40 F (20J @ +4 C), H frequency. Temperature reduced by 15 F for each 10 ksi over 65 ksi;

Manufactured to fully killed fine grain practice by Electric Arc Furnace. Welding or weld repair was not performed on this material. Mercury has not been used in the direct manufacturing of this material. Produced as continuous cast discrete plate as-rolled, unless otherwise noted in Specification. For Mexico shipments: nhc-SalesMX@Nucor.com
Yield by 0.5EUL method unless otherwise specified. Ceq = C+(Mn/6)+((Cr+Mo+V)/5)+((Cu+Ni)/15)
Pcm = C+(Si/30)+(Mn/20)+(Cu/20)+(Ni/60)+(Cr/20)+(Mo/15)+(V/10)+5B
Melted and Manufactured in the USA. ISO 9001:2008 certified (#010940) by SRI Quality System Registrar (#0985-09). PED 97/23/EC 7/2 Annex 1, Para. 4.3 Compliant. DIN 50049 3.1.B/EN 10204 3.1B(2004), DIN EN 10204 3.1(2005) compliant. For ABS grades only, Quality Assurance certificate 14-MMPQA-723

We hereby certify that the contents of this report are accurate and correct. All test results and operations performed by the material manufacturer are in compliance with the applicable specifications, including customer specifications.

T. A. Depratis

T. A. Depratis, Metallurgist

6/1/2016 5:30:50 PM

Heat Number	Shipper No	Customer PO#	Customer Name
6503350	899727	36287	AMERICAN BUILDINGS - VA

NUCOR PLATE MILL

P.O.Box 279
Winton, NC 27986
(252) 356-3700

Mill Test Report

Page 2

NUCOR It's our Nature.

Issuing Date : 03/20/2014 B/L No. : 383203 Load No. : 387239 Our Order No. : 119211/9 Cust. Order No. : VA-81287

Vehicle No: 8806

Specification : 1.0000" x 74.000" x 673.000"
AASHTO M270 GR50/345 T2&T1/ASTM A709-13a GR50/345 T2&T1/A572 50-13a

Sold To : INFRAMETALS CO PETERSBURG
580 MIDDLETON BLVD STE D100
LANGHORNE,PA 19047

Ship To : INFRA - METALS
1900 BESSEMER RD
PH 8049575900
PETERSBURG,VA 23805

Marking :

Heat No	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Al(tot)	V	Nb	Ti	N	Ca	B	Sn	CEQ	PCM
4501876	0.17	1.19	0.017	0.002	0.18	0.26	0.08	0.10	0.02	0.029	0.048	0.002	0.003		0.0021	0.0000	0.009	0.42	0.26

Tensile Test								Charpy Impacts												
Plate Serial No	Pieces	Tons	Dir.	(psi) Yield	(psi) Tensile	Elongation % in 2"	Elongation % in 8"	Dir.	(ft-lbs) 1	(%) shear	(ft-lbs) 2	(%) shear	(ft-lbs) 3	(%) shear	(ft-lbs) Ave.	(%) shear	Size	Temp (°F)	Min Ave.
4501876-07	1	7.06	T	55,200	80,700		17.0	H-L	107.1		116.2		103.5		108.9		10mm	40	15
			T	53,300	80,000		20.6	H-L	89.5		79.6		102.2		90.4		10mm	40	15

NFCM, T1 and T2, 15ft-lbs @ +40 F (20J @ +4 C), H frequency. Temperature reduced by 15 F for each 10 ksi over 65 ksi;

Manufactured to fully killed fine grain practice by Electric Arc Furnace. Welding or weld repair was not performed on this material. Mercury has not been used in the direct manufacturing of this material. Produced as continuous cast discrete plate as-rolled, unless otherwise noted in Specification.
Yield by 0.5EUL method unless otherwise specified. Ceq = C+(Mn/6)+((Cr+Mo+V)/5)+((Cu+Ni)/15)
Pcm = C+(Si/30)+(Mn/20)+(Cu/20)+(Ni/60)+(Cr/20)+(Mo/15)+(V/10)+5B
Melted and manufactured in the USA. ISO 9001:2008 certified (#008063) by SRI Quality System Registrar (#0985-09). PED 97/23/EC 7/2 Annex 1, Para. 4.3 Compliant DIN 50049 3.1.B/EN 10204 3.1B(2004), DIN EN 10204 3.1(2005) compliant. For ABS grades only, Quality Assurance certificate 09-MMPQA-546

We hereby certify that the contents of this report are accurate and correct. All test results and operations performed by the material manufacturer are in compliance with the applicable specifications, including customer specifications.

T. A. Depretis

T. A. Depretis, Metallurgist

03/20/2014 5:33:48 PM

Customer Name AMERICAN BUILDINGS - VA

Customer PO# 36267

Shipper No 899676

Heat Number 4501876

APPENDIX C   MILL TEST REPORTS FOR BOLTS

PACKING LIST

BIRMINGHAM BF FASTENER, INC.

Birmingham Fastener, Inc.
a division of the Birmingham Fastener & Supply, Inc. Group
931 Avenue W, Birmingham, Alabama 35214
800.695.3511 205.595.3511 fax 205.591.7107

Invoice Number	
5343125	
Invoice Date	Page
6/14/2016 10:20:57	1 of 2
ORDER NUMBER	
1345018	

Bill To:
AMERICAN BUILDING
P.O. BOX 800
EUFAULA, AL 36072
USA
334-688-2276

Ship To:
AMERICAN BUILDING
501 GOLDEN EAGLE DRIVE
LA CROSSE, VA 23950
USA

Attn: GIOVANNA SCREWS

Ordered By: Ms. TAMMY THOMAS

Customer ID: 200510

PO Number	Order Date	Dispatch Date	Required Date	Pick Ticket No	Salesrep
36302	6/7/2016		6/10/2016	3377522	Tommy. Peters

Customer Note: VENDOR NUMBER - 098800
Customer Note: ***** MTR'S REQUIRED WITH EACH SHIPMENT *****
Order Note: TAG JOB A16SMP22

Delivery Instructions: 2 kegs @ 215 lbs
Carrier: FED EX FGT PRIORITY ***Tracking #:*** 4000525034 wogs 1345177

Quantities: Ordered	Shipped	Remaining	Disp.	Item ID / Item Description	UOM	Unit Size
30.00	30.00	0.00		(001) 75C350A49 3/4-10 X 3 1/2 A490 HVY HEX BOLT	EA	1.0

ICN#: 941500 ***Qty:*** 30.00 EA
Manufacturer Lot ID: 701605

30.00	30.00	0.00		(002) 75CNDH/D 3/4-10 A563 DH HVY HEX NUT DOM.	EA	1.0

Finish: PLAIN ***Spec:*** ASTM A563

ICN#: 368909A ***Qty:*** 30.00 EA
Manufacturer Lot ID: 368909A

30.00	30.00	0.00		(003) 87C350A49 7/8-9 X 3 1/2 A490 HVY HEX BOLT	EA	1.0

ICN#: 361504A ***Qty:*** 30.00 EA
Manufacturer Lot ID: 361504A

30.00	30.00	0.00		(004) 87CNDH/D 7/8-9 A563 DH HVY HEX NUT DOM.	EA	1.0

Finish: PLAIN ***Spec:*** ASTM A563

ICN#: 365478A ***Qty:*** 30.00 EA
Manufacturer Lot ID: 365478A

30.00	30.00	0.00		(005) 112C425A49 1 1/8-7 X 4 1/4 A490 HVY HEX BOLT	EA	1.0

ICN#: 607988 ***Qty:*** 30.00 EA
Manufacturer Lot ID: 719183

30.00	30.00	0.00		(006) 112CNDH/D 1 1/8-7 A563 DH HVY HEX NUT DOM.	EA	1.0

Finish: PLAIN ***Spec:*** ASTM A563

ICN#: 326929A ***Qty:*** 30.00 EA
Manufacturer Lot ID: 326929A

30.00	30.00	0.00		(007) 125C450A49 1 1/4-7 X 4 1/2 A490 HVY HEX BOLT	EA	1.0

Finish: PLAIN ***Spec:*** ASTM A490

ICN#: 757320 ***Qty:*** 30.00 EA

BIRMINGHAM FASTENER, INC.
Birmingham Fastener, Inc.
a division of the Birmingham Fastener & Supply, Inc. Group
931 Avenue W, Birmingham, Alabama 35214
800.695.3511 205.595.3511 fax 205.591.7107

PACKING LIST

Invoice Number	
5343125	
Invoice Date	*Page*
6/14/2016 10:20:57	2 of 2
ORDER NUMBER	
1345018	

Quantities				*Item ID*	*UOM*	
Ordered	*Shipped*	*Remaining*	*Disp.*	*Item Description*		*Unit Size*
Manufacturer Lot ID:	757320					
30.00	30.00	0.00		(008) 125CNDH/D 1 1/4-7 A563 DH HVY HEX NUT DOM.	EA	1.0

Finish: PLAIN ***Spec:*** ASTM A563

ICN#: 359872A ***Qty:*** 30.00 EA
Manufacturer Lot ID: 359872A

Total Lines: 8
Total Pieces: 240
****Est Net Weight:*** 211.08 ***Amount may change at time of invoicing***

Checked by: ______________________ ***Ship Date:*** ______________

Pallet Count: ________ ***# of Cartons:*** ________ ***# of Kegs:*** ________ ***Gross Weight:*** ________

Certificate of Compliance

We hereby certify that the material furnished in reference to the purchase order number will meet or exceed the above assigned ASTM specifications.

Michael R. Black

Michael Black, Quality Manager

7/13/2016 1:19:54PM

TEST REPORT

LAKE ERIE PRODUCTS
CUSTOM ENGINEERED FASTENERS

Operations Center
3281 West County Road 0 NS
Frankfort, IN 46041-6966
T. 765.654.0477
F. 765.654.0857

Ship Date	09-22-12
Certification	321361*40*2
Report Date	09-22-12

Cust PO	U09788
Lot Nbr	701605
Quantity	2450
Mfg Date	09-11-12

BRIGHTON-BEST INTL - LOGAN
2100 CENTER SQUARE ROAD
LOGAN TOWNSHIP, NJ 08085

PART INFORMATION

Part Number	504232
Description	3/4-10 X 3 1/2 A490-1 HEAVY HEX STRUCTURAL DOUBLE MADE IN USA

Finish	PLAIN
Head Marking	LE A490

RAW MATERIAL ANALYSIS

Steel Heat Nbr	Steel Supplier	Steel Grade	Code	Element	Rqd Min Pct	Rqd Max Pct	Percent
CR10207610	Charter Steel	4037M SKFG LE 354-3	C	Carbon	0.40	0.43	0.41
			Mn	Manganese	0.90	1.10	1.00
			P	Phosphorus	0.000	0.020	0.011
			S	Sulfur	0.000	0.020	0.009
			SI	Silicon	0.150	0.300	0.270
			Ni	Nickel	0.00	0.10	0.05
			Cr	Chromium	0.30	0.45	0.32
			Mo	Molybdenum	0.20	0.30	0.21
			Cu	Copper	0.00	0.15	0.10
			Sn	Tin	0.000	0.010	0.008
			V	Vanadium	0.000	0.030	0.004
			Al	Aluminum	0.020	0.050	0.024
			N	Nitrogen	0.0000	0.0080	0.0080
			B	Boron	0.0000	0.0005	0.0001

Certification test results include those reported by the following laboratories:
Charter Steel, A2LA, 01-31-11
LEP Special Fasteners, Inc, ISO17025-A2LA Cert#0122.02, 05-31-12

MECHANICAL PROPERTIES

Wedge Angle	10
Proof Load	40100/120000 (lbs/Psi)

Test Performed	Required	High	Low	Average	Samples
Tensile, PSI	150000 / 170000	160000	158000	159000	3
Proof Load Elongation	0.0000 / 0.0005	0.0001	0.0000	0.0001	2
Superficial R30N	53 / 56	53	53	53	3
Core Hardness, HRC	33 / 36	34	33	33	3

LAKE ERIE PRODUCTS
CUSTOM ENGINEERED FASTENERS

TEST REPORT

Operations Center
3281 West County Road 0 NS
Frankfort, IN 46041-6966
T. 765.654.0477
F. 765.654.0857

Ship Date	09-22-12
Certification	321361*40*2
Report Date	09-22-12

Cust PO	U09788
Lot Nbr	701605
Quantity	2450
Mfg Date	09-11-12

3RIGHTON-BEST INTL - LOGAN
2100 CENTER SQUARE ROAD
LOGAN TOWNSHIP, NJ 08085

Applicable Standards, Specifications, and Sampling Schemes:

Results reported in the mechanical properties section were determined in accordance with the following test methods: ASTM A370, E18, F606/F606M, and SAE J1216. Mechanical properties were determined in accordance with ISO 898-1 and SAE J429. Dimensional properties are compliant to the applicable product standards defined within ASME B18, IFI SSTD, ISO T2 Standards Committees or Customer Specifications for non-standards. Product passed a surface discontinuity inspection following ASTM F788/788M and SAE J1061. Sampling plan is based off ASTM F1470

The listed standards, specifications, and sampling schemes are of the revision in effect on the date of manufacture unless noted otherwise. Only those standards specifically noted under "test methods" or "additional test methods" are included on LE's scope of laboratory accreditation.

Additional Information

None

This lot has been found to conform to the requirements of the above standards and specifications

We certify: The product furnished by LEP Special Fasteners was manufactured, sampled, tested, and inspected in accordance with the standards and specifications listed above and with LEP Special Fasteners Quality Manual in effect as of the date of manufacture. The above data accurately represents values provided by LEP Special Fasteners suppliers and/or values generated in one of LEP Special Fasteners A2LA accredited laboratories. Statistical process control data is on file.

This test report relates only to the sample tested above. This document may only be reproduced unaltered and may not be used for any purpose other than the purpose of certifying the same or lesser quality of the products specified herein. Reproduction, alteration or use of this document for any other purpose is prohibited, except as expressly provided in this certification. LEP Special Fasteners makes no (and disclaims all) representations, warranties and guarantees whatsoever, whether express, implied or statutory, including, without limitation, any warranty of merchantability or fitness for a particular purpose.

CERT #0122.02
"MECHANICAL FIELD OF TESTING"

Lake Erie Products

Michael A. Schwark
Quality Manager

EMAIL

CHARTER STEEL

A Division of
Charter Manufacturing Company, Inc.

1658 Cold Springs Road
Saukville, Wisconsin 53080
(262) 268-2400
1-800-437-8789
FAX (262) 268-2570

CHARTER STEEL TEST REPORT
Reverse Has Text And Codes

LEP Special Fasteners Inc.
3595 West State Road 28
Jim Cull
Frankfort,IN-46041

Cust P.O.	90136
Customer Part #	951012619C1
Charter Sales Order	10072535
Heat #	10207610
Ship Lot #	4156630
Grade	LE354 M SK FG RHQ 1-1/32
Process	HRSA
Finish Size	1-1/32

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed below and on the reverse side,and that it satisfies these requirements.

Test Results of Heat Lot# 10207610

Lab Code: 7388

CHEM %Wt	C	MN	P	S	SI	NI	CR	MO	CU	SN	V
	.41	1.00	.011	.009	.27	.05	.32	.21	.10	.008	.004

	AL	N	B	TI	NB
	.024	.0080	.0001	.002	.002

JOMINY(HRC)	JOM01	JOM02	JOM03	JOM04	JOM05	JOM06	JOM07	JOM08	JOM09	JOM10	JOM12
	57	57	57	53	50	46	44	42	40	39	35

	JOM14	JOM16	JOM18	JOM20	JOM24	JOM28	JOM32
	33	32	31	30	28	26	25

JOMINY SAMPLE TYPE ENGLISH = C DI = 3.28
CHEM. DEVIATION EXT.- GREEN =

Test Results of Rolling Lot# 1074335

	# of Tests	Min Value	Max Value	Mean Value
ROD SIZE	8	1.030	1.039	1.035
ROD OUT OF ROUND	4	.002	.005	.004

REDUCTION RATIO = 36:1

Test Results of Processing Lot# 4156287,4156630

	# of Tests	Min Value	Max Value	Mean Value	
TENSILE	1.0	79.1	79.1	79.1	TENSILE LAB = 0358-02
REDUCTION OF AREA	1	59	59	59	RA LAB = 0358-02
ROCKWELL B	1	80	80	80	RB LAB = 0358-02

NUM DECARB = 1 FREE FERRITE DECARB = .001 FREE FERR & PARTIAL DECARB = .010
CP SPHERO % LAB = 0358-02 NUM SPHERO = 1 SPHERODIZATION = 85.0

Specifications: Manufactured per Charter Steel Quality Manual Rev 9,08-01-09
Meets customer specifications with any applicable Charter Steel exceptions for the following customer documents:
Customer Document = LE 1.1 Revision = 9 Dated = 27-NOV-07

Additional Comments:

Charter Steel
Saukville, WI, USA

Rem: Load1,Fax0,Mail0

ACCREDITED
Testing Laboratory

This MTR supersedes all previously dated MTRs for this order

Janice Barnard
Manager of Quality Assurance
07/30/2012

The following statements are applicable to the material described on the front of this Test Report:

1. Except as noted, the steel supplied for this order was melted, rolled, and processed in the United States meeting DFAR's compliance.
2. Mercury was not used during the manufacture of this product, nor was the steel contaminated with mercury during processing.
3. Unless directed by the customer, there are no welds in any of the coils produced for this order.
4. The laboratory that generated the analytical or test results can be identified by the following key:

Certificate Number	Lab Code	Laboratory		Address
0358-01	7388	CSSM	Charter Steel Melting Division	1653 Cold Springs Road, Saukville, WI 53080
0358-02	8171	CSSR/ CSSP	Charter Steel Rolling/ Processing Division	1658 Cold Springs Road, Saukville, WI 53080
0358-03	123633	CSFP	Charter Steel Ohio Processing Division	6255 US Highway 23, Risingsun, OH 43457
0358-04	125544	CSCM/ CSCR	Charter Steel Cleveland	4300 E. 49th St., Cuyahoga Heights, OH 44125-1004
*	*	--	Subcontracted test performed by laboratory not in Charter Steel system	

5. When run by a Charter Steel laboratory, the following tests were performed according to the latest revisions of the specifications listed below, as noted in the Charter Steel Laboratory Quality Manual:

Test	Specification	CSSM	CSSR/CSSP	CSFP	CSCM/CSCR
Chemistry Analysis	ASTM E415; ASTM E1019	X			X
Macroetch	ASTM E381	X			X
Hardenability (Jominy)	ASTM A255; SAE J406; JIS G0561	X			X
Grain Size	ASTM E112	X	X	X	X
Tensile Test	ASTM E8; ASTM A370		X	X	X
Rockwell Hardness	ASTM E18; ASTM A370	X	X	X	X
Microstructure (spheroidization)	ASTM A892		X	X	
Inclusion Content (Methods A, E)	ASTM E45		X		X
Decarburization	ASTM E1077		X	X	X

Charter Steel has been accredited to perform all of the above tests by the American Association for Laboratory Accreditation (A2LA). These accreditations expire 01/31/13.

All other test results associated with a Charter Steel laboratory that appear on the front of this report, if any, were performed according to documented procedures developed by Charter Steel and are not accredited by A2LA.

6. The test results on the front of this report are the true values measured on the samples taken from the production lot. They do not apply to any other sample.
7. This test report cannot be reproduced or distributed except in full without the written permission of Charter Steel. The primary customer whose name and address appear on the front of this form may reproduce this test report subject to the following restrictions:
   - It may be distributed only to their customers
   - Both sides of all pages must be reproduced in full
8. This certification is given subject to the terms and conditions of sale provided in Charter Steel's acknowledgement (designated by our Sales Order number) to the customer's purchase order. Both order numbers appear on the front page of this Report.
9. Where the customer has provided a specifiction, the results on the front of this test report conform to that specification unless otherwise noted on this test report.

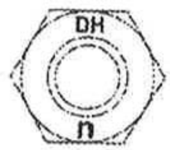
A2LA ACCREDITED Testing Laboratory

NUCOR
FASTENER DIVISION

LOT NO.
368909A

Post Office Box 6100
Saint Joe, Indiana 46785
Telephone 260/337-1600

CUSTOMER NO/NAME
126 BIRMINGHAM FASTENER — NUCOR ORDER # 960776
TEST REPORT SERIAL# FB479320 — CUST PART # 75CNDH/D
TEST REPORT ISSUE DATE 12/01/15
DATE SHIPPED 2/18/16 — CUSTOMER P.O. # 6113097
NAME OF LAB SAMPLER: KELLEY HARTER, LAB TECHNICIAN



****************CERTIFIED MATERIAL TEST REPORT****************

NUCOR PART NO	QUANTITY	LOT NO.	DESCRIPTION
175650	54000	368909A	3/4-10 GR DH HV HX NUT

MANUFACTURE DATE 11/18/15 — HEX NUT BLACK

--CHEMISTRY — MATERIAL GRADE -1045L

**CHEMISTRY COMPOSITION (WT% HEAT ANALYSIS) BY MATERIAL SUPPLIER — NUCOR STEEL - SOUTH CAROL

MATERIAL NUMBER	HEAT NUMBER	C	MN	P	S	SI
RM030397	DL15106139	.46	.66	.003	.018	.23

--MECHANICAL PROPERTIES IN ACCORDANCE WITH ASTM A563-07a

SURFACE HARDNESS (R30N)	CORE HARDNESS (RC)	PROOF LOAD 58500 LBS	TENSILE STRENGTH DEG-WEDGE (LBS)	STRESS (PSI)
N/A	28.9	PASS	N/A	N/A
N/A	29.4	PASS	N/A	N/A
N/A	30.6	PASS	N/A	N/A
N/A	28.9	PASS	N/A	N/A
N/A	29.5	PASS	N/A	N/A
AVERAGE VALUES FROM TESTS	29.5			

PRODUCTION LOT SIZE 200000 PCS

--VISUAL INSPECTION IN ACCORDANCE WITH ASTM A563-07a — 160 PCS. SAMPLED — LOT PASSED
HEAT TREATMENT - AUSTENITIZED, OIL QUENCHED & TEMPERED (MIN 800 DEG F)

--DIMENSIONS PER ASME B18.2.6-2012

CHARACTERISTIC	#SAMPLES TESTED	MINIMUM	MAXIMUM
Width Across Corners	8	1.400	1.407
Thickness	32	0.733	0.753

ALL TESTS ARE IN ACCORDANCE WITH THE LATEST REVISIONS OF THE METHODS PRESCRIBED IN THE APPLICABLE SAE AND ASTM SPECIFICATIONS. THE SAMPLES TESTED CONFORM TO THE SPECIFICATIONS AS DESCRIBED/LISTED ABOVE AND WERE MANUFACTURED FREE OF MERCURY CONTAMINATION. NO INTENTIONAL ADDITIONS OF BISMUTH, SELENIUM, TELLURIUM, OR LEAD WERE USED IN THE STEEL USED TO PRODUCE THIS PRODUCT.
THE STEEL WAS MELTED AND MANUFACTURED IN THE U.S.A. AND THE PRODUCT WAS MANUFACTURED AND TESTED IN THE U.S.A. PRODUCT COMPLIES WITH DFARS 252.225-7014. WE CERTIFY THAT THIS DATA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY. THIS CERTIFIED MATERIAL TEST REPORT RELATES ONLY TO THE ITEMS LISTED ON THIS DOCUMENT AND MAY NOT BE REPRODUCED EXCEPT IN FULL.

ACCREDITED

MECHANICAL FASTENER
CERTIFICATE NO. A2LA 0139.01
EXPIRATION DATE 12/31/15

NUCOR FASTENER
A DIVISION OF NUCOR CORPORATION

John W. Ferguson

JOHN W. FERGUSON
QUALITY ASSURANCE SUPERVISOR

30397

NUCOR

NUCOR CORPORATION
NUCOR STEEL SOUTH CAROLINA

Mill Certification
10/29/2015

MTR #: 0000101916
300 Steel Mill Road
DARLINGTON, SC 29540
(843) 393-5841
Fax: (843) 395-8701

Sold To: NUCOR FASTENER INDIANA
PO BOX 6100
ST JOE, IN 46785-0000
(800) 955-6826
Fax: (219) 337-1726

Ship To: NUCOR FASTENER
6730 COUNTY ROAD 60
ST JOE, IN 46785
(800) 955-6826
Fax: (219) 337-1722

Customer P.O.	156181	Sales Order	234003.1
Product Group	Special Bar Quality	Part Number	30001000396V780
Grade	1045L	Lot #	DL1510613901
Size	1" (1.0000) Round	Heat #	DL15106139
Product	1" (1.0000) Round 33' 1045L	B.L. Number	C1-675912
Description	1045L	Load Number	C1-356576
Customer Spec		Customer Part #	025012

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

Roll Date: 10/24/2015 Melt Date: 10/16/2015 Qty Shipped LBS: 95,881 Qty Shipped Pcs: 1,088

Melt Date: 10/16/2015

C	Mn	V	Si	S	P	Cu	Cr	Ni	Mo	Al	Cb
0.46%	0.66%	0.002%	0.23%	0.018%	0.003%	0.15%	0.10%	0.06%	0.01%	0.001%	0.002%
Pb	**Sn**	**Ca**	**B**	**Ti**	**NICUMO**						
0.003%	0.008%	0.0008%	0.0003%	0.001%	0.22						

NICUMO: Cu+Ni+Mo

Roll Date: 10/24/2015

Reduction Ratio 62 :1

ASTM E381
Surface: 2 Mid Radius: 2 Center: 2

Specification Comments:

1. WELDING OR WELD REPAIR WAS NOT PERFORMED ON THIS MATERIAL
2. MELTED AND MANUFACTURED IN THE USA
3. MERCURY, RADIUM, OR ALPHA SOURCE MATERIALS IN ANY FORM HAVE NOT BEEN USED IN THE PRODUCTION OF THIS MATERIAL

Chemistry Verification Checks

Part# 25012 RM# 30397

	Checked By	Date
Receiving OK:	297	11-10-15
Certifications OK:	375	11-10-15

James H. Blew
Division Metallurgist

Page 1 of 2

NBMG-10 January 1, 2012

NUCOR
FASTENER DIVISION

LOT NO.
361504A

Post Office Box 6100
Saint Joe, Indiana 46785
Telephone 260/337-1600

CUSTOMER NO/NAME
126 BIRMINGHAM FASTENER NUCOR ORDER # 934475
TEST REPORT SERIAL# FB464578 CUST PART # 87C350A49
TEST REPORT ISSUE DATE 6/30/15
DATE SHIPPED 7/30/15 CUSTOMER P.O. # 6095449
NAME OF LAB SAMPLER: LISA EDGAR, LAB TECHNICIAN

****************CERTIFIED MATERIAL TEST REPORT****************

NUCOR PART NO	QUANTITY	LOT NO.	DESCRIPTION
165460	500	361504A	7/8-9 X 3 1/2 A490 HVY HX STRUC SCREW PLAIN

MANUFACTURE DATE 6/23/15

--CHEMISTRY MATERIAL GRADE -4135MLV
**CHEMISTRY COMPOSITION (WT% HEAT ANALYSIS) BY MATERIAL SUPPLIER NUCOR STEEL - NEBRASKA

MATERIAL NUMBER	HEAT NUMBER	C	MN	P	S	SI	CR	MO	V
RM029842	NF15201040	.36	.92	.012	.008	.21	1.04	.18	.030

--MECHANICAL PROPERTIES IN ACCORDANCE WITH ASTM A490-14a

SURFACE HARDNESS (R30N)	CORE HARDNESS (RC)	PROOF LOAD 55450 LBS	TENSILE STRENGTH 10 DEG-WEDGE (LBS)	STRESS (PSI)
N/A	35.3	PASS	77310	173052
N/A	34.5	PASS	78690	167338
N/A	36.3			170325
AVERAGE VALUES FROM TESTS				
	35.4		78000	170238

PRODUCTION LOT SIZE 3020 PCS

--WET MAGNETIC PARTICLE INSPECTION IN ACCORDANCE WITH ASTM A490-14a 3 PCS. SAMPLED LOT PASSED

--MICROHARDNESS TEST RESULT IN ACCORDANCE WITH ASTM A490-14a
POSITION 1. 378 2. 384 3. 373
CARBURIZATION/DECARBURIZATION TEST IN ACCORDANCE WITH A490-14a LOT PASSED
HEAT TREATMENT - AUSTENITIZED, OIL QUENCHED & TEMPERED (MIN 800 DEG F)

--DIMENSIONS PER ASME B18.2.6-2012

CHARACTERISTIC	#SAMPLES TESTED	MINIMUM	MAXIMUM
Width Across Corners	4	1.620	1.623
Grip Length	4	1.91	1.96
Head Height	4	0.546	0.551
Threads	4	PASS	PASS

ALL TESTS ARE IN ACCORDANCE WITH THE LATEST REVISIONS OF THE METHODS PRESCRIBED IN THE APPLICABLE SAE AND ASTM SPECIFICATIONS. THE SAMPLES TESTED CONFORM TO THE SPECIFICATIONS AS DESCRIBED/LISTED ABOVE AND WERE MANUFACTURED FREE OF MERCURY CONTAMINATION. NO HEATS TO WHICH BISMUTH, SELENIUM, TELLURIUM, OR LEAD WAS INTENTIONALLY ADDED HAVE BEEN USED TO PRODUCE THE BOLTS.
THE STEEL WAS MELTED AND MANUFACTURED IN THE U.S.A. AND THE PRODUCT WAS MANUFACTURED AND TESTED IN THE U.S.A. PRODUCT COMPLIES WITH DFARS 252.225-7014. WE CERTIFY THAT THIS DATA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY. THIS CERTIFIED MATERIAL TEST REPORT RELATES ONLY TO THE ITEMS LISTED ON THIS DOCUMENT AND MAY NOT BE REPRODUCED EXCEPT IN FULL.

ACCREDITED

MECHANICAL FASTENER
CERTIFICATE NO. A2LA 0139.01
EXPIRATION DATE 12/31/15

NUCOR FASTENER
A DIVISION OF NUCOR CORPORATION

John W. Ferguson
JOHN W. FERGUSON
QUALITY ASSURANCE SUPERVISOR

298412

NUCOR
NUCOR CORPORATION
NUCOR STEEL NEBRASKA

Mill Certification
3/3/2015

MTR #: 0000090443
2911 East Nucor Road
NORFOLK, NE 68701
(402) 644-0200
Fax: (402) 644-0329

Sold To: NUCOR FASTENER INDIANA
PO BOX 6100
6730 COUNTY RD 60
ST JOE, IN 46785-0000
(260) 337-1600
Fax: (435) 734-4581

Ship To: NUCOR FASTENER INDIANA
COUNTY RD 60
ST JOE, IN 46785-0000

Customer P.O.	150711	Sales Order	139901.8
Product Group	Special Bar Quality	Part Number	31000890000S510
Grade	4135MLV	Lot #	NF1520104011
Size	57/64" (.8906) Round Coil	Heat #	NF15201040
Product	57/64" (.8906) Round Coil 4135MLV	B.L. Number	N1-299381
Description	4135MLV	Load Number	N1-242817
Customer Spec		Customer Part #	008014

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

Roll Date: 3/2/2015 Melt Date: 2/9/2015 Qty Shipped LBS: 42,443 Qty Shipped Pcs: 8

Melt Date: 2/9/2015

C	Mn	V	Si	S	P	Cu	Cr	Ni	Mo	Al	Cb
0.36%	0.92%	0.03%	0.21%	0.008%	0.012%	0.11%	1.04%	0.07%	0.18%	0.001%	0.005%
Pb	**Sn**	**Ca**	**B**	**Ti**	**CUNI**						
0.000%	0.007%	0.0007%	0.0002%	0.001%	0.18%						

CUNI: Cu + Ni

Austenitic fine grain by chemical analysis per the latest revision of ASTM A29

Roll Date: 3/2/2015

Decarb depth : 0.0017 Tensile 1: 129214psi Elongation 14% in 8"(% in 203.3mm)

Reduction Ratio 70 :1

Reheat Quality Comments: REVISION #1

Specification Comments: Fine Grain 5 or finer per ASTM A29-05

Sellenium,Tellurium,Lead,Bismuth or Boron were not intentionally added to this heat.

1. All manufacturing processes of the steel materials in this product, including melting, have been performed in the United States.
2. All products produced are weld free.
3. Mercury, in any form, has not been used in the production or testing of this material.
4. Test conform to ASTM A29-12, ASTM E415 and ASTM E1019-resulphurized grades or applicable customer requirements.
5. All material melted at Nucor Steel Nebraska is produced in an Electric Arc Furnace
6. Strand Cast
7. ISO-17025 LAB accreditation cert available upon request
8. Exporting Country-USA
9. Sales@nucorne.com

Jim Hill

Jim Hill
Division Metallurgist

Page 1 of 2

NBMG-10 January 1, 2012

NUCOR

NUCOR CORPORATION
NUCOR STEEL NEBRASKA

Mill Certification

3/3/2015

MTR #: 0000090443
2911 East Nucor Road
NORFOLK, NE 68701
(402) 644-0200
Fax: (402) 644-0329

Sold To: NUCOR FASTENER INDIANA
PO BOX 6100
6730 COUNTY RD 60
ST JOE, IN 46785-0000
(260) 337-1600
Fax: (435) 734-4581

Ship To: NUCOR FASTENER INDIANA
COUNTY RD 60
ST JOE, IN 46785-0000

Customer P.O.	150711	Sales Order	139901.8
Product Group	Special Bar Quality	Part Number	31000890000S510
Grade	4135MLV	Lot #	NF1520104011
Size	57/64" (.8906) Round Coil	Heat #	NF15201040
Product	57/64" (.8906) Round Coil 4135MLV	B.L. Number	N1-299381
Description	4135MLV	Load Number	N1-242817
Customer Spec		Customer Part #	008014

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

1. All manufacturing processes of the steel materials in this product, including melting, have been performed in the United States.
2. All products produced are weld free.
3. Mercury, in any form, has not been used in the production or testing of this material.
4. Test conform to ASTM A29-12, ASTM E415 and ASTM E1019-resulphurized grades or applicable customer requirements.
5. All material melted at Nucor Steel Nebraska is produced in an Electric Arc Furnace
6. Strand Cast
7. ISO-17025 LAB accreditation cert available upon request
8. Exporting Country-USA
9. Sales@nucorne.com

Chemistry Verification Checks

Part# 8014 **RM#** 29842

	Checked By	Date
Receiving OK:	297	3-18-15
Certifications OK:	375	3-18-15

Jim Hill

Jim Hill
Division Metallurgist

Page 2 of 2

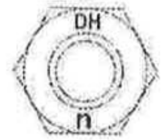
NBMG-10 January 1, 2012

NUCOR
FASTENER DIVISION

LOT NO.
365478A

Post Office Box 6100
Saint Joe, Indiana 46785
Telephone 260/337-1800

TEST REPORT SERIAL# FB469275
TEST REPORT ISSUE DATE 10/08/15
MANUFACTURE DATE 9/22/15
NAME OF LAB SAMPLER: BRUCE DELAUDER, LAB TECHNICIAN



****************CERTIFIED MATERIAL TEST REPORT****************

PART NO.	LOT NO.	DESCRIPTION
175660	365478A	7/8-9 GR DH HV HX NUT HEX NUT BLACK

--CHEMISTRY MATERIAL GRADE -1045L

**CHEMISTRY COMPOSITION (WT% HEAT ANALYSIS) BY MATERIAL SUPPLIER

MATERIAL NUMBER	HEAT NUMBER	C	MN	P	S	SI	
RM030285	NF15102777	.44	.67	.011	.011	.18	NUCOR STEEL - NEBRASKA

--MECHANICAL PROPERTIES IN ACCORDANCE WITH ASTM A563-07a

SURFACE HARDNESS (R30N)	CORE HARDNESS (RC)	PROOF LOAD 80900 LBS	TENSILE STRENGTH DEG-WEDGE (LBS)	STRESS (PSI)
N/A	28.7	PASS	N/A	N/A
N/A	31.4	PASS	N/A	N/A
N/A	32.4	PASS	N/A	N/A
N/A	31.3	PASS	N/A	N/A
N/A	30.8	PASS	N/A	N/A
AVERAGE VALUES FROM TESTS				
	30.9			

PRODUCTION LOT SIZE 133000 PCS

--VISUAL INSPECTION IN ACCORDANCE WITH ASTM A563-07a 80 PCS. SAMPLED LOT PASSED
HEAT TREATMENT - AUSTENITIZED, OIL QUENCHED & TEMPERED (MIN 800 DEG F)

--DIMENSIONS PER ASME B18.2.6-2012

CHARACTERISTIC	#SAMPLES TESTED	MINIMUM	MAXIMUM
Width Across Corners	8	1.6060	1.6110
Thickness	32	0.8610	0.8710

ALL TESTS ARE IN ACCORDANCE WITH THE LATEST REVISIONS OF THE METHODS PRESCRIBED IN THE APPLICABLE SAE AND ASTM SPECIFICATIONS. THE SAMPLES TESTED CONFORM TO THE SPECIFICATIONS AS DESCRIBED/LISTED ABOVE AND WERE MANUFACTURED FREE OF MERCURY CONTAMINATION. NO INTENTIONAL ADDITIONS OF BISMUTH, SELENIUM, TELLURIUM, OR LEAD WERE USED IN THE STEEL USED TO PRODUCE THIS PRODUCT.
THE STEEL WAS MELTED AND MANUFACTURED IN THE U.S.A. AND THE PRODUCT WAS MANUFACTURED AND TESTED IN THE U.S.A. PRODUCT COMPLIES WITH DFARS 252.225-7014. WE CERTIFY THAT THIS DATA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY. THIS CERTIFIED MATERIAL TEST REPORT RELATES ONLY TO THE ITEMS LISTED ON THIS DOCUMENT AND MAY NOT BE REPRODUCED EXCEPT IN FULL.

ACCREDITED

MECHANICAL FASTENER
CERTIFICATE NO. A2LA 0139.01
EXPIRATION DATE 12/31/15

NUCOR FASTENER
A DIVISION OF NUCOR CORPORATION

John W. Ferguson

JOHN W. FERGUSON
QUALITY ASSURANCE SUPERVISOR

30285

NUCOR

NUCOR CORPORATION
NUCOR STEEL NEBRASKA

Mill Certification

9/11/2015

MTR #: 0000132552
2911 East Nucor Road
NORFOLK, NE 68701
(402) 644-0200
Fax: (402) 644-0329

Sold To: NUCOR FASTENER INDIANA
PO BOX 6100
6730 COUNTY RD 60
ST JOE, IN 46785-0000
(260) 337-1600
Fax: (435) 734-4581

Ship To: NUCOR FASTENER INDIANA
COUNTY RD 60
ST JOE, IN 46785-0000

Customer P.O.	154337	Sales Order	143129.14
Product Group	Special Bar Quality	Part Number	30001125480V780
Grade	1045L	Lot #	NF1510277751
Size	1-1/8" (1.1250) Round	Heat #	NF15102777
Product	1-1/8" (1.1250) Round 40' 1045L	B.L. Number	N1-310461
Description	1045L	Load Number	N1-253763
Customer Spec		Customer Part #	025014

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

Roll Date: 8/28/2015 Melt Date: 8/23/2015 Qty Shipped LBS: 43,288 Qty Shipped Pcs: 320

Melt Date: 8/23/2015

C	Mn	V	Si	S	P	Cu	Cr	Ni	Mo	Al	Cb
0.44%	0.67%	0.003%	0.18%	0.011%	0.011%	0.16%	0.10%	0.06%	0.01%	0.001%	0.002%
Pb	Sn	Ca	B	Ti	NICUMO						
0.000%	0.006%	0.0006%	0.0002%	0.002%	0.23						

NICUMO: Cu+Ni+Mo

Roll Date: 8/28/2015

Reduction Ratio 44 :1

ASTM E381
Surface: 1 Mid Radius: 1 Center: 1

Specification Comments: Coarse Grain Practice

Sellenium,Tellurium,Lead,Bismuth or Boron were not intentionally added to this heat.

1. All manufacturing processes of the steel materials in this product, including melting, have been performed in the United States.
2. All products produced are weld free.
3. Mercury, in any form, has not been used in the production or testing of this material.
4. Test conform to ASTM A29-12, ASTM E415 and ASTM E1019-resulphurized grades or applicable customer requirements.
5. All material melted at Nucor Steel Nebraska is produced in an Electric Arc Furnace
6. Strand Cast
7. ISO-17025 LAB accreditation cert available upon request
8. Exporting Country-USA
9. Sales@nucorne.com

Chemistry Verification Checks

Part# 25014 RM# 30285

	Checked By	Date
Receiving OK:	297	9-14-15
Certifications OK:	375	9-14-15

Jim Hill

Jim Hill
Division Metallurgist

Page 1 of 1

NBMG-10 January 1, 2012

FONTANA FASTENERS
PLANT FRANKFORT

TEST REPORT

Operations Center
3281 West County Road 0 NS
Frankfort, IN 46041-6966
T. 765.654.0477
F. 765.654.0857

Ship Date	
Certification	374619*76*4
Report Date	10-01-13

Cust PO	6032446
Lot Nbr	719183
Quantity	1125
Mfg Date	09-24-13

Birmingham Fastener, Inc.
Hanceville Dist. Center
1100 Main Street S.E.
Hanceville, AL 35077

PART INFORMATION

Part Number	112C425A49
Description	1 1/8-7 X 4 1/4 A490-1 HEAVY HEX STRUCTURAL DOUBLE MADE IN USA

Finish	PLAIN
Head Marking	LE A490

RAW MATERIAL ANALYSIS

Steel Heat Nbr	Steel Supplier	Steel Grade	Code	Element	Rqd Min Pct	Rqd Max Pct	Percent
CR10244150	Charter Steel	50B35 SKFG	C	Carbon	0.32	0.37	0.36
			Mn	Manganese	0.95	1.10	1.02
			P	Phosphorus	0.000	0.017	0.012
			S	Sulfur	0.000	0.020	0.007
			Si	Silicon	0.150	0.300	0.250
			Ni	Nickel	0.00	0.10	0.05
			Cr	Chromium	0.50	0.65	0.56
			Mo	Molybdenum	0.10	0.15	0.11
			Cu	Copper	0.00	0.15	0.09
			Sn	Tin	0.000	0.011	0.007
			V	Vanadium	0.000	0.010	0.004
			Al	Aluminum	0.020	0.050	0.027
			N	Nitrogen	0.0000	0.0080	0.0060
			B	Boron	0.0005	0.0030	0.0028
			Ti	Titanium	0.015	0.050	0.019

Certification test results include those reported by the following laboratories:
Charter Steel, A2LA, 01-31-13
LEP Special Fasteners, Inc, ISO17025-A2LA Cert#0122.02, 05-31-12

MECHANICAL PROPERTIES

Wedge Angle	6
Proof Load	91550/120000 (lbs/Psi)

Test Performed	Required	High	Low	Average	Samples
Tensile, PSI	150000 / 170000	165000	162000	163000	3
Proof Load Elongation	0.0000 / 0.0005	0.0005	0.0000	0.0002	3
Superficial R30N	53 / 56	55	53	54	3
Core Hardness, HRC	33 / 36	35	34	34	3

FONTANA FASTENERS
PLANT FRANKFORT

TEST REPORT

Operations Center
3281 West County Road 0 NS
Frankfort, IN 46041-6966
T. 765.654.0477
F. 765.654.0857

Ship Date	
Certification	374619*76*4
Report Date	10-01-13

Cust PO	6032446
Lot Nbr	719183
Quantity	1125
Mfg Date	09-24-13

Birmingham Fastener, Inc.
Hanceville Dist. Center
1100 Main Street S.E.
Hanceville, AL 35077

Applicable Standards, Specifications, and Sampling Schemes:

Test methods are in accordance with ASTM F606-5. Thread fit and dimensional properties are compliant to ASTM B18.2.6. These bolts passed a surface discontinuity inspection per ASTM F788, were not produced from heats in which Bismuth, Selenium, Tellurium, or Lead was intentionally added, nor were they exposed to Mercury or any other metal alloy that is liquid at ambient temperature during processing or while in our possession. These bolts were manufactured in the U.S.A. from domestic material. Results reported in the mechanical properties section were determined in accordance with the following test methods: ASTM A370 E18, F606/F606M and SAE J1216. Mechanical properties were determined in accordance with ISO 898-1 and SAE J429. Dimensional properties are compliant to the applicable product standards defined within ASME B18, IFI SSTD, ISO T2 Standards Committees or Customer Specifications for non-standards. Product passed a surface discontinuity inspection following ASTM F788/788M and SAE J1061. Sampling plan is based off ASTM F1470 ASTM F1941 FE/ZN 5 Melted and Manufactured in the USA

The listed standards, specifications, and sampling schemes are of the revision in effect on the date of manufacture unless noted otherwise. Only those standards specifically noted under "test methods" or "additional test methods" are included on LE's scope of laboratory accreditation.

Additional Information

None

This lot has been found to conform to the requirements of the above standards and specifications

We certify: The product furnished by LEP Special Fasteners was manufactured, sampled, tested, and inspected in accordance with the standards and specifications listed above and with LEP Special Fasteners Quality Manual in effect as of the date of manufacture. The above data accurately represents values provided by LEP Special Fasteners suppliers and/or values generated in one of LEP Special Fasteners A2LA accredited laboratories. Statistical process control data is on file.
This test report relates only to the sample tested above. This document may only be reproduced unaltered and may not be used for any purpose other than the purpose of certifying the same or lesser quantity of the product specified herein. Reproduction, alteration or use of this document for any other purpose is prohibited, except as expressly provided in this certification. LEP Special Fasteners makes no (and disclaims all) representations, warranties and guarantees whatsoever, whether express, implied or statutory, including, without limitation, any warranty of merchantability or fitness for a particular purpose

LEP Special Products Inc

Michael Rogler
Quality Manager

ACCREDITED
CERT #0122.02
"MECHANICAL FIELD OF TESTING"

EMAIL

CHARTER STEEL

A Division of
Charter Manufacturing Company, Inc.

1658 Cold Springs Road
Saukville, Wisconsin 53080
(262) 268-2400
1-800-437-8789
FAX (262) 268-2570

CHARTER STEEL TEST REPORT
Reverse Has Text And Codes

LEP Special Fasteners Inc.
3595 West State Road 28
Jim Cull
Frankfort,IN-46041

Cust P.O.	96773
Customer Part #	951713016C1
Charter Sales Order	10089213
Heat #	10244150
Ship Lot #	4194202
Grade	50B35 M SK FG RHQ 1-3/16
Process	HRSA
Finish Size	1-3/16

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed below and on the reverse side,and that it satisfies these requirements.

Test Results of Heat Lot# 10244150

Lab Code: 7388

CHEM	C	MN	P	S	SI	NI	CR	MO	CU	SN	V
%Wt	.36	1.02	.012	.007	.250	.05	.56	.11	.09	.007	.004
	AL	N	B	TI	NB						
	.027	.0060	.0028	.019	.002						

JOMINY(HRC)	JOM01	JOM02	JOM03	JOM04	JOM05	JOM06	JOM07	JOM08	JOM09	JOM10	JOM11	JOM12
	57	56	55	55	55	55	55	54	54	54	53	52
	JOM13	JOM14	JOM15	JOM16	JOM18	JOM20	JOM22	JOM24	JOM26	JOM28	JOM30	JOM32
	51	50	48	47	44	42	41	39	37	36	36	35

JOMINY SAMPLE TYPE ENGLISH = R JOMINY LAB = 0358-01 DI = 5.16
CHEM. DEVIATION EXT.-GREEN =

Test Results of Rolling Lot# 1091159

	# of Tests	Min Value	Max Value	Mean Value
ROD SIZE	2	1.187	1.193	1.190
ROD OUT OF ROUND	1	.006	.006	.006

REDUCTION RATIO = 27:1

Test Results of Processing Lot# 4193994,4194202

	# of Tests	Min Value	Max Value	Mean Value	
ROCKWELL B	1	79	79	79	RB LAB = 0358-02

NUM DECARB = 1 FREE FERRITE DECARB = .000 FREE FERR & PARTIAL DECARB = .004
CP SPHERO % LAB = 0358-02 NUM SPHERO = 1 SPHERODIZATION = 87.5

Specifications: Manufactured per Charter Steel Quality Manual Rev 9,08-01-09
Meets customer specifications with any applicable Charter Steel exceptions for the following customer documents:
Customer Document = LE 1.1 Revision = 9 Dated = 27-NOV-07

Additional Comments:

Charter Steel
Saukville, WI, USA

Rem: Load1,Fax0,Mail0

ACCREDITED
Testing Laboratory

This MTR supersedes all previously dated MTRs for this order

Janice Barnard
Manager of Quality Assurance
03/08/2013

The following statements are applicable to the material described on the front of this Test Report:

1. Except as noted, the steel supplied for this order was melted, rolled, and processed in the United States meeting DFAR's compliance.
2. Mercury was not used during the manufacture of this product, nor was the steel contaminated with mercury during processing.
3. Unless directed by the customer, there are no welds in any of the coils produced for this order.
4. The laboratory that generated the analytical or test results can be identified by the following key:

Certificate Number	Lab Code	Laboratory		Address
0358-01	7388	CSSM	Charter Steel Melting Division	1653 Cold Springs Road, Saukville, WI 53080
0358-02	8171	CSSR/ CSSP	Charter Steel Rolling/ Processing Division	1658 Cold Springs Road, Saukville, WI 53080
0358-03	123633	CSFP	Charter Steel Ohio Processing Division	6255 US Highway 23, Risingsun, OH 43457
0358-04	125544	CSCM/ CSCR	Charter Steel Cleveland	4300 E. 49th St., Cuyahoga Heights, OH 44125-1004
*	*	--	Subcontracted test performed by laboratory not in Charter Steel system	

5. When run by a Charter Steel laboratory, the following tests were performed according to the latest revisions of the specifications listed below, as noted in the Charter Steel Laboratory Quality Manual:

Test	Specification	CSSM	CSSR/CSSP	CSFP	CSCM/CSCR
Chemistry Analysis	ASTM E415; ASTM E1019	X			X
Macroetch	ASTM E381	X			X
Hardenability (Jominy)	ASTM A255; SAE J406; JIS G0561	X			X
Grain Size	ASTM E112	X	X	X	X
Tensile Test	ASTM E8; ASTM A370		X	X	X
Rockwell Hardness	ASTM E18; ASTM A370	X	X	X	X
Microstructure (spheroidization)	ASTM A892		X	X	
nclusion Content (Methods A, E)	ASTM E45		X		X
Decarburization	ASTM E1077		X	X	X

Charter Steel has been accredited to perform all of the above tests by the American Association for Laboratory Accreditation (A2LA). These accreditations expire 01/31/15.

All other test results associated with a Charter Steel laboratory that appear on the front of this report, if any, were performed according to documented procedures developed by Charter Steel and are not accredited by A2LA.

6. The test results on the front of this report are the true values measured on the samples taken from the production lot. They do not apply to any other sample.
7. This test report cannot be reproduced or distributed except in full without the written permission of Charter Steel. The primary customer whose name and address appear on the front of this form may reproduce this test report subject to the following restrictions:
   - It may be distributed only to their customers
   - Both sides of all pages must be reproduced in full
8. This certification is given subject to the terms and conditions of sale provided in Charter Steel's acknowledgement (designated by our Sales Order number) to the customer's purchase order. Both order numbers appear on the front page of this Report.
9. Where the customer has provided a specifiction, the results on the front of this test report conform to that specification unless otherwise noted on this test report.

ACCREDITED
Testing Laboratory

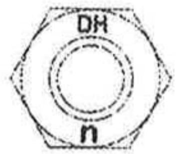
NUCOR

FASTENER DIVISION

LOT NO.
326929A

Post Office Box 6100
Saint Joe, Indiana 46785
Telephone 260/337-1600

CUSTOMER NO/NAME
9000 BIRMINGHAM-CONS/SHIPPING — NUCOR ORDER # 830310
TEST REPORT SERIAL# FB410200 — CUST PART # 112CNDH/D
TEST REPORT ISSUE DATE 7/08/13
DATE SHIPPED 9/19/13 — CUSTOMER P.O. # 6029453
NAME OF LAB SAMPLER: DAWN LEAVITT, LAB TECHNICIAN



*****************CERTIFIED MATERIAL TEST REPORT*****************

NUCOR PART NO	QUANTITY	LOT NO.	DESCRIPTION
175310	3150	326929A	1 1/8-7 GR DH HV HEX NUT
MANUFACTURE DATE 6/05/13			HEX NUT BLACK

--CHEMISTRY — MATERIAL GRADE -1045L

**CHEMISTRY COMPOSITION (WT% HEAT ANALYSIS) BY MATERIAL SUPPLIER — NUCOR STEEL - SOUTH CAROL

MATERIAL NUMBER	HEAT NUMBER		C	MN	P	S	SI
RM028265	DL13102374		.45	.65	.011	.009	.19
		MIN	.20	.60			
		MAX	.55		.040	.050	

--MECHANICAL PROPERTIES IN ACCORDANCE WITH ASTM A563-07a

SURFACE HARDNESS (R30N)	CORE HARDNESS (RC)	PROOF LOAD 133600 LBS	TENSILE STRENGTH DEG-WEDGE (LBS)	STRESS (PSI)
N/A	31.3	PASS	N/A	N/A
N/A	31.0	PASS	N/A	N/A
N/A	32.4	PASS	N/A	N/A
N/A	30.8	PASS	N/A	N/A
N/A	30.2	PASS	N/A	N/A
AVERAGE VALUES FROM TESTS	31.1	PRODUCTION LOT SIZE	57000 PCS	

--VISUAL INSPECTION IN ACCORDANCE WITH ASTM A563-07a — 80 PCS. SAMPLED — LOT PASSED
HEAT TREATMENT - AUSTENITIZED, OIL QUENCHED & TEMPERED (MIN 800 DEG F)

--DIMENSIONS PER ASME B18.2.6-2010

CHARACTERISTIC	#SAMPLES TESTED	MINIMUM	MAXIMUM
Width Across Corners	8	2.0250	2.0290
Thickness	32	1.1060	1.1340

ALL TESTS ARE IN ACCORDANCE WITH THE LATEST REVISIONS OF THE METHODS PRESCRIBED IN THE APPLICABLE SAE AND ASTM SPECIFICATIONS. THE SAMPLES TESTED CONFORM TO THE SPECIFICATIONS AS DESCRIBED/LISTED ABOVE AND WERE MANUFACTURED FREE OF MERCURY CONTAMINATION. NO INTENTIONAL ADDITIONS OF BISMUTH, SELENIUM, TELLURIUM, OR LEAD WERE USED IN THE STEEL USED TO PRODUCE THIS PRODUCT.
THE STEEL WAS MELTED AND MANUFACTURED IN THE U.S.A. AND THE PRODUCT WAS MANUFACTURED AND TESTED IN THE U.S.A. PRODUCT COMPLIES WITH DFARS 252.225-7014. WE CERTIFY THAT THIS DATA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY. THIS CERTIFIED MATERIAL TEST REPORT RELATES ONLY TO THE ITEMS LISTED ON THIS DOCUMENT AND MAY NOT BE REPRODUCED EXCEPT IN FULL.

A2LA
ACCREDITED

MECHANICAL FASTENER
CERTIFICATE NO. A2LA 0139.01
EXPIRATION DATE 12/31/13

NUCOR FASTENER
A DIVISION OF NUCOR CORPORATION

John W. Ferguson

JOHN W. FERGUSON
QUALITY ASSURANCE SUPERVISOR

28265

NUCOR

NUCOR CORPORATION
NUCOR STEEL SOUTH CAROLINA

Mill Certification

5/21/2013

300 Steel Mill Road
DARLINGTON, SC 29540
(843) 393-5841
Fax: (843) 395-8701

Sold To: NUCOR FASTENER INDIANA
PO BOX 6100
ST JOE, IN 46785-0000
(800) 955-6826
Fax: (219) 337-1726

Ship To: NUCOR FASTENER
6730 COUNTY ROAD 60
ST JOE, IN 46785
(800) 955-6826
Fax: (219) 337-1722

Customer P.O.	137238	Sales Order	179051.3
Product Group	Special Bar Quality	Part Number	30001437480V780
Grade	1045L	Lot #	DL1310237401
Size	1-7/16" (1.4375) Round	Heat #	DL13102374
Product	1-7/16" (1.4375) Round 40' 1045L	B.L. Number	C1-606088
Description	1045L	Load Number	C1-287232
Customer Spec		Customer Part #	025018

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

Roll Date: 5/17/2013 Melt Date: 4/23/2013 Qty Shipped LBS: 38,662 Qty Shipped Pcs: 175

C	Mn	V	Si	S	P	Cu	Cr	Ni	Mo	Al	Cb
0.45%	0.65%	0.005%	0.19%	0.009%	0.011%	0.16%	0.09%	0.07%	0.01%	0.002%	0.002%
Pb	Sn	Ca	B	Ti	NICUMO						
0.003%	0.013%	0.0004%	0.0004%	0.001%	0.24						

NICUMO: Cu+Ni+Mo

Reduction Ratio 30 :1

ASTM E381
Surface: 2 Mid Radius: **2** Center: **2**

Specification Comments: CHEMICAL ANALYSIS WAS PERFORMED BY NUCOR NE L.A.B. ACREDITIED CHEMICAL TESTING, CERT L-2232 EXPIRES 12-16-2012 ALL MATERIAL PRODUCED BY NUCOR SC IS EAF MELTED MATERIAL TESTED IN CONFORMANCE WITH ASTM A29-05, AND E415-08

1. WELDING OR WELD REPAIR WAS NOT PERFORMED ON THIS MATERIAL
2. MELTED AND MANUFACTURED IN THE USA
3. MERCURY, RADIUM, OR ALPHA SOURCE MATERIALS IN ANY FORM HAVE NOT BEEN USED IN THE PRODUCTION OF THIS MATERIAL

Chemistry Verification Checks

Part# 25018 **RM#** 28265

	Checked By	Date
Receiving OK:	297	5-29-13
Certifications OK:	375	10-13-13

James H. Blew
Division Metallurgist

NBMG-10 January 1, 2012

FONTANA FASTENERS

PLANT FRANKFORT

TEST REPORT

Operations Center
3281 West County Road 0 NS
Frankfort, IN 46041-6966
T. 765.654.0477
F. 765.654.0857

Ship Date	
Certification	518139*4*1
Report Date	03-29-16

Cust PO	6116857
Lot Nbr	757320
Quantity	90
Mfg Date	09-17-15

Birmingham Fastener, Inc.
Hanceville Dist. Center
1100 Main Street S.E.
HANCEVILLE, AL 35077

PART INFORMATION

Part Number	125C425A49
Description	1 1/4-7 X 4 1/2 F3125 A490-1 HEAVY HEX STRUCTURAL DOUBLE MADE IN USA

Finish	PLAIN
Head Marking	A490 LE USA

RAW MATERIAL ANALYSIS

Steel Heat Nbr	Steel Supplier	Steel Grade	Code	Element	Rqd Min Pct	Rqd Max Pct	Percent
CR10384510	Charter Steel	50B35 SKFG	C	Carbon	0.32	0.37	0.35
			Mn	Manganese	0.95	1.10	1.01
			P	Phosphorus	0.000	0.017	0.007
			S	Sulfur	0.000	0.020	0.008
			Si	Silicon	0.150	0.300	0.250
			Ni	Nickel	0.00	0.10	0.05
			Cr	Chromium	0.50	0.65	0.58
			Mo	Molybdenum	0.10	0.15	0.11
			Cu	Copper	0.00	0.15	0.09
			Sn	Tin	0.000	0.011	0.008
			V	Vanadium	0.000	0.010	0.004
			Al	Aluminum	0.020	0.050	0.027
			N	Nitrogen	0.0000	0.0080	0.0070
			B	Boron	0.0005	0.0030	0.0025
			Ti	Titanium	0.015	0.050	0.021

Certification test results include those reported by the following laboratories:
Charter Steel, A2LA, 01-31-15
Fontana Fasteners, Inc., ISO17025-A2LA Cert#0122.02, 05-31-16

MECHANICAL PROPERTIES

Wedge Angle	6
Proof Load	116300/120000 (lbs/Psi)

Test Performed	Required	High	Low	Average	Samples
Tensile, PSI	150000 / 170000	166000	159000	162222	9
Proof Load Elongation	0.0000 / 0.0005	0.0003	0.0000	0.0000	9
Superficial R30N	53 / 56	55	53	54	9
Core Hardness, HRC	33 / 36	34	34	34	9

FONTANA FASTENERS

PLANT FRANKFORT LE

TEST REPORT

Operations Center
3281 West County Road 0 NS
Frankfort, IN 46041-6966
T. 765.654.0477
F. 765.654.0857

Ship Date	
Certification	518139*4*1
Report Date	03-29-16

Cust PO	6116857
Lot Nbr	757320
Quantity	90
Mfg Date	09-17-15

Birmingham Fastener, Inc.
Hanceville Dist. Center
1100 Main Street S.E.
HANCEVILLE, AL 35077

Applicable Standards, Specifications, and Sampling Schemes:

For Non Structural Product:Test methods in accordance to ASTMF606/ISO 898-1 where applicable.Sampling plan in accordance to ASTM F1470. Thread fit and dimensional properties are compliant to ASME B18.2.1. Mechanical properties are according to SAE J429 or ISO 898-1 were applicable. Bolts passed a surface discontinuity inspection per ASTM F788. These bolts were manufactured in the USA from domestic material and were not produced from heat in which Bismuth, Selenium, Tellurim, or Lead was intentionally added. Bolts were not exposed to Mercury or any other metal alloy that is liquid at ambient temperature during processing or while in our possession.

The listed standards, specifications, and sampling schemes are of the revision in effect on the date of manufacture unless noted otherwise. Only those standards specifically noted under "test methods" or "additional test methods" are included on LE's scope of laboratory accreditation.

Additional Information

None

This lot has been found to conform to the requirements of the above standards and specifications

We certify: The product furnished by Fontana Fasteners, Inc. was manufactured, sampled, tested, and inspected in accordance with the standards and specifications listed above and with Fontana Fasteners, Inc. Quality Manual in effect as of the date of manufacture. The above data accurately represents values provided by Fontana Fasteners, Inc. suppliers and/or values generated in one of Fontana Fasteners, Inc. A2LA accredited laboratories.
This test report relates only to the sample tested above. This document may only be reproduced unaltered and may not be used for any purpose other than the purpose of certifying the same or lesser quantity of the product specified herein. Reproduction, alteration or use of this document for any other purpose is prohibited, except as expressly provided in this certification. Fontana Fasteners, Inc. makes no (and disclaims all) representations, warranties and guarantees whatsoever, whether express, implied or statutory, including, without limitation, any warranty of merchantability or fitness for a particular purpose.

Fontana Fasteners, Inc.

Rick Hall

Rick Hall
Quality Manager

ACCREDITED

CERT #0122.02
"MECHANICAL FIELD OF TESTING"

CHARTER STEEL

A Division of
Charter Manufacturing Company, Inc.

EMAIL

1658 Cold Springs Road
Saukville, Wisconsin 53080
(262) 268-2400
1-800-437-8789
Fax (262) 268-2570

CHARTER STEEL TEST REPORT

Melted in USA Manufactured in USA

Cust P.O.	111346
Customer Part #	951713016C1
Charter Sales Order	10154725
Heat #	10384510
Ship Lot #	4348539
Grade	50B35 M SK FG RHQ 1-3/16
Process	HRSA
Finish Size	1-3/16
Ship date	15-JUN-15

Fontana Fasteners Inc
3595 West State Road 28
Frankfort,IN-46041

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed below and that it satisfies these requirements. The recording of false, fictitious and fraudulent statements or entries on this document may be punishable as a felony under federal statute.

Test results of Heat Lot # 10384510

Lab Code: 7388

CHEM	C	MN	P	S	SI	NI	CR	MO	CU	SN	V
%Wt	.35	1.01	.007	.008	.250	.05	.58	.11	.09	.008	.004
	AL	N	B	TI	NB						
	.027	.0070	.0025	.021	.002						

JOMINY(HRC)

J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J11	J12
55	54	53	53	53	53	53	52	51	51	49	48
J13	J14	J15	J16	J18	J20	J22	J24	J26	J28	J30	J32
47	44	43	42	40	37	36	34	34	33	32	31

JOMINY LAB=0358-01 **JOMINY SAMPLE TYPE ENGLISH=R** **DI=5.18**

Test results of Rolling Lot # 1158289

	# of Tests	Min Value	Max Value	Mean Value
ROD SIZE (Inch)	2	1.185	1.190	1.188
ROD OUT OF ROUND (Inch)	1	.005	.005	.005

REDUCTION RATIO=27:1

Test results of Processing Lot #4348399, 4348539

	# of Tests	Min Value	Max Value	Mean Value	
ROCKWELL B (HRBW)	2	82	82	82	RB LAB = 0358-02

NUM DECARB=2 **FREE FERRITE DECARB (Inch)=.000** **FREE FERR & PARTIAL DECARB (Inch)=.007**
NUM SPHERO=2 **SPHERODIZATION (%)=83.8** **CP SPHERO % LAB=0358-02**

Specifications: **Manufactured per Charter Steel Quality Manual Rev Date 9/12/12**
Meets customer specifications with any applicable Charter Steel exceptions for the following customer documents:
Customer Document = LE 1.1 Revision = 9 Dated = 27-NOV-07

Additional Comments:

Charter Steel
Saukville, WI, USA

Rem: Load1,Fax0,Mail0

ACCREDITED
Testing Laboratory

This MTR supersedes all previously dated MTRs for this order

Janice Barnard

Janice Barnard
Manager of Quality Assurance
Printed Date : 06/15/2015

The following statements are applicable to the material described on the front of this Test Report:

1. Except as noted, the steel supplied for this order was melted, rolled, and processed in the United States meeting DFARS compliance, LEEDS compliance, REACH compliance, ROHS-WEEE compliance, and Conflict Materials Restrictions.
2. Mercury was not used during the manufacture of this product, nor was the steel contaminated with mercury during processing.
3. Unless directed by the customer, there are no welds in any of the coils produced for this order.
4. The laboratory that generated the analytical or test results can be identified by the following key:

Certificate Number	Lab Code	Laboratory		Address
0358-01	7388	**CSSM**	Charter Steel Melting Division	1658 Cold Springs Road, Saukville, WI 53080
0358-02	8171	**CSSR/ CSSP**	Charter Steel Rolling/ Processing Division	1658 Cold Springs Road, Saukville, WI 53080
0358-03	123633	**CSFP**	Charter Steel Ohio Processing Division	6255 US Highway 23, Rising Sun, OH 43457
0358-04	125544	**CSCM/ CSCR**	Charter Steel Cleveland	4300 E. 49th St., Cuyahoga Heights, OH 44125-1004
*	*	--	Subcontracted test performed by laboratory not in Charter Steel System	

5. When run by a Charter Steel laboratory, the following tests were performed according to the latest revisions of the specifications listed below, as noted in the Charter Steel Laboratory Quality Manual:

Test	Specifications	CSSM	CSSR/ CSSP	CSFP	CSCM/ CSCR
Chemistry Analysis	ASTM E415; ASTM E1019	X			X
Macroetch	ASTM E381	X			X
Hardenability (Jominy)	ASTM A255; SAE J406; JIS G0561	X			X
Grain Size	ASTM E112	X	X	X	X
Tensile Test	ASTM E8; ASTM A370		X	X	X
Rockwelll Hardness	ASTM E18; ASTM A370	X	X	X	X
Microstructure (spheroidization)	ASTM A892		X	X	
Inclusion Content (Methods A, E)	ASTM E45		X		X
Decarburization	ASTM E1077		X	X	X

Charter Steel has been accredited to perform all of the above tests by the American Association for Laboratory Accreditation (A2LA). These accreditations expire 03/31/17. All other test results associated with a Charter Steel laboratory that appear on the front of this report, if any, were performed according to documented procedures developed by Charter Steel and are not accredited by A2LA.

6. The test results on the front of this report are the true values measured on the samples taken from the production lot. They do not apply to any other sample.
7. This test report cannot be reproduced or distributed except in full without the written permission of Charter Steel. The primary customer whose name and address appear on the front of this form may reproduce this test report subject to the following restrictions:
   - It may be distributed only to their customers
   - Both sides of all pages must be reproduced in full
8. This certification is given subject to the terms and conditions of sale provided in Charter Steel's acknowledgement (designated by our Sales Order number) to the customer's purchase order. Both order numbers appear on the front page of this Report.
9. Where the customer has provided a specification, the results on the front of this test report conform to that specification unless otherwise noted on this test report.

ACCREDITED
Testing Laboratory

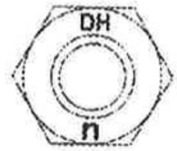
NUCOR

FASTENER DIVISION

LOT NO.
359872A

Post Office Box 6100
Saint Joe, Indiana 46785
Telephone 260/337-1600

CUSTOMER NO/NAME
9000 BIRMINGHAM-CONS/SHIPPING — NUCOR ORDER # 936595
TEST REPORT SERIAL# FB461408 — CUST PART # 125CNDH/D
TEST REPORT ISSUE DATE 5/29/15
DATE SHIPPED 8/18/15 — CUSTOMER P.O. # 6089912
NAME OF LAB SAMPLER: RYAN UNGER, LAB TECHNICIAN



****************CERTIFIED MATERIAL TEST REPORT****************

NUCOR PART NO	QUANTITY	LOT NO.	DESCRIPTION
175410	5400	359872A	1 1/4-7 GR DH HV HEX NUT
MANUFACTURE DATE 5/13/15			HEX NUT BLACK

--CHEMISTRY — MATERIAL GRADE -1045L

MATERIAL NUMBER	HEAT NUMBER	C	MN	P	S	SI	
		**CHEMISTRY COMPOSITION (WT% HEAT ANALYSIS) BY MATERIAL SUPPLIER					NUCOR STEEL - SOUTH CAROL
RM029955	DL15102171	.44	.65	.005	.016	.20	

--MECHANICAL PROPERTIES IN ACCORDANCE WITH ASTM A563-07a

SURFACE HARDNESS (R30N)	CORE HARDNESS (RC)	PROOF LOAD 169600 LBS	TENSILE STRENGTH DEG-WEDGE (LBS)	STRESS (PSI)
N/A	28.2	PASS	N/A	N/A
N/A	27.6	PASS	N/A	N/A
N/A	28.9	PASS	N/A	N/A
AVERAGE VALUES FROM TESTS	28.2			

PRODUCTION LOT SIZE 12200 PCS

--VISUAL INSPECTION IN ACCORDANCE WITH ASTM A563-07a — 50 PCS. SAMPLED — LOT PASSED
HEAT TREATMENT - AUSTENITIZED, OIL QUENCHED & TEMPERED (MIN 800 DEG F)

--DIMENSIONS PER ASME B18.2.6-2012

CHARACTERISTIC	#SAMPLES TESTED	MINIMUM	MAXIMUM
Width Across Corners	8	2.233	2.240
Thickness	32	1.209	1.233

ALL TESTS ARE IN ACCORDANCE WITH THE LATEST REVISIONS OF THE METHODS PRESCRIBED IN THE APPLICABLE SAE AND ASTM SPECIFICATIONS. THE SAMPLES TESTED CONFORM TO THE SPECIFICATIONS AS DESCRIBED/LISTED ABOVE AND WERE MANUFACTURED FREE OF MERCURY CONTAMINATION. NO INTENTIONAL ADDITIONS OF BISMUTH, SELENIUM, TELLURIUM, OR LEAD WERE USED IN THE STEEL USED TO PRODUCE THIS PRODUCT.
THE STEEL WAS MELTED AND MANUFACTURED IN THE U.S.A. AND THE PRODUCT WAS MANUFACTURED AND TESTED IN THE U.S.A. PRODUCT COMPLIES WITH DFARS 252.225-7014. WE CERTIFY THAT THIS DATA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY. THIS CERTIFIED MATERIAL TEST REPORT RELATES ONLY TO THE ITEMS LISTED ON THIS DOCUMENT AND MAY NOT BE REPRODUCED EXCEPT IN FULL.

A2LA ACCREDITED

MECHANICAL FASTENER
CERTIFICATE NO. A2LA 0139.01
EXPIRATION DATE 12/31/15

NUCOR FASTENER
A DIVISION OF NUCOR CORPORATION

John W. Ferguson

JOHN W. FERGUSON
QUALITY ASSURANCE SUPERVISOR

Page 1 of 1

29955

NUCOR

NUCOR CORPORATION
NUCOR STEEL SOUTH CAROLINA

Mill Certification
4/19/2015

MTR #: 0000071920
300 Steel Mill Road
DARLINGTON, SC 29540
(843) 393-5841
Fax: (843) 395-8701

Sold To: NUCOR FASTENER INDIANA
PO BOX 6100
ST JOE, IN 46785-0000
(800) 955-6826
Fax: (219) 337-1726

Ship To: NUCOR FASTENER
6730 COUNTY ROAD 60
ST JOE, IN 46785
(800) 955-6826
Fax: (219) 337-1722

Customer P.O.	151396	Sales Order	221925.4
Product Group	Special Bar Quality	Part Number	30001562480V780
Grade	1045L	Lot #	DL1510217101
Size	1-9/16" (1.5625) Round	Heat #	DL15102171
Product	1-9/16" (1.5625) Round 40' 1045L	B.L. Number	C1-660170
Description	1045L	Load Number	C1-339493
Customer Spec		Customer Part #	025020

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

Roll Date: 4/14/2015 Melt Date: 4/10/2015 Qty Shipped LBS: 156,600 Qty Shipped Pcs: 600

Melt Date: 4/10/2015

C	Mn	V	Si	S	P	Cu	Cr	Ni	Mo	Al	Cb
0.44%	0.65%	0.003%	0.20%	0.016%	0.005%	0.15%	0.08%	0.06%	0.01%	0.003%	0.004%
Pb	Sn	Ca	B	Ti	NICUMO						
0.004%	0.009%	0.0018%	0.0004%	0.001%	0.22						

NICUMO: Cu+Ni+Mo

Roll Date: 4/14/2015

Reduction Ratio 25 :1

ASTM E381
Surface: 2 Mid Radius: 2 Center: 2

Specification Comments:

1. WELDING OR WELD REPAIR WAS NOT PERFORMED ON THIS MATERIAL
2. MELTED AND MANUFACTURED IN THE USA
3. MERCURY, RADIUM, OR ALPHA SOURCE MATERIALS IN ANY FORM HAVE NOT BEEN USED IN THE PRODUCTION OF THIS MATERIAL

Chemistry Verification Checks

Part# 25020 RM# 29955

	Checked By	Date
Receiving OK:	297	4-30-15
Certifications OK:	375	4-30-15

James H. Blew
Division Metallurgist

NBMG-10 January 1, 2012

FONTANA FASTENERS
PLANT FRANKFORT

TEST REPORT

Operations Center
3281 West County Road 0 NS
Frankfort, IN 46041-6966
T. 765.654.0477
F. 765.654.0857

Ship Date	
Certification	518139'4'1
Report Date	03-29-16

Cust PO	6116857
Lot Nbr	759070
Quantity	720
Mfg Date	10-29-15

Birmingham Fastener, Inc.
Hanceville Dist. Center
1100 Main Street S.E.
HANCEVILLE, AL 35077

PART INFORMATION

Part Number	125C425A49	Finish	PLAIN
Description	1 1/4-7 X 4 1/2 F3125 A490-1 HEAVY HEX STRUCTURAL DOUBLE MADE IN USA	Head Marking	A490 LE USA

RAW MATERIAL ANALYSIS

Steel Heat Nbr	Steel Supplier	Steel Grade	Code	Element	Rqd Min Pct	Rqd Max Pct	Percent
CR10392100	Charter Steel	50B35 SKFG	C	Carbon	0.32	0.37	0.36
			Mn	Manganese	0.95	1.10	1.03
			P	Phosphorus	0.000	0.017	0.009
			S	Sulfur	0.000	0.020	0.012
			Si	Silicon	0.150	0.300	0.250
			Ni	Nickel	0.00	0.10	0.05
			Cr	Chromium	0.50	0.65	0.57
			Mo	Molybdenum	0.10	0.15	0.11
			Cu	Copper	0.00	0.15	0.09
			Sn	Tin	0.000	0.011	0.007
			V	Vanadium	0.000	0.010	0.004
			Al	Aluminum	0.020	0.050	0.025
			N	Nitrogen	0.0000	0.0080	0.0060
			B	Boron	0.0005	0.0030	0.0027
			Ti	Titanium	0.015	0.050	0.021

Certification test results include those reported by the following laboratories:
Charter Steel, A2LA, 01-31-15
Fontana Fasteners, Inc., ISO17025-A2LA Cert#0122.02, 05-31-16

MECHANICAL PROPERTIES

Wedge Angle	6
Proof Load	116300/120000 (lbs/Psi)

Test Performed	Required	High	Low	Average	Samples
Tensile, PSI	150000 / 170000	152000	151000	151500	6
Proof Load Elongation	0.0000 / 0.0005	0.0003	0.0001	0.0001	6
Superficial R30N	53 / 56	54	53	54	6
Core Hardness, HRC	33 / 36	34	34	34	6

FONTANA FASTENERS

FRANKFORT

TEST REPORT

Operations Center
3281 West County Road 0 NS
Frankfort, IN 46041-6966
T. 765.654.0477
F. 765.654.0867

Ship Date	
Certification	518139*4*1
Report Date	03-29-16

Cust PO	6116857
Lot Nbr.	759070
Quantity	720
Mfg Date	10-29-15

Birmingham Fastener, Inc.
Hanceville Dist. Center
1100 Main Street S.E.
HANCEVILLE, AL 35077

Applicable Standards, Specifications, and Sampling Schemes

For Non Structural Product:Test methods in accordance to ASTMF606/ISO 898-1 where applicable.Sampling plan in accordance to ASTM F1470. Thread fit and dimensional properties are compliant to ASME B18.2.1, Mechanical properties are according to SAE J429 or ISO 898-1 were applicable. Bolts passed a surface discontinuity inspection per ASTM F788. These bolts were manufactured in the USA from domestic material and were not produced from heat in which Bismuth, Selenium, Tellurim, or Lead was intentionally added. Bolts were not exposed to Mercury or any other metal alloy that is liquid at ambient temperature during processing or while in our possession.

The listed standards, specifications, and sampling schemes are of the revision in effect on the date of manufacture unless noted otherwise. Only those standards specifically noted under "test methods" or "additional test methods" are included on LE's scope of laboratory accreditation.

Additional Information

None

This lot has been found to conform to the requirements of the above standards and specifications

We certify: The product furnished by Fontana Fasteners, Inc. was manufactured, sampled, tested, and inspected in accordance with the standards and specifications listed above and with Fontana Fasteners, Inc. Quality Manual in effect as of the date of manufacture. The above data accurately represents values provided by Fontana Fasteners, Inc. suppliers and/or values generated in one of Fontana Fasteners, Inc. A2LA accredited laboratories.
This test report relates only to the sample listed above. This document may only be reproduced unaltered and may not be used for any purpose other than the purpose of certifying the same or lesser quantity of the product specified herein. Reproduction, alteration or use of this document for any other purpose is prohibited, except as expressly provided in this certification. Fontana Fasteners, Inc. makes no (and disclaims all) representations, warranties and guarantees whatsoever, whether express, implied or statutory, including without limitation, any warranty of merchantability or fitness for a particular purpose

Fontana Fasteners, Inc.

Rick Hall
Quality Manager

ACCREDITED CERT #0122.02
"MECHANICAL FIELD OF TESTING"

CHARTER STEEL
A Division of Charter Manufacturing Company, Inc.

EMAIL

1658 Cold Springs Road
Saukville, Wisconsin 53080
(262) 268-2400
1-800-437-8789
Fax (262) 268-2570

CHARTER STEEL TEST REPORT

Melted in USA Manufactured In USA

Fontana Fasteners Inc
3595 West State Road 28
Frankfort,IN-46041

Cust P.O.	111345
Customer Part #	951712937C1
Charter Sales Order	10161339
Heat #	10392100
Ship Lot #	4361769
Grade	50B35 M SK FG RHQ 1-5/32
Process	HRSA
Finish Size	1-5/32
Ship date	02-SEP-15

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed below and that it satisfies these requirements. The recording of false, fictitious and fraudulent statements or entries on this document may be punishable as a felony under federal statute.

Test results of Heat Lot # 10392100

Lab Code: 7388

CHEM	C	MN	P	S	SI	NI	CR	MO	CU	SN	V
%Wt	.36	1.03	.009	.012	.250	.05	.57	.11	.09	.007	.004
	AL	N	B	TI	NB	SB	AS				
	.025	.0060	.0027	.021	.002	.003	.005				

JOMINY(HRC)

J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J11	J12
54	53	53	52	52	52	52	52	51	49	49	48
J13	J14	J15	J16	J18	J20	J22	J24	J26	J28	J30	J32
47	45	44	43	40	39	38	37	35	35	34	33

JOMINY LAB=0358-01 JOMINY SAMPLE TYPE ENGLISH=R DI=5.28

Test results of Rolling Lot # 1161902

	# of Tests	Min Value	Max Value	Mean Value
ROD SIZE (Inch)	4	1.153	1.162	1.157
ROD OUT OF ROUND (Inch)	2	.008	.009	.009

REDUCTION RATIO=29:1

Test results of Processing Lot #4359895, 4361769

	# of Tests	Min Value	Max Value	Mean Value	
ROCKWELL B (HRBW)	1	79	79	79	RB LAB = 0358-02

NUM DECARB=1
NUM SPHERO=1

FREE FERRITE DECARB (Inch)=.000
SPHERODIZATION (%)=85.0

FREE FERR & PARTIAL DECARB (Inch)=.005
CP SPHERO % LAB=0358-02

Specifications: Manufactured per Charter Steel Quality Manual Rev Date 9/12/12
Meets customer specifications with any applicable Charter Steel exceptions for the following customer documents:
Customer Document = LE 1.1 Revision = 9 Dated = 27-NOV-07

Additional Comments:

Melt Source:
Charter Steel
Saukville, WI, USA

Rem: Load1,Fax0,Mail0

ACCREDITED
Testing Laboratory

This MTR supersedes all previously dated MTRs for this order

Janice Barnard
Manager of Quality Assurance
Printed Date 09/02/2015

The following statements are applicable to the material described on the front of this Test Report:

1. Except as noted, the steel supplied for this order was melted, rolled, and processed in the United States meeting DFARS compliance, LEEDS compliance, REACH compliance, ROHS-WEEE compliance, and Conflict Materials Restrictions.
2. Mercury was not used during the manufacture of this product, nor was the steel contaminated with mercury during processing.
3. Unless directed by the customer, there are no welds in any of the coils produced for this order.
4. The laboratory that generated the analytical or test results can be identified by the following key:

Certificate Number	Lab Code	Laboratory		Address
0358-01	7388	CSSM	Charter Steel Melting Division	1658 Cold Springs Road, Saukville, WI 53080
0358-02	8171	CSSR/ CSSP	Charter Steel Rolling/ Processing Division	1658 Cold Springs Road, Saukville, WI 53080
0358-03	123633	CSFP	Charter Steel Ohio Processing Division	6255 US Highway 23, Rising Sun, OH 43457
0358-04	125544	CSCM/ CSCR	Charter Steel Cleveland	4300 E. 49th St., Cuyahoga Heights, OH 44125-1004
*	*	--	Subcontracted test performed by laboratory not in Charter Steel System	

5. When run by a Charter Steel laboratory, the following tests were performed according to the latest revisions of the specifications listed below, as noted in the Charter Steel Laboratory Quality Manual:

Test	Specifications	CSSM	CSSR/ CSSP	CSFP	CSCM/ CSCR
Chemistry Analysis	ASTM E415; ASTM E1019	X			X
Macroetch	ASTM E381	X			X
Hardenability (Jominy)	ASTM A255; SAE J406; JIS G0561	X			X
Grain Size	ASTM E112	X	X	X	X
Tensile Test	ASTM E8; ASTM A370		X	X	X
Rockwelll Hardness	ASTM E18; ASTM A370	X	X	X	X
Microstructure (spheroidization)	ASTM A892		X	X	
Inclusion Content (Methods A, E)	ASTM E45		X		X
Decarburization	ASTM E1077		X	X	X

Charter Steel has been accredited to perform all of the above tests by the American Association for Laboratory Accreditation (A2LA). These accreditations expire 03/31/17. All other test results associated with a Charter Steel laboratory that appear on the front of this report, if any, were performed according to documented procedures developed by Charter Steel and are not accredited by A2LA.

6. The test results on the front of this report are the true values measured on the samples taken from the production lot. They do not apply to any other sample.
7. This test report cannot be reproduced or distributed except in full without the written permission of Charter Steel. The primary customer whose name and address appear on the front of this form may reproduce this test report subject to the following restrictions:
   - It may be distributed only to their customers
   - Both sides of all pages must be reproduced in full
8. This certification is given subject to the terms and conditions of sale provided in Charter Steel's acknowledgement (designated by our Sales Order number) to the customer's purchase order. Both order numbers appear on the front page of this Report.
9. Where the customer has provided a specification, the results on the front of this test report conform to that specification unless otherwise noted on this test report.

A2LA ACCREDITED
Testing Laboratory

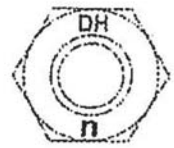
NUCOR

FASTENER DIVISION

LOT NO.
359467A

Post Office Box 6100
Saint Joe, Indiana 46785
Telephone 260/337-1600

CUSTOMER NO/NAME
9000 BIRMINGHAM-CONS/SHIPPING — NUCOR ORDER # 936595
TEST REPORT SERIAL# FB459583 — CUST PART # 125CNDH/D
TEST REPORT ISSUE DATE 5/08/15
DATE SHIPPED 8/18/15 — CUSTOMER P.O. # 6089912
NAME OF LAB SAMPLER: JOSEPH BYERLY, LAB TECHNICIAN



****************CERTIFIED MATERIAL TEST REPORT****************

NUCOR PART NO	QUANTITY	LOT NO.	DESCRIPTION
175410	19130	359467A	1 1/4-7 GR DH HV HEX NUT
MANUFACTURE DATE 4/14/15			HEX NUT BLACK

--CHEMISTRY — MATERIAL GRADE -1045L

**CHEMISTRY COMPOSITION (WT% HEAT ANALYSIS) BY MATERIAL SUPPLIER — NUCOR STEEL - SOUTH CAROL

MATERIAL NUMBER	HEAT NUMBER	C	MN	P	S	SI
RM029704	DL15100152	.47	.71	.011	.023	.18

--MECHANICAL PROPERTIES IN ACCORDANCE WITH ASTM A563-07a

SURFACE HARDNESS (R30N)	CORE HARDNESS (RC)	PROOF LOAD 169600 LBS	TENSILE STRENGTH DEG-WEDGE (LBS)	STRESS (PSI)
N/A	29.6	PASS	N/A	N/A
N/A	28.8	PASS	N/A	N/A
N/A	27.8	PASS	N/A	N/A
N/A	29.0	PASS	N/A	N/A
N/A	29.6	PASS	N/A	N/A
AVERAGE VALUES FROM TESTS	29.0			

PRODUCTION LOT SIZE 31900 PCS

--VISUAL INSPECTION IN ACCORDANCE WITH ASTM A563-07a — 50 PCS. SAMPLED — LOT PASSED
HEAT TREATMENT - AUSTENITIZED, OIL QUENCHED & TEMPERED (MIN 800 DEG F)

--DIMENSIONS PER ASME B18.2.6-2012

CHARACTERISTIC	#SAMPLES TESTED	MINIMUM	MAXIMUM
Width Across Corners	8	2.233	2.243
Thickness	32	1.211	1.245

ALL TESTS ARE IN ACCORDANCE WITH THE LATEST REVISIONS OF THE METHODS PRESCRIBED IN THE APPLICABLE SAE AND ASTM SPECIFICATIONS. THE SAMPLES TESTED CONFORM TO THE SPECIFICATIONS AS DESCRIBED/LISTED ABOVE AND WERE MANUFACTURED FREE OF MERCURY CONTAMINATION. NO INTENTIONAL ADDITIONS OF BISMUTH, SELENIUM, TELLURIUM, OR LEAD WERE USED IN THE STEEL USED TO PRODUCE THIS PRODUCT.
THE STEEL WAS MELTED AND MANUFACTURED IN THE U.S.A. AND THE PRODUCT WAS MANUFACTURED AND TESTED IN THE U.S.A. PRODUCT COMPLIES WITH DFARS 252.225-7014. WE CERTIFY THAT THIS DATA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY. THIS CERTIFIED MATERIAL TEST REPORT RELATES ONLY TO THE ITEMS LISTED ON THIS DOCUMENT AND MAY NOT BE REPRODUCED EXCEPT IN FULL.

A2LA ACCREDITED

MECHANICAL FASTENER
CERTIFICATE NO. A2LA 0139.01
EXPIRATION DATE 12/31/15

NUCOR FASTENER
A DIVISION OF NUCOR CORPORATION

John W. Ferguson

JOHN W. FERGUSON
QUALITY ASSURANCE SUPERVISOR

29704

NUCOR

NUCOR CORPORATION
NUCOR STEEL SOUTH CAROLINA

Mill Certification
1/20/2015

MTR #: 0000058445
300 Steel Mill Road
DARLINGTON, SC 29540
(843) 393-5841
Fax: (843) 395-8701

Sold To: NUCOR FASTENER INDIANA
PO BOX 6100
ST JOE, IN 46785-0000
(800) 955-6826
Fax: (219) 337-1726

Ship To: NUCOR FASTENER
6730 COUNTY ROAD 60
ST JOE, IN 46785
(800) 955-6826
Fax: (219) 337-1722

Customer P.O.	149739	Sales Order	215661.5
Product Group	Special Bar Quality	Part Number	30001562460V760
Grade	1045L	Lot #	DL1510015201
Size	1-9/16" (1.5625) Round	Heat #	DL15100152
Product	1-9/16" (1.5625) Round 40' 1045L	B.L. Number	C1-652992
Description	1045L	Load Number	C1-332493
Customer Spec		Customer Part #	025020

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

Roll Date: 1/12/2015 Melt Date: 1/6/2015 Qty Shipped LBS: 178,524 Qty Shipped Pcs: 684

Melt Date: 1/6/2015

C	Mn	V	Si	S	P	Cu	Cr	Ni	Mo	Al	Cb
0.47%	0.71%	0.002%	0.18%	0.023%	0.011%	0.13%	0.05%	0.04%	0.01%	0.001%	0.003%

Pb	Sn	Ca	B	Ti	NICUMO
0.003%	0.006%	0.0014%	0.0004%	0.001%	0.18

NICUMO: Cu+Ni+Mo

Roll Date: 1/12/2015

Reduction Ratio 25 :1

ASTM E381
Surface: 2 Mid Radius: 2 Center: 2

Specification Comments:

1. WELDING OR WELD REPAIR WAS NOT PERFORMED ON THIS MATERIAL
2. MELTED AND MANUFACTURED IN THE USA
3. MERCURY, RADIUM, OR ALPHA SOURCE MATERIALS IN ANY FORM HAVE NOT BEEN USED IN THE PRODUCTION OF THIS MATERIAL

Chemistry Verification Checks

Part# 25020 RM# 29704

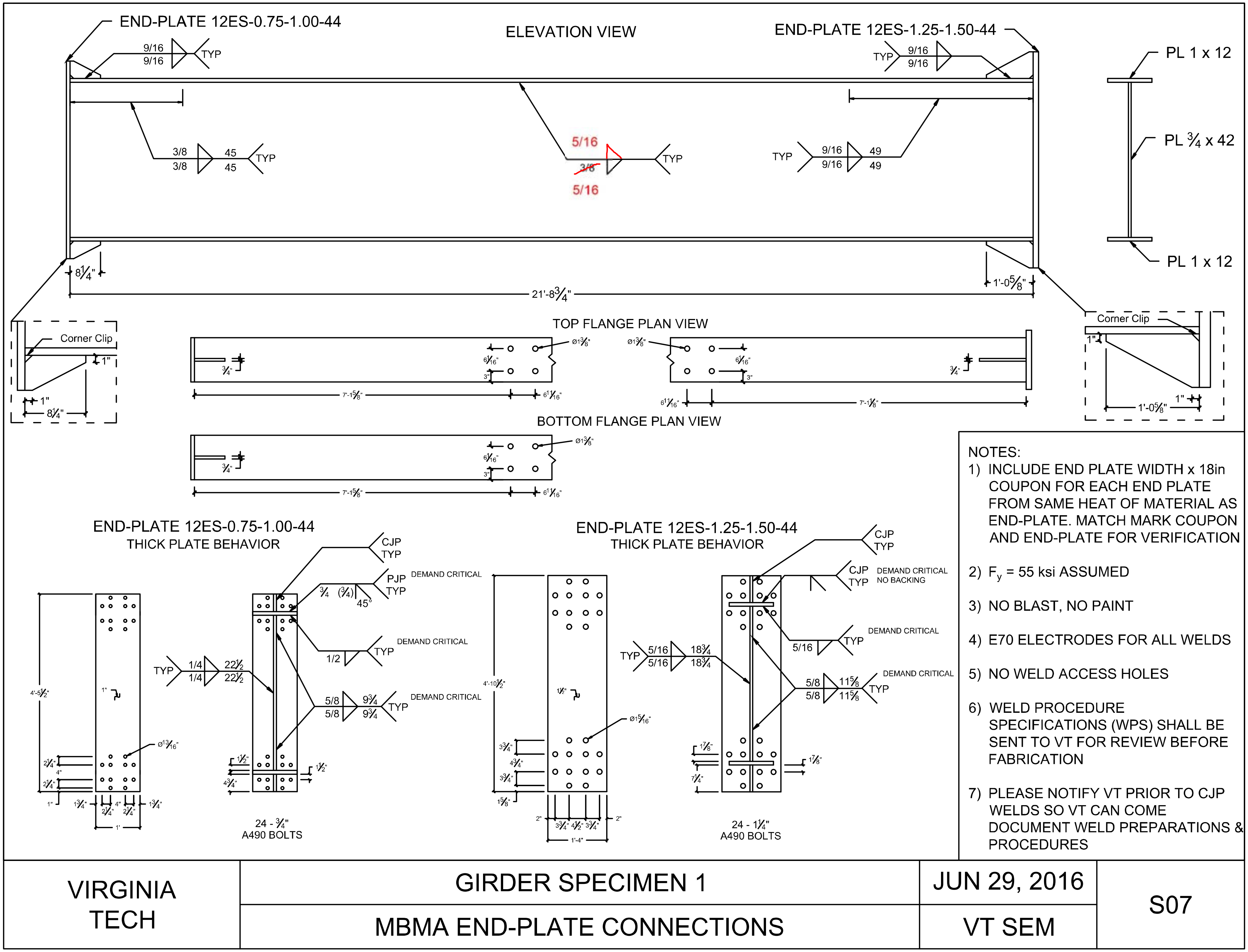
	Checked By	Date
Receiving OK:	297	2-9-15
Certifications OK:	375	2-9-15

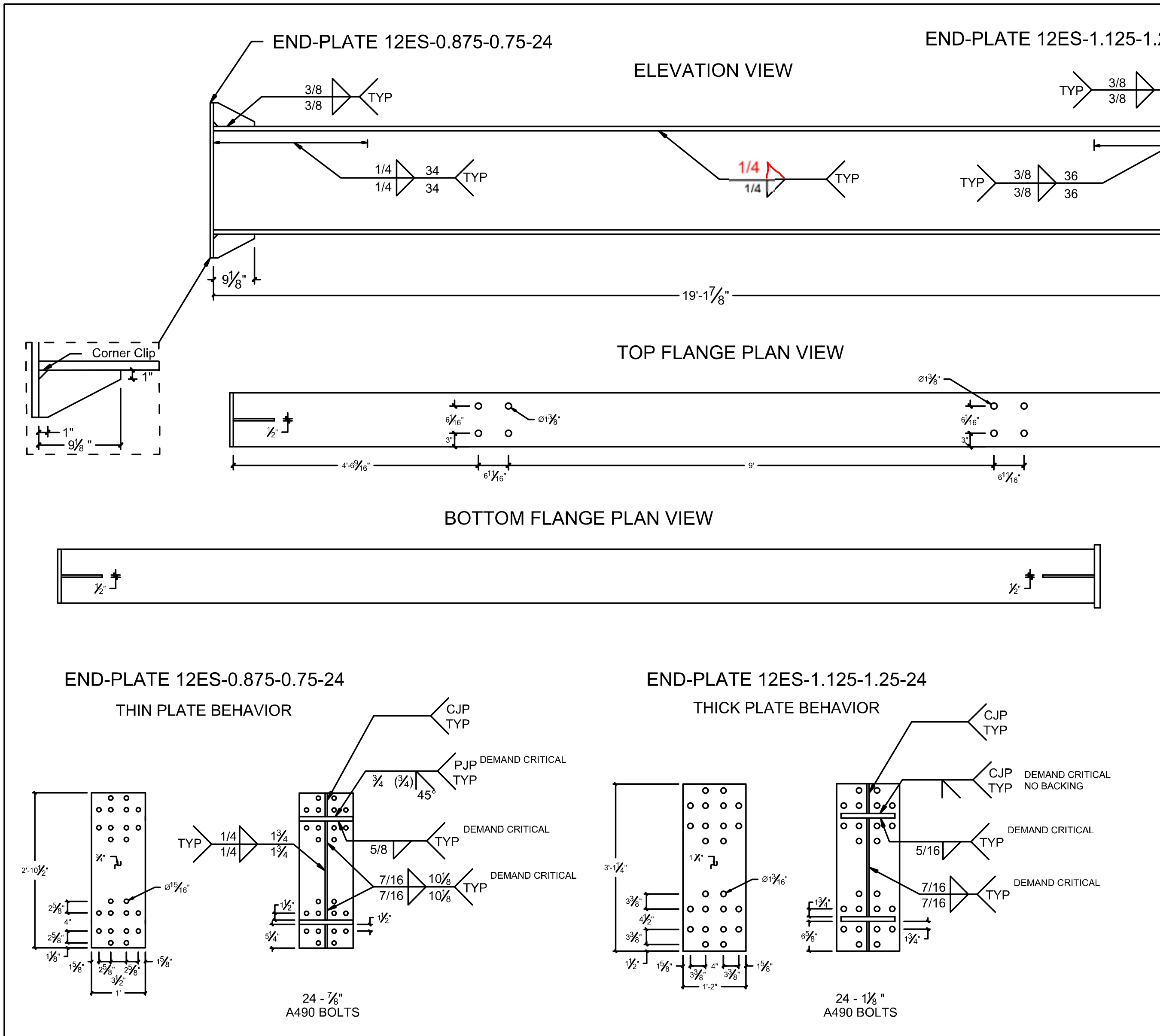
James H. Blew
Division Metallurgist

Page 1 of 1

NSMG-10 January 1, 2012

APPENDIX D FABRICATION DRAWINGS

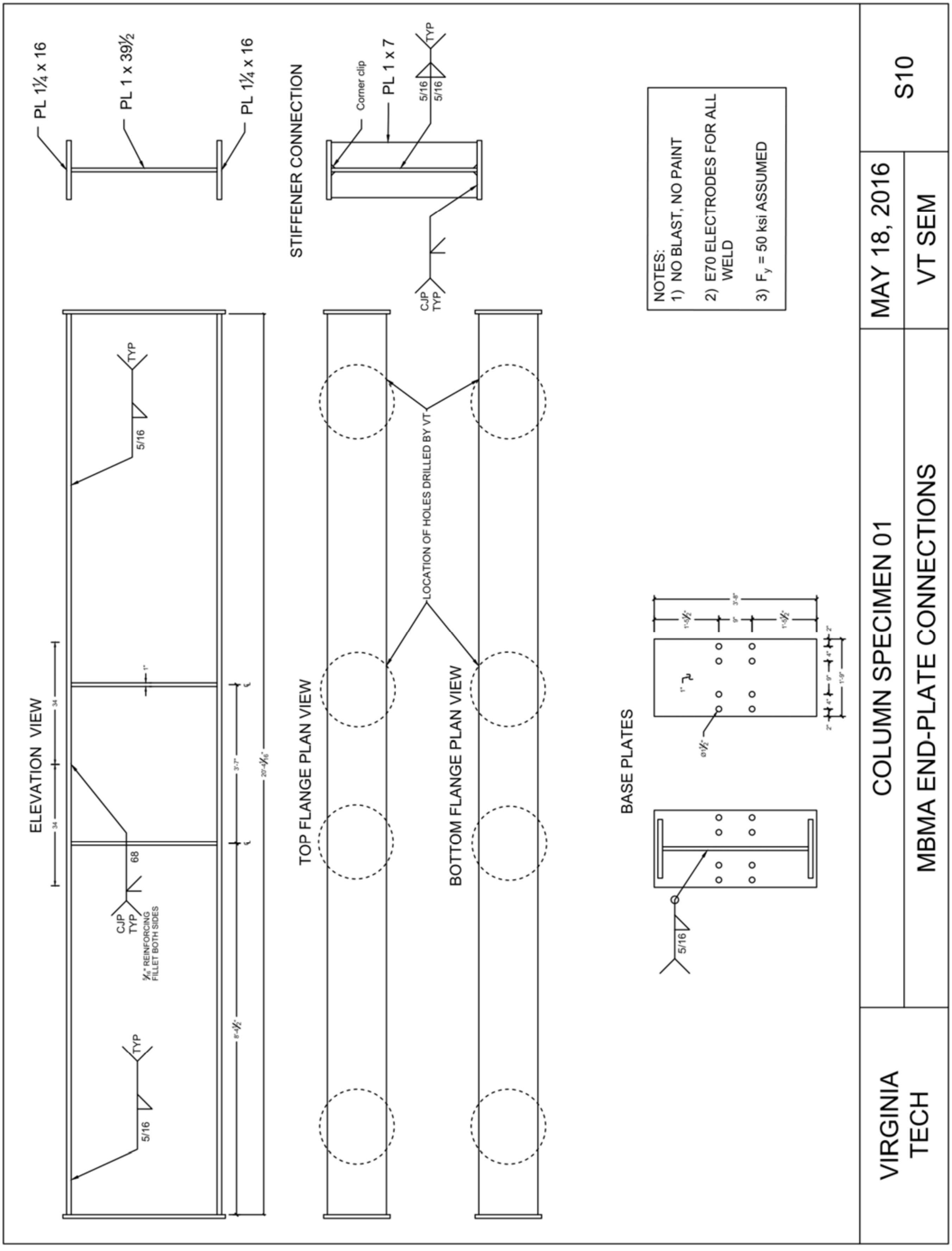

END-PLATE 12ES-0.75-1.00-44
ELEVATION VIEW
END-PLATE 12ES-1.25-1.50-44
PL 1 x 12
PL ¾ x 42
PL 1 x 12
21'-8¾"
8¼"
1'-0⅝"
Corner Clip
TOP FLANGE PLAN VIEW
BOTTOM FLANGE PLAN VIEW
END-PLATE 12ES-0.75-1.00-44
THICK PLATE BEHAVIOR
24 - ¾" A490 BOLTS
END-PLATE 12ES-1.25-1.50-44
THICK PLATE BEHAVIOR
24 - 1¼" A490 BOLTS
DEMAND CRITICAL
DEMAND CRITICAL NO BACKING
CJP TYP
PJP TYP
NOTES:
1) INCLUDE END PLATE WIDTH x 18in COUPON FOR EACH END PLATE FROM SAME HEAT OF MATERIAL AS END-PLATE. MATCH MARK COUPON AND END-PLATE FOR VERIFICATION
2) F_y = 55 ksi ASSUMED
3) NO BLAST, NO PAINT
4) E70 ELECTRODES FOR ALL WELDS
5) NO WELD ACCESS HOLES
6) WELD PROCEDURE SPECIFICATIONS (WPS) SHALL BE SENT TO VT FOR REVIEW BEFORE FABRICATION
7) PLEASE NOTIFY VT PRIOR TO CJP WELDS SO VT CAN COME DOCUMENT WELD PREPARATIONS & PROCEDURES
VIRGINIA TECH
GIRDER SPECIMEN 1
MBMA END-PLATE CONNECTIONS
JUN 29, 2016
VT SEM
S07

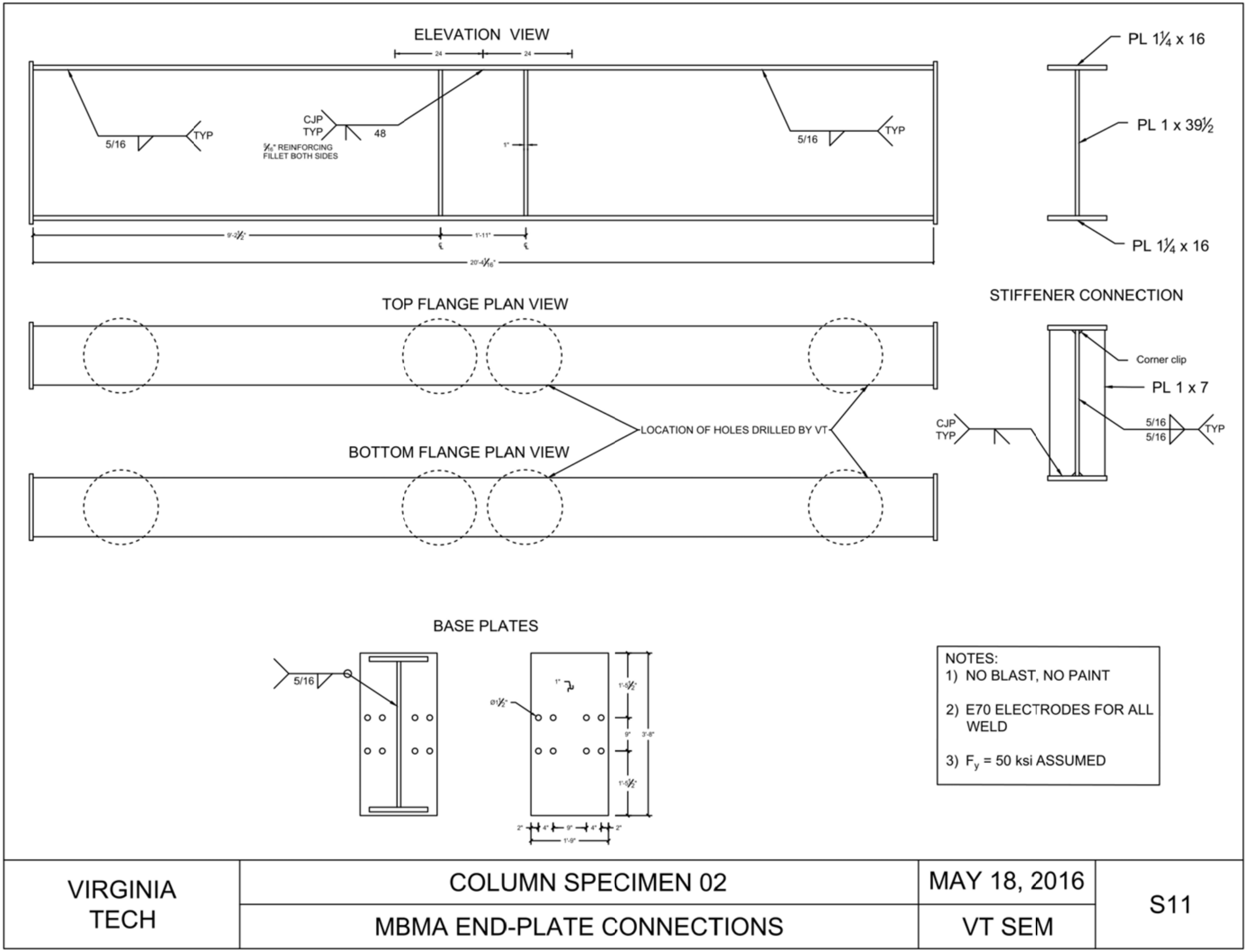


NOTES:

1) INCLUDE END PLATE WIDTH x 18in COUPON FOR EACH END PLATE FROM SAME HEAT OF MATERIAL AS END-PLATE. MATCH MARK COUPON AND END-PLATE FOR VERIFICATION
2) F_y = 55 ksi ASSUMED
3) NO BLAST, NO PAINT
4) E70 ELECTRODES FOR ALL WELDS
5) NO WELD ACCESS HOLES
6) WELD PROCEDURE SPECIFICATIONS (WPS) SHALL BE SENT TO VT FOR REVIEW BEFORE FABRICATION
7) PLEASE NOTIFY VT PRIOR TO CJP WELDS SO VT CAN COME DOCUMENT WELD PREPARATIONS & PROCEDURES

VIRGINIA TECH	GIRDER SPECIMEN 2	JUN 29, 2016	S08
	MBMA END-PLATE CONNECTIONS	VT SEM	


PL 1¼ x 16
PL 1 x 39½
PL 1¼ x 16
STIFFENER CONNECTION
Corner clip
PL 1 x 7
5/16
5/16
TYP
CJP
TYP
NOTES:
1) NO BLAST, NO PAINT
2) E70 ELECTRODES FOR ALL WELD
3) F_y = 50 ksi ASSUMED
ELEVATION VIEW
34
34
1"
3'-7"
20'-4 1/16"
8'-4½"
5/16
TYP
68
CJP
TYP
5/16" REINFORCING FILLET BOTH SIDES
TOP FLANGE PLAN VIEW
BOTTOM FLANGE PLAN VIEW
LOCATION OF HOLES DRILLED BY VT
BASE PLATES
Ø1½"
5/16
VIRGINIA TECH
COLUMN SPECIMEN 01
MBMA END-PLATE CONNECTIONS
MAY 18, 2016
VT SEM
S10


ELEVATION VIEW
24
24
CJP
TYP
48
1/8" REINFORCING FILLET BOTH SIDES
5/16
TYP
1"
9'-3½"
1'-11"
20'-4⅟16"
PL 1¼ x 16
PL 1 x 39½
PL 1¼ x 16
TOP FLANGE PLAN VIEW
BOTTOM FLANGE PLAN VIEW
LOCATION OF HOLES DRILLED BY VT
STIFFENER CONNECTION
Corner clip
PL 1 x 7
CJP
TYP
5/16
5/16
TYP
BASE PLATES
5/16
Ø1½"
1"
1'-5½"
9"
3'-8"
1'-5½"
2"
4"
9"
4"
2"
1'-9"
NOTES:
1) NO BLAST, NO PAINT
2) E70 ELECTRODES FOR ALL WELD
3) Fy = 50 ksi ASSUMED
VIRGINIA TECH
COLUMN SPECIMEN 02
MBMA END-PLATE CONNECTIONS
MAY 18, 2016
VT SEM
S11

APPENDIX E WELDING PROCEDURE SPECIFICATIONS FOR BEAM SPECIMENS

BLUESCOPE BUILDINGS

BlueScope Buildings North America, Inc.
Welding Procedure Data Sheet

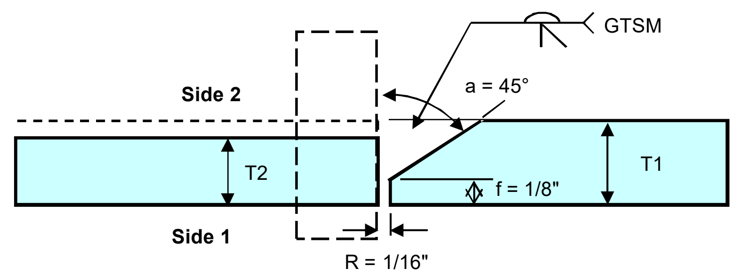
WPS No: G-BTC-SBG-2b	Revision 0	Date: 13-Mar-2015	Ref Stds: W47.1-2009 / W59-2013 / D1.1-2010

Prepared By: Skip Hyder / Tim Brown — **Type:** Semi-Auto
Welding Process(es) **GMAW** — WPS Ref #WS-01
Prequalified per: D1.1 Fig 3.4; B-U4b-GF & TC-U4b-GF and W59 Fig 10.7; B-U4a-G & TC-U4a-G

JOINT

Type: **Butt, T, or Corner Joint**

Backing: No — Double-Side Weld
Penetration: CJP - Complete Joint Penetration
Back Gouge: Yes
Method: Air Carbon Arc
Thickness ETT (in): 3/8" to Unlimited



BASE METALS

AWS Steel: Group II to Group II
W59 Steel: Group 3 to Group 3

POSITION

D1.1 Position of Groove: F / H / V (up)
CSA W59 Position of Groove: F / H

FILLER METALS:	CSA	AWS
Specification:	W48-2006	A5.18 - 2005
Classification:	G 49A 3C S6	ER70S-6

Wire Diam (in): 0.045" (Lincoln L59)

ELECTRICAL CHARACTERISTICS

Transfer (GMAW): Spray
Current: DCEP

SHIELDING:

GAS: 90% Argon / 10% CO2
Flow Rate: 35-45 CFH
Gas Cup Size: 0.6875"

TECHNIQUE

Stringer — # of Electordes: 1
Multi-pass weld
Contact Tube to Work Distance: 3/4"
Interpass Cleaning: Chipping & wire brush

WELDING PROCEDURE

Weld Size (ETT)	Layer	Pass	Process	Diam (in)	WFS (in/M)		Volts	Travel (in/M)	Other Notes
1/4"	1	1	GMAW	0.045"	460		28	25	Side 1
Plate T	All	All	GMAW	0.045"	460		28	25	Side 2

PREHEAT:

Thickness	Temperature
Up to 3/4"	32 deg F
Over 3/4" to 1-1/2"	50 deg F
Over 1-1/2" to 2-1/2"	150 deg F
Over 2-1/2"	225 deg F

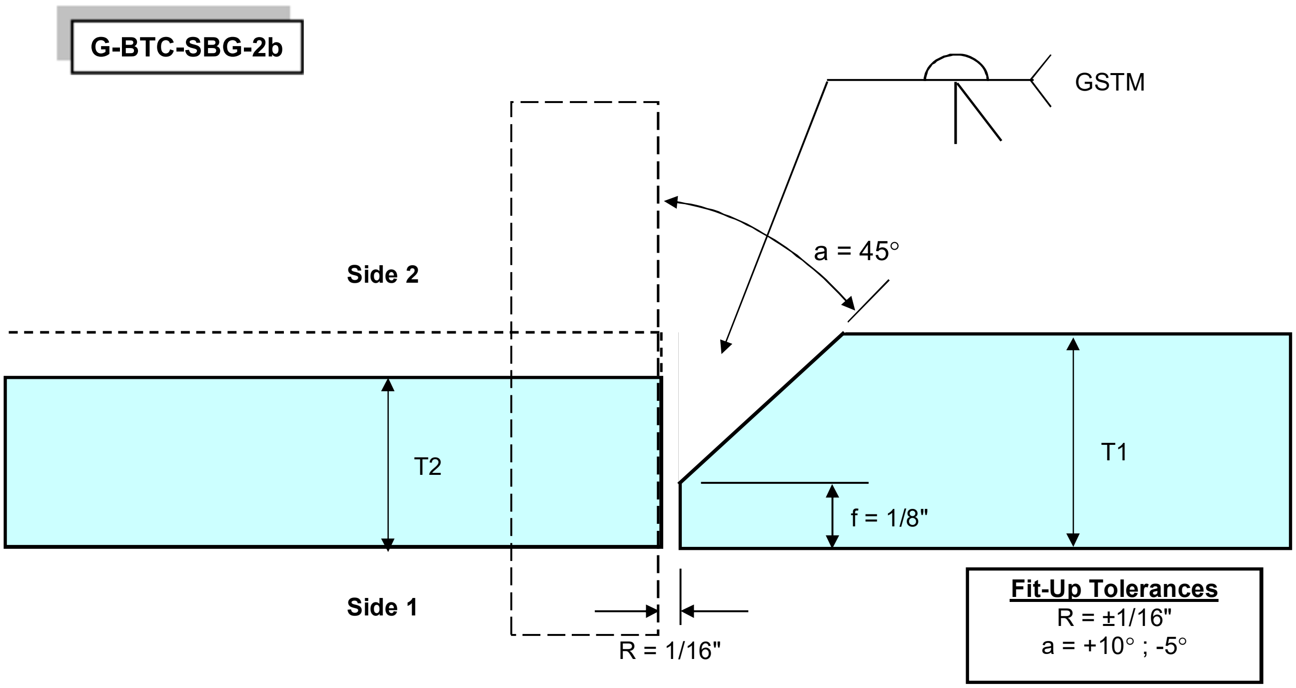
Interpass Temp: Min 50° F Max 550° F

CWB Accepted
CWB
April 08, 2015
Valid only if welding consumables are certified by the CWB

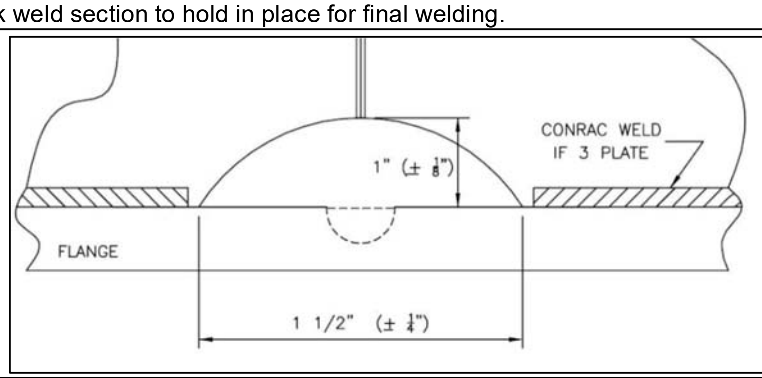
Company Authorization

C.D. Hyder
PROFESSIONAL ENGINEER
C.D. HYDER
#172574
BRITISH COLUMBIA
ENGINEER

JOINT DETAIL



GENERAL NOTES:

1	Amps or Wirefeed speed variance allowed; ± 10% . Check with meter. (Range 414 - 506 in/min)
2	Voltage variance allowed; ± 7% . Check with meter. (Range 26 - 30 volts)
3	Travel speed variance allowed; ± 25% (19 - 31 in/min)
4	When base metal temperature is below 32° F, the base metal shall be preheated to at least 70° F and maintained during welding for thickness' up to and including 3/4".
5	Use forehand technique.
6	Keep gas cup free from excessive spatter.
7	**For tubular shapes:** Bevel one edge before assembly. Tack backing ring to HSS or cap plate to hold in place. Slide HSS over ring leaving 1/4" gap root opening. Tack weld section to hold in place for final welding.
8	For hot rolled shape butt splices and 3-plate sections with unspliced flanges already welded to webs: Cut weld access hole for root weld application. Access hole may be left open. **NOTE:** Web access hole is NOT required for corner joints on endplates. 

BLUESCOPE BUILDINGS

BlueScope Buildings North America, Inc.
Welding Procedure Data Sheet

WPS No: G-BTC-SBG-PJP-2 Revision **0** Date: **17-Mar-2015** Ref Stds: **W47.1-2009 / W59-2013 / D1.1-2010**

Prepared By: Skip Hyder / Tim Brown **Type:** Semi-Auto

Welding Process(es): **GMAW** WPS Ref #WS-01

Prequalified per: D1.1 Fig 3.3; BTC-P4-GF and W59 Fig 10.8; B-P4b-G & TC-P4b-G

JOINT

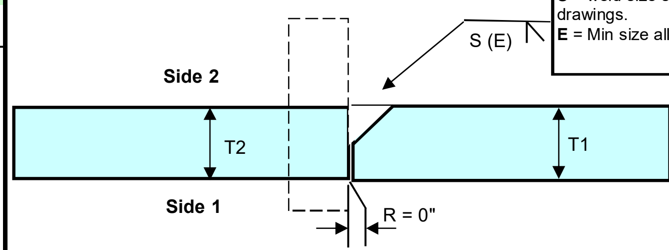
Type: **Butt, T, or Corner Joint**

Backing: No Single-Side Weld

Penetration: PJP - Partial Joint Penetration

Thickness ETT (in): 3/8" to Unlimited

Use larger of S or E
S = weld size shown on drawings.
E = Min size allowed.



BASE METALS

AWS Steel: Group II to Group II
W59 Steel: Group 3 to Group 3

POSITION

D1.1 Position of Groove: Flat / Horizontal
CSA W59 Position of Groove: Flat / Horizontal

FILLER METALS:	**CSA**	**AWS**
Specification:	W48-2006	A5.18 - 2005
Classification:	G 49A 3C S6	ER70S-6
Wire Diam (in):	0.045" (Lincoln L59)	

ELECTRICAL CHARACTERISTICS

Transfer (GMAW): Spray
Current: DCEP

SHIELDING:

GAS: 90% Argon / 10% CO2
Flow Rate: 35-45 CFH
Gas Cup Size: 0.6875"

TECHNIQUE

Stringer # of Electordes: 1
Multi-pass weld

Contact Tube to Work Distance: 3/4"
Interpass Cleaning: Chipping & wire brush

WELDING PROCEDURE

Weld Size (ETT)	Layer	Pass	Process	Diam (in)	WFS (in/M)		Volts	Travel (in/M)	Other Notes
3/16"	1	1	GMAW	.045"	460		28	35	1/4" > T1 ≤ 1/2"
1/4"	1	1	GMAW	.045"	460		28	25	3/8" ≥ T1 ≤ 3/4"
5/16"	1	1	GMAW	.045"	460		28	15	7/16" ≥ T1 ≤ 1-1/2"
3/8"	All	All	GMAW	.045"	460		28	25	1/2" ≥ T1 ≤ 2-1/4"
1/2"	All	All	GMAW	.045"	460		28	25	5/8" ≥ T1 ≤ 6"
≥ 5/8"	All	All	GMAW	.045"	460		28	25	T1 ≥ S + 1/8"

PREHEAT:

Thickness	**Temperature**
Up to 3/4"	32 deg F
Over 3/4" to 1-1/2"	50 deg F
Over 1-1/2" to 2-1/2"	150 deg F
Over 2-1/2"	225 deg F
Interpass Temp:	Min 50° F Max 550° F

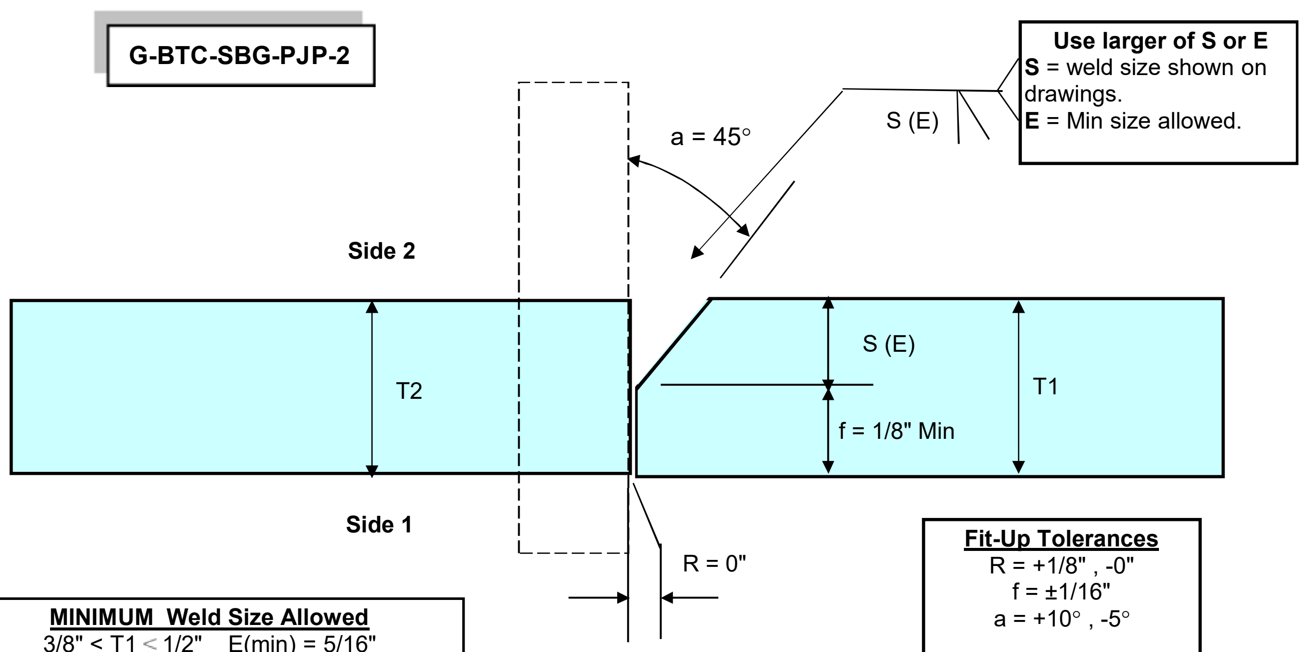
CWB Accepted
April 08, 2015
Valid only if welding consumables are certified by the CWB

Company Authorization

C.D. Hyder
PROFESSIONAL ENGINEER C.D. HYDER #112674 BRITISH COLUMBIA

BlueScope Buildings North America, Inc.
Welding Procedure Data Sheet

JOINT DETAIL



MINIMUM Weld Size Allowed

3/8" < T1 ≤ 1/2" E(min) = 5/16"
1/2" < T1 ≤ 3/4" E(min) = 3/8"
3/4" < T1 ≤ 1-1/2" E(min) = 7/16"
1-1/2" < T1 ≤ 2-1/4" E(min) = 1/2"

Ref D1.1 Table 3.4 and W59 Table 4.3

GENERAL NOTES:

1	Amps or Wirefeed speed variance allowed; ± 10% . Check with meter. (Range 414 - 506 in/min)
2	Voltage variance allowed; ± 7% . Check with meter. (Range 26 - 30 volts)
3	Travel speed variance allowed; ± 25%
4	When base metal temperature is below 32° F, the base metal shall be preheated to at least 70° F and maintained during welding for thickness' up to and including 3/4".
5	Use forehand technique.
6	Keep gas cup free from excessive spatter.

BLUESCOPE BUILDINGS

BlueScope Buildings North America, Inc.
Welding Procedure Data Sheet

WPS No: G-Fillet-3	Revision 0	Date: 4-May-2015	Ref Stds: W47.1-2009 / W59-2013 / D1.1-2010

Prepared By: Skip Hyder / Tim Brown — Type: Semi-Auto
Welding Process(es) **GMAW** — WPS Ref #WS-01
Prequalified per: D1.1 Section 3.9 and W59 Clause 10.1.3.3

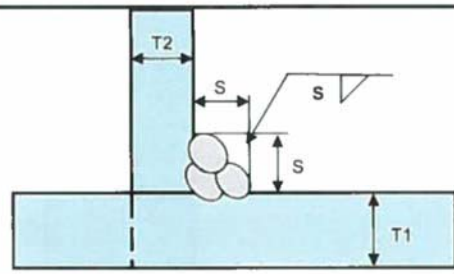
JOINT

Type: **T, Corner Joint, or Lap Joint**

Backing: No — Single-Side Weld

Penetration: Fillet Weld

Fillet (in): 3/16" to 5/8"



BASE METALS

AWS Steel: Group II to Group II
W59 Steel: Group 3 to Group 3

POSITION

AWS D1.1 Position of Groove: F / H / V (up)
CSA W59 Position of Groove: F / H / V (up)

FILLER METALS:	**CSA**	**AWS**
Specification:	W48-2006	A5.18 - 2005
Classification:	G 49A 3C S6	ER70S-6
Wire Diam (in):	0.045" (Lincoln L59)	

ELECTRICAL CHARACTERISTICS

Transfer (GMAW): Spray
Current: DCEP

SHIELDING:

GAS: 90% Argon / 10% CO2
Flow Rate: 35-45 CFH
Gas Cup Size: 0.6875"

TECHNIQUE

Stringer — # of Electordes: 1
Single pass or Multi-pass weld

Contact Tube to Work Distance: 3/4"
Interpass Cleaning: Chipping & wire brush

WELDING PROCEDURE

Weld Size (ETT)	Layer	Pass	Process	Diam (in)	WFS (in/M)		Volts	Travel (in/M)	Other Notes
1/8"	1	1	GMAW	0.045"	380		27	45	
3/16"	1	1	GMAW	0.045"	380		27	32	
1/4"	1	1	GMAW	0.045"	380		27	18	
5/16"	1	1	GMAW	0.045"	380		27	12	
3/8"	1	1	GMAW	0.045"	380		27	24	
	2	1	GMAW	0.045"	380		27	24	
7/16"	1	1	GMAW	0.045"	380		27	18	
	2	1	GMAW	0.045"	380		27	18	
1/2" & 5/8"	1	1	GMAW	0.045"	380		27	14	
	2	1	GMAW	0.045"	380		27	14	
	2	2	GMAW	0.045"	380		27	14	

PREHEAT:

Thickness	**Temperature**
Up to 3/4"	32 deg F
Over 3/4" to 1-1/2"	50 deg F
Over 1-1/2" to 2-1/2"	150 deg F
Over 2-1/2"	225 deg F
Interpass Temp:	Min 50° F Max 550° F

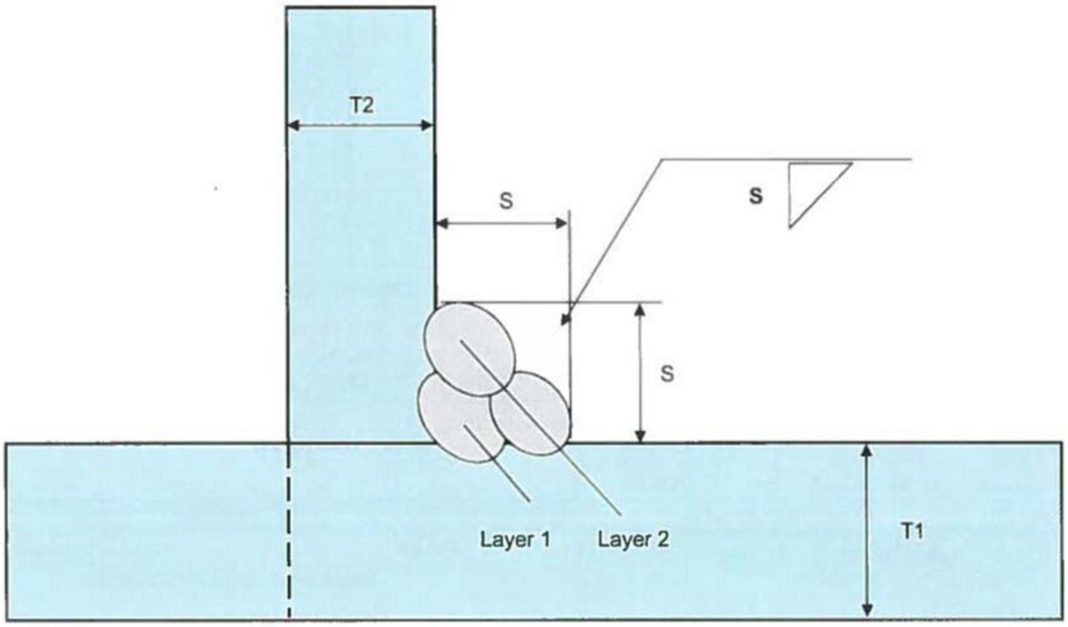
CWB
CWB Accepted
May 07, 2015
Valid only if welding consumables are certified by the CWB

Company Authorization

C.D. Hyder
PROFESSIONAL ENGINEER
C.D. HYDER
#112374
BRITISH COLUMBIA

JOINT DETAIL

G-Fillet-2



GENERAL NOTES:

1	Amps or Wirefeed speed variance allowed; ± 10% . Check with meter. (Range 345 - 418 in/min)
2	Voltage variance allowed; ± 7% . Check with meter. (Range 24 - 29 volts)
3	Travel speed variance allowed; ± 25%
4	When base metal temperature is below 32° F, the base metal shall be preheated to at least 70° F and maintained during welding for thickness' up to and including 3/4".
5	Use forehand technique.
6	Keep gas cup free from excessive spatter.

BLUESCOPE BUILDINGS

BlueScope Buildings North America, Inc.
Welding Procedure Data Sheet

WPS No: S-Fillet-1 | Revision **2** | Date: **6-Jul-2015** | Ref Stds: **W47.1-2009 / W59-2013 / D1.1-2010**

Prepared By: Skip Hyder | **Type:** Machine

Welding Process(es) **SAW - Pull Thru Welder (Conrac)** | WPS Ref #WS-01

Prequalified per: D1.1 Section 3.9 and W59 Clause 10.1.3.3

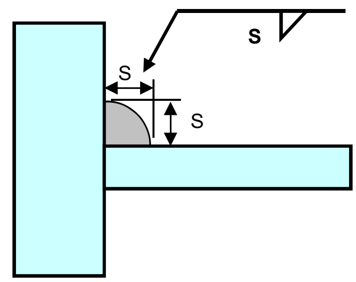
JOINT

Type: **T Joint - Fillet**

Backing: No | Single-Side Weld

Penetration: Fillet Weld

Fillet: 1/8" - 5/16"



BASE METALS

AWS Steel: Group II to Group II

W59 Steel: Group 3 to Group 3

POSITION

AWS D1.1 Position of Groove: 2F - Flat / Horizontal

CSA W59 Position of Groove: 2F - Flat / Horizontal

FILLER METALS:	CSA	AWS
Specification:	W48-2006	A5.17 - 2007
Classification:	F49A2-EM13K-H8	F7A0-EM13K-H8
Wire Diam (in):	Twin 0.045" (1.1 mm) Lincoln L50	

SHIELDING:	CSA	AWS
Flux:	See Note 5	See Note 5
Flux (Class):	F49A2-EM13K-H8	F7A0-EM13K-H8

ELECTRICAL CHARACTERISTICS

Current: DCEP

TECHNIQUE

Stringer | # of Electordes: 2

Single Pass Weld

Electrode Spacing:

Longitudinal: 0.128" | Lateral: 0.352" | Angle: 50 deg

Contact Tube to Work Distance: 5/8" - 3/4"

WELDING PROCEDURE

Weld Size (ETT)	Layer	Pass	Process	Diam (in)	WFS (in/M)	Amps	Volts	Travel (in/M)	Other Notes
1/8"	1	1	SAW	2 - .045"	420	600	27	115	
5/32"	1	1	SAW	2 - .045"	450	640	28	110	
3/16"	1	1	SAW	2 - .045"	450	640	28	84	
1/4"	1	1	SAW	2 - .045"	450	640	30	47	
5/16"	1	1	SAW	2 - .045"	450	640	30	30	

PREHEAT:

Thickness	Temperature
Up to 3/4"	32 deg F
Over 3/4" to 1-1/2"	50 deg F
Over 1-1/2" to 2-1/2"	150 deg F
Over 2-1/2"	225 deg F

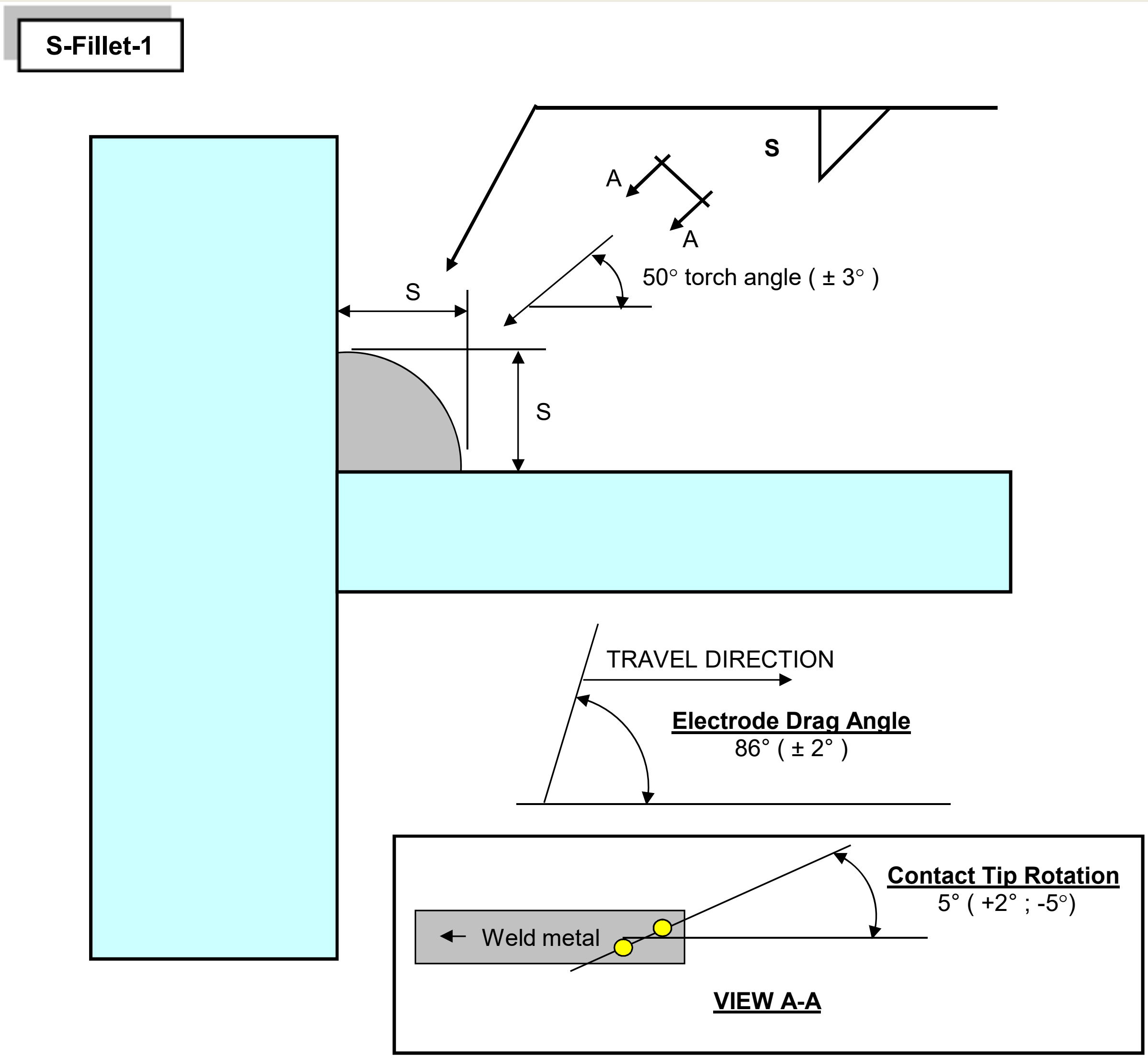
Interpass Temp: Min 50° F Max 550° F

BBNA Authorization

CWB Authorization

BlueScope Buildings North America, Inc.
Welding Procedure Data Sheet

JOINT DETAIL



GENERAL NOTES:

1	Wire feed speed variance allowed; ± 10% . Check with meter.
2	Voltage variance allowed; ± 7% . Check with meter.
3	Travel speed variance allowed; ± 15%
4	When base metal temperature is below 32° F, the base metal shall be preheated to at least 70° F and maintained during welding for thickness' up to and including 3/4".
5	**Certified** recycled flux (T-781) or Lincoln 781 virgin flux may be used as specified in WS-01-Welding Procedure Specification

A. Scope

1) This specification covers welding and related operations for steel fabrication in BlueScope Buildings North America (BBNA) manufacturing facilities. Projects shipping to jurisdictions requiring certification shall be shipped from facilities that are certified or approved by that jurisdiction.

Table 1 – BlueScope Buildings North America Manufacturing Facilities

Facility	CWB Certification	USA Approved Fabricator
Annville, PA	BUTMC1	IAS AC472: MB-177
Evansville, WI	VARPR1	IAS AC472: MB-171
St. Joseph, MO	VPSTJ1	IAS AC472: MB-156 City of Houston, TX
Visalia, CA	BUTLE1	IAS AC472: MB-147 City of Los Angeles, CA: FB00031 Clark County Nevada City of Phoenix, AZ
Laurinburg, NC	BUTMA1	IAS AC472: MB-169
Monterrey, Mexico	BUTME1	IAS AC472: MB-179
Rainsville, AL	--	IAS AC472: MB-161

2) This specification covers welding and related operations in accordance with the following standards:

a) American Welding Society D1.1-2010 – Structural Welding Code-Steel
b) American Welding Society D1.3-2008 – Structural Welding Code-Sheet Steel
c) Canadian Standards Association W47.1-2009 – Certification of Companies for Fusion Welding
d) Canadian Standards Association W59-2013 – Welded Steel Construction (Metal Arc Welding)
e) American Welding Society (AWS) D1.8-2009 – Structural Welding Code-Seismic Supplement

3) Welding Procedure Data Sheets (WPDS's); WS-02 Fabrication Quality, Workmanship, and Repairs; and WS-03 Quality Assurance and Inspection form an essential part of this specification.
4) Project specific variations from WPDS's and Weld Standards must be specified on fabrication/shop documents.

B. Personnel

1) BlueScope will designate qualified **Welding Engineer(s)** responsible for the company wide welding design, principles, and practices as required by CSA W47.1. Weld engineers shall have qualifications as required by W47.1.

2) Each manufacturing facility that does welding operations under the jurisdiction of Canadian Welding Bureau (CWB) shall have qualified **Welding Supervisor(s)** with a minimum welding-related experience as required by W47.1.

3) Each manufacturing facility (any form of fabrication) shall designate a **Quality Manager** responsible for assuring that company Quality Systems criteria are implemented and enforced and meet the quality requirements listed below as applicable;

   a) IAS AC472 Accreditation Criteria.

   b) CWB Welding Certification.

   c) CSA A660 Certification.

   d) All State and local jurisdiction quality criteria and procedures as applicable to each facility.

4) Each manufacturing facility that does welding operations shall have **In-House Quality Control Inspector(s)** who are **Certified Welding Inspectors** (CWI).

5) All **welding personnel** shall be qualified welders in accordance with AWS and/or CWB criteria. Welders shall be tested and qualified to perform all welding procedures required by their specific duties. Welders shall not perform production welds for procedures or positions they have not been properly tested and qualified to do as defined by AWS and/or CWB as applicable.

   a) The welding shall be done in the position as limited by the welding procedure data sheet (WPDS) and as allowed by individual welder qualifications. Lower difficulty positions are allowed in addition to the position indicated on the WPDS.

C. Base Metal

1) The base metal shall conform to the following specifications or their equivalent:

   a) ASTM Standards

   b) CSA Standards

2) Base metals are grouped into material specification “steel groups” for each weld process and electrode type;

   a) AWS D1.1 Section 3.3 for ASTM designated steels ≥ 1/8” thick.

   b) AWS D1.3 Section 1.4 for ASTM designated steels < 1/8” thick.

   c) CSA W59 Clause 3.2 for ASTM or CSA designated steels ≥ 1/8” thick.

3) Base Metal Groups are limited to those indicated on the WPDS’s. Steel groups equal to or lower than indicated on the WPDS may be welded with specified electrodes.

4) Base Metal Thickness' covered by this specification are as specified on WPDS's.

D. Weld Filler Metal (Electrodes), Flux, and Gas

1) **Electrodes** shall be in accordance with the requirements of AWS A5 – Filler Metal Specifications, and/or CSA Standard W48 – Filler Metals and Allied Materials for Metal Arc Welding.

a) Electrodes shall be tested and/or certified by manufacturer meeting:

(i) Minimum CVN = 20 ft-lbs @ -20°F and,

(ii) Minimum CVN = 40 ft-lbs @ +20°F

b) Electrodes shall be limited to those specified in the WPDS's. All electrodes shall be "low-hydrogen" electrodes with diffusible hydrogen designation of H8 or less as determined per AWS A4.3.[1]

c) Electrodes shall be delivered and stored in suitable condition that will keep them dry and free from surface rust and foreign material.

2) **Flux** used for SAW shall be dry and free from contamination of dirt, mill scale, or other foreign material. Flux shall be purchased in packages capable of being stored under normal conditions for at least 6 months without such storage affecting its welding characteristics or weld metal properties. Flux from damaged packages that have been exposed to free moisture shall be discarded or shall be dried before use in shallow layers (2" maximum) at a minimum temperature of 500° F for at least 1 hour or at time and temperature conditions as recommended by the manufacturer. Flux fused in welding shall not be reused.

a) Flux may be reclaimed, recycled, tested, and repackaged per AWS A5.17 and CSA W48 requirements. Test reports, and/or manufacturer's certifications of recycled flux shall be submitted to Quality Manager and packaging must be in accordance with AWS A5.17 and CSA W48. For processing details see Quality System Procedure #PRMF0701-Crushed Slag Procurement, Processing, and Utilization in SAW Processes.

3) **Shielding Gas** shall be a welding grade having a dew point of -40° F or lower. The shielding gas/electrode combination shall be as shown on the accepted WPDS.

a) Welding with the GMAW processes shall NOT be performed in winds exceeding 3 MPH.[2]

E. Welding Procedures. Only documented and approved weld procedures may be used in BlueScope Buildings facilities.

[1] AWS D1.8 Sec. 6.3.2 and W59 Clause 5.2
[2] AWS D1.8 Sec 6.2.2.1

1) Pre-qualified welds shall be documented on BBNA welding procedure data sheets (WPDS) and approved by the Welding Engineer.

2) Non pre-qualified welds shall be documented on BBNA welding procedure data sheets (WPDS) and approved by the Welding Engineer. Welds shall be tested and qualified by Procedure Qualification Reports (PQR) per AWS and/or CWB requirements.

3) Weld position shall be done in the position indicated on the WPDS, or of lesser difficulty.
   a) Welders shall not weld in any position they have not been qualified for.
   b) When making temporary welds for repairs, no out-of-position welding is permitted.

4) Preheat temperatures shall apply as shown on the WPDS.
   a) When the base metal temperature is below 32°F (0°C), preheat the base metal to at least 70°F (20°C) and maintain this temperature during welding. Preheat shall consist of heating a band of the base metal approx. 3" (8 cm) on either side of the joint and in advance of the welding.
   b) Preheating may be accomplished by means of flame torches or by means of electrical or induction heaters.

5) Interpass temperatures shall not exceed 550°F (300°C).[3] Interpass temperatures must be maintained within the minimum and maximum temperatures indicated on the WPDS. Temperatures shall be measured approx. 3" (8 cm) either side of the joint.
   a) After welding, weldments shall be allowed to cool to ambient temperature, without external quench media being applied. Do not cool using water or by immediate placement in frigid conditions which will cause a rapid change in temperature.

6) Electrical Characteristics
   a) Welding current shall be direct current (reverse polarity) using a constant voltage type power supply. The range of parameters will be as per the electrode manufacturer's instructions, or as determined by charts in this specification, and will be shown on the WPDS.
   b) For SAW processes, the current used shall be either direct current (DC) or alternating current (AC) as indicated on the WPDS.

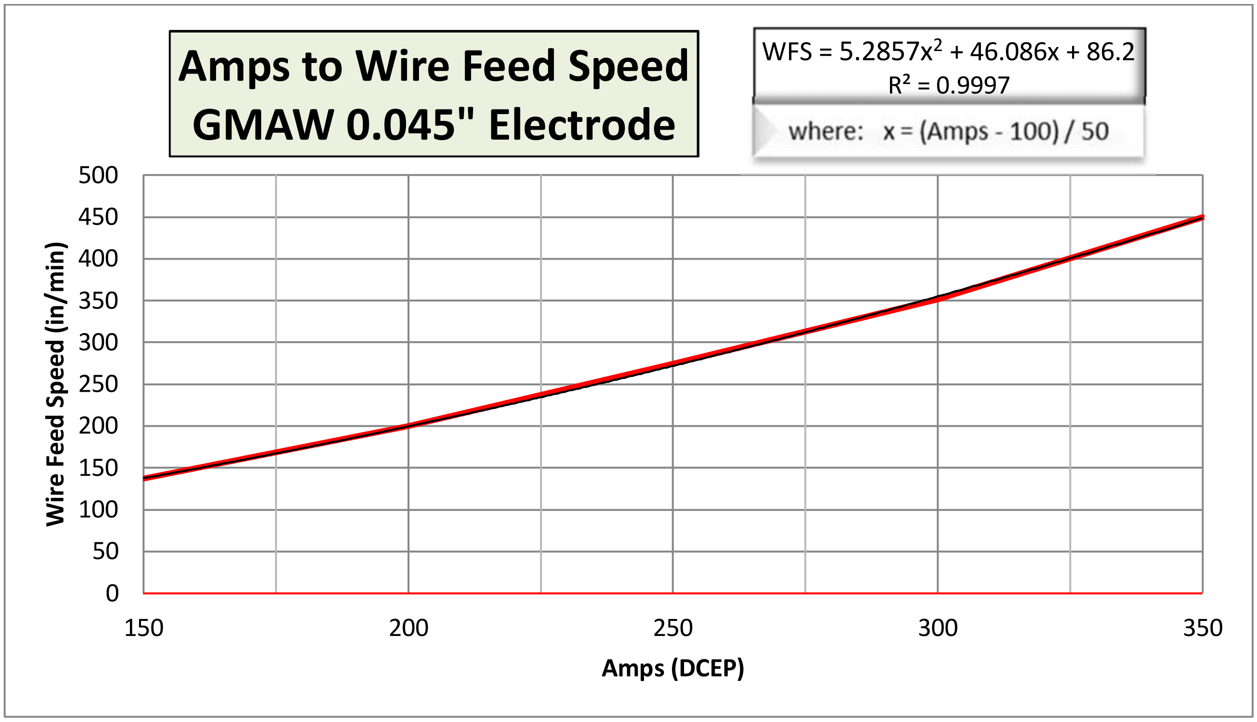
7) Heat Treatment and Stress Relieving is not applicable to weldments under this specification.

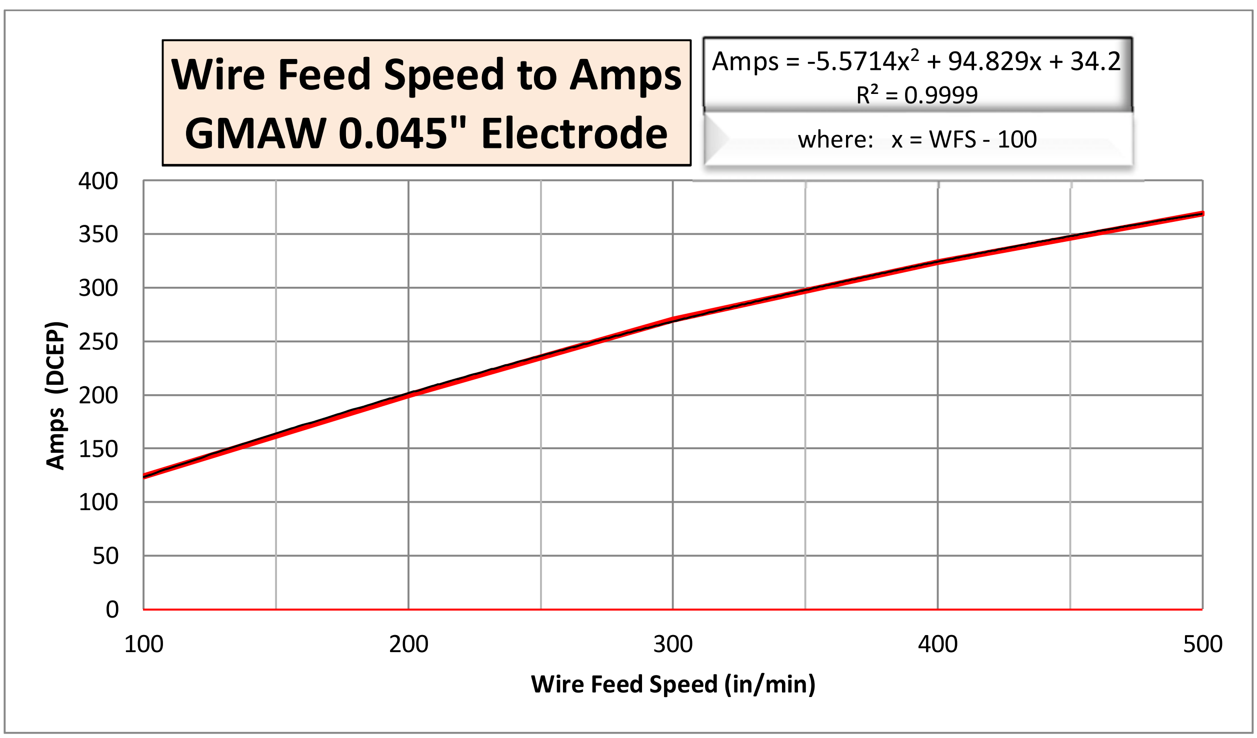
F. Welding Technique and Settings

1) The correct amperage and voltage, speed of travel, thickness of layers, number of passes, position, material electrodes, and any special instructions will be shown on WPDS. Guidelines below may be used to determine required variables as required.

[3] AWS D1.8; Section 6.5

2) Wire Feed Speed vs Welding Current. Below are charts and equations that can be used to determine required AMPs when given WFS and visa-versa.





3) Heat Input calculations are occasionally required by contract documents. Use the following method to provide Heat Input[4] values for joints when specified by contract. Data needed for variables shown below can be found on WPDS.

$$H = (60 * E * I) / (1000 * S)$$

Where:

H = heat input, (kilo-joules/in)
E = arc voltage, (Volts)
I = current (Amps)
S = travel speed, (in/min)

G. Preparation, Quality & Workmanship

1) All welding and its related operations shall be performed in accordance with good shop practice and within the recommendations and limitations outlined in AWS D1.1, AWSD1.3, or CSA W59 as applicable. See **WS-02** - Fabrication Quality, and Workmanship for more details.

2) Cleaning of previously deposited layers shall be done by wire brushing or by mechanical means, if necessary, to remove all traces of slag from deposit, before the next successive bead is applied, and prior to painting.

H. Groove Welds

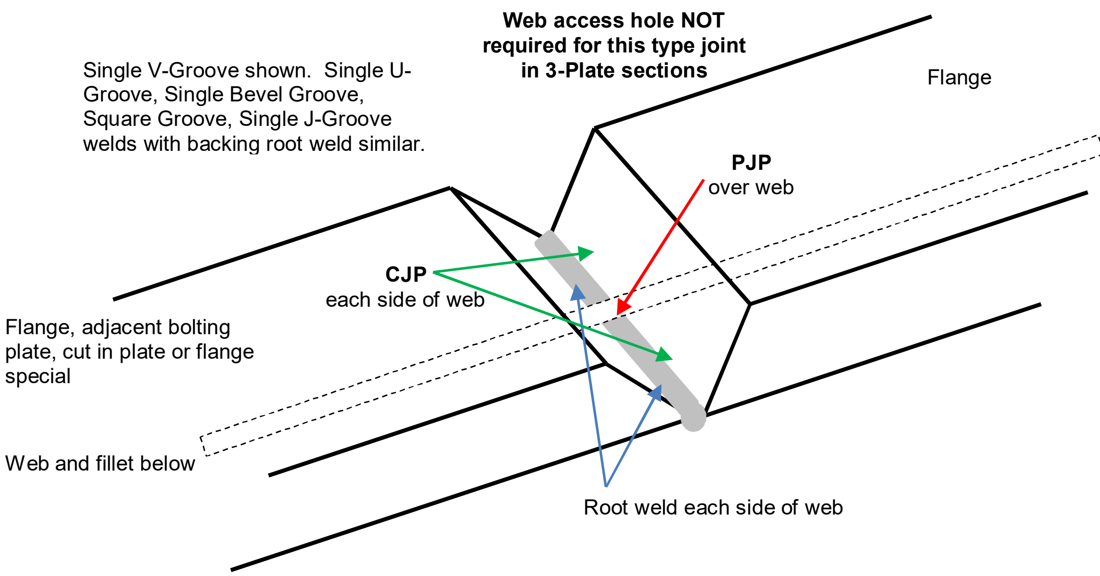
1) When back gouging is required on the underside of a full penetration weld joint, the root of the joint shall be chipped, ground or arc-gouged to produce a groove contour substantially conforming to the pre-qualified single U-joint, and its depth shall be adequate to ensure complete penetration into the previously deposited sound weld metal as indicated on WPDS.

2) Reinforcement of CJP groove butt welds and outside corner joints is permitted to a maximum of 1/8”. This is the added thickness of the weld face over the surface of the plate. Any weld metal exceeding this must be removed by grinding. A smooth transition into adjoining weld metal must be maintained. T-joints and inside of corner joints reinforcement is not limited. Welds shall have a gradual transition to the plane of the base metal. See D1.1 Section 5.24.3, Table 5.9 & 5.10, and Figure 5.4 for more details.

3) Access Holes in webs of beams, when provided, are intended to provide clearance for weld gun or backing bar. Unless indicated otherwise on engineering shop drawings, the following guidelines apply.

a) CJP Butt Welds w/ weld metal backer do not require access holes for 3-Plate built-up sections.

[4] AISC Design Guide #21; Section 7.6.9

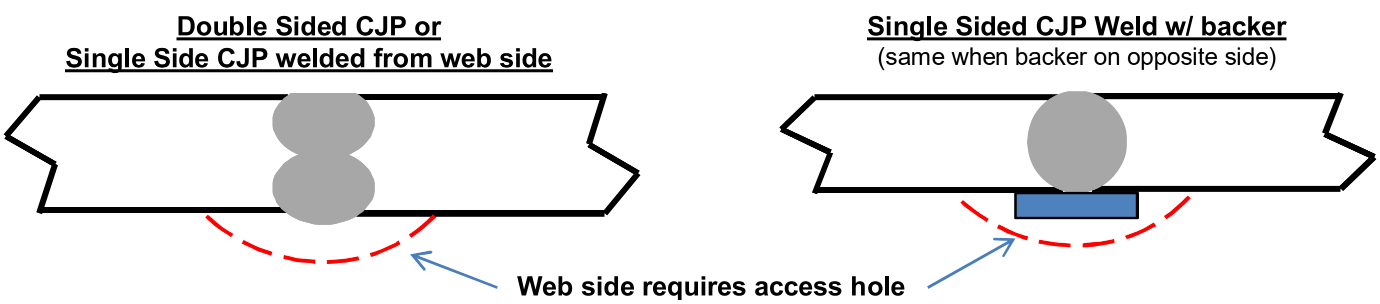


<u>3-Plate section flange butt weld with weld metal Backer</u>

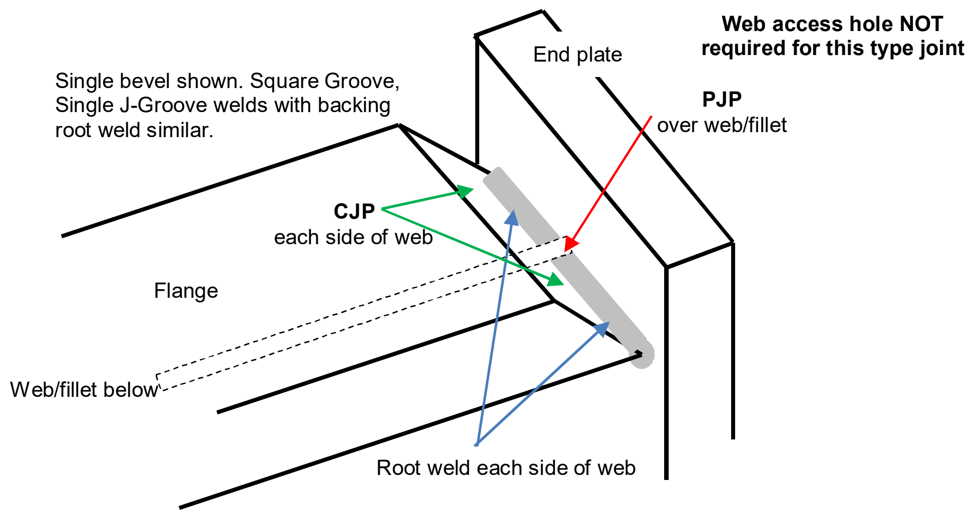
The joint is not CJP completely across the joint. The portion of weld over the web is a PJP weld. However the joint strength is adequate to resist required loads. This procedure **IS NOT** permitted for CJP splices of hot rolled I-shaped sections.

Note: When UT testing is done at the center of the weld (at the web) indications of incomplete fusion are often seen and are acceptable for these type joints.

b) Butt Welds w/ backer bars, or when welds are finished from the web side, access holes are required for 3-plate sections. Access holes are <u>always required for CJP splices in hot roll I-shaped</u> sections unless approved otherwise by engineering.



c) CJP welds to end plates (e.g.- moment resisting endplates, cap plates, base plates, etc.) with weld metal backer do not require access holes.

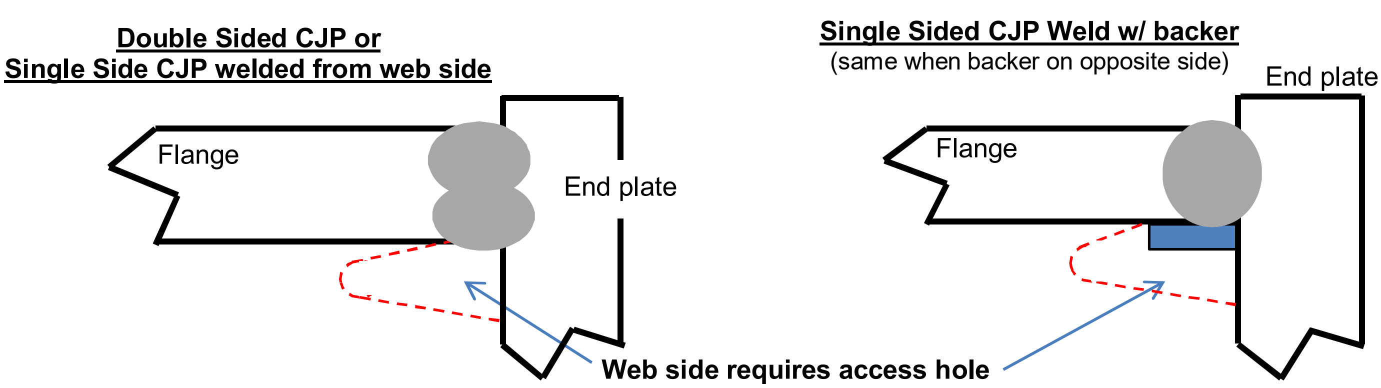


CJP weld to endplate with weld metal Backer

The joint is not CJP completely across the joint. The portion of weld over the web is a PJP weld.[5] However the joint strength is adequate to resist required loads and performs better than same joint with access holes.[6] This procedure is permitted for CJP welds to endplates for 3-plate and hot roll I-shape sections.

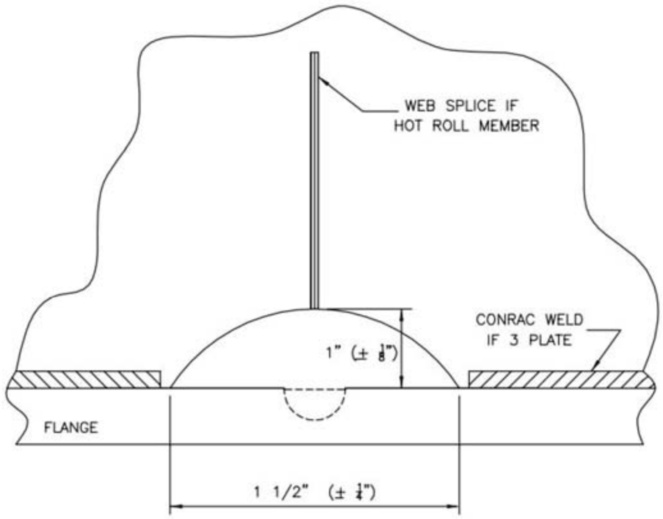
Note: When UT testing is done at the center of the weld (at the web) indications of incomplete fusion are often seen and are acceptable for these type joints.

d) CJP welds to endplates w/ backer bars, or when welds are finished from the web side, access holes are required.

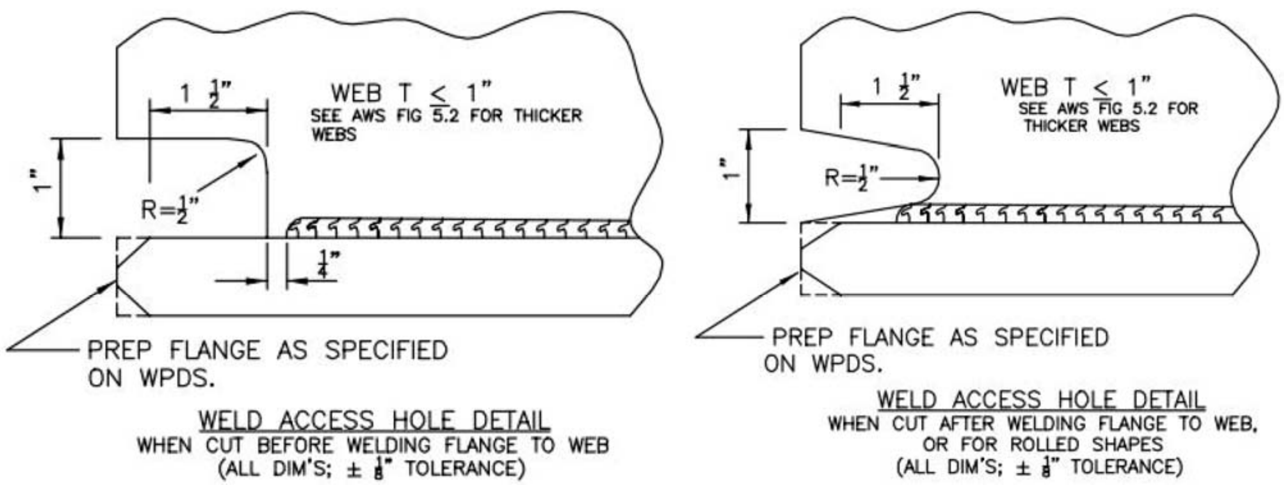


[5] AISC Steel Construction Manual 14th Ed; Part 12 – Extended End-Plate FR Moment Connections Design Assumptions Note 8.
[6] AISC Design Guide 4; Sec. 1.3.4 and AISC Design Guide 16; Section 1.2.4

I. **Weld access hole profiles** shall be made as shown below.[7]



Butt Splice Access Hole
(Access holes may be left open)



Access Hole Profile at CJP Endplate Joints
(Access holes may be left open)

J. Weld Applications

1) Welds designated with no length specified shall be applied from blockage to blockage.
2) Oversizing of fillet welds is not good practice. Overwelding may cause heat distortion and increased costs. Fillet welds that are oversized are restricted to no more than 1/16” of either fillet leg.

[7] AISC-360 Sec J1.6; AWS D1.1 Sec. 5.17; AWS D1.8 Sec. 6.10; and W59 Clause 5.3.8

3) Minimum size of fillets welds shall not be less than called for on engineering documents.

   a) Minimum weld sizes established by engineering are based on AWS D1.1 Table 5.8 which is based on the thinner part joined when using low hydrogen electrodes.

   b) CSA W59 Table 4.4 allows the minimum weld size based on the thinner part jointed when low hydrogen electrodes are used and minimum pre-heat criteria are applied as required on the WPDS.

4) Random shop splices (i.e.- CJP butt joints) are allowed anywhere in any weldment at shop discretion (either built-up 3-Plate or Hot Rolled sections), unless noted otherwise on shop documents. See **WS-02** for instructions when random butt splices occur at or near holes.

   There is no specified minimum distance between CJP splices and other welded connections. Welded clips, cuts, and pin connected bolt holes may be applied over CJP splices. Before welding or cutting, grind weld reinforcement smooth to allow flush fit up of clips or bolted parts.

   Butt splices are not permitted within 4” of holes in moment resisting connections. See WS-02 for details.

5) Weld designation with no length specified on clips and plates with no blockage shall be stopped ¼” from each end to prevent notching. Weld terminations shall be as illustration below.

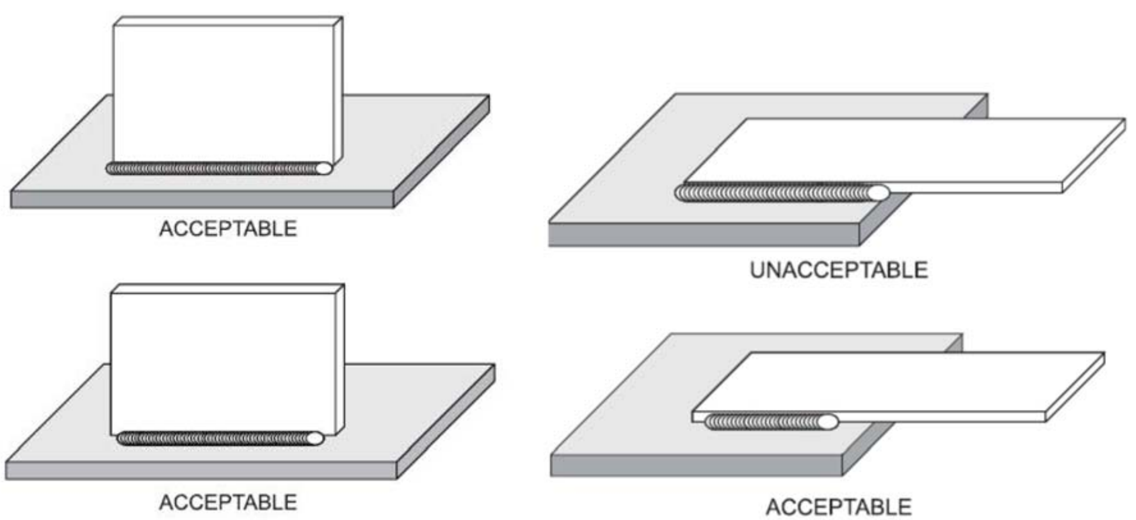


Figure 3–18. Acceptable weld terminations. *Figure 3–19. Weld termination for lap joints.*

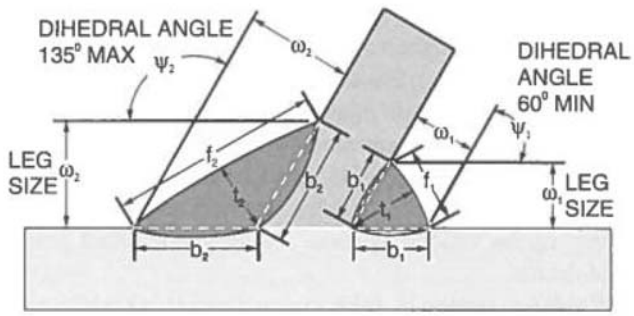
6) Weld tabs, extension bars, and run-off plates are recommended for CJP multi-pass welds. Tabs shall be made of the same material as the base metal.

a) Weld run-on/run-off distance shall be sufficient to provide as extension of the joint preparation in such a manner that the full groove weld profile is sound from edge to edge of connected plates. Remove run-off plates and grind edge smooth.

(i) Minimum run-on/run-off shall be at least equal to the thickness of the parts joined, but not less than 1 inch [8] unless indicated otherwise on WPDS.

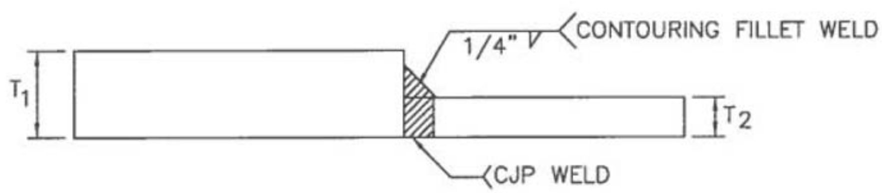
7) Skewed T-Joints

a) When welding skewed T-Joints the weld size indicated on shop drawings is the Leg Size shown in illustration below.



8) Thickness and Width Transitions

a) In **USA** thickness and width transitions are not required[9] unless specified on engineering shop drawings. When thickness transitions are not provided, apply a ¼" contouring fillet weld as shown.

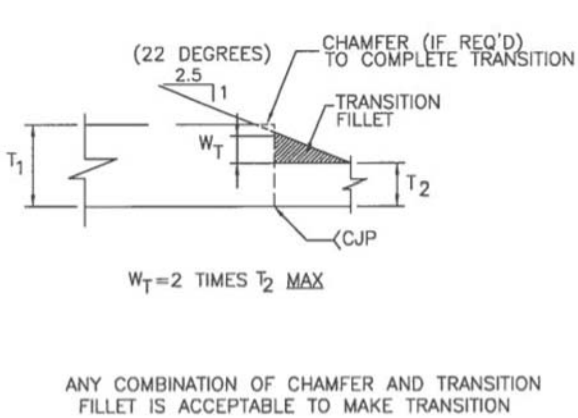


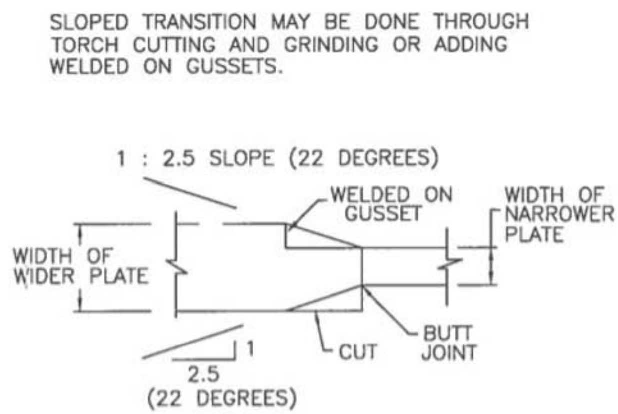
b) In **CANADA** thickness and width transitions are required at CJP butt spliced joints.[10]

(i) Exception: At joints where flanges butt into cut in bolting plates (e.g.- haunches, cut in plates for moment connections for interior columns or canopies), width transitions are not required.

c) When transitions are required they shall be furnished per AWS and W59 criteria as shown below.

[8] AWS D1.8 Sec. 6.11.1
[9] AWS D1.1 Sec. 2.8.1
[10] W59 Clause 11.4.8





Thickness Transition **Width Transition**

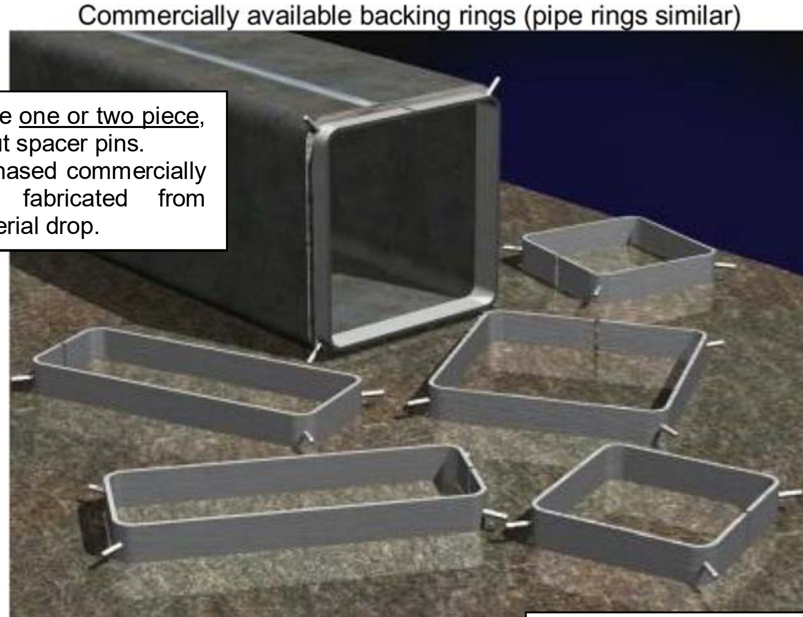
9) Temporary welds and tack welds need NOT be removed unless required by the engineer, or as noted below.

   a) Tack welds SHALL be removed from "**cyclically loaded**" members such as crane beams and crane support brackets (corbels). Remove either by grinding or gouging. All surface tack welds must also be removed either by grinding, gouging, or chiseling.

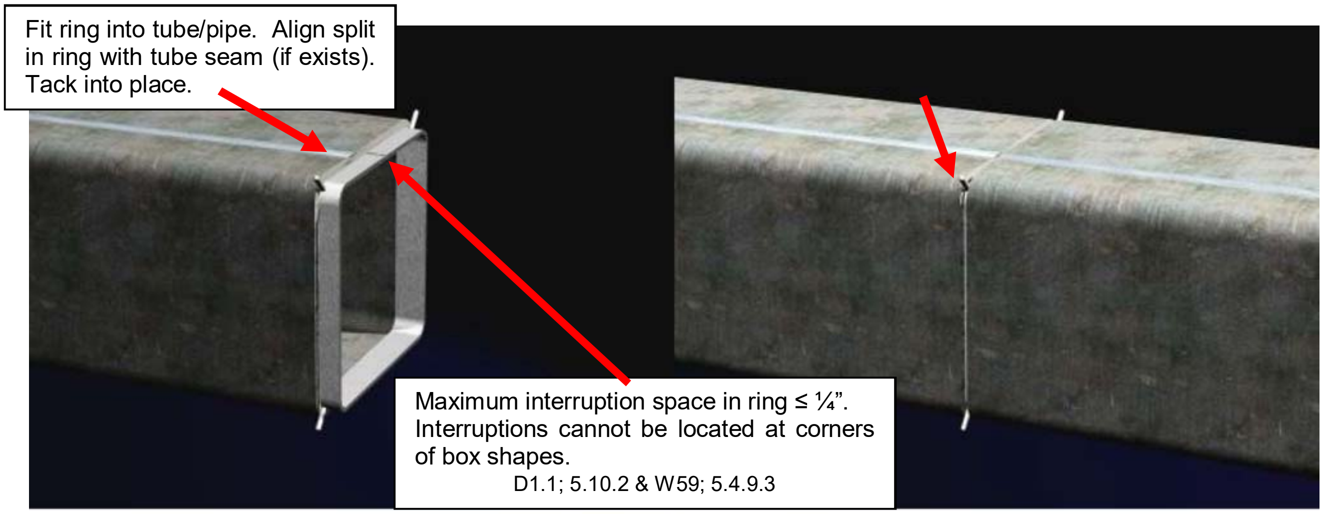
10) Tubular Butt Joints (pipe or box shapes) shall be made using backing rings as illustrated below. Root gaps shall be held as indicated on applicable WPDS. Backing rings are commercially available rings (illustrated below) or may be fabricated from tubular drop. Backing rings must fit flush against inside of pipe or box shape.

Commercially available backing rings (pipe rings similar)

Rings must be one or two piece, with or without spacer pins. May be purchased commercially (shown) or fabricated from pipe/box material drop.



As root weld approaches spacer pin (if present), stop and strike the spacer pin off and continue welding until root pass is complete.



K. Protected Zones

1) Protected zones are zones that are required by Code to be free of any bolt holes, weep holes, access holes, any welded on parts, or accidental arc strikes. Protected zones will be indicated on shop drawings when required.

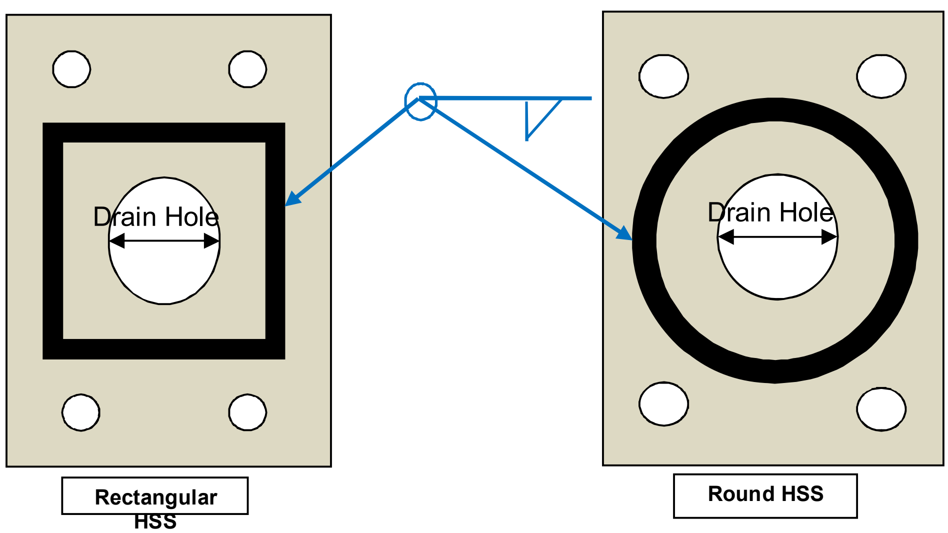
   a) CJP butt welds that fall within the protected zone are allowed. Weld metal to make such joints shall meet requirements of demand critical welds.

L. Seal Welding

1) When project contract specify “seal welding”, engineering shall indicate the requirement for seal welding on the fabrication release documents. See **PRCF0108** for more details. When seal welding is specified, welders shall weld all joints such that any two plates joined shall be sealed all around with weld metal. Complete all structurally required welds as shown on the fabrication drawings. Then weld all remaining

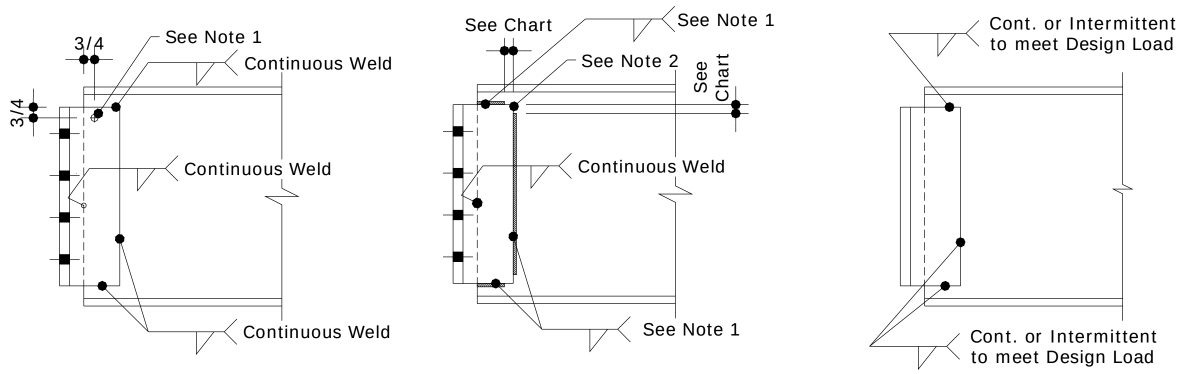
"unwelded" with minimum sized welds[11]. Seal weld quality shall meet all visual inspection criteria per AWS D1.1.

- Flange to web welds shall be continuously welded both sides of web.
- All clips, stiffeners, etc. shall be welded all around including capping the ends of the joined plates.
- End plates, cap plates, and base plates shall be fully welded both sides and capped.
- For HSS sections, end plates shall be detailed by engineering to allow for drainage as follows if hot dip galvanized. If fab drawing does not indicate this detail, contact engineering for details.



- Angle clips shall be specified as shown below depending on galvanize class. If engineering does not specify criteria on fab drawing, contact engineering for details.

[11] See AWS Table 5.8 or W59 Table 4.4, as appropriate, for minimum fillet weld sizes.



Class I / Class II
(If no vent is required) / (If vent is required)

Class III

M. Inspection & Traceability

1) Visual inspection is required by Code to be done on 100% of all weldments.[12] Acceptance criteria shall satisfy **AWS D1.1 Table 6.1** or **W59 Clause 11.5.4.1**. Visual inspection may be done by the CWI, or his designee. The Quality Manager and CWI retain responsibility for weld quality.

   a) Weldments shall be considered statically loaded members except as follows;

      (i) Crane brackets and crane runway beams are always cyclically loaded and shall be inspected to the visual inspection criteria for cyclically loaded members per AWS Table 6.1 or W59 Clause 12.5.4.1. or,

      (ii) When specified as cyclically loaded members on shop drawings.

2) All weldments shall be identified by welding personnel by use of an identifying number, letter or symbol for the purpose of traceability.[13] The welders ID on the weldment signifies that he has visually inspected his work and certifies it as acceptable per BBNA quality standards.

3) NDT inspection requirements beyond visual are not required by Code. NDT testing shall be done when required by contract specification. Specific inspection procedure and acceptance criteria shall be stipulated by contract and agreed to by BBNA.

   a) When the contract, or Authority Having Jurisdiction calls for In-Plant NDT inspection and/or testing, see WS-03 – Quality Assurance Plan for more details.

4) Quality Manager is responsible for maintaining all weld quality inspection and test records as required to satisfy regulatory audit requirements and/or contract documents.

[12] IAS AC472 Section 5.1.2.5; AWS D1.1, Section 6.9; CSA W59 Clause 7
[13] IAS AC472 Section 5.1.3.2; AWS D1.1, Section 6.5; CSA W59 Clause 7

N. WPDS Numbering System

1) BBNA WPDS's will follow the numbering system below.
   a) Example designations are as follows:
      (i) GA-G-FVG-1 (D1.3 light gage welding)
      (ii) G-B-SUG-1 (D1.1 or W59 structural welding)
2) Light Gage Welding governed by AWS D1.3
   a) Preceded by **GA-**
3) Weld Process Designation

G	(GMAW) Gas Metal Arc Welding
S	(SAW) Submerged Arc Welding
F	(FCAW) Flux Cored Arc Welding
R	(RW) Resistance Welding
SM	(SMAW) Shielded Metal Arc Welding
RS	(RSW) Resistance Spot Welding

4) Joint Types

B	Butt joint
C	Corner joint
T	T-joint
BC	Butt or Corner joint
TC	T- or Corner joint
BTC	Butt, T-, or Corner joint
BT	Butt or T joint
Lap	Lap joint
Fillet	Fillet weld
Plug	Plug weld
EF	Edge flange
AS	Arc spot

5) Weld Types (preparation types)

SG	Square Groove	**DUG**	Double U Groove
SVG	Single V Groove	**SJG**	Single J Groove
DVG	Double V Groove	**DJG**	Double J Groove
SBG	Single Bevel Groove	**FBG**	Flare Bevel Groove

DBG	Double Bevel Groove	**FVG**	Flare V Groove
SUG	Single U Groove		

6) Extended designators;

a) **-PJP** for **partial joint penetration** welds

b) Sequence number to allow for similar procedures that have the same weld designation. Follow WPDS ID number with **-1, -2, -3**, etc. for designations that are the same.

Document Revision History

REV. #	DATE	NAME	DESCRIPTION
0	12/18/2009	S.Hyder	Original document.
1	05-Apr-2010	S.Hyder	1.) Clarified workmanship criteria for weld starts and stops, and tack welds. 2.) Revised WPDS designation definition. 3.) Added new spec info at request of CWB.
2	23-Nov-2010	S.Hyder	1) Moved workmanship criteria to WS-02 - Fabrication Quality, Workmanship, & Repairs. 2) Added more detail to inspection rules and added reference to new WS-03 - Quality Assurance & Inspection. 3) Editorial changes made to clear up instructions.
3	17-Oct-2011	S.Hyder	1) Added clarification to Section D allowing equivalent Canadian steels to be used.
4	06-Aug-2012	S.Hyder	1) Added welding specs for tubular butt joints. 2) Added clarification on reinforcement welds. 3) Added clarification on random shop splices. 4) Added clarifications as required by CWB.
5	11-Nov-2013	S.Hyder	1) Clarified and expanded "random shop splice" rules (Sec N.4). 2) Clarified that CJP butt splices are not allowed in bolting plates unless approved.
6	19-Oct-2015	S.Hyder	1) Added new section to address rules for access holes. 2) Added criteria for CVN toughness for electrodes. 3) Added rules for thickness and width transitions.
7	01-Dec-2015	S.Hyder	Editorial rearrangement and cleanup of information in WS-01 and WS-02 to separate "Specifications" from "Quality, Workmanship, and Repairs". No changes to procedures.

APPENDIX F WELDING PROCEDURE SPECIFICATIONS FOR COLUMN SPECIMENS

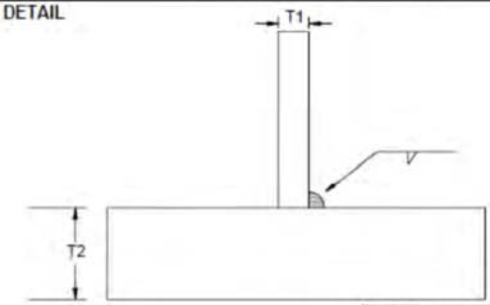
American Buildings Company
Welding Procedure Specification

WPS No. **AB4-F1** Rev **9**
Revised By: **Allen O'Steen, CWI** Date **1/31/2009**
Issued By: **Vernon Hopper, CWI** Date **2/23/1995**

Welding Process (es) **GMAW** Ref. Standards: **AWS D1.1 & CSA W47.1** Manual ☐ Machine ☐
Supporting PQR(s) Prequalified ☑ SemiAuto ☑ Automatic ☐

Joint
Type **Fillet, T- or Corner Joint**
Backing Yes ☐ No ☐ Single Weld ☑ Double Weld ☑
Back Material **N/A**
Root Opening (R) **0"** Root Face (f) **N/A**
Groove Angle (α) **N/A** Groove Radius (r) **N/A**
Back Gouge Yes ☐ No ☑
Method **N/A**

JOINT DETAIL



Tolerances As Fit-Up
R = +1/8, -0
α = N/A
f = N/A
r = N/A

Base Metals

	T1		T2
Material Specification	See Notes	to	A572
Type or Grade	Grade 55	to	Grade 55
Thickness: Groove		to	
Fillet	1/8" to Unl	to	1/4" to Unl
Diameter (Pipe)		to	

Filler Metals

	1	2
Specification	AWS A5.18	CSA W48.06
Classification	ER70S-3	B-G 49A2 CG3

Shielding
Flux **N/A** Electrode-Flux (Class) **N/A**
Shield Gas **Argon / CO2** Flow Rate **40 cfh**
Composition **92% Argon / 8% CO2** Cup Size **5/8"**

Position
Groove **N/A** Vertical Up ☐ Down ☐
Fillet **2F**

Electrical Characteristics
GMAW Transfer Short Circuit ☐ Globular ☐ Spray ☑
Current AC ☐ DCEP ☑ DCEN ☐ Pulsed Yes ☐ No ☑
Other

Coatings
Coating Type **None Allowed** Coating Thickness **N/A**

Technique
Bead Type **Stringer** Multi-pass or Single Pass **Single**
Travel Angle **15 Deg Push** Contact Tip to Work Distance **3/4"**
Number of Electrodes **1**
Electrode Spacing Longitudinal **N/A** Lateral **N/A**
Peening **Not Permitted** Cleaning **N/A**

Preheat
Preheat Temp (min) **32 F**
Thickness Up to 3/4" **32 F**
Over 3/4" to 1 1/2" **50 F**
Over 1 1/2" to 2 1/2" **150 F**
Over 2 1/2" **225 F**

Interpass Temperature
Min. **N/A** Max. **N/A**

Postweld Heat Treatment
Required Yes ☐ No ☑
Tem **N/A** Time **N/A**

Layer	Passes	Process	FillerMetal	Dia	Current	Amps	WFS	Volts	Travel	Other
1 Layer	1 Pass	GMAW	ER70S-3	.045"	DCEP	280 amps	430 wfs	27 v	30 ipm	For 1/8" fillet
1 Layer	1 Pass	GMAW	ER70S-3	.045"	DCEP	300 amps	460 wfs	29 v	26 ipm	For 3/16" fillet
1 Layer	1 Pass	GMAW	ER70S-3	.045"	DCEP	300 amps	460 wfs	29 v	18 ipm	For 1/4" fillet
1 Layer	1 Pass	GMAW	ER70S-3	.045"	DCEP	300 amps	460 wfs	29 v	14 ipm	For 5/16" fillet
1 Layer	1 Pass	GMAW	ER70S-3	.045"	DCEP	300 amps	460 wfs	29 v	10 ipm	For 3/8" fillet

Notes

1. Per D1.1-2004, Table 3.7, Prequalified WPS Requirements, the maximum single pass fillet for the GMAW process in the horizontal position shall be 3/8".
2. Per Table 4.8, D1.1-2004, this WPS is qualified for any combination of Group I and II steels provided they are within the WPS specified thickness ranges. A1011 SS Grade 55 shall cover 1/8" to 3/16" thickness while A572 Grade 55 shall cover > 3/16" thickness".

CWB Acceptance
Welding Procedure Data Sheet
CWB Accepted to CSA W47.1
SEP 21 2009
Acceptance valid only when Welding Consumables certified by C.W.B.

Certification
COMMONWEALTH OF VIRGINIA
PAUL KLIM
Lic. No. 006893
PROFESSIONAL ENGINEER

Month	Day	Year
08	24	09

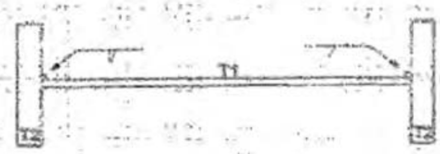
American Buildings Company
Welding Procedure Specification

WPS No. AB3-Tractor LT-1 Rev 0
Revised By: Date
Issued By: Allen O'Steen, CWI Date 4/25/2008

Welding Process (es) SAW Ref. Standards: AWS D1.1 & CSA W47.1 Manual ☐ Machine ☑
Supporting PQR(s) Prequalified ☑ SemiAuto ☐ Automatic ☐

Joint
Type Fillet, T-Joint
Backing Yes ☐ No ☐ Single Weld ☑ Double Weld ☑
Back Materia N/A
Root Opening (R) 0" Root Face (f) N/A
Groove Angle (a) N/A Groove Radius (r) N/A
Back Gouge Yes ☐ No ☑
Method N/A

JOINT DETAIL



T1 & T2 See Notes

Base Metals T1 T2
Material Specification A572 to A529
Type or Grade Grade 55 to Grade 55
Thickness: Groove to
Fillet 1/8" to Unl to 1/4" to Unl
Diameter (Pipe) to

Filler Metals 1 2
Specification AWS A5.17 CSA W48.06
Classification EM-13K Lincoln ER49S-3

Coatings
Coating Type None Allowed Coating Thickness N/A

Shielding
Electrode-Flux (Class) F7A0-EM13K
F4A3-EM13K
Shield Gas N/A Flow Rate N/A
Composition N/A Cup Size N/A

Technique
Bead Type Stringer Multi-pass or Single Pass Single
Travel Angle 10 - 15 Contact Tip to Work Distance 1"
Number of Electrodes 1
Electrode Spacing Longitudinal N/A Lateral N/A
Peening Not Permitted Cleaning N/A

Position
Groove N/A Vertical Up ☐ Down ☐
Fillet 2F

Preheat
Preheat Temp (min) 32 F
Thickness Up to 3/4" 32 F
Over 3/4" to 1 1/2" 50 F
Over 1 1/2" to 2 1/2" 150 F
Over 2 1/2" 225 F

Interpass Temperature
Min. N/A Max. N/A

Electrical Characteristics
GMAW Transfer Short Circuit ☐ Globular ☐ Spray ☐
Current AC ☐ DCEP ☑ DCEN ☐ Pulsed Yes ☐ No ☑
Other

Postweld Heat Treatment
Required Yes ☐ No ☑
Temp N/A Time N/A

Layer	Passes	Process	Filler Metal	Dia	Current	Amps	WFS	Volts	Travel	Other
1	1	SAW	EM13K	5/32"	DCEP	750	82 ipm	31	20 ipm	5/16" Fillet

Notes
1. Maximum single side autoweld fillet is 5/16".
2. For use when T1= or >5/16" and T2= or >3/4"
3. WPS utilized for SAW process using the Lincoln LT-7 Tractor Welder
4. LT-7 Control Knob Settings:
   A. Set Knob to 7.8 for 31 volts.
   B. Set Knob to 3 for 750 amps.
   C. Set Travel Knob @ 9 for 20 ipm.

Welding CWB Acceptance Sheet
CWB Accepted to CSA W47.1
MAY 1 6 2008
Acceptance valid only when Welding Consumables certified by C.W.B. (CL 11.8.1 CSA W47.1) mlb
AWS Compliant ☑

Company Authorization
PAUL KLIM
Lic. No. 006893
PROFESSIONAL ENGINEER

Month	Day	Year
04	29	08

ESSENTIAL VARIABLES RANGE FOR SAW FILLETS				
WPS AB3-Tractor LT-1		SAW	ELECTRODE= 5/32"	
Material Thickness	AMPS	WFS (ipm)	VOLTS	T.SPEED (ipm)
5/16"	630 - 770	46.8 - 57.2	29.3 - 33.7	15.3 - 20.7
TABLE VALUES FROM=>	Amps(+/- 10%)	WFS(+/- 10%)	Volts(+/- 7%)	Travel(+/- 15%)

American Buildings Company
Welding Procedure Specification

WPS No.	AB4-G3	Rev	8
Revised By:	Allen O'Steen, CWI	Date	1/31/2009
Issued By:	Vernon Hopper, CWI	Date	9/25/1996

Revisions to AB4-G3 Printed On: 2/23/2010 10:14:00 AM

Rev #	Date	By	Description
1	1/25/1999	C. Moshell	1. Added revisions sheet
			2. Changed base metals section to match joint detail, Thickness for groove welds, unlimited to unlimited
2	3/19/1999	C. Moshell	1. Clarification of layer / pass sequence
			2. Changed base metals section to reflect thickness range of established weld layering and pass sequence. Revised joint detail to match
3	3/3/2000	C. Moshell	1. Typographical changes
			2. Weld parameter changes to contact tube to work distance, wfs, volts, and travel. Added amperage readings.
			3. Changed position of groove to allow for 2G and 1G positions.
4	8/3/2000	C. Moshell	1. Corrected Arcworks software error in “as fit-up” root opening joint tolerance
5	5/10/2001	C. Moshell	1. Extended low end of qualification range to 3/16” minimum thickness to cover light W-shapes.
6	3/24/2004	C. Moshell	1. Added CSA standards information
			2. Changed material from A572 Grade 50 to A572 Grade 55
			3. Changed format of WPS data sheet
			4. Changed gas flow rate from 50 cfh to 40 cfh. Change limited material thickness to 1" for T1. New WPS(s) added for thicker materials
7	9/15/2008	A. O' Steen	Updated CSA W48.01 to CSA W48.06 & Updated Sketch.
8	1/31/2009	A. O'Steen	Changed Root Opening and Electrical Characteristics from Globular to Spray.

ESSENTIAL VARIABLES RANGE for FLANGE CORNER WELD

WPS AB4-G3 — GMAW — ELECTRODE = 0.045"

Material Thickness	AMPS	WFS (ipm)	VOLTS	T.SPEED (ipm)
All T (Side 1)	270 - 330	414 - 506	27.0 - 31.0	15.3 - 20.7
3/16"- 1/4" (Side 2)	270 - 330	414 - 506	27.0 - 31.0	15.3 - 20.7
5/16" (Side 2)	270 - 330	414 - 506	27.0 - 31.0	10.2 -13.8
3/8" (Side 2)	270 - 330	414 - 506	27.0 - 31.0	13.6 -18.4
1/2" (Side 2)	270 - 330	414 - 506	27.0 - 31.0	11.9 - 16.1
5/8" (Side 2)	270 - 330	414 - 506	27.0 - 31.0	10.2 - 13.8
3/4" (Side 2)	270 - 330	414 - 506	27.0 - 31.0	11.9 - 16.1
1" (Side 2)	270 - 330	414 - 506	27.0 - 31.0	11.9 - 16.1
TABLE VALUES FROM=>	Amps(+/- 10%)	WFS(+/- 10%)	Volts(+/- 7%)	Travel(+/- 15%)

Recommended ESSENTIAL VARIABLES for FLANGE CORNER WELD				
WPS AB4-G3		GMAW	ELECTRODE = 0.045"	
Material Thickness	AMPS	WFS (ipm)	VOLTS	T.SPEED (ipm)
All T (Side 1)	300	460	29	18
3/16" (Side 2)	300	415	29	20.7
1/4" (Side 2)	300	460	29	18
5/16" (Side 2)	300	480	29	12
3/8" (Side 2)	300	450	29	16
1/2" (Side 2)	300	415	29	14
5/8" (Side 2)	300	495	29	12
3/4" (Side 2)	300	495	29	12.5
1" (Side 2)	300	500	29	13

American Buildings Company
Welding Procedure Specification

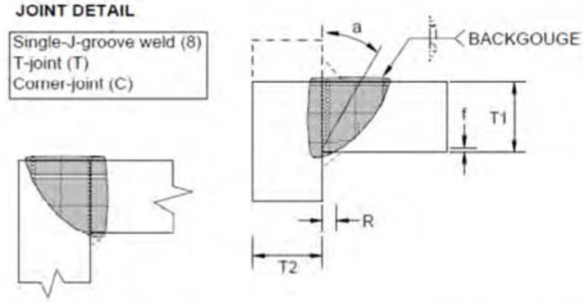
WPS No. **AB4-G3 .052** Rev **0**
Revised By: **Dave Volk CWI** Date **4/30/2014**
Issued By: **Dave Volk, CWI** Date **4/30/2014**

Welding Process (es) **GMAW** Ref. Standards: **AWS D1.1 & CSA W47.1** Manual ☐ Machine ☐
Supporting PQR(s) Prequalified ☑ SemiAuto ☑ Automatic ☐

Joint
Type **Single J, T- or Corner Joint**
Backing Yes ☐ No ☑ Single Weld ☐ Double Weld ☑
Back Material **N/A**
Root Opening (R) **0" to 1/8"** Root Face (f) **1/8"**
Groove Angle (α) **30** Groove Radius (r) **3/8"**
Back Gouge Yes ☑ No ☐
Method **Air Arc**

JOINT DETAIL

Single-J-groove weld (8)
T-joint (T)
Corner-joint (C)



T1 = 3/16" to 2"
T2 = unlimited

Tolerances As Fit-Up
R = + 1/16, - 1/8
α = + 10°, -5°
f = Not Limited
r = ± 1/16

Base Metals

	T1		T2
Material Specification	**A572**	to	**A572**
Type or Grade	**Grade 55**	to	**Grade 55**
Thickness: Groove	**3/16" to 1"**	to	**3/16" to Unl**
Fillet		to	
Diameter (Pipe)		to	

Filler Metals

	1	2
Specification	**AWS A5.18**	**CSA W48.06**
Classification	**ER70S-3**	**B-G 49A2**

Shielding
Flux **N/A** Electrode-Flux (Class) **N/A**
Shield Gas **Argon / CO2** Flow Rate **40 cfh**
Composition **92% Argon / 8% CO2** Cup Size **5/8"**

Position
Groove **1G or 2G** Vertical Up ☐ Down ☐
Fillet

Electrical Characteristics
GMAW Transfer Short Circut ☐ Globular ☐ Spray ☐
Current AC ☐ DCEP ☐ DCEN ☐ Pulsed Yes ☐ No ☑
Other

Coatings
Coating Type **None Allowed** Coating Thickness **N/A**

Technique
Bead Type **Stringer** Multi-pass or Single Pass **Both**
Travel Angle **15 Deg Push** Contact Tip to Work Distance **3/4"**
Number of Electrodes **1**
Electrode Spacing Longitudinal **N/A** Lateral **N/A**
Peening **Not Permitted** Cleaning **Wire brush as required**

Preheat
Preheat Temp (min) **32 F**
Thickness Up to 3/4" **32 F**
Over 3/4" to 1 1/2" **50F**
Over 1 1/2" to 2 1/2" **150 F**
Over 2 1/2" **225 F**

Interpass Temperature
Min. **32 F** Max. **550 F**

Postweld Heat Treatment
Required Yes ☐ No ☑
Tem **N/A** Time **N/A**

Layer	Passes	Process	FillerMetal	Dia	Current	Amps	WFS	Volts	Travel	Other
1 Layer	1 Pass	GMAW	ER70S-3	.052	DCEP	290 amps	320 ipm	26.6 v	24 ipm	Side 1 for all T
1 Layer	1 Pass	GMAW	ER70S-3	.052	DCEP	290 amps	320 ipm	26.6 v	24 ipm	Side 2 for 3/16" & 1/4" T
1 Layer	1 Pass	GMAW	ER70S-3	.052	DCEP	290 amps	320 ipm	26.6 v	24 ipm	Side 2 for 5/15" T
2 Layers	2 Passes	GMAW	ER70S-3	.052	DCEP	290 amps	320 ipm	26.6 v	24 ipm	Side 2 for 3/8" T
2 Layers	3 Passes	GMAW	ER70S-3	.052	DCEP	290 amps	320 ipm	26.6 v	21 ipm	Side 2 for 1/2" T
2 Layers	3 Passes	GMAW	ER70S-3	.052	DCEP	290 amps	320 ipm	26.6 v	20 ipm	Side 2 for 5/8" T
3 Layers	4 Passes	GMAW	ER70S-3	.052	DCEP	290 amps	320 ipm	26.6 v	17 ipm	Side 2 for 3/4" T
5 Layers	6 Passes	GMAW	ER70S-3	.052	DCEP	290 amps	320 ipm	26.6 v	14 imp	Side 2 for 1" T

Notes

Reinforcing fillets should be sized according to Chart A in Standard Weld Manual page 13.
Notes from D1.1 2010 Edition
(1) Not prequalified for GMAW, short circuiting transfer nor GTAW. (4) Backgouge root to sound metal before welding second side.
(7) If fillet welds are used in statically loaded structures to reinforce groove welds in corner and T-joints, these shall be equal to 1/4 T1, but need not exceed 3/8 inch. Groove welds in corner and T-joints of cyclically loaded structures shall be reinforced with fillet welds equal to 1/4 T1, but not more than 3/8 inch. (ABF-F1).
(10) The orientation in the joint may vary from 135 to 180 deg for butt joints, 45 to 135 deg for corner joints, or 45 to 90 deg for T-joints.
(11) For corner joints, the outside groove preparation may be in either or both members, provided the basic groove configuration is not changed and adequate edge distance is maintained to support the welding operation without excessive edge melting

CWB Acceptance

AWS Compliant

AWS
John Perl
CWI 08050431
QC1 EXP. 5/1/2017

Company Authorization

Month	Day	Year

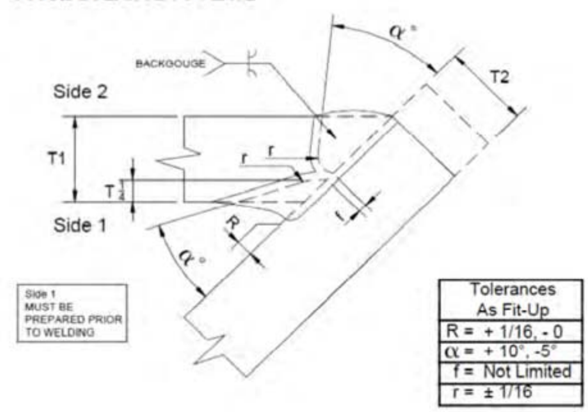
American Buildings Company
Welding Procedure Specification

WPS No. **AB4-G3b** Rev **2**
Revised By: **Allen O'Steen, CWI** Date **1/31/2009**
Issued By: **Chris Moshell, CWI** Date **1/5/2002**

Welding Process (es) **GMAW** Ref. Standards: **AWS D1.1 & CSA W47.1** Manual ☐ Machine ☐
Supporting PQR(s) Prequalified ☑ SemiAuto ☑ Automatic ☐

Joint
Type **Double J, T- or Corner Joint**
Backing Yes ☐ No ☑ Single Weld ☐ Double Weld ☑
Back Material **N/A**
Root Opening (R) **0"** Root Face (f) **1/8"**
Groove Angle (α) **30** Groove Radius (r) **3/8"**
Back Gouge Yes ☑ No ☐
Method **Air Arc**

OUTSIDE FLANGE TO PLATE



Base Metals

	T1		T2
Material Specification	A572	to	A572
Type or Grade	Grade 55	to	Grade 55
Thickness: Groove	1 1/4" to 2 1/4"	to	Unl
Fillet		to	
Diameter (Pipe)		to	

Filler Metals

	1	2
Specification	AWS A5.18	CSA W48.06
Classification	ER70S-3	B-G 49A2 CG3

Coatings
Coating Type **None Allowed** Coating Thickness **N/A**

Shielding
Flux **N/A** Electrode-Flux (Class) **N/A**
Shield Gas **Argon / CO2** Flow Rate **40 cfh**
Composition **92% Argon / 8% CO2** Cup Size **5/8"**

Technique
Bead Type **Stringer** Multi-pass or Single Pass **Multiple**
Travel Angle **15 Deg Push** Contact Tip to Work Distance **3/4"**
Number of Electrodes **1**
Electrode Spacing Longitudinal **N/A** Lateral **N/A**
Peening **Not Permitted** Cleaning **Wire brush as required**

Position
Groove **1G or 2G** Vertical Up ☐ Down ☐
Fillet **N/A**

Preheat
Preheat Temp (min) **32 F**
Thickness Up to 3/4" **32 F**
Over 3/4" to 1 1/2" **50 F**
Over 1 1/2" to 2 1/2" **150 F**
Over 2 1/2" **N/A**

Interpass Temperature
Min. **32 F** Max. **550 F**

Postweld Heat Treatment
Required Yes ☐ No ☑
Tem **N/A** Time **N/A**

Electrical Characteristics
GMAW Transfer Short Circut ☐ Globular ☐ Spray ☑
Current AC ☐ DCEP ☑ DCEN ☐ Pulsed Yes ☐ No ☑
Other

Layer	Passes	Process	FillerMetal	Dia	Current	Amps	WFS	Volts	Travel	Other
2 Layers	5 Passes	GMAW	ER70S-3	.052	DCEP	300 Amps	430 ipm	29 v	14 ipm	Side 1 1 1/4" to 1 3/4" T
5 Layers	9 Passes	GMAW	ER70S-3	.052	DCEP	300 Amps	430 ipm	29 v	14 ipm	Side 2 for T1 = 1 1/4"
5 Layers	12 Passes	GMAW	ER70S-3	.052"	DCEP	300 Amps	430 ipm	29 v	14 ipm	Side 2 for T1 = 1 1/2"
5 Layers	12 Passes	GMAW	ER70S-3	.052"	DCEP	300 Amps	430 ipm	29 v	14 ipm	Side 2 for T1 = 1 3/4"
7 Layers	14 Passes	GMAW	ER70S-3	.052"	DCEP	300 Amps	430 ipm	29 v	14 ipm	Side 1 for T1 = 2" to 2 1/4"
7 Layers	16 Passes	GMAW	ER70S-3	.052"	DCEP	300 Amps	430 ipm	29 v	14 ipm	Side 2 for T1 = 2"
8 Layers	20 Passes	GMAW	ER70S-3	.052	DCEP	300 Amps	430 ipm	29 v	14 ipm	Side 2 for T1 = 2 1/4"

Notes
1. This procedure is for flange to plate.
2. Maximum root pass thickness is 3/8". Maximum fill pass thickness is 1/4". If the layer width is greater than 5/8", then split the layer.
3. Preheat temperature determined based on thickest part of joint.

Notes from D1.1-2004:
(1) Not prequalified for GMAW using short circuiting transfer
(4) Backgouge root to sound metal before welding second side.
(8) [...unequal depth grooves..] the depth of the shallower groove shall be no less than 1/4 of the thickness of the thinner part joined.
(10) The orientation of members may vary from 45 deg to 135 deg.
(11) For corner joints, the outside groove preparation may be in either or both members, provided the basic groove configuration is not changed and adequate edge distance is maintained to support the welding operation without excessive edge melting.

CWB Acceptance
Welding Procedure Data Sheet
CWB Accepted to CSA W47.1
SEP 21 2009
Acceptance valid only when Welding Consumables certified by C.W.B. (Cl. 11.6.1, CSA W47.1)
AWS Compliant

AWS QC1 CHRISTOPHER S. MOSHELL CWI

Company Authorization
COMMONWEALTH OF VIRGINIA
PAUL KLIM
Lic. No. 006893
PROFESSIONAL ENGINEER

Month		Day		Year	
0	8	2	4	0	9

APPENDIX G SPECIMEN DESIGN CALCULATIONS

VIRGINIA TECH	44" DEEP BEAM 12ES-0.75-1.00-44	DATE 12/21/2016	MADE TAS	SHEET NO. 1 of 6

Limit States: Bolt Tension Rupture Without Prying Action

Plate Girder Material Properties

Elastic Modulus	E	29000	ksi	Yield Strength	F_y	55	ksi
Expected Yield Ratio	R_y	1.1		Ultimate Strength	F_u	70	ksi

Plate Girder Dimensions

Flange Width	b_f	12	in	Web Plate Height	h	42	in
Flange Thickness	t_f	1	in	Unbraced Length	L_B	12	in
Web Thickness	t_w	0.75	in	Column Face to CL Force	L_h	14.4375	ft
Min Length of Stiffener	$L_{st,min}$	8.227	in	Depth	d	44	in
Length of Stiffener	L_{st}	8.25	in				

End Plate Dimensions & Properties

Yield Strength	F_{py}	55	ksi	Flange to Outer Bolt CL	p_{fo}	1.5000	in
Width	b_p	12	in	Flange to Inner Bolt CL	p_{fi}	1.5000	in
Thickness	t_p	1.000	in	Bolt Spacing	p_b	2.250	in
Inner Bolt Gage	g	4.000	in	Outer Bolt to Edge	d_e	1.000	in
Outer Bolt Gage	g_o	2.250	in	Top of Flange to Edge	p_{ext}	4.7500	in

Bolt Properties

Bolt Grade		A490		Bolt Diameter	d_b	0.750	in
Tensile Strength	F_t	113	ksi	Bolt Pretension	T_b	35	kip
No. of Bolts at Each Flange	n_s	12				AISC 360-10 Table J3.1	

Resistance Factors

Bolt Tension & Shear	ϕ	0.75	End Plate Yielding	ϕ_b	0.9
Bolt Shear - Slip Critical	ϕ_{sc}	1	Shear	ϕ_v	0.9

Summary of Limit State & Capacities

						25% Bolt Over Strength	
Nominal Moment Section Capacity	M_n	39316	k-in	3276	k-ft		
Plastic Moment Section Capacity	M_p	46571	k-in	3881	k-ft		
End-Plate Yielding	M_{pl}	36733	k-in	3061	k-ft		
Bolt Tension Rupture Without Prying	M_{np}	25760	k-in	2147	k-ft	2683	k-ft
Bolt Tension Rupture With Prying	M_q	20119	k-in	1677	k-ft	2096	k-ft
Maximum Strain Hardening Plastic Moment	M_{pr}	58214	k-in	4851	k-ft		
Hardened Plastic Capacity	M_f	61497	k-in	5125	k-ft		
Moment Ratio	M_{pl}/M_f	0.597	> 1.11 (see Eqn 6.10-5 AISC 358-10)				
Moment Ratio	M_{np}/M_f	0.419	> 1.11 (see Eqn 6.10-4 AISC 358-10)				
Is Section Highly Ductile?	Yes						
Bolt Shear - Slip Critical	$\phi_{sc}R_n$	219.5	kip	Ok			
Bolt Shear Capacity	$\phi_v R_n$	270.4	kip	Ok			
Shear Capacity	$\phi_v V_n$	980.1	kip	Ok			
Concentrated Forces Controlling Capacity	$\phi R_{n.min}$	274.0	kip	No bearing stiffeners required			
Total Actuator Force	V_u	185.9	kip				

Plate Girder Properties

	Width		Height		Area		Distance From Centroid of Shape to Top of Plate Girder		Distance From Centroid of Shape to Centroid of Plate Girder	
Top Flange	12	in	1	in	12	in^2	0.5	in	21.5	in
Web	0.75	in	42	in	31.5	in^2	22	in	0	in
Bottom Flange	12	in	1	in	12	in^2	43.5	in	21.5	in

	Centroid	22.000	in	Measured from top of plate girder.
	Elastic Neutral Axis	22.000	in	Measured from top of plate girder.
	Plastic Neutral Axis	22.000	in	Measured from top of plate girder. Assume is in web.
	h_c	42.000	in	
	h_p	42.000	in	
Cross-Sectional Area	A	55.500	in^2	
Elastic Section Modulus	S_{xc}	714.841	in^3	
Plastic Section Modulus	Z_x	846.750	in^3	
Moment of Inertia, X-Axis	I_x	15726.500	in^4	
Moment of Inertia, Y-Axis	I_y	289.477	in^4	
Radius of Gyration, X-Axis	r_x	16.833	in	
Radius of Gyration, Y-Axis	r_y	2.284	in	
Tosional Constant	J	14.047	in^4	
	r_{ts}	2.951	in	

Slenderness

K_c 0.5345

F_L 38.5 ksi Verify that conditions are valid for F_L=0.7*F_y

	λ	λ_p	λ_r	
Flange	6.000	8.726	19.062	COMPACT
Web*	56.000	86.339	130.886	COMPACT

*Based upon doubly symmetric cross section

Highly Ductile Section Check AISC 341-10 Table D1.1

(b/t) flange = 0.3 sqrt (E/F_y)	6.89	Compression Ratio C_a	0	
$b_f/2t_f$	6.00	(h/t_w) Web Limit	56.26	2.45*sqrt(E/F_y)(1-0.93C_a)
Is Flange OK?	Yes	h/t_w	56.00	Assumes C_a < 0.125
		Is Web OK?	Yes	

Determine RPG

a_w	2.63	h_c/t_w	56.000
5.7*(E/Fy)^1/2	130.89	RPG	1.000

Limit State Analysis

Compression Flange Strength	F_{CR}	55.00	ksi

Yield

F_{CR}	55.00	ksi

Flange Local Buckling

F_{CR}	55.00	ksi

Lateral Torsional Buckling

F_{CR}	55.00

I_{CY}	144.25	in^4	A	17.25	in^2
r_t	2.89		C_b	1.00	
L_P	92.30	in	L_R	272.00	in

NO LTB

Bend-Buckling of the Web

F_{CR}	55.00		(IF Applicable) F_{CR}	302.93	ksi
$5.7*SQRT(E/F_{CR})$	56.00	=	h/tw	56.00	

NOT APPLICABLE

Plate Girder Moment Capacities

Nominal Moment Section Capacity	M_n	39316	k·in	3276.4	k·ft
Plastic Moment Section Capacity	M_p	46571	k-in	3880.9	k-ft

Moment at Column Face AISC 358-10 Sec. 2.4-3

Peak Connection Strength Factor	$C_{pr} = (F_y+F_u)/(2F_y)$	1.136	
Maximum Strain Hardening Plastic Moment	$M_{pr} = C_{pr}R_yF_yZ_x$	58214	k-in
Maximum Shear Force	$V_u = M_{pr}/(L_h\text{-}S_h)$	355.0	kip
Stiffener Height	p_{ext}	4.75	in
Length of Stiffener	L_{st}	8.250	in
Column Face to Plastic Hinge	$S_h = L_{st} + t_p$	9.250	in
Hardened Plastic Capacity	$M_f = M_{pr} + V_uS_h$	61497	k-in
	(AISC 358-10 6.10-1)		

Bolt Shear - Slip Critical

Mean Slip Coefficient: Class A	μ	0.3	
Force in Flange	F_{flange}	731.8	kip
Nominal Capacity	R_n	219.5	kip
Design Capacity	$\phi_{sc}R_n$	219.5	kip

Bolt Shear Capacity AISC 360-10 Sec. J3-6

Nominal Shear Strength	F_{nv}	68	ksi	AISC 360-10 Table J3.2
Nominal Capacity per bolt	R_n	30.0	kip	Group B bolts, when treads not excluded
Design Capacity	$\phi_v n_s R_n$	270.4	kip	

End Plate Yielding

distance, s	3.46	in	Which Case?	Case 3
γ	1		Type:	EXTENDED

Bolt Distance to Flange CL	h_1	47.250	in
	h_2	45.000	in
	h_3	41.000	in
	h_4	38.750	in
Bolt Distance to Flange CL	d_1	47.250	in
	d_2	45.000	in
	d_3	41.000	in
	d_4	38.750	in

Case 1

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_1}{2\cdot d_e} + \frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot(4\cdot d_e + 3\cdot p_b) + h_2\cdot(p_b + 4\cdot p_{fo}) + h_3\cdot(4\cdot p_{fi} + 3\cdot p_b) + h_4\cdot(p_b + 4\cdot s)\right] + g$$

Case 2

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_1}{s} + \frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot(4\cdot s + 3\cdot p_b) + h_2\cdot(p_b + 4\cdot p_{fo}) + h_3\cdot(4\cdot p_{fi} + 3\cdot p_b) + h_4\cdot(p_b + 4\cdot s)\right] + g$$

Case 3

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot(4\cdot d_e + 3\cdot p_b) + h_2\cdot(p_b + 4\cdot p_{fo}) + h_3\cdot(4\cdot p_{fi} + 3\cdot p_b) + h_4\cdot(p_b + 4\cdot s) + 4\cdot d_e^{2}\right] + \frac{3}{2}\cdot g$$

Case 1	Y_1	810.1	in
Case 2	Y_2	808.4	in
Case 3	Y_3	667.9	in
Controlling Case	Y	667.9	in
$M_{pl} = F_{py}\, t_p^{2}\, Y$	36733	k-in	

Bolt Rupture Without Prying Action

Bolt Force	P_t	49.9	kip	$P_t = \pi d_b^2 F_t / 4$
Moment Capacity	M_{np}	25760	k-in	

Bolt Rupture With Prying Action

Bolt Distribution Factors

α_1	1.0	β_1	1.0	γ_1	1.0	δ_1	1.0
α_2	-	β_2	0.5	γ_2	0.75	δ_2	-

Tributary Widths (inches)

$w_{\alpha1}$	3.125	$w_{\beta1}$	3.125	$w_{\gamma1}$	3.125	$w_{\delta1}$	3.125
$w'_{\alpha1}$	2.3125	$w'_{\beta1}$	2.3125	$w'_{\gamma1}$	2.3125	$w'_{\delta1}$	2.3125
		$w_{\beta2}$	2.875	$w_{\gamma2}$	2.875		
		$w'_{\beta2}$	2.0625	$w'_{\gamma2}$	2.0625		

a_o	3.250	in	a_i	8.643	in

$F_{o\alpha1}$	44.43	$F_{o\beta1}$	44.43	$F_{i\gamma1}$	44.43	$F_{i\delta1}$	44.43
		$F_{o\beta2}$	40.65	$F_{i\gamma2}$	40.65		

Prying Forces (kips)

$Q_{max,\alpha1}$	7.79	$Q_{max,\beta1}$	7.79	$Q_{max,\gamma1}$	2.93	$Q_{max,\delta1}$	2.93
		$Q_{max,\beta2}$	6.84	$Q_{max,\gamma2}$	2.57		

Prying In All Rows	20119	k-in
Prying In Rows Near Flange	18516	k-in
Prying in Row Outside of Flange	16773	k-in
Prying in Row Inside of Flange	17510	k-in
No Prying	15768	k-in
M_q	20119	k-in

Intermediate Stiffeners Check AISC 360-10 Chapt. G

Web Plate Shear Buckling Coefficient	k_v	5		
Web Shear Coefficient	C_v	1		
Web Area = $t_w d$	A_w	33	in^2	
Nominal Shear Capacity	V_n	1089.0	kip	
Design Shear Capacity	$\phi_v V_n$	980.1	kip	
Ultimate Shear	V_u	185.9	kip	Ok

Bearing Stiffeners - Concentrated Forces Checks AISC 360-10 Sec. J10

Flange Local Bending *From Yield Line Analysis*

Hole Spacing across Web	g	6.0625	in
Hole Spacing // Web	p_b	6.6875	in
Yield Line Spacing	s	3.865	in
Yield Line Parameter	Y	8.339	kip
Resistance Factor	ϕ_{FLB}	0.9	
Design Capacity	$\phi_{FLB} R_{n.FLB}$	412.8	kip

VIRGINIA TECH	44" DEEP BEAM 12ES-0.75-1.00-44	DATE 12/21/2016	MADE TAS	SHEET NO. 6 of 6

Web Local Yielding

Length of Bearing	l_b	18	in
Outside Flange Surface to Fillet Toe	k	1.0	in
Nominal Capacity	$R_{n.WLY}$	948.8	kip
Resistance Factor	ϕ_{WLY}	1.0	
Design Capacity	$\phi_{WLY}R_{n.WLY}$	948.8	kip

Web Crippling

Nominal Capacity	$R_{n.WC}$	1179.4	kip
Resistance Factor	ϕ_{WC}	0.75	
Design Capacity	$\phi_{WC}R_{n.WC}$	884.5	kip

Web Sidesway Buckling

WSB Coefficient	C_r	480000	ksi
Nominal Capacity	$R_{n.WSB}$	8064000.0	kip
Resistance Factor	ϕ_{WSB}	0.85	
Design Capacity	$\phi_{WSB}R_{n.WSB}$	6854400.0	kip

Web Compression Buckling

Nominal Capacity	$R_{n.WCB}$	304.5	kip
Resistance Factor	ϕ_{WCB}	0.9	
Design Capacity	$\phi_{WCB}R_{n.WCB}$	274.0	kip

Final Check

Controlling Capacity	$\phi R_{n.min}$	274.0	kip	
Ultimate Force	F_u	185.9	kip	No bearing stiffeners required

VIRGINIA TECH	44" DEEP BEAM 12ES-1.25-1.50-44	DATE 12/21/2016	MADE TAS	SHEET NO. 1 of 6

Limit States: Plastic Hinging of the Beam

Plate Girder Material Properties

Elastic Modulus	E	29000	ksi	Yield Strength	F_y	55	ksi
Expected Yield Ratio	R_y	1.1		Ultimate Strength	F_u	70	ksi

Plate Girder Dimensions

Flange Width	b_f	12	in	Web Plate Height	h	42	in
Flange Thickness	t_f	1	in	Unbraced Length	L_B	12	in
Web Thickness	t_w	0.75	in	Column Face to CL Force	L_h	14.4375	ft
Min Length of Stiffener	$L_{st,min}$	12.557	in	Depth	d	44	in
Length of Stiffener	L_{st}	12.625	in				

End Plate Dimensions & Properties

Yield Strength	F_{py}	55	ksi	Flange to Outer Bolt CL	p_{fo}	1.8750	in
Width	b_p	16	in	Flange to Inner Bolt CL	p_{fi}	1.8750	in
Thickness	t_p	1.500	in	Bolt Spacing	p_b	3.750	in
Inner Bolt Gage	g	4.500	in	Outer Bolt to Edge	d_e	1.625	in
Outer Bolt Gage	g_o	3.750	in	Top of Flange to Edge	p_{ext}	7.2500	in

Bolt Properties

Bolt Grade		A490		Bolt Diameter	d_b	1.250	in
Tensile Strength	F_t	113	ksi	Bolt Pretension	T_b	102	kip
No. of Bolts at Each Flange	n_s	12				AISC 360-10 Table J3.1	

Resistance Factors

Bolt Tension & Shear	ϕ	0.75	End Plate Yielding	ϕ_b	0.9
Bolt Shear - Slip Critical	ϕ_{sc}	1	Shear	ϕ_v	0.9

Summary of Limit State & Capacities

Nominal Moment Section Capacity	M_n	39316	k-in	3276	k-ft		
Plastic Moment Section Capacity	M_p	46571	k-in	3881	k-ft	25% Bolt Over	
End-Plate Yielding	M_{pl}	94486	k-in	7874	k-ft	Strength	
Bolt Tension Rupture Without Prying	M_{np}	71555	k-in	5963	k-ft	7454	k-ft
Bolt Tension Rupture With Prying	M_q	58459	k-in	4872	k-ft	6089	k-ft
Maximum Strain Hardening Plastic Moment	M_{pr}	58214	k-in	4851	k-ft		
Hardened Plastic Capacity	M_f	63382	k-in	5282	k-ft		
Moment Ratio	M_{pl}/M_f	1.491	> 1.11 (see Eqn 6.10-5 AISC 358-10)				
Moment Ratio	M_{np}/M_f	1.129	> 1.11 (see Eqn 6.10-4 AISC 358-10)				
Is Section Highly Ductile?	Yes						
Bolt Shear - Slip Critical	$\phi_{sc}R_n$	432.1	kip		Ok		
Bolt Shear Capacity	$\phi_v R_n$	751.0	kip		Ok		
Shear Capacity	$\phi_v V_n$	980.1	kip		Ok		
Concentrated Forces Controlling Capacity	$\phi R_{n.min}$	274.0	kip	* No bearing stiffeners required			
Total Actuator Force	V_u	365.8	kip	* Total of two actuators			

VIRGINIA TECH	44" DEEP BEAM 12ES-1.25-1.50-44	DATE 12/21/2016	MADE TAS	SHEET NO. 2 of 6

Plate Girder Properties

	Width		Height		Area		Distance From Centroid of Shape to Top of Plate Girder		Distance From Centroid of Shape to Centroid of Plate Girder	
Top Flange	12	in	1	in	12	in^2	0.5	in	21.5	in
Web	0.75	in	42	in	31.5	in^2	22	in	0	in
Bottom Flange	12	in	1	in	12	in^2	43.5	in	21.5	in

Centroid		22.000	in	Measured from top of plate girder.
Elastic Neutral Axis		22.000	in	Measured from top of plate girder.
Plastic Neutral Axis		22.000	in	Measured from top of plate girder. Assume is in web.
	h_c	42.000	in	
	h_p	42.000	in	
Cross-Sectional Area	A	55.500	in^2	
Elastic Section Modulus	S_{xc}	714.841	in^3	
Plastic Section Modulus	Z_x	846.750	in^3	
Moment of Inertia, X-Axis	I_x	15726.500	in^4	
Moment of Inertia, Y-Axis	I_y	289.477	in^4	
Radius of Gyration, X-Axis	r_x	16.833	in	
Radius of Gyration, Y-Axis	r_y	2.284	in	
Tosional Constant	J	14.047	in^4	
	r_{ts}	2.951	in	

Slenderness

K_c 0.5345

F_L 38.5 ksi Verify that conditions are valid for F_L=0.7*F_y

	λ	λ_p	λ_r	
Flange	6.000	8.726	19.062	COMPACT
Web*	56.000	86.339	130.886	COMPACT

*Based upon doubly symmetric cross section

Highly Ductile Section Check AISC 341-10 Table D1.1

(b/t) flange = 0.3 sqrt (E/F_y)	6.89	Compression Ratio C_a	0	
$b_f/2t_f$	6.00	(h/t_w) Web Limit	56.26	2.45*sqrt(E/F_y)(1-0.93C_a)
Is Flange OK?	Yes	h/t_w	56.00	Assumes C_a < 0.125
		Is Web OK?	Yes	

Determine RPG

a_w	2.63	h_c/t_w	56.000
5.7*(E/Fy)^1/2	130.89	RPG	1.000

Limit State Analysis

Compression Flange Strength	F_{CR}	55.00	ksi

Yield

F_{CR}	55.00	ksi

Flange Local Buckling

F_{CR}	55.00	ksi

Lateral Torsional Buckling

F_{CR}	55.00

I_{CY}	144.25	in^4	A	17.25	in^2
r_t	2.89		C_b	1.00	
L_P	92.30	in	L_R	272.00	in

NO LTB

Bend-Buckling of the Web

F_{CR}	55.00	(IF Applicable) F_{CR}		302.93	ksi
$5.7*SQRT(E/F_{CR})$	56.00	=	h/tw	56.00	

NOT APPLICABLE

Plate Girder Moment Capacities

Nominal Moment Section Capacity	M_n	39316	k·in	3276.4	k·ft
Plastic Moment Section Capacity	M_p	46571	k-in	3880.9	k-ft

Moment at Column Face AISC 358-10 Sec. 2.4-3

Peak Connection Strength Factor	$C_{pr} = (F_y+F_u)/(2F_y)$	1.136	
Maximum Strain Hardening Plastic Moment	$M_{pr} = C_{pr}R_yF_yZ_x$	58214	k-in
Maximum Shear Force	$V_u = M_{pr}/(L_h-S_h)$	365.8	kip
Stiffener Height	p_{ext}	7.25	in
Length of Stiffener	L_{st}	12.625	in
Column Face to Plastic Hinge	$S_h = L_{st} + t_p$	14.125	in
Hardened Plastic Capacity	$M_f = M_{pr} + V_uS_h$ (AISC 358-10 6.10-1)	63382	k-in

Bolt Shear - Slip Critical

Mean Slip Coefficient: Class A	μ	0.3	
Force in Flange	F_{flange}	1440.5	kip
Nominal Capacity	R_n	432.1	kip
Design Capacity	$\phi_{sc}R_n$	432.1	kip

Bolt Shear Capacity AISC 360-10 Sec. J3-6

Nominal Shear Strength	F_{nv}	68	ksi	AISC 360-10 Table J3.2
Nominal Capacity per bolt	R_n	83.4	kip	Group B bolts, when treads not excluded
Design Capacity	$\phi_v n_s R_n$	751.0	kip	

End Plate Yielding

distance, s	4.24	in	Which Case?	Case 3
γ	1		Type:	EXTENDED

Bolt Distance to Flange CL	h_1	49.125	in
	h_2	45.375	in
	h_3	40.625	in
	h_4	36.875	in
Bolt Distance to Flange CL	d_1	49.125	in
	d_2	45.375	in
	d_3	40.625	in
	d_4	36.875	in

Case 1

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_1}{2\cdot d_e} + \frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot(4\cdot d_e + 3\cdot p_b) + h_2\cdot(p_b + 4\cdot p_{fo}) + h_3\cdot(4\cdot p_{fi} + 3\cdot p_b) + h_4\cdot(p_b + 4\cdot s)\right] + g$$

Case 2

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_1}{s} + \frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot(4\cdot s + 3\cdot p_b) + h_2\cdot(p_b + 4\cdot p_{fo}) + h_3\cdot(4\cdot p_{fi} + 3\cdot p_b) + h_4\cdot(p_b + 4\cdot s)\right] + g$$

Case 3

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot(4\cdot d_e + 3\cdot p_b) + h_2\cdot(p_b + 4\cdot p_{fo}) + h_3\cdot(4\cdot p_{fi} + 3\cdot p_b) + h_4\cdot(p_b + 4\cdot s) + 4\cdot d_e^2\right] + \frac{3}{2}\cdot g$$

Case 1	Y_1	885.0	in
Case 1	Y_2	913.9	in
Case 3	Y_3	763.5	in
Controlling Case	Y	763.5	in
$M_{pl} = F_{py}\, t_p^2\, Y$	94486	k-in	

Bolt Rupture Without Prying Action

Bolt Force	P_t	138.7	kip	$P_t = \pi d_b^2 F_t / 4$
Moment Capacity	M_{np}	71555	k-in	

Bolt Rupture With Prying Action

Bolt Distribution Factors

α_1	1.0	β_1	1.0	γ_1	1.0	δ_1	1.0
α_2	-	β_2	0.5	γ_2	0.75	δ_2	-

Tributary Widths (inches)

$w_{\alpha1}$	4.125	$w_{\beta1}$	4.125	$w_{\gamma1}$	4.125	$w_{\delta1}$	4.125
$w'_{\alpha1}$	2.8125	$w'_{\beta1}$	2.8125	$w'_{\gamma1}$	2.8125	$w'_{\delta1}$	2.8125
		$w_{\beta2}$	3.875	$w_{\gamma2}$	3.875		
		$w'_{\beta2}$	2.5625	$w'_{\gamma2}$	2.5625		

a_o	5.375	in	a_i	6.277	in

$F_{o\alpha1}$	106.53	$F_{o\beta1}$	106.53	$F_{i\gamma1}$	106.53	$F_{i\delta1}$	106.53
		$F_{o\beta2}$	99.73	$F_{i\gamma2}$	99.73		

Prying Forces (kips)

$Q_{max,\alpha1}$	9.81	$Q_{max,\beta1}$	9.81	$Q_{max,\gamma1}$	8.40	$Q_{max,\delta1}$	8.40
		$Q_{max,\beta2}$	8.50	$Q_{max,\gamma2}$	7.28		

Prying In All Rows	58459	k-in
Prying In Rows Near Flange	53735	k-in
Prying in Row Outside of Flange	49647	k-in
Prying in Row Inside of Flange	50020	k-in
No Prying	45932	k-in
M_q	58459	k-in

Intermediate Stiffeners Check AISC 360-10 Chapt. G

Web Plate Shear Buckling Coefficient	k_v	5		
Web Shear Coefficient	C_v	1		
Web Area = $t_w d$	A_w	33	in^2	
Nominal Shear Capacity	V_n	1089.0	kip	
Design Shear Capacity	$\phi_v V_n$	980.1	kip	
Ultimate Shear	V_u	365.8	kip	Ok

Bearing Stiffeners - Concentrated Forces Checks AISC 360-10 Sec. J10

Flange Local Bending *From Yield Line Analysis*

Hole Spacing across Web	g	6.0625	in
Hole Spacing // Web	p_b	6.6875	in
Yield Line Spacing	s	3.865	in
Yield Line Parameter	Y	8.339	kip
Resistance Factor	ϕ_{FLB}	0.9	
Design Capacity	$\phi_{FLB}R_{n.FLB}$	412.8	kip

VIRGINIA TECH	44" DEEP BEAM 12ES-1.25-1.50-44	DATE 12/21/2016	MADE TAS	SHEET NO. 6 of 6

Web Local Yielding

Length of Bearing	l_b	18	in
Outside Flange Surface to Fillet Toe	k	1.0	in
Nominal Capacity	$R_{n.WLY}$	948.8	kip
Resistance Factor	ϕ_{WLY}	1.0	
Design Capacity	$\phi_{WLY}R_{n.WLY}$	948.8	kip

Web Crippling

Nominal Capacity	$R_{n.WC}$	1179.4	kip
Resistance Factor	ϕ_{WC}	0.75	
Design Capacity	$\phi_{WC}R_{n.WC}$	884.5	kip

Web Sidesway Buckling

WSB Coefficient	C_r	480000	ksi
Nominal Capacity	$R_{n.WSB}$	8064000.0	kip
Resistance Factor	ϕ_{WSB}	0.85	
Design Capacity	$\phi_{WSB}R_{n.WSB}$	6854400.0	kip

Web Compression Buckling

Nominal Capacity	$R_{n.WCB}$	304.5	kip
Resistance Factor	ϕ_{WCB}	0.9	
Design Capacity	$\phi_{WCB}R_{n.WCB}$	274.0	kip

Final Check

Controlling Capacity	$\phi R_{n.min}$	274.0	kip	
Ultimate Force	F_u	182.9	kip	No bearing stiffeners required
				* Total of two actuators

VIRGINIA TECH	24" DEEP BEAM 12ES-0.875-0.75-24	DATE 12/21/2016	MADE TAS	SHEET NO. 1 of 6

Limit States: End Plate Yielding & Bolt Rupture With Prying Action

Plate Girder Material Properties

Elastic Modulus	E	29000	ksi	Yield Strength	F_y	55	ksi
Expected Yield Ratio	R_y	1.1		Ultimate Strength	F_u	70	ksi

Plate Girder Dimensions

Flange Width	b_f	12	in	Web Plate Height	h	22	in
Flange Thickness	t_f	1	in	Unbraced Length	L_B	12	in
Web Thickness	t_w	0.5	in	Column Face to CL Force	L_h	14.4375	ft
Min Length of Stiffener	$L_{st,min}$	9.093	in	Depth	d	24	in
Length of Stiffener	L_{st}	9.125	in				

End Plate Dimensions & Properties

Yield Strength	F_{py}	55	ksi	Flange to Outer Bolt CL	p_{fo}	1.5000	in
Width	b_p	12	in	Flange to Inner Bolt CL	p_{fi}	1.5000	in
Thickness	t_p	0.750	in	Bolt Spacing	p_b	2.625	in
Inner Bolt Gage	g	3.500	in	Outer Bolt to Edge	d_e	1.125	in
Outer Bolt Gage	g_o	2.625	in	Top of Flange to Edge	p_{ext}	5.2500	in

Bolt Properties

Bolt Grade		A490		Bolt Diameter	d_b	0.875	in
Tensile Strength	F_t	113	ksi	Bolt Pretension	T_b	49	kip
No. of Bolts at Each Flange	n_s	12				AISC 360-10 Table J3.1	

Resistance Factors

Bolt Tension & Shear	ϕ	0.75	End Plate Yielding	ϕ_b	0.9
Bolt Shear - Slip Critical	ϕ_{sc}	1	Shear	ϕ_v	1

Summary of Limit State & Capacities

						25% Bolt Over Strength	
Nominal Moment Section Capacity	M_n	16590	k-in	1383	k-ft		
Plastic Moment Section Capacity	M_p	18508	k-in	1542	k-ft		
End-Plate Yielding	M_{pl}	11855	k-in	988	k-ft		
Bolt Tension Rupture Without Prying	M_{np}	18754	k-in	1563	k-ft	1954	k-ft
Bolt Tension Rupture With Prying	M_q	14872	k-in	1239	k-ft	1549	k-ft
Maximum Strain Hardening Plastic Moment	M_{pr}	23134	k-in	1928	k-ft		
Hardened Plastic Capacity	M_f	24533	k-in	2044	k-ft		
Moment Ratio	M_{pl}/M_f	0.483	> 1.11 (see Eqn 6.10-5 AISC 358-10)				
Moment Ratio	M_{np}/M_f	0.764	> 1.11 (see Eqn 6.10-4 AISC 358-10)				
Is Section Highly Ductile?	Yes						
Bolt Shear - Slip Critical	$\phi_{sc}R_n$	232.4	kip		Ok		
Bolt Shear Capacity	$\phi_v R_n$	368.0	kip		Ok		
Shear Capacity	$\phi_v V_n$	396.0	kip		Ok		
Concentrated Forces Controlling Capacity	$\phi R_{n.min}$	155.0	kip		No bearing stiffeners required		
Total Actuator Force	V_u	107.3	kip				

Plate Girder Properties

	Width		Height		Area		Distance From Centroid of Shape to Top of Plate Girder		Distance From Centroid of Shape to Centroid of Plate Girder	
Top Flange	12	in	1	in	12	in^2	0.5	in	11.5	in
Web	0.5	in	22	in	11	in^2	12	in	0	in
Bottom Flange	12	in	1	in	12	in^2	23.5	in	11.5	in

	Centroid	12.000	in	Measured from top of plate girder.
	Elastic Neutral Axis	12.000	in	Measured from top of plate girder.
	Plastic Neutral Axis	12.000	in	Measured from top of plate girder. Assume is in web.
	h_c	22.000	in	
	h_p	22.000	in	
Cross-Sectional Area	A	35.000	in^2	
Elastic Section Modulus	S_{xc}	301.639	in^3	
Plastic Section Modulus	Z_x	336.500	in^3	
Moment of Inertia, X-Axis	I_x	3619.667	in^4	
Moment of Inertia, Y-Axis	I_y	288.229	in^4	
Radius of Gyration, X-Axis	r_x	10.170	in	
Radius of Gyration, Y-Axis	r_y	2.870	in	
Tosional Constant	J	8.958	in^4	
	r_{ts}	3.315	in	

Slenderness

K_c 0.6030

F_L 38.5 ksi Verify that conditions are valid for F_L=0.7*F_y

	λ	λ_p	λ_r	
Flange	6.000	8.726	20.247	COMPACT
Web*	44.000	86.339	130.886	COMPACT

*Based upon doubly symmetric cross section

Highly Ductile Section Check AISC 341-10 Table D1.1

(b/t) flange = 0.3 sqrt (E/F_y)	6.89	Compression Ratio C_a	0	
$b_f/2t_f$	6.00	(h/t_w) Web Limit	56.26	2.45*sqrt(E/F_y)(1-0.93C_a)
Is Flange OK?	Yes	h/t_w	44.00	Assumes C_a < 0.125
		Is Web OK?	Yes	

Determine RPG

a_w	0.92	h_c/t_w	44.000
5.7*(E/Fy)^1/2	130.89	RPG	1.000

Limit State Analysis

Compression Flange Strength	F_{CR}	55.00	ksi

Yield

F_{CR}	55.00	ksi

Flange Local Buckling

F_{CR}	55.00	ksi

Lateral Torsional Buckling

F_{CR}	55.00

I_{CY}	144.04	in^4	A	13.83	in^2
r_t	3.23		C_b	1.00	
L_P	115.98	in	L_R	343.49	in

NO LTB

Bend-Buckling of the Web

F_{CR}	55.00		(IF Applicable) F_{CR}	490.70	ksi
5.7*SQRT(E/F_{CR})	44.00	=	h/tw	44.00	

NOT APPLICABLE

Plate Girder Moment Capacities

Nominal Moment Section Capacity	M_n	16590	k·in	1382.5	k·ft
Plastic Moment Section Capacity	M_p	18508	k-in	1542.3	k-ft

Moment at Column Face AISC 358-10 Sec. 2.4-3

Peak Connection Strength Factor	$C_{pr} = (F_y+F_u)/(2F_y)$	1.136	
Maximum Strain Hardening Plastic Moment	$M_{pr} = C_{pr}R_yF_yZ_x$	23134	k-in
Maximum Shear Force	$V_u = M_{pr}/(L_h\text{-}S_h)$	141.6	kip
Stiffener Height	p_{ext}	5.25	in
Length of Stiffener	L_{st}	9.125	in
Column Face to Plastic Hinge	$S_h = L_{st} + t_p$	9.875	in
Hardened Plastic Capacity	$M_f = M_{pr} + V_uS_h$	24533	k-in
	(AISC 358-10 6.10-1)		

Bolt Shear - Slip Critical

Mean Slip Coefficient: Class A	μ	0.3	
Force in Flange	F_{flange}	774.6	kip
Nominal Capacity	R_n	232.4	kip
Design Capacity	$\phi_{sc}R_n$	232.4	kip

Bolt Shear Capacity AISC 360-10 Sec. J3-6

Nominal Shear Strength	F_{nv}	68	ksi	AISC 360-10 Table J3.2
Nominal Capacity per bolt	R_n	40.9	kip	Group B bolts, when treads not excluded
Design Capacity	$\phi_v n_s R_n$	368.0	kip	

End Plate Yielding

distance, s	3.24	in	Which Case?	Case 3
γ	1		Type:	EXTENDED

Bolt Distance to Flange CL	h_1	27.625	in
	h_2	25.000	in
	h_3	21.000	in
	h_4	18.375	in
Bolt Distance to Flange CL	d_1	27.625	in
	d_2	25.000	in
	d_3	21.000	in
	d_4	18.375	in

Case 1

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_1}{2\cdot d_e} + \frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot\left(4\cdot d_e + 3\cdot p_b\right) + h_2\cdot\left(p_b + 4\cdot p_{fo}\right) + h_3\cdot\left(4\cdot p_{fi} + 3\cdot p_b\right) + h_4\cdot\left(p_b + 4\cdot s\right)\right] + g$$

Case 2

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_1}{s} + \frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot\left(4\cdot s + 3\cdot p_b\right) + h_2\cdot\left(p_b + 4\cdot p_{fo}\right) + h_3\cdot\left(4\cdot p_{fi} + 3\cdot p_b\right) + h_4\cdot\left(p_b + 4\cdot s\right)\right] + g$$

Case 3

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot\left(4\cdot d_e + 3\cdot p_b\right) + h_2\cdot\left(p_b + 4\cdot p_{fo}\right) + h_3\cdot\left(4\cdot p_{fi} + 3\cdot p_b\right) + h_4\cdot\left(p_b + 4\cdot s\right) + 4\cdot d_e^{\,2}\right] + \frac{3}{2}\cdot g$$

Case 1	Y_1	457.4	in
Case 2	Y_2	468.2	in
Case 3	Y_3	383.2	in
Controlling Case	Y	383.2	in
$M_{pl} = F_{py}\, t_p^{\,2}\, Y$	11855	k-in	

Bolt Rupture Without Prying Action

Bolt Force	P_t	67.9	kip	$P_t = \pi d_b^2 F_t / 4$
Moment Capacity	M_{np}	18754	k-in	

Bolt Rupture With Prying Action

Bolt Distribution Factors

α_1	1.0	β_1	1.0	γ_1	1.0	δ_1	1.0
α_2	-	β_2	0.5	γ_2	0.75	δ_2	-

Tributary Widths (inches)

$w_{\alpha1}$	3.0625	$w_{\beta1}$	3.0625	$w_{\gamma1}$	3.0625	$w_{\delta1}$	3.0625
$w'_{\alpha1}$	2.125	$w'_{\beta1}$	2.125	$w'_{\gamma1}$	2.125	$w'_{\delta1}$	2.125
		$w_{\beta2}$	2.9375	$w_{\gamma2}$	2.9375		
		$w'_{\beta2}$	2	$w'_{\gamma2}$	2		

a_o	2.234	in	a_i	2.234	in

$F_{o\alpha1}$	27.14	$F_{o\beta1}$	27.14	$F_{i\gamma1}$	27.14	$F_{i\delta1}$	27.14
		$F_{o\beta2}$	26.08	$F_{i\gamma2}$	26.08		

Prying Forces (kips)

$Q_{max,\alpha1}$	6.21	$Q_{max,\beta1}$	6.21	$Q_{max,\gamma1}$	6.21	$Q_{max,\delta1}$	6.21
		$Q_{max,\beta2}$	5.79	$Q_{max,\gamma2}$	5.79		

Prying In All Rows	14872	k-in
Prying In Rows Near Flange	13700	k-in
Prying in Row Outside of Flange	12750	k-in
Prying in Row Inside of Flange	12734	k-in
No Prying	11785	k-in
M_q	14872	k-in

Intermediate Stiffeners Check AISC 360-10 Chapt. G

Web Plate Shear Buckling Coefficient	k_v	5		
Web Shear Coefficient	C_v	1		
Web Area = $t_w d$	A_w	12	in^2	
Nominal Shear Capacity	V_n	396.0	kip	
Design Shear Capacity	$\phi_v V_n$	396.0	kip	
Ultimate Shear	V_u	107.3	kip	Ok

Bearing Stiffeners - Concentrated Forces Checks AISC 360-10 Sec. J10

Flange Local Bending *From Yield Line Analysis*

Hole Spacing across Web	g	6.0625	in
Hole Spacing // Web	p_b	6.6875	in
Yield Line Spacing	s	3.999	in
Yield Line Parameter	Y	8.156	kip
Resistance Factor	ϕ_{FLB}	0.9	
Design Capacity	$\phi_{FLB} R_{n.FLB}$	403.7	kip

Web Local Yielding

Length of Bearing	l_b	18	in
Outside Flange Surface to Fillet Toe	k	1.0	in
Nominal Capacity	$R_{n.WLY}$	632.5	kip
Resistance Factor	ϕ_{WLY}	1.0	
Design Capacity	$\phi_{WLY}R_{n.WLY}$	632.5	kip

Web Crippling

Nominal Capacity	$R_{n.WC}$	641.4	kip
Resistance Factor	ϕ_{WC}	0.75	
Design Capacity	$\phi_{WC}R_{n.WC}$	481.0	kip

Web Sidesway Buckling

WSB Coefficient	C_r	480000	ksi
Nominal Capacity	$R_{n.WSB}$	4224000.0	kip
Resistance Factor	ϕ_{WSB}	0.85	
Design Capacity	$\phi_{WSB}R_{n.WSB}$	3590400.0	kip

Web Compression Buckling

Nominal Capacity	$R_{n.WCB}$	172.2	kip
Resistance Factor	ϕ_{WCB}	0.9	
Design Capacity	$\phi_{WCB}R_{n.WCB}$	155.0	kip

Final Check

Controlling Capacity	$\phi R_{n.min}$	155.0	kip	
Ultimate Force	F_u	107.3	kip	No bearing stiffeners required

VIRGINIA TECH	24" DEEP BEAM 12ES-1.125-1.25-24	DATE 12/21/2016	MADE TAS	SHEET NO. 1 of 6

Limit States: Plastic Hinging of the Beam

Plate Girder Material Properties

Elastic Modulus	E	29000	ksi	Yield Strength	F_y	55	ksi
Expected Yield Ratio	R_y	1.1		Ultimate Strength	F_u	70	ksi

Plate Girder Dimensions

Flange Width	b_f	12	in	Web Plate Height	h	22	in
Flange Thickness	t_f	1	in	Unbraced Length	L_B	12	in
Web Thickness	t_w	0.5	in	Column Face to CL Force	L_h	14.4375	ft
Min Length of Stiffener	$L_{st,min}$	11.475	in	Depth	d	24	in
Length of Stiffener	L_{st}	11.5	in				

End Plate Dimensions & Properties

Yield Strength	F_{py}	55	ksi	Flange to Outer Bolt CL	p_{fo}	1.7500	in
Width	b_p	14	in	Flange to Inner Bolt CL	p_{fi}	1.7500	in
Thickness	t_p	1.250	in	Bolt Spacing	p_b	3.375	in
Inner Bolt Gage	g	4.000	in	Outer Bolt to Edge	d_e	1.500	in
Outer Bolt Gage	g_o	3.375	in	Top of Flange to Edge	p_{ext}	6.6250	in

Bolt Properties

Bolt Grade		A490		Bolt Diameter	d_b	1.125	in
Tensile Strength	F_t	113	ksi	Bolt Pretension	T_b	80	kip
No. of Bolts at Each Flange	n_s	12				AISC 360-10 Table J3.1	

Resistance Factors

Bolt Tension & Shear	ϕ	0.75	End Plate Yielding	ϕ_b	0.9
Bolt Shear - Slip Critical	ϕ_{sc}	1	Shear	ϕ_v	1

Summary of Limit State & Capacities

Nominal Moment Section Capacity	M_n	16590	k-in	1383	k-ft		
Plastic Moment Section Capacity	M_p	18508	k-in	1542	k-ft	25% Bolt Over	
End-Plate Yielding	M_{pl}	33931	k-in	2828	k-ft	Strength	
Bolt Tension Rupture Without Prying	M_{np}	31001	k-in	2583	k-ft	3229	k-ft
Bolt Tension Rupture With Prying	M_q	25280	k-in	2107	k-ft	2633	k-ft
Maximum Strain Hardening Plastic Moment	M_{pr}	23134	k-in	1928	k-ft		
Hardened Plastic Capacity	M_f	24972	k-in	2081	k-ft		
Moment Ratio	M_{pl}/M_f	1.359	> 1.11 (see Eqn 6.10-5 AISC 358-10)				
Moment Ratio	M_{np}/M_f	1.241	> 1.11 (see Eqn 6.10-4 AISC 358-10)				
Is Section Highly Ductile?	Yes						
Bolt Shear - Slip Critical	$\phi_{sc}R_n$	312.2	kip	Ok			
Bolt Shear Capacity	$\phi_v R_n$	608.3	kip	Ok			
Shear Capacity	$\phi_v V_n$	396.0	kip	Ok			
Concentrated Forces Controlling Capacity	$\phi R_{n.min}$	155.0	kip	No bearing stiffeners required			
Total Actuator Force	V_u	144.1	kip				

VIRGINIA TECH	24" DEEP BEAM 12ES-1.125-1.25-24	DATE 12/21/2016	MADE TAS	SHEET NO. 2 of 6

Plate Girder Properties

	Width		Height		Area		Distance From Centroid of Shape to Top of Plate Girder		Distance From Centroid of Shape to Centroid of Plate Girder	
Top Flange	12	in	1	in	12	in^2	0.5	in	11.5	in
Web	0.5	in	22	in	11	in^2	12	in	0	in
Bottom Flange	12	in	1	in	12	in^2	23.5	in	11.5	in

	Centroid	12.000	in	Measured from top of plate girder.
	Elastic Neutral Axis	12.000	in	Measured from top of plate girder.
	Plastic Neutral Axis	12.000	in	Measured from top of plate girder. Assume is in web.
	h_c	22.000	in	
	h_p	22.000	in	
Cross-Sectional Area	A	35.000	in^2	
Elastic Section Modulus	S_{xc}	301.639	in^3	
Plastic Section Modulus	Z_x	336.500	in^3	
Moment of Inertia, X-Axis	I_x	3619.667	in^4	
Moment of Inertia, Y-Axis	I_y	288.229	in^4	
Radius of Gyration, X-Axis	r_x	10.170	in	
Radius of Gyration, Y-Axis	r_y	2.870	in	
Tosional Constant	J	8.958	in^4	
	r_{ts}	3.315	in	

Slenderness

K_c 0.6030

F_L 38.5 ksi Verify that conditions are valid for F_L=0.7*F_y

	λ	λ_p	λ_r	
Flange	6.000	8.726	20.247	COMPACT
Web*	44.000	86.339	130.886	COMPACT

*Based upon doubly symmetric cross section

Highly Ductile Section Check AISC 341-10 Table D1.1

(b/t) flange = 0.3 sqrt (E/F_y)	6.89	Compression Ratio C_a	0	
$b_f/2t_f$	6.00	(h/t_w) Web Limit	56.26	2.45*sqrt(E/F_y)(1-0.93C_a)
Is Flange OK?	Yes	h/t_w	44.00	Assumes C_a < 0.125
		Is Web OK?	Yes	

Determine RPG

a_w	0.92	h_c/t_w	44.000
5.7*(E/Fy)^1/2	130.89	RPG	1.000

Limit State Analysis

Compression Flange Strength	F_{CR}	55.00	ksi
Yield			
	F_{CR}	55.00	ksi
Flange Local Buckling			
	F_{CR}	55.00	ksi
Lateral Torsional Buckling			
	F_{CR}	55.00	

I_{CY}	144.04	in^4	A	13.83	in^2
r_t	3.23		C_b	1.00	
L_P	115.98	in	L_R	343.49	in

NO LTB

Bend-Buckling of the Web

F_{CR}	55.00	(IF Applicable) F_{CR}		490.70	ksi
5.7*SQRT(E/F_{CR})	44.00	=	h/tw	44.00	

NOT APPLICABLE

Plate Girder Moment Capacities

Nominal Moment Section Capacity	M_n	16590	k·in	1382.5	k·ft
Plastic Moment Section Capacity	M_p	18508	k-in	1542.3	k-ft

Moment at Column Face AISC 358-10 Sec. 2.4-3

Peak Connection Strength Factor	$C_{pr} = (F_y+F_u)/(2F_y)$	1.136	
Maximum Strain Hardening Plastic Moment	$M_{pr} = C_{pr}R_yF_yZ_x$	23134	k-in
Maximum Shear Force	$V_u = M_{pr}/(L_h\text{-}S_h)$	144.1	kip
Stiffener Height	p_{ext}	6.625	in
Length of Stiffener	L_{st}	11.500	in
Column Face to Plastic Hinge	$S_h = L_{st} + t_p$	12.750	in
Hardened Plastic Capacity	$M_f = M_{pr} + V_uS_h$	24972	k-in
	(AISC 358-10 6.10-1)		

Bolt Shear - Slip Critical

Mean Slip Coefficient: Class A	μ	0.3	
Force in Flange	F_{flange}	1040.5	kip
Nominal Capacity	R_n	312.2	kip
Design Capacity	$\phi_{sc}R_n$	312.2	kip

Bolt Shear Capacity AISC 360-10 Sec. J3-6

Nominal Shear Strength	F_{nv}	68	ksi	AISC 360-10 Table J3.2
Nominal Capacity per bolt	R_n	67.6	kip	Group B bolts, when treads not excluded
Design Capacity	$\phi_v n_s R_n$	608.3	kip	

End Plate Yielding

Distance, s	3.742	in	Which Case?	Case 3
γ	1		Type:	EXTENDED

Bolt Distance to Flange CL	h_1	28.625	in
	h_2	25.250	in
	h_3	20.750	in
	h_4	17.375	in
Bolt Distance to Flange CL	d_1	28.625	in
	d_2	25.250	in
	d_3	20.750	in
	d_4	17.375	in

Case 1

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_1}{2\cdot d_e} + \frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot\left(4\cdot d_e + 3\cdot p_b\right) + h_2\cdot\left(p_b + 4\cdot p_{fo}\right) + h_3\cdot\left(4\cdot p_{fi} + 3\cdot p_b\right) + h_4\cdot\left(p_b + 4\cdot s\right)\right] + g$$

Case 2

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_1}{s} + \frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot\left(4\cdot s + 3\cdot p_b\right) + h_2\cdot\left(p_b + 4\cdot p_{fo}\right) + h_3\cdot\left(4\cdot p_{fi} + 3\cdot p_b\right) + h_4\cdot\left(p_b + 4\cdot s\right)\right] + g$$

Case 3

$$Y = \frac{b_p}{2}\cdot\left(\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2}\right) + \frac{1}{2\cdot g}\cdot\left[h_1\cdot\left(4\cdot d_e + 3\cdot p_b\right) + h_2\cdot\left(p_b + 4\cdot p_{fo}\right) + h_3\cdot\left(4\cdot p_{fi} + 3\cdot p_b\right) + h_4\cdot\left(p_b + 4\cdot s\right) + 4\cdot {d_e}^2\right] + \frac{3}{2}\cdot g$$

Case 1	Y_1	462.0	in
Case 2	Y_2	480.8	in
Case 3	Y_3	394.8	in
Controlling Case	Y	394.8	in
$M_{pl} = F_{py}\, t_p^2\, Y$	33931	k-in	

Bolt Rupture Without Prying Action

Bolt Force	P_t	112.3	kip	$P_t = \pi d_b^2 F_t / 4$
Moment Capacity	M_{np}	31001	k-in	

Bolt Rupture With Prying Action

Bolt Distribution Factors

α_1	1.0	β_1	1.0	γ_1	1.0	δ_1	1.0
α_2	-	β_2	0.5	γ_2	0.75	δ_2	-

Tributary Widths (inches)

$w_{\alpha1}$	3.6875	$w_{\beta1}$	3.6875	$w_{\gamma1}$	3.6875	$w_{\delta1}$	3.6875
$w'_{\alpha1}$	2.5	$w'_{\beta1}$	2.5	$w'_{\gamma1}$	2.5	$w'_{\delta1}$	2.5
		$w_{\beta2}$	3.3125	$w_{\gamma2}$	3.3125		
		$w'_{\beta2}$	2.125	$w'_{\gamma2}$	2.125		

a_o	4.875	in	a_i	4.966	in

$F_{o\alpha1}$	72.06	$F_{o\beta1}$	72.06	$F_{i\gamma1}$	72.06	$F_{i\delta1}$	72.06
		$F_{o\beta2}$	64.46	$F_{i\gamma2}$	64.46		

Prying Forces (kips)

$Q_{max,\alpha1}$	7.57	$Q_{max,\beta1}$	7.57	$Q_{max,\gamma1}$	7.44	$Q_{max,\delta1}$	7.44
		$Q_{max,\beta2}$	6.04	$Q_{max,\gamma2}$	5.93		

Prying In All Rows	25280	k-in
Prying In Rows Near Flange	22998	k-in
Prying in Row Outside of Flange	21144	k-in
Prying in Row Inside of Flange	21084	k-in
No Prying	19230	k-in
M_q	25280	k-in

Intermediate Stiffeners Check — AISC 360-10 Chapt. G

Web Plate Shear Buckling Coefficient	k_v	5		
Web Shear Coefficient	C_v	1		
Web Area = $t_w d$	A_w	12	in^2	
Nominal Shear Capacity	V_n	396.0	kip	
Design Shear Capacity	$\phi_v V_n$	396.0	kip	
Ultimate Shear	V_u	144.1	kip	Ok

Bearing Stiffeners - Concentrated Forces Checks — AISC 360-10 Sec. J10

Flange Local Bending — *From Yield Line Analysis*

Hole Spacing across Web	g	6.0625	in
Hole Spacing // Web	p_b	6.6875	in
Yield Line Spacing	s	3.999	in
Yield Line Parameter	Y	8.156	kip
Resistance Factor	ϕ_{FLB}	0.9	
Design Capacity	$\phi_{FLB}R_{n.FLB}$	403.7	kip

Web Local Yielding

Length of Bearing	l_b	18	in
Outside Flange Surface to Fillet Toe	k	1.0	in
Nominal Capacity	$R_{n.WLY}$	632.5	kip
Resistance Factor	ϕ_{WLY}	1.0	
Design Capacity	$\phi_{WLY}R_{n.WLY}$	632.5	kip

Web Crippling

Nominal Capacity	$R_{n.WC}$	641.4	kip
Resistance Factor	ϕ_{WC}	0.75	
Design Capacity	$\phi_{WC}R_{n.WC}$	481.0	kip

Web Sidesway Buckling

WSB Coefficient	C_r	480000	ksi
Nominal Capacity	$R_{n.WSB}$	4224000.0	kip
Resistance Factor	ϕ_{WSB}	0.85	
Design Capacity	$\phi_{WSB}R_{n.WSB}$	3590400.0	kip

Web Compression Buckling

Nominal Capacity	$R_{n.WCB}$	172.2	kip
Resistance Factor	ϕ_{WCB}	0.9	
Design Capacity	$\phi_{WCB}R_{n.WCB}$	155.0	kip

Final Check

Controlling Capacity	$\phi R_{n.min}$	155.0	kip	
Ultimate Force	F_u	144.1	kip	No bearing stiffeners required

44 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12.00$ in

Flange Thickness: $t_f := 1.00$ in

Web Thickness: $t_w := 0.75$ in

Total Beam Depth: $d := 44.00$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 42.00$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 31.5$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 22$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 43.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 21.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 21.5$ in

Plastic Section Modulus: $Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f}\right) + \left(\frac{A_w}{2}\right) \cdot \left(\frac{h_w}{4}\right)\right] = 847$ in^3

Section Nominal Moment Capacity: $M_n := 39316$ kip·in

Actuator Input:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Beam 1 - End Plate 12ES-0.75-1.00-44

Thick End-Plate Behavior

Limit State: Rupture of Bolts without Prying Action

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

End Plate Thickness: $t_p := 1.00$ in

End Plate Width: $b_p := 12.00$ in

Inner Bolt Gage: $g := 4.00$ in

Outer Bolt Gage: $g_o := 2.25$ in

Flange to Outer Bolt CL: $p_{fo} := 1.50$ in

Flange to Inner Bolt CL: $p_{fi} := 1.50$ in

Bolt Spacing: $p_b := 2.25$ in

Outer Bolt to Edge: $d_e := 1.00$ in

Top of Flange to Edge: $p_{ext} := p_{fo} + p_b + d_e = 4.75$ in

Length of Stiffener: $L_{st} := 8.25$ in

Top of Compression Flange to Tension Face Bolt Rows:

Outermost Row: $h_1 := d + p_{fo} + p_b = 47.75$ in

Outside of Flange: $h_2 := d + p_{fo} = 45.5$ in

Inside of Flange: $h_3 := d - t_f - p_{fi} = 41.5$ in

Innermost Row: $h_4 := h_2 - p_b = 43.25$ in

Centroid of Compression Flange to Tension Face Bolt Rows:

Outermost Row: $d_1 := d - \frac{t_f}{2} + p_{fo} + p_b = 47.25$ in

Outside of Flange: $d_2 := d - \frac{t_f}{2} + p_{fo} = 45$ in

Inside of Flange: $d_3 := d - \frac{3 \cdot t_f}{2} - p_{fi} = 41$ in

Innermost Row: $d_4 := d_2 - p_b = 42.75$ in

Bolts:

Bolt Grade: A490

Nominal Tensile Strength: $F_t := 113$ ksi

Bolt Diameter: $d_b := 0.75$ in

Bolt Pretension: $T_b := 35$ kip (AISC 360-10 Table J3.1)

$\phi_d := 1.00$ $\phi_n := 0.90$ $\phi_r := 0.75$

Bolt Rupture Without Prying Action:

$\Sigma d_n := 2\cdot\left(d_1 + 2\cdot d_2 + 2\cdot d_3 + d_4\right) = 524$ in

Bolt Force: $P_t := \frac{\pi}{4}\cdot d_b^{\,2}\cdot F_t = 49.9$ kip

$M_{np} := P_t\cdot\Sigma d_n = 26159$ kip·in

End-Plate Yield:

$s := \frac{1}{2}\cdot\sqrt{b_p\cdot g} = 3.464$ in

$$Y := \frac{b_p}{2}\cdot\left(\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2}\right) + \frac{1}{2\cdot g}\cdot\left[\begin{matrix} h_1\cdot\left(4\cdot d_e + 3\cdot p_b\right) + h_2\cdot\left(p_b + 4\cdot p_{fo}\right) \dots \\ + h_3\cdot\left(4\cdot p_{fi} + 3\cdot p_b\right) + h_4\cdot\left(p_b + 4\cdot s\right) + 4\cdot d_e^{\,2} \end{matrix}\right] + \frac{3}{2}\cdot g = 691 \quad \text{in}$$

$M_{pl} := F_{py}\cdot t_p^{\,2}\cdot Y = 37989$ kip·in

Bolt Rupture With Prying Action:

Bolt Distribution Factors: $\alpha_1 := 1.0$ $\beta_1 := 1.0$ $\gamma_1 := 1.0$ $\delta_1 := 1.0$

$\beta_2 := 0.5$ $\gamma_2 := 0.75$

Tributary Widths:

$w_{\alpha 1} := \frac{\left(g + g_o\right)}{2} = 3.125$ in

$w'_{\alpha 1} := w_{\alpha 1} - \left(d_b + \frac{1}{16}\right) = 2.313$ in

$w_{\beta 1} := \frac{\left(g + g_o\right)}{2} = 3.125$ in

$w'_{\beta 1} := w_{\beta 1} - \left(d_b + \frac{1}{16}\right) = 2.313$ in

$w_{\beta 2} := \frac{\left(b_p - g - g_o\right)}{2} = 2.875$ in

$w'_{\beta 2} := w_{\beta 2} - \left(d_b + \frac{1}{16}\right) = 2.063$ in

$$w_{\gamma1} := \frac{(g + g_o)}{2} = 3.125 \quad \text{in} \qquad w_{\gamma2} := \frac{(b_p - g - g_o)}{2} = 2.875 \quad \text{in}$$

$$w'_{\gamma1} := w_{\gamma1} - \left(d_b + \frac{1}{16}\right) = 2.313 \quad \text{in} \qquad w'_{\gamma2} := w_{\gamma2} - \left(d_b + \frac{1}{16}\right) = 2.063 \quad \text{in}$$

$$w_{\delta1} := \frac{(g + g_o)}{2} = 3.125 \quad \text{in}$$

$$w'_{\delta1} := w_{\delta1} - \left(d_b + \frac{1}{16}\right) = 2.313 \quad \text{in}$$

$$a_o := \min\left(\left[\begin{array}{c} 3.682\cdot\left(\frac{t_p}{d_b}\right)^3 - 0.085 \\ p_{ext} - p_{fo} \end{array}\right]\right) = 3.25 \quad \text{in} \qquad a_i := 3.682\cdot\left(\frac{t_p}{d_b}\right)^3 - 0.085 = 8.643 \quad \text{in}$$

$$F_{o\alpha1} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\alpha1} + 0.8\cdot w'_{\alpha1}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fo}} = 44.427 \quad \text{kip}$$

$$F_{o\beta1} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\beta1} + 0.8\cdot w'_{\beta1}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fo}} = 44.427 \quad \text{kip}$$

$$F_{o\beta2} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\beta2} + 0.8\cdot w'_{\beta2}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fo}} = 40.646 \quad \text{kip}$$

$$F_{i\gamma1} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\gamma1} + 0.8\cdot w'_{\gamma1}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fi}} = 44.427 \quad \text{kip}$$

$$F_{i\gamma2} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\gamma2} + 0.8\cdot w'_{\gamma2}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fi}} = 40.646 \quad \text{kip}$$

$$F_{i\delta1} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\delta1} + 0.8\cdot w'_{\delta1}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fi}} = 44.427 \quad \text{kip}$$

Prying Forces:

$$Q_{max.\alpha1} := \frac{w'_{\alpha1}\cdot t_p^{\,2}}{4\cdot a_o}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{o\alpha1}}{w'_{\alpha1}\cdot t_p}\right)^2} = 7.79 \qquad \text{kip}$$

$$Q_{max.\beta1} := \frac{w'_{\beta1}\cdot t_p^{\,2}}{4\cdot a_o}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{o\beta1}}{w'_{\beta1}\cdot t_p}\right)^2} = 7.79 \qquad \text{kip}$$

$$Q_{max.\beta2} := \frac{w'_{\beta2}\cdot t_p^{\,2}}{4\cdot a_o}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{o\beta2}}{w'_{\beta2}\cdot t_p}\right)^2} = 6.842 \qquad \text{kip}$$

$$Q_{max.\gamma1} := \frac{w'_{\gamma1}\cdot t_p^{\,2}}{4\cdot a_i}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{i\gamma1}}{w'_{\gamma1}\cdot t_p}\right)^2} = 2.929 \qquad \text{kip}$$

$$Q_{max.\gamma2} := \frac{w'_{\gamma2}\cdot t_p^{\,2}}{4\cdot a_i}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{i\gamma2}}{w'_{\gamma2}\cdot t_p}\right)^2} = 2.573 \qquad \text{kip}$$

$$Q_{max.\delta1} := \frac{w'_{\delta1}\cdot t_p^{\,2}}{4\cdot a_i}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{i\delta1}}{w'_{\delta1}\cdot t_p}\right)^2} = 2.929 \qquad \text{kip}$$

$$M_q := \max\left[\left[\begin{array}{c}
2\cdot\alpha_1\cdot\left(P_t - Q_{max.\alpha1}\right)\cdot d_1 + 2\cdot\left[\beta_1\cdot\left(P_t - Q_{max.\beta1}\right) + \beta_2\cdot\left(P_t - Q_{max.\beta2}\right)\right]\cdot d_2 \;... \\
+\,2\cdot\left[\gamma_1\cdot\left(P_t - Q_{max.\gamma1}\right) + \gamma_2\cdot\left(P_t - Q_{max.\gamma2}\right)\right]\cdot d_3 + 2\cdot\delta_1\cdot\left(P_t - Q_{max.\delta1}\right)\cdot d_4 \\
2\cdot\alpha_1\cdot T_b\cdot d_1 + 2\cdot\left[\beta_1\cdot\left(P_t - Q_{max.\beta1}\right) + \beta_2\cdot\left(P_t - Q_{max.\beta2}\right)\right]\cdot d_2 \;... \\
+\,2\cdot\left[\gamma_1\cdot\left(P_t - Q_{max.\gamma1}\right) + \gamma_2\cdot\left(P_t - Q_{max.\gamma2}\right)\right]\cdot d_3 + 2\cdot\delta_1\cdot T_b\cdot d_4 \\
2\cdot\alpha_1\cdot T_b\cdot d_1 + 2\cdot\left[\beta_1\cdot\left(P_t - Q_{max.\beta1}\right) + \beta_2\cdot\left(P_t - Q_{max.\beta2}\right)\right]\cdot d_2 \;... \\
+\,2\cdot\left(\gamma_1 + \gamma_2\right)\cdot T_b\cdot d_3 + 2\cdot\delta_1\cdot T_b\cdot d_4 \\
2\cdot\alpha_1\cdot T_b\cdot d_1 + 2\cdot\left(\beta_1 + \beta_2\right)\cdot T_b\cdot d_2 \;... \\
+\,2\cdot\left[\gamma_1\cdot\left(P_t - Q_{max.\gamma1}\right) + \gamma_2\cdot\left(P_t - Q_{max.\gamma2}\right)\right]\cdot d_3 + 2\cdot\delta_1\cdot T_b\cdot d_4 \\
2\cdot T_b\cdot\left[\left(\alpha_1\right)\cdot d_1 + \left(\beta_1 + \beta_2\right)\cdot d_2 + \left(\gamma_1 + \gamma_2\right)\cdot d_3 + \left(\delta_1\right)\cdot d_4\right]
\end{array}\right]\right] = 20495 \quad \text{kip}\cdot\text{in}$$

<u>Moment at Column Face</u>:

Peak Connection Strength Factor: $C_{pr} := \min\left[\frac{\left(F_y + F_u\right)}{2 \cdot F_y}, 1.2\right] = 1.136$ AISC 358-10 (2.4.3-2)

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 46571$ $\text{kip} \cdot \text{in}$

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 58214$ $\text{kip} \cdot \text{in}$

AISC 358-10 (2.4.3-1)

Column Face to Plastic Hinge: $S_h := L_{st} + t_p = 9.25$ in

Maximum Shear Force: $V_u := \frac{M_{pr}}{L_h - S_h} = 354.964$ kip

Hardened Plastic Capacity: $M_f := M_{pr} + V_u \cdot S_h = 61497$ $\text{kip} \cdot \text{in}$

AISC 358-10 (6.10-1)

Summary of Limit States & Capacities

Beam 1 - End Plate 12ES-0.75-1.00-44

Limit State: Rupture of Bolts without Prying Action

	kip·in	kip·ft	
Nominal Moment Capacity of Rafter	$M_n = 39316$	$\frac{M_n}{12} = 3276$	
Plastic Moment Capacity of Rafter	$M_p = 46571$	$\frac{M_p}{12} = 3881$	
Moment Capacity for End-Plate Yielding	$M_{pl} = 37989$	$\frac{M_{pl}}{12} = 3166$	
Moment Capacity for Bolt Tenion Rupture without Prying	$M_{np} = 26159$	$\frac{M_{np}}{12} = 2180$	$\frac{1.25 \cdot M_{np}}{12} = 2725$
Moment Capacity for Bolt Rupture with Prying Action	$M_q = 20495$	$\frac{M_q}{12} = 1708$	$\frac{1.25 \cdot M_q}{12} = 2135$
Maximum Plastic Moment Including Strain Hardening	$M_{pr} = 58214$	$\frac{M_{pr}}{12} = 4851$	
Hardened Plastic Capacity	$M_f = 61497$	$\frac{M_f}{12} = 5125$	

$$d_{b.req} := \sqrt{\frac{4 \cdot M_f}{\pi \cdot \phi_n \cdot F_t \cdot \Sigma d_n}} = 1.212 \quad \text{in}$$

AISC 358-10 (6.10-4)

$$t_{p.req} := \sqrt{\frac{1.11 \cdot M_f}{\phi_d \cdot F_{py} \cdot Y}} = 1.34 \quad \text{in}$$

AISC 358-10 (6.10-5)

Rafter Input:

$b_f = 12.00$ in

$t_f = 1.00$ in

$t_w = 0.75$ in

$d = 44.00$ in

End Plate Input:

$t_p = 1.00$ in

Bolt Pretension:

$T_b = 35$ kip

(AISC 360-10 Table J3.1)

Bolts:

Bolt Grade A490

$F_t = 113$ ksi

$d_b = 0.75$ in

44 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12.00$ in

Flange Thickness: $t_f := 1.00$ in

Web Thickness: $t_w := 0.75$ in

Total Beam Depth: $d := 44.00$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 42.00$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 31.5$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 22$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 43.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 21.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 21.5$ in

Plastic Section Modulus: $Z_x := 2 \cdot \left[A_{top.f} \cdot (C_w - C_{top.f}) + \left(\frac{A_w}{2}\right) \cdot \left(\frac{h_w}{4}\right)\right] = 847$ in^3

Section Nominal Moment Capacity: $M_n := 39316$ $kip \cdot in$

Actuator Input:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Beam 1 - End Plate 12ES-1.25-1.50-44

Thick End-Plate Behavior

Limit State: Plastic Hinging of Beam

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

End Plate Thickness: $t_p := 1.50$ in

End Plate Width: $b_p := 16.00$ in

Inner Bolt Gage: $g := 4.50$ in

Outer Bolt Gage: $g_o := 3.75$ in

Flange to Outer Bolt CL: $p_{fo} := 1.875$ in

Flange to Inner Bolt CL: $p_{fi} := 1.875$ in

Bolt Spacing: $p_b := 3.75$ in

Outer Bolt to Edge: $d_e := 1.625$ in

Top of Flange to Edge: $p_{ext} := p_{fo} + p_b + d_e = 7.25$ in

Length of Stiffener: $L_{st} := 12.625$ in

Top of Compression Flange to Tension Face Bolt Rows:

Outermost Row: $h_1 := d + p_{fo} + p_b = 49.625$ in

Outside of Flange: $h_2 := d + p_{fo} = 45.875$ in

Inside of Flange: $h_3 := d - t_f - p_{fi} = 41.125$ in

Innermost Row: $h_4 := h_2 - p_b = 42.125$ in

Centroid of Compression Flange to Tension Face Bolt Rows:

Outermost Row: $d_1 := d - \frac{t_f}{2} + p_{fo} + p_b = 49.125$ in

Outside of Flange: $d_2 := d - \frac{t_f}{2} + p_{fo} = 45.375$ in

Inside of Flange: $d_3 := d - \frac{3 \cdot t_f}{2} - p_{fi} = 40.625$ in

Innermost Row: $d_4 := d_2 - p_b = 41.625$ in

Bolts:

Bolt Grade: A490

Nominal Tensile Strength: $F_t := 113$ ksi

Bolt Diameter: $d_b := 1.25$ in

Bolt Pretension: $T_b := 102$ kip (AISC 360-10 Table J3.1)

$\phi_d := 1.00$ $\phi_n := 0.90$ $\phi_r := 0.75$

Bolt Rupture Without Prying Action:

$\Sigma d_n := 2\cdot\left(d_1 + 2\cdot d_2 + 2\cdot d_3 + d_4\right) = 525.5$ in

Bolt Force: $P_t := \frac{\pi}{4}\cdot {d_b}^2\cdot F_t = 138.7$ kip

$M_{np} := P_t\cdot \Sigma d_n = 72872$ kip·in

End-Plate Yield:

$s := \frac{1}{2}\cdot\sqrt{b_p\cdot g} = 4.243$ in

$$Y := \frac{b_p}{2}\cdot\left(\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2}\right) + \frac{1}{2\cdot g}\cdot\left[\begin{array}{l} h_1\cdot\left(4\cdot d_e + 3\cdot p_b\right) + h_2\cdot\left(p_b + 4\cdot p_{fo}\right) \ldots \\ + h_3\cdot\left(4\cdot p_{fi} + 3\cdot p_b\right) + h_4\cdot\left(p_b + 4\cdot s\right) + 4\cdot {d_e}^2 \end{array}\right] + \frac{3}{2}\cdot g = 792 \quad \text{in}$$

$M_{pl} := F_{py}\cdot {t_p}^2\cdot Y = 98063$ kip·in

Bolt Rupture With Prying Action:

Bolt Distribution Factors: $\alpha_1 := 1.0$ $\beta_1 := 1.0$ $\gamma_1 := 1.0$ $\delta_1 := 1.0$

$\beta_2 := 0.5$ $\gamma_2 := 0.75$

Tributary Widths:

$w_{\alpha 1} := \frac{\left(g + g_o\right)}{2} = 4.125$ in

$w'_{\alpha 1} := w_{\alpha 1} - \left(d_b + \frac{1}{16}\right) = 2.813$ in

$w_{\beta 1} := \frac{\left(g + g_o\right)}{2} = 4.125$ in

$w'_{\beta 1} := w_{\beta 1} - \left(d_b + \frac{1}{16}\right) = 2.813$ in

$w_{\beta 2} := \frac{\left(b_p - g - g_o\right)}{2} = 3.875$ in

$w'_{\beta 2} := w_{\beta 2} - \left(d_b + \frac{1}{16}\right) = 2.563$ in

$$w_{\gamma 1} := \frac{(g + g_o)}{2} = 4.125 \quad \text{in}$$

$$w_{\gamma 2} := \frac{(b_p - g - g_o)}{2} = 3.875 \quad \text{in}$$

$$w'_{\gamma 1} := w_{\gamma 1} - \left(d_b + \frac{1}{16}\right) = 2.813 \quad \text{in}$$

$$w'_{\gamma 2} := w_{\gamma 2} - \left(d_b + \frac{1}{16}\right) = 2.563 \quad \text{in}$$

$$w_{\delta 1} := \frac{(g + g_o)}{2} = 4.125 \quad \text{in}$$

$$w'_{\delta 1} := w_{\delta 1} - \left(d_b + \frac{1}{16}\right) = 2.813 \quad \text{in}$$

$$a_o := \min\left(\begin{bmatrix} 3.682 \cdot \left(\frac{t_p}{d_b}\right)^3 - 0.085 \\ p_{ext} - p_{fo} \end{bmatrix}\right) = 5.375 \quad \text{in}$$

$$a_i := 3.682 \cdot \left(\frac{t_p}{d_b}\right)^3 - 0.085 = 6.277 \quad \text{in}$$

$$F_{o\alpha 1} := \frac{t_p^{\,2} \cdot F_{py} \cdot (0.85 \cdot w_{\alpha 1} + 0.8 \cdot w'_{\alpha 1}) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fo}} = 106.534 \quad \text{kip}$$

$$F_{o\beta 1} := \frac{t_p^{\,2} \cdot F_{py} \cdot (0.85 \cdot w_{\beta 1} + 0.8 \cdot w'_{\beta 1}) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fo}} = 106.534 \quad \text{kip}$$

$$F_{o\beta 2} := \frac{t_p^{\,2} \cdot F_{py} \cdot (0.85 \cdot w_{\beta 2} + 0.8 \cdot w'_{\beta 2}) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fo}} = 99.728 \quad \text{kip}$$

$$F_{i\gamma 1} := \frac{t_p^{\,2} \cdot F_{py} \cdot (0.85 \cdot w_{\gamma 1} + 0.8 \cdot w'_{\gamma 1}) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fi}} = 106.534 \quad \text{kip}$$

$$F_{i\gamma 2} := \frac{t_p^{\,2} \cdot F_{py} \cdot (0.85 \cdot w_{\gamma 2} + 0.8 \cdot w'_{\gamma 2}) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fi}} = 99.728 \quad \text{kip}$$

$$F_{i\delta 1} := \frac{t_p^{\,2} \cdot F_{py} \cdot (0.85 \cdot w_{\delta 1} + 0.8 \cdot w'_{\delta 1}) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fi}} = 106.534 \quad \text{kip}$$

Prying Forces:

$$Q_{max.\alpha 1} := \frac{w'_{\alpha 1}\cdot t_p^{\,2}}{4\cdot a_o}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{o\alpha 1}}{w'_{\alpha 1}\cdot t_p}\right)^2} = 9.815 \qquad \text{kip}$$

$$Q_{max.\beta 1} := \frac{w'_{\beta 1}\cdot t_p^{\,2}}{4\cdot a_o}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{o\beta 1}}{w'_{\beta 1}\cdot t_p}\right)^2} = 9.815 \qquad \text{kip}$$

$$Q_{max.\beta 2} := \frac{w'_{\beta 2}\cdot t_p^{\,2}}{4\cdot a_o}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{o\beta 2}}{w'_{\beta 2}\cdot t_p}\right)^2} = 8.504 \qquad \text{kip}$$

$$Q_{max.\gamma 1} := \frac{w'_{\gamma 1}\cdot t_p^{\,2}}{4\cdot a_i}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{i\gamma 1}}{w'_{\gamma 1}\cdot t_p}\right)^2} = 8.404 \qquad \text{kip}$$

$$Q_{max.\gamma 2} := \frac{w'_{\gamma 2}\cdot t_p^{\,2}}{4\cdot a_i}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{i\gamma 2}}{w'_{\gamma 2}\cdot t_p}\right)^2} = 7.281 \qquad \text{kip}$$

$$Q_{max.\delta 1} := \frac{w'_{\delta 1}\cdot t_p^{\,2}}{4\cdot a_i}\cdot\sqrt{F_{py}^{\,2} - 3\cdot\left(\frac{F_{i\delta 1}}{w'_{\delta 1}\cdot t_p}\right)^2} = 8.404 \qquad \text{kip}$$

$$M_q := \max\left[\begin{array}{c}
2\cdot\alpha_1\cdot\left(P_t - Q_{max.\alpha 1}\right)\cdot d_1 + 2\cdot\left[\beta_1\cdot\left(P_t - Q_{max.\beta 1}\right) + \beta_2\cdot\left(P_t - Q_{max.\beta 2}\right)\right]\cdot d_2 \cdots \\
+ 2\cdot\left[\gamma_1\cdot\left(P_t - Q_{max.\gamma 1}\right) + \gamma_2\cdot\left(P_t - Q_{max.\gamma 2}\right)\right]\cdot d_3 + 2\cdot\delta_1\cdot\left(P_t - Q_{max.\delta 1}\right)\cdot d_4 \\
2\cdot\alpha_1\cdot T_b\cdot d_1 + 2\cdot\left[\beta_1\cdot\left(P_t - Q_{max.\beta 1}\right) + \beta_2\cdot\left(P_t - Q_{max.\beta 2}\right)\right]\cdot d_2 \cdots \\
+ 2\cdot\left[\gamma_1\cdot\left(P_t - Q_{max.\gamma 1}\right) + \gamma_2\cdot\left(P_t - Q_{max.\gamma 2}\right)\right]\cdot d_3 + 2\cdot\delta_1\cdot T_b\cdot d_4 \\
2\cdot\alpha_1\cdot T_b\cdot d_1 + 2\cdot\left[\beta_1\cdot\left(P_t - Q_{max.\beta 1}\right) + \beta_2\cdot\left(P_t - Q_{max.\beta 2}\right)\right]\cdot d_2 \cdots \\
+ 2\cdot\left(\gamma_1 + \gamma_2\right)\cdot T_b\cdot d_3 + 2\cdot\delta_1\cdot T_b\cdot d_4 \\
2\cdot\alpha_1\cdot T_b\cdot d_1 + 2\cdot\left(\beta_1 + \beta_2\right)\cdot T_b\cdot d_2 \cdots \\
+ 2\cdot\left[\gamma_1\cdot\left(P_t - Q_{max.\gamma 1}\right) + \gamma_2\cdot\left(P_t - Q_{max.\gamma 2}\right)\right]\cdot d_3 + 2\cdot\delta_1\cdot T_b\cdot d_4 \\
2\cdot T_b\cdot\left[\left(\alpha_1\right)\cdot d_1 + \left(\beta_1 + \beta_2\right)\cdot d_2 + \left(\gamma_1 + \gamma_2\right)\cdot d_3 + \left(\delta_1\right)\cdot d_4\right]
\end{array}\right] = 59696 \quad \text{kip}\cdot\text{in}$$

<u>Moment at Column Face</u>:

Peak Connection Strength Factor: $C_{pr} := \min\left[\frac{(F_y + F_u)}{2 \cdot F_y}, 1.2\right] = 1.136$ AISC 358-10 (2.4.3-2)

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 46571 \quad kip \cdot in$

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 58214 \quad kip \cdot in$

AISC 358-10 (2.4.3-1)

Column Face to Plastic Hinge: $S_h := L_{st} + t_p = 14.125 \quad in$

Maximum Shear Force: $V_u := \frac{M_{pr}}{L_h - S_h} = 365.839 \quad kip$

Hardened Plastic Capacity: $M_f := M_{pr} + V_u \cdot S_h = 63382 \quad kip \cdot in$

AISC 358-10 (6.10-1)

Summary of Limit States & Capacities

Beam 1 - End Plate 12ES-1.25-1.50-44

Limit State: Plastic Hinging of Beam

	kip·in	kip·ft	
Nominal Moment Capacity of Rafter	$M_n = 39316$	$\frac{M_n}{12} = 3276$	
Plastic Moment Capacity of Rafter	$M_p = 46571$	$\frac{M_p}{12} = 3881$	
Moment Capacity for End-Plate Yielding	$M_{pl} = 98063$	$\frac{M_{pl}}{12} = 8172$	
Moment Capacity for Bolt Tenion Rupture without Prying	$M_{np} = 72872$	$\frac{M_{np}}{12} = 6073$	$\frac{1.25 \cdot M_{np}}{12} = 7591$
Moment Capacity for Bolt Rupture with Prying Action	$M_q = 59696$	$\frac{M_q}{12} = 4975$	$\frac{1.25 \cdot M_q}{12} = 6218$
Maximum Plastic Moment Including Strain Hardening	$M_{pr} = 58214$	$\frac{M_{pr}}{12} = 4851$	
Hardened Plastic Capacity	$M_f = 63382$	$\frac{M_f}{12} = 5282$	

$$d_{b.req} := \sqrt{\frac{4 \cdot M_f}{\pi \cdot \phi_n \cdot F_t \cdot \Sigma d_n}} = 1.229 \quad \text{in}$$

AISC 358-10 (6.10-4)

$$t_{p.req} := \sqrt{\frac{1.11 \cdot M_f}{\phi_d \cdot F_{py} \cdot Y}} = 1.271 \quad \text{in}$$

AISC 358-10 (6.10-5)

Rafter Input:

$b_f = 12.00$ in

$t_f = 1.00$ in

$t_w = 0.75$ in

$d = 44.00$ in

End Plate Input:

$t_p = 1.50$ in

Bolt Pretension:

$T_b = 102$ kip

(AISC 360-10 Table J3.1)

Bolts:

Bolt Grade A490

$F_t = 113$ ksi

$d_b = 1.25$ in

24 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12.00$ in

Flange Thickness: $t_f := 1.00$ in

Web Thickness: $t_w := 0.50$ in

Total Beam Depth: $d := 24.00$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 22.00$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 11$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 12$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 23.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 11.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 11.5$ in

Plastic Section Modulus: $Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f}\right) + \left(\frac{A_w}{2}\right) \cdot \left(\frac{h_w}{4}\right)\right] = 337$ in^3

Section Nominal Moment Capacity: $M_n := 16590$ kip·in

Actuator Input:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Beam 2 - End Plate 12ES-0.875-0.75-24

Thin End-Plate Behavior

Limit States: End Plate Yielding & Bolt Fracture with Prying Action

End Plate Input:

Yield Strength:	$F_{py} := 55$	ksi
End Plate Thickness:	$t_p := 0.75$	in
End Plate Width:	$b_p := 12.00$	in
Inner Bolt Gage:	$g := 3.50$	in
Outer Bolt Gage:	$g_o := 2.625$	in
Flange to Outer Bolt CL:	$p_{fo} := 1.50$	in
Flange to Inner Bolt CL:	$p_{fi} := 1.50$	in
Bolt Spacing:	$p_b := 2.625$	in
Outer Bolt to Edge:	$d_e := 1.125$	in
Top of Flange to Edge:	$p_{ext} := p_{fo} + p_b + d_e = 5.25$	in
Length of Stiffener:	$L_{st} := 9.125$	in

Top of Compression Flange to Tension Face Bolt Rows:

Outermost Row:	$h_1 := d + p_{fo} + p_b = 28.125$	in
Outside of Flange:	$h_2 := d + p_{fo} = 25.5$	in
Inside of Flange:	$h_3 := d - t_f - p_{fi} = 21.5$	in
Innermost Row:	$h_4 := h_2 - p_b = 22.875$	in

Centroid of Compression Flange to Tension Face Bolt Rows:

Outermost Row:	$d_1 := d - \frac{t_f}{2} + p_{fo} + p_b = 27.625$	in
Outside of Flange:	$d_2 := d - \frac{t_f}{2} + p_{fo} = 25$	in
Inside of Flange:	$d_3 := d - \frac{3 \cdot t_f}{2} - p_{fi} = 21$	in
Innermost Row:	$d_4 := d_2 - p_b = 22.375$	in

Bolts:

Bolt Grade: A490

Nominal Tensile Strength: $F_t := 113$ ksi

Bolt Diameter: $d_b := 0.875$ in

Bolt Pretension: $T_b := 49$ kip (AISC 360-10 Table J3.1)

$\phi_d := 1.00$ $\phi_n := 0.90$ $\phi_r := 0.75$

Bolt Rupture Without Prying Action:

$\Sigma d_n := 2 \cdot (d_1 + 2 \cdot d_2 + 2 \cdot d_3 + d_4) = 284$ in

Bolt Force: $P_t := \frac{\pi}{4} \cdot d_b^{\,2} \cdot F_t = 67.9$ kip

$M_{np} := P_t \cdot \Sigma d_n = 19298$ kip·in

End-Plate Yield:

$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g} = 3.24$ in

$$Y := \frac{b_p}{2} \cdot \left(\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2} \right) + \frac{1}{2 \cdot g} \cdot \left[\begin{array}{l} h_1 \cdot (4 \cdot d_e + 3 \cdot p_b) + h_2 \cdot (p_b + 4 \cdot p_{fo}) \ldots \\ + h_3 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_4 \cdot (p_b + 4 \cdot s) + 4 \cdot d_e^{\,2} \end{array} \right] + \frac{3}{2} \cdot g = 408 \quad \text{in}$$

$M_{pl} := F_{py} \cdot t_p^{\,2} \cdot Y = 12623$ kip·in

Bolt Rupture With Prying Action:

Bolt Distribution Factors: $\alpha_1 := 1.0$ $\beta_1 := 1.0$ $\gamma_1 := 1.0$ $\delta_1 := 1.0$

$\beta_2 := 0.5$ $\gamma_2 := 0.75$

Tributary Widths:

$w_{\alpha 1} := \frac{(g + g_o)}{2} = 3.063$ in

$w'_{\alpha 1} := w_{\alpha 1} - \left(d_b + \frac{1}{16} \right) = 2.125$ in

$w_{\beta 1} := \frac{(g + g_o)}{2} = 3.063$ in

$w'_{\beta 1} := w_{\beta 1} - \left(d_b + \frac{1}{16} \right) = 2.125$ in

$w_{\beta 2} := \frac{(b_p - g - g_o)}{2} = 2.938$ in

$w'_{\beta 2} := w_{\beta 2} - \left(d_b + \frac{1}{16} \right) = 2$ in

$$w_{\gamma 1} := \frac{(g + g_o)}{2} = 3.063 \quad \text{in}$$

$$w_{\gamma 2} := \frac{(b_p - g - g_o)}{2} = 2.938 \quad \text{in}$$

$$w'_{\gamma 1} := w_{\gamma 1} - \left(d_b + \frac{1}{16}\right) = 2.125 \quad \text{in}$$

$$w'_{\gamma 2} := w_{\gamma 2} - \left(d_b + \frac{1}{16}\right) = 2 \quad \text{in}$$

$$w_{\delta 1} := \frac{(g + g_o)}{2} = 3.063 \quad \text{in}$$

$$w'_{\delta 1} := w_{\delta 1} - \left(d_b + \frac{1}{16}\right) = 2.125 \quad \text{in}$$

$$a_o := \min\left(\begin{bmatrix} 3.682 \cdot \left(\frac{t_p}{d_b}\right)^3 - 0.085 \\ p_{ext} - p_{fo} \end{bmatrix}\right) = 2.234 \quad \text{in}$$

$$a_i := 3.682 \cdot \left(\frac{t_p}{d_b}\right)^3 - 0.085 = 2.234 \quad \text{in}$$

$$F_{o\alpha 1} := \frac{t_p^{\,2} \cdot F_{py} \cdot \left(0.85 \cdot w_{\alpha 1} + 0.8 \cdot w'_{\alpha 1}\right) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fo}} = 27.143 \quad \text{kip}$$

$$F_{o\beta 1} := \frac{t_p^{\,2} \cdot F_{py} \cdot \left(0.85 \cdot w_{\beta 1} + 0.8 \cdot w'_{\beta 1}\right) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fo}} = 27.143 \quad \text{kip}$$

$$F_{o\beta 2} := \frac{t_p^{\,2} \cdot F_{py} \cdot \left(0.85 \cdot w_{\beta 2} + 0.8 \cdot w'_{\beta 2}\right) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fo}} = 26.079 \quad \text{kip}$$

$$F_{i\gamma 1} := \frac{t_p^{\,2} \cdot F_{py} \cdot \left(0.85 \cdot w_{\gamma 1} + 0.8 \cdot w'_{\gamma 1}\right) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fi}} = 27.143 \quad \text{kip}$$

$$F_{i\gamma 2} := \frac{t_p^{\,2} \cdot F_{py} \cdot \left(0.85 \cdot w_{\gamma 2} + 0.8 \cdot w'_{\gamma 2}\right) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fi}} = 26.079 \quad \text{kip}$$

$$F_{i\delta 1} := \frac{t_p^{\,2} \cdot F_{py} \cdot \left(0.85 \cdot w_{\delta 1} + 0.8 \cdot w'_{\delta 1}\right) + \frac{\pi \cdot d_b^{\,3} \cdot F_t}{8}}{4 \cdot p_{fi}} = 27.143 \quad \text{kip}$$

Prying Forces:

$$Q_{max.\alpha1} := \frac{w'_{\alpha1} \cdot t_p^{\,2}}{4 \cdot a_o} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{o\alpha1}}{w'_{\alpha1} \cdot t_p}\right)^2} = 6.21 \qquad \text{kip}$$

$$Q_{max.\beta1} := \frac{w'_{\beta1} \cdot t_p^{\,2}}{4 \cdot a_o} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{o\beta1}}{w'_{\beta1} \cdot t_p}\right)^2} = 6.21 \qquad \text{kip}$$

$$Q_{max.\beta2} := \frac{w'_{\beta2} \cdot t_p^{\,2}}{4 \cdot a_o} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{o\beta2}}{w'_{\beta2} \cdot t_p}\right)^2} = 5.795 \qquad \text{kip}$$

$$Q_{max.\gamma1} := \frac{w'_{\gamma1} \cdot t_p^{\,2}}{4 \cdot a_i} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{i\gamma1}}{w'_{\gamma1} \cdot t_p}\right)^2} = 6.21 \qquad \text{kip}$$

$$Q_{max.\gamma2} := \frac{w'_{\gamma2} \cdot t_p^{\,2}}{4 \cdot a_i} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{i\gamma2}}{w'_{\gamma2} \cdot t_p}\right)^2} = 5.795 \qquad \text{kip}$$

$$Q_{max.\delta1} := \frac{w'_{\delta1} \cdot t_p^{\,2}}{4 \cdot a_i} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{i\delta1}}{w'_{\delta1} \cdot t_p}\right)^2} = 6.21 \qquad \text{kip}$$

$$M_q := \max\left[\begin{array}{c}
2 \cdot \alpha_1 \cdot \left(P_t - Q_{max.\alpha1}\right) \cdot d_1 + 2 \cdot \left[\beta_1 \cdot \left(P_t - Q_{max.\beta1}\right) + \beta_2 \cdot \left(P_t - Q_{max.\beta2}\right)\right] \cdot d_2 \;\ldots \\
+ 2 \cdot \left[\gamma_1 \cdot \left(P_t - Q_{max.\gamma1}\right) + \gamma_2 \cdot \left(P_t - Q_{max.\gamma2}\right)\right] \cdot d_3 + 2 \cdot \delta_1 \cdot \left(P_t - Q_{max.\delta1}\right) \cdot d_4 \\
2 \cdot \alpha_1 \cdot T_b \cdot d_1 + 2 \cdot \left[\beta_1 \cdot \left(P_t - Q_{max.\beta1}\right) + \beta_2 \cdot \left(P_t - Q_{max.\beta2}\right)\right] \cdot d_2 \;\ldots \\
+ 2 \cdot \left[\gamma_1 \cdot \left(P_t - Q_{max.\gamma1}\right) + \gamma_2 \cdot \left(P_t - Q_{max.\gamma2}\right)\right] \cdot d_3 + 2 \cdot \delta_1 \cdot T_b \cdot d_4 \\
2 \cdot \alpha_1 \cdot T_b \cdot d_1 + 2 \cdot \left[\beta_1 \cdot \left(P_t - Q_{max.\beta1}\right) + \beta_2 \cdot \left(P_t - Q_{max.\beta2}\right)\right] \cdot d_2 \;\ldots \\
+ 2 \cdot \left(\gamma_1 + \gamma_2\right) \cdot T_b \cdot d_3 + 2 \cdot \delta_1 \cdot T_b \cdot d_4 \\
2 \cdot \alpha_1 \cdot T_b \cdot d_1 + 2 \cdot \left(\beta_1 + \beta_2\right) \cdot T_b \cdot d_2 \;\ldots \\
+ 2 \cdot \left[\gamma_1 \cdot \left(P_t - Q_{max.\gamma1}\right) + \gamma_2 \cdot \left(P_t - Q_{max.\gamma2}\right)\right] \cdot d_3 + 2 \cdot \delta_1 \cdot T_b \cdot d_4 \\
2 \cdot T_b \cdot \left[\left(\alpha_1\right) \cdot d_1 + \left(\beta_1 + \beta_2\right) \cdot d_2 + \left(\gamma_1 + \gamma_2\right) \cdot d_3 + \left(\delta_1\right) \cdot d_4\right]
\end{array}\right] = 15366 \quad \text{kip} \cdot \text{in}$$

Moment at Column Face:

Peak Connection Strength Factor: $C_{pr} := \min\left[\frac{(F_y + F_u)}{2 \cdot F_y}, 1.2\right] = 1.136$ AISC 358-10 (2.4.3-2)

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 18508$ kip·in

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 23134$ kip·in

AISC 358-10 (2.4.3-1)

Column Face to Plastic Hinge: $S_h := L_{st} + t_p = 9.875$ in

Maximum Shear Force: $V_u := \frac{M_{pr}}{L_h - S_h} = 141.603$ kip

Hardened Plastic Capacity: $M_f := M_{pr} + V_u \cdot S_h = 24533$ kip·in

AISC 358-10 (6.10-1)

Summary of Limit States & Capacities

Beam 2 - End Plate 12ES-0.875-0.75-24

Limit States: End Plate Yielding & Bolt Fracture with Prying Action

	kip·in	kip·ft	
Nominal Moment Capacity of Rafter	$M_n = 16590$	$\frac{M_n}{12} = 1383$	
Plastic Moment Capacity of Rafter	$M_p = 18508$	$\frac{M_p}{12} = 1542$	
Moment Capacity for End-Plate Yielding	$M_{pl} = 12623$	$\frac{M_{pl}}{12} = 1052$	
Moment Capacity for Bolt Tenion Rupture without Prying	$M_{np} = 19298$	$\frac{M_{np}}{12} = 1608$	$\frac{1.25 \cdot M_{np}}{12} = 2010$
Moment Capacity for Bolt Rupture with Prying Action	$M_q = 15366$	$\frac{M_q}{12} = 1280$	$\frac{1.25 \cdot M_q}{12} = 1601$
Maximum Plastic Moment Including Strain Hardening	$M_{pr} = 23134$	$\frac{M_{pr}}{12} = 1928$	
Hardened Plastic Capacity	$M_f = 24533$	$\frac{M_f}{12} = 2044$	

$$d_{b.req} := \sqrt{\frac{4 \cdot M_f}{\pi \cdot \phi_n \cdot F_t \cdot \Sigma d_n}} = 1.04 \quad \text{in}$$

AISC 358-10 (6.10-4)

$$t_{p.req} := \sqrt{\frac{1.11 \cdot M_f}{\phi_d \cdot F_{py} \cdot Y}} = 1.102 \quad \text{in}$$

AISC 358-10 (6.10-5)

Rafter Input:

$b_f = 12.00$ in

$t_f = 1.00$ in

$t_w = 0.50$ in

$d = 24.00$ in

End Plate Input:

$t_p = 0.75$ in

Bolt Pretension:

$T_b = 49$ kip

(AISC 360-10 Table J3.1)

Bolts:

Bolt Grade A490

$F_t = 113$ ksi

$d_b = 0.875$ in

24 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12.00$ in

Flange Thickness: $t_f := 1.00$ in

Web Thickness: $t_w := 0.50$ in

Total Beam Depth: $d := 24.00$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 22.00$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 11$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \dfrac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \dfrac{h_w}{2} = 12$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \dfrac{t_f}{2} = 23.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 11.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 11.5$ in

Plastic Section Modulus: $Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f} \right) + \left(\dfrac{A_w}{2} \right) \cdot \left(\dfrac{h_w}{4} \right) \right] = 337$ in^3

Section Nominal Moment Capacity: $M_n := 16590$ kip·in

Actuator Input:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \dfrac{1}{4} \right) = 173.25$ in

Beam 2 - End Plate 12ES-1.125-1.25-24

Thick End-Plate Behavior

Limit State: Plastic Hinging of Beam

End Plate Input:

Yield Strength:	$F_{py} := 55$	ksi
End Plate Thickness:	$t_p := 1.25$	in
End Plate Width:	$b_p := 14.00$	in
Inner Bolt Gage:	$g := 4.00$	in
Outer Bolt Gage:	$g_o := 3.375$	in
Flange to Outer Bolt CL:	$p_{fo} := 1.75$	in
Flange to Inner Bolt CL:	$p_{fi} := 1.75$	in
Bolt Spacing:	$p_b := 3.375$	in
Outer Bolt to Edge:	$d_e := 1.50$	in
Top of Flange to Edge:	$p_{ext} := p_{fo} + p_b + d_e = 6.625$	in
Stiffener Length:	$L_{st} := 11.5$	in

Top of Compression Flange to Tension Face Bolt Rows:

Outermost Row:	$h_1 := d + p_{fo} + p_b = 29.125$	in
Outside of Flange:	$h_2 := d + p_{fo} = 25.75$	in
Inside of Flange:	$h_3 := d - t_f - p_{fi} = 21.25$	in
Innermost Row:	$h_4 := h_2 - p_b = 22.375$	in

Centroid of Compression Flange to Tension Face Bolt Rows:

Outermost Row:	$d_1 := d - \frac{t_f}{2} + p_{fo} + p_b = 28.625$	in
Outside of Flange:	$d_2 := d - \frac{t_f}{2} + p_{fo} = 25.25$	in
Inside of Flange:	$d_3 := d - \frac{3 \cdot t_f}{2} - p_{fi} = 20.75$	in
Innermost Row:	$d_4 := d_2 - p_b = 21.875$	in

Bolts:

Bolt Grade: A490

Nominal Tensile Strength: $F_t := 113$ ksi

Bolt Diameter: $d_b := 1.125$ in

Bolt Pretension: $T_b := 80$ kip (AISC 360-10 Table J3.1)

$\phi_d := 1.00$ $\phi_n := 0.90$ $\phi_r := 0.75$

Bolt Rupture Without Prying Action:

$\Sigma d_n := 2\cdot\left(d_1 + 2\cdot d_2 + 2\cdot d_3 + d_4\right) = 285$ in

Bolt Force: $P_t := \frac{\pi}{4}\cdot d_b^{\,2}\cdot F_t = 112.3$ kip

$M_{np} := P_t\cdot\Sigma d_n = 32012$ kip·in

End-Plate Yield:

$s := \frac{1}{2}\cdot\sqrt{b_p\cdot g} = 3.742$ in

$$Y := \frac{b_p}{2}\cdot\left(\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2}\right) + \frac{1}{2\cdot g}\cdot\begin{bmatrix} h_1\cdot\left(4\cdot d_e + 3\cdot p_b\right) + h_2\cdot\left(p_b + 4\cdot p_{fo}\right) \ldots \\ + h_3\cdot\left(4\cdot p_{fi} + 3\cdot p_b\right) + h_4\cdot\left(p_b + 4\cdot s\right) + 4\cdot d_e^{\,2} \end{bmatrix} + \frac{3}{2}\cdot g = 422 \quad \text{in}$$

$M_{pl} := F_{py}\cdot t_p^{\,2}\cdot Y = 36298$ kip·in

Bolt Rupture With Prying Action:

Bolt Distribution Factors: $\alpha_1 := 1.0$ $\beta_1 := 1.0$ $\gamma_1 := 1.0$ $\delta_1 := 1.0$

$\beta_2 := 0.5$ $\gamma_2 := 0.75$

Tributary Widths:

$w_{\alpha 1} := \frac{\left(g + g_o\right)}{2} = 3.688$ in

$w'_{\alpha 1} := w_{\alpha 1} - \left(d_b + \frac{1}{16}\right) = 2.5$ in

$w_{\beta 1} := \frac{\left(g + g_o\right)}{2} = 3.688$ in

$w'_{\beta 1} := w_{\beta 1} - \left(d_b + \frac{1}{16}\right) = 2.5$ in

$w_{\beta 2} := \frac{\left(b_p - g - g_o\right)}{2} = 3.313$ in

$w'_{\beta 2} := w_{\beta 2} - \left(d_b + \frac{1}{16}\right) = 2.125$ in

$$w_{\gamma 1} := \frac{(g + g_o)}{2} = 3.688 \quad \text{in} \qquad w_{\gamma 2} := \frac{(b_p - g - g_o)}{2} = 3.313 \quad \text{in}$$

$$w'_{\gamma 1} := w_{\gamma 1} - \left(d_b + \frac{1}{16}\right) = 2.5 \quad \text{in} \qquad w'_{\gamma 2} := w_{\gamma 2} - \left(d_b + \frac{1}{16}\right) = 2.125 \quad \text{in}$$

$$w_{\delta 1} := \frac{(g + g_o)}{2} = 3.688 \quad \text{in}$$

$$w'_{\delta 1} := w_{\delta 1} - \left(d_b + \frac{1}{16}\right) = 2.5 \quad \text{in}$$

$$a_o := \min\left(\left[\begin{matrix} 3.682\cdot\left(\frac{t_p}{d_b}\right)^3 - 0.085 \\ p_{ext} - p_{fo} \end{matrix}\right]\right) = 4.875 \quad \text{in} \qquad a_i := 3.682\cdot\left(\frac{t_p}{d_b}\right)^3 - 0.085 = 4.966 \quad \text{in}$$

$$F_{o\alpha 1} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\alpha 1} + 0.8\cdot w'_{\alpha 1}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fo}} = 72.06 \quad \text{kip}$$

$$F_{o\beta 1} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\beta 1} + 0.8\cdot w'_{\beta 1}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fo}} = 72.06 \quad \text{kip}$$

$$F_{o\beta 2} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\beta 2} + 0.8\cdot w'_{\beta 2}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fo}} = 64.463 \quad \text{kip}$$

$$F_{i\gamma 1} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\gamma 1} + 0.8\cdot w'_{\gamma 1}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fi}} = 72.06 \quad \text{kip}$$

$$F_{i\gamma 2} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\gamma 2} + 0.8\cdot w'_{\gamma 2}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fi}} = 64.463 \quad \text{kip}$$

$$F_{i\delta 1} := \frac{t_p^{\,2}\cdot F_{py}\cdot(0.85\cdot w_{\delta 1} + 0.8\cdot w'_{\delta 1}) + \frac{\pi\cdot d_b^{\,3}\cdot F_t}{8}}{4\cdot p_{fi}} = 72.06 \quad \text{kip}$$

Prying Forces:

$$Q_{max.\alpha1} := \frac{w'_{\alpha1} \cdot t_p^{\,2}}{4 \cdot a_o} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{o\alpha1}}{w'_{\alpha1} \cdot t_p}\right)^2} = 7.575 \qquad \text{kip}$$

$$Q_{max.\beta1} := \frac{w'_{\beta1} \cdot t_p^{\,2}}{4 \cdot a_o} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{o\beta1}}{w'_{\beta1} \cdot t_p}\right)^2} = 7.575 \qquad \text{kip}$$

$$Q_{max.\beta2} := \frac{w'_{\beta2} \cdot t_p^{\,2}}{4 \cdot a_o} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{o\beta2}}{w'_{\beta2} \cdot t_p}\right)^2} = 6.04 \qquad \text{kip}$$

$$Q_{max.\gamma1} := \frac{w'_{\gamma1} \cdot t_p^{\,2}}{4 \cdot a_i} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{i\gamma1}}{w'_{\gamma1} \cdot t_p}\right)^2} = 7.436 \qquad \text{kip}$$

$$Q_{max.\gamma2} := \frac{w'_{\gamma2} \cdot t_p^{\,2}}{4 \cdot a_i} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{i\gamma2}}{w'_{\gamma2} \cdot t_p}\right)^2} = 5.929 \qquad \text{kip}$$

$$Q_{max.\delta1} := \frac{w'_{\delta1} \cdot t_p^{\,2}}{4 \cdot a_i} \cdot \sqrt{F_{py}^{\,2} - 3 \cdot \left(\frac{F_{i\delta1}}{w'_{\delta1} \cdot t_p}\right)^2} = 7.436 \qquad \text{kip}$$

$$M_q := \max\left[\begin{array}{c}
2 \cdot \alpha_1 \cdot \left(P_t - Q_{max.\alpha1}\right) \cdot d_1 + 2 \cdot \left[\beta_1 \cdot \left(P_t - Q_{max.\beta1}\right) + \beta_2 \cdot \left(P_t - Q_{max.\beta2}\right)\right] \cdot d_2 \cdots \\
+ 2 \cdot \left[\gamma_1 \cdot \left(P_t - Q_{max.\gamma1}\right) + \gamma_2 \cdot \left(P_t - Q_{max.\gamma2}\right)\right] \cdot d_3 + 2 \cdot \delta_1 \cdot \left(P_t - Q_{max.\delta1}\right) \cdot d_4 \\
2 \cdot \alpha_1 \cdot T_b \cdot d_1 + 2 \cdot \left[\beta_1 \cdot \left(P_t - Q_{max.\beta1}\right) + \beta_2 \cdot \left(P_t - Q_{max.\beta2}\right)\right] \cdot d_2 \cdots \\
+ 2 \cdot \left[\gamma_1 \cdot \left(P_t - Q_{max.\gamma1}\right) + \gamma_2 \cdot \left(P_t - Q_{max.\gamma2}\right)\right] \cdot d_3 + 2 \cdot \delta_1 \cdot T_b \cdot d_4 \\
2 \cdot \alpha_1 \cdot T_b \cdot d_1 + 2 \cdot \left[\beta_1 \cdot \left(P_t - Q_{max.\beta1}\right) + \beta_2 \cdot \left(P_t - Q_{max.\beta2}\right)\right] \cdot d_2 \cdots \\
+ 2 \cdot \left(\gamma_1 + \gamma_2\right) \cdot T_b \cdot d_3 + 2 \cdot \delta_1 \cdot T_b \cdot d_4 \\
2 \cdot \alpha_1 \cdot T_b \cdot d_1 + 2 \cdot \left(\beta_1 + \beta_2\right) \cdot T_b \cdot d_2 \cdots \\
+ 2 \cdot \left[\gamma_1 \cdot \left(P_t - Q_{max.\gamma1}\right) + \gamma_2 \cdot \left(P_t - Q_{max.\gamma2}\right)\right] \cdot d_3 + 2 \cdot \delta_1 \cdot T_b \cdot d_4 \\
2 \cdot T_b \cdot \left[\left(\alpha_1\right) \cdot d_1 + \left(\beta_1 + \beta_2\right) \cdot d_2 + \left(\gamma_1 + \gamma_2\right) \cdot d_3 + \left(\delta_1\right) \cdot d_4\right]
\end{array}\right] = 26224 \quad \text{kip} \cdot \text{in}$$

<u>Moment at Column Face</u>:

Peak Connection Strength Factor: $C_{pr} := \min\left[\frac{(F_y + F_u)}{2 \cdot F_y}, 1.2\right] = 1.136$ AISC 358-10 (2.4.3-2)

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 18508$ kip·in

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 23134$ kip·in

AISC 358-10 (2.4.3-1)

Column Face to Plastic Hinge: $S_h := L_{st} + t_p = 12.75$ in

Maximum Shear Force: $V_u := \frac{M_{pr}}{L_h - S_h} = 144.139$ kip

Hardened Plastic Capacity: $M_f := M_{pr} + V_u \cdot S_h = 24972$ kip·in

AISC 358-10 (6.10-1)

Summary of Limit States & Capacities

Beam 2 - End Plate 12ES-1.125-1.25-24

Limit State: Plastic Hinging of Beam

	kip·in	kip·ft	
Nominal Moment Capacity of Rafter	$M_n = 16590$	$\frac{M_n}{12} = 1383$	
Plastic Moment Capacity of Rafter	$M_p = 18508$	$\frac{M_p}{12} = 1542$	
Moment Capacity for End-Plate Yielding	$M_{pl} = 36298$	$\frac{M_{pl}}{12} = 3025$	
Moment Capacity for Bolt Tenion Rupture without Prying	$M_{np} = 32012$	$\frac{M_{np}}{12} = 2668$	$\frac{1.25 \cdot M_{np}}{12} = 3335$
Moment Capacity for Bolt Rupture with Prying Action	$M_q = 26224$	$\frac{M_q}{12} = 2185$	$\frac{1.25 \cdot M_q}{12} = 2732$
Maximum Plastic Moment Including Strain Hardening	$M_{pr} = 23134$	$\frac{M_{pr}}{12} = 1928$	
Hardened Plastic Capacity	$M_f = 24972$	$\frac{M_f}{12} = 2081$	

$$d_{b.req} := \sqrt{\frac{4 \cdot M_f}{\pi \cdot \phi_n \cdot F_t \cdot \Sigma d_n}} = 1.047 \quad \text{in}$$

AISC 358-10 (6.10-4)

$$t_{p.req} := \sqrt{\frac{1.11 \cdot M_f}{\phi_d \cdot F_{py} \cdot Y}} = 1.092 \quad \text{in}$$

AISC 358-10 (6.10-5)

Rafter Input:

$b_f = 12.00$ in

$t_f = 1.00$ in

$t_w = 0.50$ in

$d = 24.00$ in

End Plate Input:

$t_p = 1.25$ in

Bolt Pretension:

$T_b = 80$ kip

(AISC 360-10 Table J3.1)

Bolts:

Bolt Grade A490

$F_t = 113$ ksi

$d_b = 1.125$ in

44 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12$ in

Flange Thickness: $t_f := 1$ in

Web Thickness: $t_w := 0.75$ in

Total Beam Depth: $d := 44$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 42$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 31.5$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \dfrac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \dfrac{h_w}{2} = 22$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \dfrac{t_f}{2} = 43.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 21.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 21.5$ in

Plastic Section Modulus: $$Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f} \right) + \left(\frac{A_w}{2} \right) \cdot \left(\frac{h_w}{4} \right) \right] = 846.8 \quad in^3$$

Moment of Inertia: $$I_x := \frac{1}{12} \cdot t_w \cdot h_w^{\,3} + 2 \left(\frac{1}{12} \cdot b_f \cdot t_f^{\,3} + b_f \cdot t_f \cdot CtC_{top.f}^{\,2} \right) = 15727 \quad in^4$$

Elastic Section Modulus: $$S_x := \frac{I_x}{C_w} = 714.84 \quad in^3$$

Peak Connection Strength Factor: $C_{pr} := \frac{(F_y + F_u)}{2 \cdot F_y} = 1.136$

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 46571$ kip·in

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 58214$ kip·in

Actuator Input:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Bolt Gage: $g_{act} := 6 + \frac{1}{16} = 6.0625$ in

Bolt Spacing: $p_{b.act} := 6 + \frac{11}{16} = 6.6875$ in

Beam 1 - End Plate 12ES-0.75-1.00-44

Limit State: Rupture of Bolts without Prying Action

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.00$ in

End Plate Width: $b_p := 12.00$ in

Inner Bolt Gage: $g := 4.00$ in

Outer Bolt Gage: $g_o := 2.25$ in

Flange to Outer Bolt CL: $p_{fo} := 1.50$ in

Flange to Inner Bolt CL: $p_{fi} := 1.50$ in

Bolt Spacing: $p_b := 2.25$ in

Outer Bolt to Edge: $d_e := 1.00$ in

Top of Flange to Edge: $p_{ext} := p_{fo} + p_b + d_e = 4.75$ in

Bolts:

$d_b := 0.75$ in

$T_b := 35$ kip

$F_t := 113$ ksi

Stiffeners:

$L_{st} := 8.25$ in

$S_h := L_{st} + t_p = 9.25$ in

Actuator Force:

$M_{np} = M_u := 1.25 \cdot (25760) = 32200$ kip·in   Increased for 25% bolt over strength

$$V_u := \frac{M_u}{L_h} = 185.9 \quad \text{kip}$$

Check Bolts:

AISC Design Guide 4 - Chap. 2.1-4
Only bolts at compression flange take shear force

Slip Critical:

Flange Force: $F_{flange} := \dfrac{M_u}{d} = 731.8$ kip

$\phi_{sc} := 1.00$

Mean Slip Coefficient: $\mu_{class.A} := 0.3$ AISC 360-10 J3-8

Nominal Strength: $R_{n.sc} := \mu_{class.A} \cdot F_{flange} = 219.5$ kip

Design Strength: $\phi_{sc} \cdot R_{n.sc} = 219.5$ kip

Shear Capacity of Bolts:

$\phi_v := 0.75$

Number of Bolts at Compression Flange: $n_s := 12$ bolts

Nominal Shear Strength: $F_{nv} := 68$ ksi AISC 360-10 Table J3.2

Nominal Strength: $R_{nv.bolt} := \frac{\pi}{4} \cdot d_b^{\ 2} \cdot F_{nv} = 30.04 \ \frac{kip}{bolt}$

Design Strength: $\phi_v \cdot n_s \cdot R_{nv.bolt} = 270.4$ kip

Required Strength: $V_u = 185.9$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \left(\phi_{sc} \cdot R_{n.sc} \geq V_u\right) \wedge \left(\phi_v \cdot n_s \cdot R_{nv.bolt} \geq V_u\right) \\ \text{"No Good"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Bearing & Tearout: End Plate $\phi_{bt} := 0.75$ AISC 360-10 J3-10

Clear Distances:

$L_{C.a1} := d_e - 0.5 \cdot \left(d_b + 0.0625\right) = 0.594$ in

$L_{C.b1} := p_b - \left(d_b + 0.0625\right) = 1.438$ in

$L_{C.b2} := p_b + d_e - 0.5 \cdot \left(d_b + 0.0625\right) = 2.844$ in

$L_{C.c1} := p_{fi} + p_{fo} + t_f - \left(d_b + 0.0625\right) = 3.188$ in

$L_{C.c2} := L_{C.c1} = 3.188$ in

$L_{C.d1} := p_b - \left(d_b + 0.0625\right) = 1.438$ in

Capacity Per Bolt Hole:

$r_{a1} := \min\left(1.2 \cdot L_{C.a1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 50$ kip

$r_{b1} := \min\left(1.2 \cdot L_{C.b1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 121$ kip

$r_{b2} := \min\left(1.2 \cdot L_{C.b2} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 126$ kip

$r_{c1} := \min\left(1.2 \cdot L_{C.c1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 126$ kip

$r_{c2} := \min\left(1.2 \cdot L_{C.c2} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 126$ kip

$r_{d1} := \min\left(1.2 \cdot L_{C.d1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 121$ kip

Nominal Strength: $R_{n.bt} := 2 \cdot \left(r_{a1} + r_{b1} + r_{b2} + r_{c1} + r_{c2} + r_{d1}\right) = 1338.8$ kip

Design Strength: $\phi_{bt} \cdot R_{n.bt} = 1004.1$ kip

Required Strength: $V_u = 185.9$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi_{bt} \cdot R_{n.bt} \geq V_u \\ \text{"No Good"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Intermediate Stiffeners: AISC 360-10 Chap. G

$\frac{h_w}{t_w} = 56$ $\qquad 2.46 \cdot \sqrt{\frac{E}{F_y}} = 56.488$ $\qquad$ AISC 360-10 Sect. G2-2

$$\left| \begin{array}{l} \text{"No transverse stiffeners required"} \quad \text{if} \ \frac{h_w}{t_w} \le 2.46 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"Transverse stiffeners required"} \quad \text{otherwise} \end{array} \right. = \text{"No transverse stiffeners required"}$$

Web Plate Shear Buckling Coefficient: $k_v := 5$

Web Shear Coefficient:

$$C_v := \left| \begin{array}{ll} 1.0 \ \text{if} \ \frac{h_w}{t_w} \le 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} & \text{(G2-3)} \\ \dfrac{1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}}}{\frac{h_w}{t_w}} \ \text{if} \ 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} < \frac{h_w}{t_w} \le 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} & \text{(G2-4)} \\ \dfrac{1.51 \cdot k_v \cdot E}{\left(\frac{h_w}{t_w}\right)^2 \cdot F_y} \ \text{if} \ \frac{h_w}{t_w} > 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} & \text{(G2-5)} \end{array} \right.$$

$C_v = 1$

$$\phi_v := \left| \begin{array}{l} 1.0 \ \text{if} \ \frac{h_w}{t_w} \le 2.24 \cdot \sqrt{\frac{E}{F_y}} \\ 0.9 \ \text{otherwise} \end{array} \right. = 0.9$$

Area of Web: $A_v := d \cdot t_w = 33$ in^2

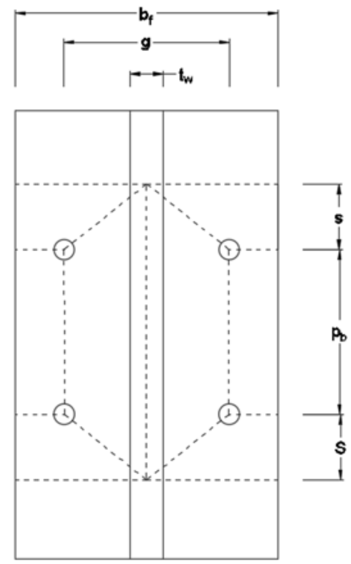
Nominal Strength: $V_n := 0.6 \cdot F_y \cdot A_v \cdot C_v = 1089$ kip

Design Strength: $\phi_v \cdot V_n = 980.1$ kip

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if} \ \phi_v \cdot V_n \ge V_u \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

<u>Bearing Stiffeners</u>: Check Concentrated Force From Actuator

<u>Flange Local Bending</u>: From Yield Line Analysis



$\phi_{FLB} := 0.9$

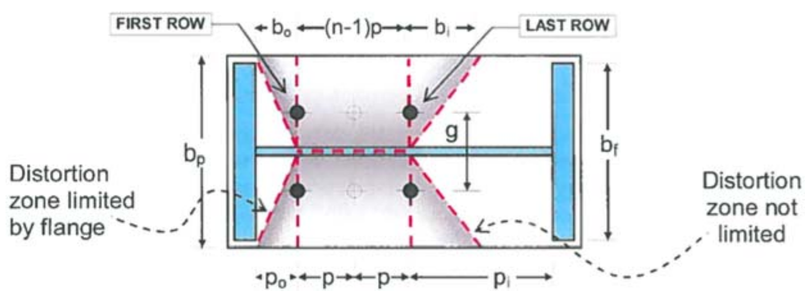
$$s := \frac{1}{2}\cdot\sqrt{b_f\cdot g_{act}} = 4.265 \quad \text{in}$$

$$Y := \frac{\left(4\cdot s^2 + b_f\cdot g_{act} + 2\cdot p_{b.act}\cdot s\right)}{s\cdot\left(g_{act}\right)} = 7.834$$

Nominal Strength: $R_{n.FLB} := F_y\cdot t_f^{\,2}\cdot Y = 430.9 \quad \text{kip}$

Design Strength: $\phi_{FLB}\cdot R_{n.FLB} = 387.8 \quad \text{kip}$

<u>Flange Local Bending</u>: BlueScope Buildings Yield Line Analysis



$$b := \frac{b_f}{\sqrt{2}} = 8.485 \quad \text{in}$$

$b_i := b = 8.485 \quad \text{in}$

$b_o := b = 8.485 \quad \text{in}$

Number of Bolt Rows: $n := 2$

$$Y := \begin{cases} \dfrac{1}{g_{act}}\cdot\left[\dfrac{b_f^{\,2}}{2}\cdot\left(\dfrac{1}{b_o} + \dfrac{1}{b_i}\right) + b_o + b_i + (n-1)\cdot p_{b.act}\right] & \text{if } p_{b.act} \le \dfrac{b_f}{\sqrt{2}} \\ \dfrac{1}{g_{act}}\cdot\left(2\cdot\dfrac{b_f^{\,2}}{b} + 4\cdot b\right) & \text{otherwise} \end{cases} = 6.702$$

Nominal Strength: $R_{n.BS} := F_y\cdot t_f^{\,2}\cdot Y = 368.6 \quad \text{kip}$

Design Strength: $\phi_{FLB}\cdot R_{n.BS} = 331.7 \quad \text{kip}$

Flange Local Bending: AISCM Chap. 9 Prying Action

$$\tan\left(\frac{\pi}{4}\right) = \frac{\left(\frac{p}{2}\right)}{g_{act} - \frac{t_w}{2}} \quad \text{-->} \quad p := 2\cdot\tan\left(\frac{\pi}{4}\right)\cdot\left(\frac{g_{act}}{2} - \frac{t_w}{2}\right)$$

Tributary Length: $p = 5.3125$ in

Bolt Hole Diameter: $\delta_1 := 1.25$ in

Bolt Diameter: $d_{b.act} := 1.125$ in

Bolt CL to the Face of the Web: $b := \frac{g_{act}}{2} - \frac{t_w}{2} = 2.65625$ in

Bolt Hole Edge to the Face of the Web: $b_1 := b - \frac{d_{b.act}}{2} = 2.09375$ in

Bolt Spacing: $S := p_{b.act} = 6.6875$ in

Required Strength per Bolt: $T := \frac{1}{4}\cdot V_u = 46.465$ kip 1/4 because 4 bolts

$$t_{min.FLB} := \sqrt{\frac{4\cdot T\cdot b_1}{\phi_{FLB}\cdot p\cdot F_u}} = 1.078 \text{ in}$$ AISCM (9-20a)

$$\delta := 1 - \frac{\delta_1}{p} = 0.76471$$ AISCM (9-24)

Bolt CL to Edge of Fitting: $a := \left(\frac{b_f - t_w}{2}\right) - b = 2.969$ in

$$a_1 := \min\left(a + \frac{d_{b.act}}{2}, 1.25\cdot b + \frac{d_{b.act}}{2}\right) = 3.5313 \text{ in}$$

$$\rho := \frac{b_1}{a_1} = 0.59292$$ AISCM (9-26)

Available Tension per Bolt: $B := 0.75\cdot\frac{\pi}{4}\cdot d_{b.act}^{\,2}\cdot F_t = 84.243$ kip

$$\beta := \frac{1}{\rho}\cdot\left(\frac{B}{T} - 1\right) = 1.371$$ AISCM (9-25)

Thickness Required to Develop Available Strength of the Bolt Without Prying:

$$t_c := \sqrt{\frac{4\cdot B\cdot b_1}{\phi_{FLB}\cdot p\cdot F_u}} = 1.452 \text{ in}$$ AISCM (9-30a)

Ratio of Moment at the Face of the Web to the Moment at the Bolt Line:

$$\alpha := \frac{1}{\delta}\cdot\left[\frac{T}{B}\cdot\left(\frac{t_c}{t_f}\right)^2 - 1\right] = 0.2128$$ AISCM (9-29)

Prying Force per Bolt: $$q := B\cdot\left[\delta\cdot\alpha\cdot\rho\cdot\left(\frac{t_f}{t_c}\right)^2\right] = 3.855 \quad \text{kip}$$ AISCM (9-28)

Value of α that either maximizes the bolt available tensile strength for a given thickness or minimizes the thickness required for a given bolt available tensile strength

$$\alpha_1 := \frac{1}{\delta\cdot(1+\rho)}\cdot\left[\left(\frac{t_c}{t_f}\right)^2 - 1\right] = 0.91$$ AISCM (9-35)

$$Q := \begin{cases} 1 & \text{if } \alpha_1 < 0 \\ \left(\frac{t_f}{t_c}\right)^2\cdot(\delta+1) & \text{if } \alpha_1 > 1 \\ \left(\frac{t_f}{t_c}\right)^2\cdot\left(\delta\cdot\alpha_1 + 1\right) & \text{otherwise} \end{cases}$$

AISCM (9-32)

AISCM (9-34)

AISCM (9-33)

$Q = 0.804$

Available Tensile Strength per Bolt Including Prying Action: $T_{avail} := B\cdot Q = 67.76 \quad \text{kip}$ AISCM (9-31)

Design Strength: $\phi R_{n.pry} := 4\cdot T_{avail} = 271 \quad \text{kip}$

Web Local Yielding: $\phi_{WLY} := 1.00$ AISC 360-10 Chap. J10

Length of Bearing $l_b := 18 \quad \text{in}$

Outside Flange Surface to Fillet Toe $k := t_f = 1 \quad \text{in}$

Nominal Strength: $R_{n.WLY} := t_w\cdot F_y\cdot\left(5\cdot k + l_b\right) = 948.8 \quad \text{kip}$

Design Strength: $\phi_{WLY}\cdot R_{n.WLY} = 948.8 \quad \text{kip}$

Web Crippling: $\phi_{WC} := 0.75$

Nominal Strength: $$R_{n.WC} := 0.8\cdot t_w^{\,2}\cdot\left[1 + 3\cdot\frac{l_b}{d}\cdot\left(\frac{t_w}{t_f}\right)^{1.5}\right]\cdot\sqrt{\frac{E\cdot F_y\cdot t_f}{t_w}} = 1179 \quad \text{kip}$$

Design Strength: $\phi_{WC}\cdot R_{n.WC} = 884.5 \quad \text{kip}$

Web Sidesway Buckling: $\phi_{WSB} := 0.85$

Compression Flange Not Restrained Against Rotation $L_b := 15 \cdot 12 = 180$ in

$C_r := 480000$ ksi

Nominal Strength: $R_{n.WSB} := \dfrac{C_r \cdot t_w^{\,3} \cdot t_f}{h_w^{\,2}} \cdot \left[0.4 \cdot \left[\dfrac{\left(\dfrac{h_w}{t_w}\right)}{\left(\dfrac{L_b}{b_f}\right)}\right]^3\right] = 2389.3$ kip

Design Strength: $\phi_{WSB} \cdot R_{n.WSB} = 2030.9$ kip

$$\begin{cases} \text{"WSB applies"} & \text{if } \dfrac{\left(\dfrac{h_w}{t_w}\right)}{\left(\dfrac{L_b}{b_f}\right)} \le 1.7 \\ \text{"WSB does not apply"} & \text{otherwise} \end{cases} = \text{"WSB does not apply"}$$

Web Compression Buckling: $\phi_{WCB} := 0.9$

Nominal Strength: $R_{n.WCB} := \dfrac{24 \cdot t_w^{\,3} \cdot \sqrt{E \cdot F_y}}{h_w} = 304.5$ kip

Design Strength: $\phi_{WCB} \cdot R_{n.WCB} = 274$ kip

Concentrated Force Strength:

$\phi R_{n.min} := \min\left(\phi_{FLB} \cdot R_{n.FLB}, \phi_{FLB} \cdot R_{n.BS}, \phi R_{n.pry}, \phi_{WLY} \cdot R_{n.WLY}, \phi_{WC} \cdot R_{n.WC}, \phi_{WSB} \cdot R_{n.WSB}, \phi_{WCB} \cdot R_{n.WC}\right.$

Design Strength: $\phi R_{n.min} = 271$ kip

Required Strength: $F_{ul} := V_u = 185.9$ kip

$$\begin{cases} \text{"Need bearing stiffeners"} & \text{if } \phi R_{n.min} < F_{ul} \\ \text{"Ok"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Beam 1 - End Plate 12ES-1.25-1.50-44

Limit State: Plastic Hinging of the Beam

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.50$ in

End Plate Width: $b_p := 16.00$ in

Inner Bolt Gage: $g := 4.50$ in

Outer Bolt Gage: $g_o := 3.75$ in

Flange to Outer Bolt CL: $p_{fo} := 1.875$ in

Flange to Inner Bolt CL: $p_{fi} := 1.875$ in

Bolt Spacing: $p_b := 3.75$ in

Outer Bolt to Edge: $d_e := 1.625$ in

Top of Flange to Edge: $p_{ext} := p_{fo} + p_b + d_e = 7.25$ in

Bolts:

$d_b := 1.25$ in

$T_b := 102$ kip

$F_t := 113$ ksi

Stiffeners:

$L_{st} := 12.625$ in

$S_h := L_{st} + t_p = 14.125$ in

Actuator Force:

$M_f = \quad M_u := 63382$ kip·in

$$V_u := \frac{M_u}{L_h} = 365.8 \quad \text{kip}$$

Check Bolts:

AISC Design Guide 4 - Chap. 2.1-4
Only bolts at compression flange take shear force

Slip Critical:

Flange Force: $F_{flange} := \dfrac{M_u}{d} = 1440.5$ kip

$\phi_{sc} := 1.00$

Mean Slip Coefficient: $\mu_{class.A} := 0.3$ AISC 360-10 J3-8

Nominal Strength: $R_{n.sc} := \mu_{class.A} \cdot F_{flange} = 432.1$ kip

Design Strength: $\phi_{sc} \cdot R_{n.sc} = 432.1$ kip

Shear Capacity of Bolts:

$\phi_v := 0.75$

Number of Bolts at Compression Flange: $n_s := 12$ bolts

Nominal Shear Strength: $F_{nv} := 68$ ksi AISC 360-10 Table J3.2

Nominal Strength: $R_{nv.bolt} := \frac{\pi}{4} \cdot d_b^{\,2} \cdot F_{nv} = 83.45 \ \frac{\text{kip}}{\text{bolt}}$

Design Strength: $\phi_v \cdot n_s \cdot R_{nv.bolt} = 751$ kip

Required Strength: $V_u = 365.8$ kip

$$\left|\begin{array}{l} \text{"Ok"} \quad \text{if } \left(\phi_{sc} \cdot R_{n.sc} \geq V_u\right) \wedge \left(\phi_v \cdot n_s \cdot R_{nv.bolt} \geq V_u\right) \\ \text{"No Good"} \quad \text{otherwise} \end{array}\right. = \text{"Ok"}$$

Bearing & Tearout: End Plate $\phi_{bt} := 0.75$ AISC 360-10 J3-10

Clear Distances:

$L_{C.a1} := d_e - 0.5 \cdot \left(d_b + 0.0625\right) = 0.969$ in

$L_{C.b1} := p_b - \left(d_b + 0.0625\right) = 2.438$ in

$L_{C.b2} := p_b + d_e - 0.5 \cdot \left(d_b + 0.0625\right) = 4.719$ in

$L_{C.c1} := p_{fi} + p_{fo} + t_f - \left(d_b + 0.0625\right) = 3.438$ in

$L_{C.c2} := L_{C.c1} = 3.438$ in

$L_{C.d1} := p_b - \left(d_b + 0.0625\right) = 2.438$ in

Capacity Per Bolt Hole:

$r_{a1} := \min\left(1.2 \cdot L_{C.a1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 122$ kip

$r_{b1} := \min\left(1.2 \cdot L_{C.b1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 307$ kip

$r_{b2} := \min\left(1.2 \cdot L_{C.b2} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 315$ kip

$r_{c1} := \min\left(1.2 \cdot L_{C.c1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 315$ kip

$r_{c2} := \min\left(1.2 \cdot L_{C.c2} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 315$ kip

$r_{d1} := \min\left(1.2 \cdot L_{C.d1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 307$ kip

Nominal Strength: $R_{n.bt} := 2 \cdot \left(r_{a1} + r_{b1} + r_{b2} + r_{c1} + r_{c2} + r_{d1}\right) = 3362.6$ kip

Design Strength: $\phi_{bt} \cdot R_{n.bt} = 2522$ kip

Required Strength: $V_u = 365.8$ kip

$$\left|\begin{array}{l} \text{"Ok"} \quad \text{if } \phi_{bt} \cdot R_{n.bt} \geq V_u \\ \text{"No Good"} \quad \text{otherwise} \end{array}\right. = \text{"Ok"}$$

Intermediate Stiffeners: AISC 360-10 Chap. G

$\frac{h_w}{t_w} = 56$ $\quad 2.46 \cdot \sqrt{\frac{E}{F_y}} = 56.488$ AISC 360-10 Sect. G2-2

$$\left| \begin{array}{ll} \text{"No transverse stiffeners required"} & \text{if } \frac{h_w}{t_w} \le 2.46 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"Transverse stiffeners required"} & \text{otherwise} \end{array} \right. = \text{"No transverse stiffeners required"}$$

Web Plate Shear Buckling Coefficient: $k_v := 5$

Web Shear Coefficient:

$$C_v := \left| \begin{array}{ll} 1.0 & \text{if } \frac{h_w}{t_w} \le 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} \quad \text{(G2-3)} \\ \frac{1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}}}{\frac{h_w}{t_w}} & \text{if } 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} < \frac{h_w}{t_w} \le 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} \quad \text{(G2-4)} \\ \frac{1.51 \cdot k_v \cdot E}{\left(\frac{h_w}{t_w}\right)^2 \cdot F_y} & \text{if } \frac{h_w}{t_w} > 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} \quad \text{(G2-5)} \end{array} \right.$$

$C_v = 1$

$$\phi_v := \left| \begin{array}{ll} 1.0 & \text{if } \frac{h_w}{t_w} \le 2.24 \cdot \sqrt{\frac{E}{F_y}} \\ 0.9 & \text{otherwise} \end{array} \right. = 0.9$$

Area of Web: $A_v := d \cdot t_w = 33$ in^2

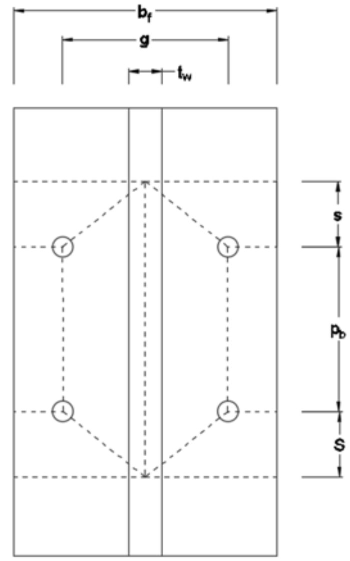
Nominal Strength: $V_n := 0.6 \cdot F_y \cdot A_v \cdot C_v = 1089$ kip

Design Strength: $\phi_v \cdot V_n = 980.1$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi_v \cdot V_n \ge V_u \\ \text{"No Good"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

<u>Bearing Stiffeners</u>: Check Concentrated Force From Actuator

<u>Flange Local Bending</u>: From Yield Line Analysis



$$\phi_{FLB} := 0.9$$

$$s := \frac{1}{2}\cdot\sqrt{b_f \cdot g_{act}} = 4.265 \quad \text{in}$$

$$Y := \frac{\left(4\cdot s^2 + b_f\cdot g_{act} + 2\cdot p_{b.act}\cdot s\right)}{s\cdot\left(g_{act}\right)} = 7.834$$

Nominal Strength: $R_{n.FLB} := F_y \cdot t_f^{\,2} \cdot Y = 430.9 \quad \text{kip}$

Design Strength: $\phi_{FLB} \cdot R_{n.FLB} = 387.8 \quad \text{kip}$

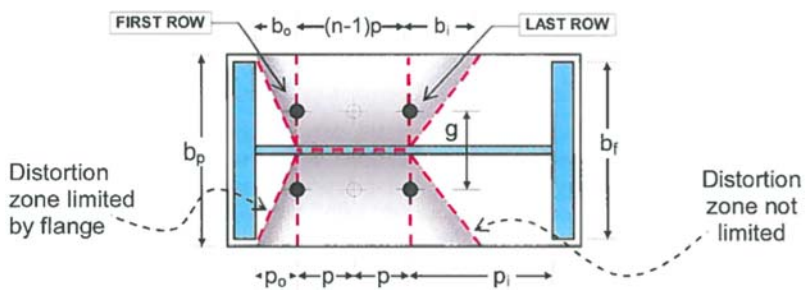
<u>Flange Local Bending</u>: BlueScope Buildings Yield Line Analysis

$$b := \frac{b_f}{\sqrt{2}} = 8.485 \quad \text{in}$$

$$b_i := b = 8.485 \quad \text{in}$$

$$b_o := b = 8.485 \quad \text{in}$$

Number of Bolt Rows: $n := 2$



$$Y := \begin{cases} \dfrac{1}{g_{act}}\cdot\left[\dfrac{b_f^{\,2}}{2}\cdot\left(\dfrac{1}{b_o} + \dfrac{1}{b_i}\right) + b_o + b_i + (n-1)\cdot p_{b.act}\right] & \text{if } p_{b.act} \le \dfrac{b_f}{\sqrt{2}} \\ \dfrac{1}{g_{act}}\cdot\left(2\cdot\dfrac{b_f^{\,2}}{b} + 4\cdot b\right) & \text{otherwise} \end{cases} = 6.702$$

Nominal Strength: $R_{n.BS} := F_y \cdot t_f^{\,2} \cdot Y = 368.6 \quad \text{kip}$

Design Strength: $\phi_{FLB} \cdot R_{n.BS} = 331.7 \quad \text{kip}$

Flange Local Bending: AISCM Chap. 9 Prying Action

$$\tan\left(\frac{\pi}{4}\right) = \frac{\left(\frac{p}{2}\right)}{g_{act} - \frac{t_w}{2}} \quad \text{-->} \quad p := 2\cdot\tan\left(\frac{\pi}{4}\right)\cdot\left(\frac{g_{act}}{2} - \frac{t_w}{2}\right)$$

Tributary Length: $p = 5.3125$ in

Bolt Hole Diameter: $\delta_1 := 1.25$ in

Bolt Diameter: $d_{b.act} := 1.125$ in

Bolt CL to the Face of the Web: $b := \frac{g_{act}}{2} - \frac{t_w}{2} = 2.65625$ in

Bolt Hole Edge to the Face of the Web: $b_1 := b - \frac{d_{b.act}}{2} = 2.09375$ in

Bolt Spacing: $S := p_{b.act} = 6.6875$ in

Required Strength per Bolt: $T := \frac{1}{4}\left(\frac{1}{2}\cdot V_u\right) = 45.73$ kip

1/4 because 4 bolts
1/2 because 2 actuators

$$t_{min.FLB} := \sqrt{\frac{4\cdot T\cdot b_1}{\phi_{FLB}\cdot p\cdot F_u}} = 1.07 \text{ in}$$ AISCM (9-20a)

$$\delta := 1 - \frac{\delta_1}{p} = 0.76471$$ AISCM (9-24)

Bolt CL to Edge of Fitting: $a := \left(\frac{b_f - t_w}{2}\right) - b = 2.969$ in

$$a_1 := \min\left(a + \frac{d_{b.act}}{2}, 1.25\cdot b + \frac{d_{b.act}}{2}\right) = 3.5313 \text{ in}$$

$$\rho := \frac{b_1}{a_1} = 0.59292$$ AISCM (9-26)

Available Tension per Bolt: $B := 0.75\cdot\frac{\pi}{4}\cdot d_{b.act}{}^2\cdot F_t = 84.243$ kip

$$\beta := \frac{1}{\rho}\cdot\left(\frac{B}{T} - 1\right) = 1.42$$ AISCM (9-25)

Thickness Required to Develop Available Strength of the Bolt Without Prying:

$$t_c := \sqrt{\frac{4\cdot B\cdot b_1}{\phi_{FLB}\cdot p\cdot F_u}} = 1.452 \text{ in}$$ AISCM (9-30a)

Ratio of Moment at the Face of the Web to the Moment at the Bolt Line:

$$\alpha := \frac{1}{\delta}\cdot\left[\frac{T}{B}\cdot\left(\frac{t_c}{t_f}\right)^2 - 1\right] = 0.1887$$ AISCM (9-29)

Prying Force per Bolt: $$q := B\cdot\left[\delta\cdot\alpha\cdot\rho\cdot\left(\frac{t_f}{t_c}\right)^2\right] = 3.42 \quad \text{kip}$$ AISCM (9-28)

Value of α that either maximizes the bolt available tensile strength for a given thickness or minimizes the thickness required for a given bolt available tensile strength

$$\alpha_1 := \frac{1}{\delta\cdot(1+\rho)}\cdot\left[\left(\frac{t_c}{t_f}\right)^2 - 1\right] = 0.91$$ AISCM (9-35)

$$Q := \begin{cases} 1 & \text{if } \alpha_1 < 0 \\ \left(\frac{t_f}{t_c}\right)^2\cdot(\delta+1) & \text{if } \alpha_1 > 1 \\ \left(\frac{t_f}{t_c}\right)^2\cdot(\delta\cdot\alpha_1+1) & \text{otherwise} \end{cases}$$

AISCM (9-32)

AISCM (9-34)

AISCM (9-33)

$Q = 0.804$

Available Tensile Strength per Bolt Including Prying Action: $T_{avail} := B\cdot Q = 67.76 \quad \text{kip}$ AISCM (9-31)

Design Strength: $\phi R_{n.pry} := 4\cdot T_{avail} = 271 \quad \text{kip}$

Web Local Yielding: $\phi_{WLY} := 1.00$ AISC 360-10 Chap. J10

Length of Bearing $l_b := 18 \quad \text{in}$

Outside Flange Surface to Fillet Toe $k := t_f = 1 \quad \text{in}$

Nominal Strength: $R_{n.WLY} := t_w\cdot F_y\cdot(5\cdot k + l_b) = 948.8 \quad \text{kip}$

Design Strength: $\phi_{WLY}\cdot R_{n.WLY} = 948.8 \quad \text{kip}$

Web Crippling: $\phi_{WC} := 0.75$

Nominal Strength: $$R_{n.WC} := 0.8\cdot t_w^2\cdot\left[1 + 3\cdot\frac{l_b}{d}\cdot\left(\frac{t_w}{t_f}\right)^{1.5}\right]\cdot\sqrt{\frac{E\cdot F_y\cdot t_f}{t_w}} = 1179 \quad \text{kip}$$

Design Strength: $\phi_{WC}\cdot R_{n.WC} = 884.5 \quad \text{kip}$

<u>Web Sidesway Buckling</u>: $\phi_{WSB} := 0.85$

Compression Flange Not Restrained Against Rotation $L_b := 15 \cdot 12 = 180$ in

$C_r := 480000$ ksi

Nominal Strength: $R_{n.WSB} := \frac{C_r \cdot t_w^{\,3} \cdot t_f}{h_w^{\,2}} \cdot \left[0.4 \cdot \left[\frac{\left(\frac{h_w}{t_w}\right)}{\left(\frac{L_b}{b_f}\right)} \right]^3 \right] = 2389.3$ kip

Design Strength: $\phi_{WSB} \cdot R_{n.WSB} = 2030.9$ kip

$$\left| \begin{array}{ll} \text{"WSB applies"} & \text{if } \frac{\left(\frac{h_w}{t_w}\right)}{\left(\frac{L_b}{b_f}\right)} \le 1.7 \\ \text{"WSB does not apply"} & \text{otherwise} \end{array} \right. = \text{"WSB does not apply"}$$

<u>Web Compression Buckling</u>: $\phi_{WCB} := 0.9$

Nominal Strength: $R_{n.WCB} := \frac{24 \cdot t_w^{\,3} \cdot \sqrt{E \cdot F_y}}{h_w} = 304.5$ kip

Design Strength: $\phi_{WCB} \cdot R_{n.WCB} = 274$ kip

<u>Concentrated Force Strength</u>:

$\phi R_{n.min} := \min(\phi_{FLB} \cdot R_{n.FLB}, \phi_{FLB} \cdot R_{n.BS}, \phi R_{n.pry}, \phi_{WLY} \cdot R_{n.WLY}, \phi_{WC} \cdot R_{n.WC}, \phi_{WSB} \cdot R_{n.WSB}, \phi_{WCB} \cdot R_{n.WC}$

Design Strength: $\phi R_{n.min} = 271$ kip

Required Strength: $F_{ul} := \frac{V_u}{2} = 182.9$ kip 1/2 because 2 actuators

$$\left| \begin{array}{ll} \text{"Need bearing stiffeners"} & \text{if } \phi R_{n.min} < F_{ul} \\ \text{"Ok"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

24 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12.00$ in

Flange Thickness: $t_f := 1.00$ in

Web Thickness: $t_w := 0.50$ in

Total Beam Depth: $d := 24.00$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 22.00$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 11$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 12$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 23.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 11.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 11.5$ in

Plastic Section Modulus: $Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f}\right) + \left(\frac{A_w}{2}\right) \cdot \left(\frac{h_w}{4}\right)\right] = 336.5$ in^3

Moment of Inertia: $I_x := \frac{1}{12} \cdot t_w \cdot h_w^{\ 3} + 2\left(\frac{1}{12} \cdot b_f \cdot t_f^{\ 3} + b_f \cdot t_f \cdot CtC_{top.f}^{\ 2}\right) = 3620$ in^4

Elastic Section Modulus: $S_x := \frac{I_x}{C_w} = 301.64$ in^3

Peak Connection Strength Factor: $C_{pr} := \frac{(F_y + F_u)}{2 \cdot F_y} = 1.136$

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 18508$ kip·in

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 23134$ kip·in

Actuator Input:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Bolt Gage: $g_{act} := 6 + \frac{1}{16} = 6.0625$ in

Bolt Spacing: $p_{b.act} := 6 + \frac{11}{16} = 6.6875$ in

Beam 2 - End Plate 12ES-0.875-0.75-24

Limit States: End Plate Yielding & Rupture of Bolts With Prying Action

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 0.75$ in

End Plate Width: $b_p := 12.00$ in

Inner Bolt Gage: $g := 3.50$ in

Outer Bolt Gage: $g_o := 2.625$ in

Flange to Outer Bolt CL: $p_{fo} := 1.50$ in

Flange to Inner Bolt CL: $p_{fi} := 1.50$ in

Bolt Spacing: $p_b := 2.625$ in

Outer Bolt to Edge: $d_e := 1.125$ in

Top of Flange to Edge: $p_{ext} := p_{fo} + p_b + d_e = 5.25$ in

Bolts:

$d_b := 0.875$ in

$T_b := 49$ kip

$F_t := 113$ ksi

Stiffeners:

$L_{st} := 9.125$ in

$S_h := L_{st} + t_p = 9.875$ in

Actuator Force:

M_q = $M_u := 1.25 \cdot (14872) = 18590$ kip·in   Increased for 25% bolt over strength

$$V_u := \frac{M_u}{L_h} = 107.3 \quad \text{kip}$$

Check Bolts:

AISC Design Guide 4 - Chap. 2.1-4
Only bolts at compression flange take shear force

Slip Critical:

Flange Force: $F_{flange} := \frac{M_u}{d} = 774.6$ kip

$\phi_{sc} := 1.00$

Mean Slip Coefficient: $\mu_{class.A} := 0.3$ AISC 360-10 J3-8

Nominal Strength: $R_{n.sc} := \mu_{class.A} \cdot F_{flange} = 232.4$ kip

Design Strength: $\phi_{sc} \cdot R_{n.sc} = 232.4$ kip

Shear Capacity of Bolts:

$\phi_v := 0.75$

Number of Bolts at Compression Flange: $n_s := 12$ bolts

Nominal Shear Strength: $F_{nv} := 68$ ksi AISC 360-10 Table J3.2

Nominal Strength: $R_{nv.bolt} := \frac{\pi}{4} \cdot d_b^{\ 2} \cdot F_{nv} = 40.89$ $\frac{\text{kip}}{\text{bolt}}$

Design Strength: $\phi_v \cdot n_s \cdot R_{nv.bolt} = 368$ kip

Required Strength: $V_u = 107.3$ kip

$$\begin{array}{|ll} \text{"Ok"} & \text{if } \left(\phi_{sc} \cdot R_{n.sc} \ge V_u\right) \wedge \left(\phi_v \cdot n_s \cdot R_{nv.bolt} \ge V_u\right) \\ \text{"No Good"} & \text{otherwise} \end{array} = \text{"Ok"}$$

Bearing & Tearout: End Plate $\phi_{bt} := 0.75$ AISC 360-10 J3-10

Clear Distances:

$L_{C.a1} := d_e - 0.5 \cdot \left(d_b + 0.0625\right) = 0.656$ in

$L_{C.b1} := p_b - \left(d_b + 0.0625\right) = 1.688$ in

$L_{C.b2} := p_b + d_e - 0.5 \cdot \left(d_b + 0.0625\right) = 3.281$ in

$L_{C.c1} := p_{fi} + p_{fo} + t_f - \left(d_b + 0.0625\right) = 3.063$ in

$L_{C.c2} := L_{C.c1} = 3.063$ in

$L_{C.d1} := p_b - \left(d_b + 0.0625\right) = 1.688$ in

Capacity Per Bolt Hole:

$r_{a1} := \min\left(1.2 \cdot L_{C.a1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 41$ kip

$r_{b1} := \min\left(1.2 \cdot L_{C.b1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 106$ kip

$r_{b2} := \min\left(1.2 \cdot L_{C.b2} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 110$ kip

$r_{c1} := \min\left(1.2 \cdot L_{C.c1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 110$ kip

$r_{c2} := \min\left(1.2 \cdot L_{C.c2} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 110$ kip

$r_{d1} := \min\left(1.2 \cdot L_{C.d1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 106$ kip

Nominal Strength: $R_{n.bt} := 2 \cdot \left(r_{a1} + r_{b1} + r_{b2} + r_{c1} + r_{c2} + r_{d1}\right) = 1169.4$ kip

Design Strength: $\phi_{bt} \cdot R_{n.bt} = 877.1$ kip

Required Strength: $V_u = 107.3$ kip

$$\begin{array}{|ll} \text{"Ok"} & \text{if } \phi_{bt} \cdot R_{n.bt} \ge V_u \\ \text{"No Good"} & \text{otherwise} \end{array} = \text{"Ok"}$$

Intermediate Stiffeners: AISC 360-10 Chap. G

$\frac{h_w}{t_w} = 44$ $\qquad 2.46 \cdot \sqrt{\frac{E}{F_y}} = 56.488$ $\qquad$ AISC 360-10 Sect. G2-2

$$\begin{array}{|l} \text{"No transverse stiffeners required"} \quad \text{if } \frac{h_w}{t_w} \le 2.46 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"Transverse stiffeners required"} \quad \text{otherwise} \end{array} = \text{"No transverse stiffeners required"}$$

Web Plate Shear Buckling Coefficient: $k_v := 5$

Web Shear Coefficient:

$$C_v := \begin{array}{|ll} 1.0 \quad \text{if } \frac{h_w}{t_w} \le 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} & \text{(G2-3)} \\ \frac{1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}}}{\frac{h_w}{t_w}} \quad \text{if } 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} < \frac{h_w}{t_w} \le 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} & \text{(G2-4)} \\ \frac{1.51 \cdot k_v \cdot E}{\left(\frac{h_w}{t_w}\right)^2 \cdot F_y} \quad \text{if } \frac{h_w}{t_w} > 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} & \text{(G2-5)} \end{array}$$

$C_v = 1$

$$\phi_v := \begin{array}{|l} 1.0 \quad \text{if } \frac{h_w}{t_w} \le 2.24 \cdot \sqrt{\frac{E}{F_y}} \\ 0.9 \quad \text{otherwise} \end{array} = 1$$

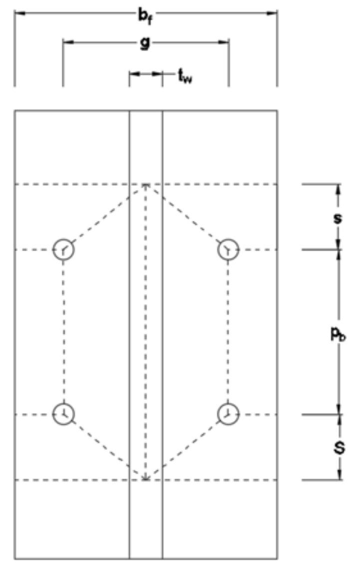
Area of Web: $A_v := d \cdot t_w = 12$ $\quad in^2$

Nominal Strength: $V_n := 0.6 \cdot F_y \cdot A_v \cdot C_v = 396$ $\quad$ kip

Design Strength: $\phi_v \cdot V_n = 396$ $\quad$ kip

$$\begin{array}{|l} \text{"Ok"} \quad \text{if } \phi_v \cdot V_n \ge V_u \\ \text{"No Good"} \quad \text{otherwise} \end{array} = \text{"Ok"}$$

Bearing Stiffeners: Check Concentrated Force From Actuator



Flange Local Bending: From Yield Line Analysis

$$\phi_{FLB} := 0.9$$

$$s := \frac{1}{2} \cdot \sqrt{b_f \cdot g_{act}} = 4.265 \quad \text{in}$$

$$Y := \frac{\left(4 \cdot s^2 + b_f \cdot g_{act} + 2 \cdot p_{b.act} \cdot s\right)}{s \cdot \left(g_{act}\right)} = 7.834$$

Nominal Strength: $R_{n.FLB} := F_y \cdot t_f^{\,2} \cdot Y = 430.9 \quad \text{kip}$

Design Strength: $\phi_{FLB} \cdot R_{n.FLB} = 387.8 \quad \text{kip}$

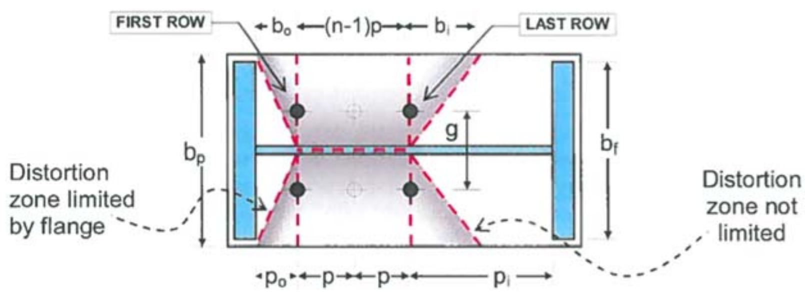
Flange Local Bending: BlueScope Buildings Yield Line Analysis

$$b := \frac{b_f}{\sqrt{2}} = 8.485 \quad \text{in}$$

$$b_i := b = 8.485 \quad \text{in}$$

$$b_o := b = 8.485 \quad \text{in}$$

Number of Bolt Rows: $n := 2$



$$Y := \begin{cases} \dfrac{1}{g_{act}} \cdot \left[\dfrac{b_f^{\,2}}{2} \cdot \left(\dfrac{1}{b_o} + \dfrac{1}{b_i}\right) + b_o + b_i + (n-1) \cdot p_{b.act}\right] & \text{if } p_{b.act} \le \dfrac{b_f}{\sqrt{2}} \\ \dfrac{1}{g_{act}} \cdot \left(2 \cdot \dfrac{b_f^{\,2}}{b} + 4 \cdot b\right) & \text{otherwise} \end{cases} = 6.702$$

Nominal Strength: $R_{n.BS} := F_y \cdot t_f^{\,2} \cdot Y = 368.6 \quad \text{kip}$

Design Strength: $\phi_{FLB} \cdot R_{n.BS} = 331.7 \quad \text{kip}$

Flange Local Bending: AISCM Chap. 9 Prying Action

$$\tan\left(\frac{\pi}{4}\right) = \frac{\left(\frac{p}{2}\right)}{g_{act} - \frac{t_w}{2}} \quad \text{-->} \quad p := 2\cdot\tan\left(\frac{\pi}{4}\right)\cdot\left(\frac{g_{act}}{2} - \frac{t_w}{2}\right)$$

Tributary Length: $p = 5.5625$ in

Bolt Hole Diameter: $\delta_1 := 1.25$ in

Bolt Diameter: $d_{b.act} := 1.125$ in

Bolt CL to the Face of the Web: $b := \frac{g_{act}}{2} - \frac{t_w}{2} = 2.78125$ in

Bolt Hole Edge to the Face of the Web: $b_1 := b - \frac{d_{b.act}}{2} = 2.21875$ in

Bolt Spacing: $S := p_{b.act} = 6.6875$ in

Required Strength per Bolt: $T := \frac{1}{4}\cdot V_u = 26.825$ kip   1/4 because 4 bolts

$$t_{min.FLB} := \sqrt{\frac{4\cdot T\cdot b_1}{\phi_{FLB}\cdot p\cdot F_u}} = 0.824 \text{ in}$$ AISCM (9-20a)

$$\delta := 1 - \frac{\delta_1}{p} = 0.77528$$ AISCM (9-24)

Bolt CL to Edge of Fitting: $a := \left(\frac{b_f - t_w}{2}\right) - b = 2.969$ in

$$a_1 := \min\left(a + \frac{d_{b.act}}{2}, 1.25\cdot b + \frac{d_{b.act}}{2}\right) = 3.5313 \text{ in}$$

$$\rho := \frac{b_1}{a_1} = 0.62832$$ AISCM (9-26)

Available Tension per Bolt: $B := 0.75\cdot\frac{\pi}{4}\cdot d_{b.act}^{\ 2}\cdot F_t = 84.243$ kip

$$\beta := \frac{1}{\rho}\cdot\left(\frac{B}{T} - 1\right) = 3.407$$ AISCM (9-25)

Thickness Required to Develop Available Strength of the Bolt Without Prying:

$$t_c := \sqrt{\frac{4\cdot B\cdot b_1}{\phi_{FLB}\cdot p\cdot F_u}} = 1.461 \text{ in}$$ AISCM (9-30a)

Ratio of Moment at the Face of the Web to the Moment at the Bolt Line:

$$\alpha := \frac{1}{\delta}\cdot\left[\frac{T}{B}\cdot\left(\frac{t_c}{t_f}\right)^2 - 1\right] = -0.4136$$ AISCM (9-29)

Prying Force per Bolt: $$q := B\cdot\left[\delta\cdot\alpha\cdot\rho\cdot\left(\frac{t_f}{t_c}\right)^2\right] = -7.955 \quad \text{kip}$$ AISCM (9-28)

Value of α that either maximizes the bolt available tensile strength for a given thickness or minimizes the thickness required for a given bolt available tensile strength

$$\alpha_1 := \frac{1}{\delta\cdot(1+\rho)}\cdot\left[\left(\frac{t_c}{t_f}\right)^2 - 1\right] = 0.898$$ AISCM (9-35)

$$Q := \begin{cases} 1 & \text{if } \alpha_1 < 0 \\ \left(\frac{t_f}{t_c}\right)^2\cdot(\delta+1) & \text{if } \alpha_1 > 1 \\ \left(\frac{t_f}{t_c}\right)^2\cdot\left(\delta\cdot\alpha_1 + 1\right) & \text{otherwise} \end{cases}$$

AISCM (9-32)

AISCM (9-34)

AISCM (9-33)

$Q = 0.795$

Available Tensile Strength per Bolt Including Prying Action: $T_{avail} := B\cdot Q = 66.97 \quad \text{kip}$ AISCM (9-31)

Design Strength: $\phi R_{n.pry} := 4\cdot T_{avail} = 267.9 \quad \text{kip}$

Web Local Yielding: $\phi_{WLY} := 1.00$ AISC 360-10 Chap. J10

Length of Bearing $l_b := 18 \quad \text{in}$

Outside Flange Surface to Fillet Toe $k := t_f = 1 \quad \text{in}$

Nominal Strength: $R_{n.WLY} := t_w\cdot F_y\cdot\left(5\cdot k + l_b\right) = 632.5 \quad \text{kip}$

Design Strength: $\phi_{WLY}\cdot R_{n.WLY} = 632.5 \quad \text{kip}$

Web Crippling: $\phi_{WC} := 0.75$

Nominal Strength: $$R_{n.WC} := 0.8\cdot t_w^{\,2}\cdot\left[1 + 3\cdot\frac{l_b}{d}\cdot\left(\frac{t_w}{t_f}\right)^{1.5}\right]\cdot\sqrt{\frac{E\cdot F_y\cdot t_f}{t_w}} = 641 \quad \text{kip}$$

Design Strength: $\phi_{WC}\cdot R_{n.WC} = 481 \quad \text{kip}$

Web Sidesway Buckling: $\phi_{WSB} := 0.85$

Compression Flange Not Restrained Against Rotation $L_b := 15 \cdot 12 = 180$ in

$C_r := 480000$ ksi

Nominal Strength: $R_{n.WSB} := \dfrac{C_r \cdot t_w^{\;3} \cdot t_f}{h_w^{\;2}} \cdot \left[0.4 \cdot \left[\dfrac{\left(\dfrac{h_w}{t_w}\right)}{\left(\dfrac{L_b}{b_f}\right)}\right]^3\right] = 1251.6$ kip

Design Strength: $\phi_{WSB} \cdot R_{n.WSB} = 1063.8$ kip

$$\left| \begin{array}{ll} \text{"WSB applies"} & \text{if } \dfrac{\left(\dfrac{h_w}{t_w}\right)}{\left(\dfrac{L_b}{b_f}\right)} \le 1.7 \\ \text{"WSB does not apply"} & \text{otherwise} \end{array} \right. = \text{"WSB does not apply"}$$

Web Compression Buckling: $\phi_{WCB} := 0.9$

Nominal Strength: $R_{n.WCB} := \dfrac{24 \cdot t_w^{\;3} \cdot \sqrt{E \cdot F_y}}{h_w} = 172.2$ kip

Design Strength: $\phi_{WCB} \cdot R_{n.WCB} = 155$ kip

Concentrated Force Strength:

$\phi R_{n.min} := \min\left(\phi_{FLB} \cdot R_{n.FLB}, \phi_{FLB} \cdot R_{n.BS}, \phi R_{n.pry}, \phi_{WLY} \cdot R_{n.WLY}, \phi_{WC} \cdot R_{n.WC}, \phi_{WSB} \cdot R_{n.WSB}, \phi_{WCB} \cdot R_{n.WC}\right.$

Design Strength: $\phi R_{n.min} = 155$ kip

Required Strength: $F_{ul} := V_u = 107.3$ kip

$$\left| \begin{array}{ll} \text{"Need bearing stiffeners"} & \text{if } \phi R_{n.min} < F_{ul} \\ \text{"Ok"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Beam 2 - End Plate 12ES-1.125-1.25-24

Limit State: Plastic Hinging of the Beam

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.25$ in

End Plate Width: $b_p := 14.00$ in

Inner Bolt Gage: $g := 4.00$ in

Outer Bolt Gage: $g_o := 3.375$ in

Flange to Outer Bolt CL: $p_{fo} := 1.75$ in

Flange to Inner Bolt CL: $p_{fi} := 1.75$ in

Bolt Spacing: $p_b := 3.375$ in

Outer Bolt to Edge: $d_e := 1.50$ in

Top of Flange to Edge: $p_{ext} := p_{fo} + p_b + d_e = 6.625$ in

Bolts:

$d_b := 1.125$ in

$T_b := 80$ kip

$F_t := 113$ ksi

Stiffeners:

$L_{st} := 11.5$ in

$S_h := L_{st} + t_p = 12.75$ in

Actuator Force:

M_f = $M_u := 24972$ kip·in

$$V_u := \frac{M_u}{L_h} = 144.1 \quad \text{kip}$$

Check Bolts:

AISC Design Guide 4 - Chap. 2.1-4
Only bolts at compression flange take shear force

Slip Critical:

Flange Force: $F_{flange} := \dfrac{M_u}{d} = 1040.5$ kip

$\phi_{sc} := 1.00$

Mean Slip Coefficient: $\mu_{class.A} := 0.3$ AISC 360-10 J3-8

Nominal Strength: $R_{n.sc} := \mu_{class.A} \cdot F_{flange} = 312.1$ kip

Design Strength: $\phi_{sc} \cdot R_{n.sc} = 312.1$ kip

Shear Capacity of Bolts:

$\phi_v := 0.75$

Number of Bolts at Compression Flange: $n_s := 12$ bolts

Nominal Shear Strength: $F_{nv} := 68$ ksi AISC 360-10 Table J3.2

Nominal Strength: $R_{nv.bolt} := \frac{\pi}{4} \cdot d_b^{\ 2} \cdot F_{nv} = 67.59 \ \frac{kip}{bolt}$

Design Strength: $\phi_v \cdot n_s \cdot R_{nv.bolt} = 608.3$ kip

Required Strength: $V_u = 144.1$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \left(\phi_{sc} \cdot R_{n.sc} \ge V_u\right) \wedge \left(\phi_v \cdot n_s \cdot R_{nv.bolt} \ge V_u\right) \\ \text{"No Good"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Bearing & Tearout: End Plate $\phi_{bt} := 0.75$ AISC 360-10 J3-10

Clear Distances:

$L_{C.a1} := d_e - 0.5 \cdot \left(d_b + 0.0625\right) = 0.906$ in

$L_{C.b1} := p_b - \left(d_b + 0.0625\right) = 2.188$ in

$L_{C.b2} := p_b + d_e - 0.5 \cdot \left(d_b + 0.0625\right) = 4.281$ in

$L_{C.c1} := p_{fi} + p_{fo} + t_f - \left(d_b + 0.0625\right) = 3.313$ in

$L_{C.c2} := L_{C.c1} = 3.313$ in

$L_{C.d1} := p_b - \left(d_b + 0.0625\right) = 2.188$ in

Capacity Per Bolt Hole:

$r_{a1} := \min\left(1.2 \cdot L_{C.a1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 95$ kip

$r_{b1} := \min\left(1.2 \cdot L_{C.b1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 230$ kip

$r_{b2} := \min\left(1.2 \cdot L_{C.b2} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 236$ kip

$r_{c1} := \min\left(1.2 \cdot L_{C.c1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 236$ kip

$r_{c2} := \min\left(1.2 \cdot L_{C.c2} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 236$ kip

$r_{d1} := \min\left(1.2 \cdot L_{C.d1} \cdot t_p \cdot F_{pu}, 2.4 \cdot d_b \cdot t_p \cdot F_{pu}\right) = 230$ kip

Nominal Strength: $R_{n.bt} := 2 \cdot \left(r_{a1} + r_{b1} + r_{b2} + r_{c1} + r_{c2} + r_{d1}\right) = 2526.6$ kip

Design Strength: $\phi_{bt} \cdot R_{n.bt} = 1894.9$ kip

Required Strength: $V_u = 144.1$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi_{bt} \cdot R_{n.bt} \ge V_u \\ \text{"No Good"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Intermediate Stiffeners: AISC 360-10 Chap. G

$\frac{h_w}{t_w} = 44$ $2.46 \cdot \sqrt{\frac{E}{F_y}} = 56.488$ AISC 360-10 Sect. G2-2

$$\begin{cases} \text{"No transverse stiffeners required"} & \text{if } \frac{h_w}{t_w} \le 2.46 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"Transverse stiffeners required"} & \text{otherwise} \end{cases} = \text{"No transverse stiffeners required"}$$

Web Plate Shear Buckling Coefficient: $k_v := 5$

Web Shear Coefficient:

$$C_v := \begin{cases} 1.0 & \text{if } \frac{h_w}{t_w} \le 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} & \text{(G2-3)} \\ \dfrac{1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}}}{\frac{h_w}{t_w}} & \text{if } 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} < \frac{h_w}{t_w} \le 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} & \text{(G2-4)} \\ \dfrac{1.51 \cdot k_v \cdot E}{\left(\frac{h_w}{t_w}\right)^2 \cdot F_y} & \text{if } \frac{h_w}{t_w} > 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} & \text{(G2-5)} \end{cases}$$

$C_v = 1$

$$\phi_v := \begin{cases} 1.0 & \text{if } \frac{h_w}{t_w} \le 2.24 \cdot \sqrt{\frac{E}{F_y}} \\ 0.9 & \text{otherwise} \end{cases} = 1$$

Area of Web: $A_v := d \cdot t_w = 12$ in^2

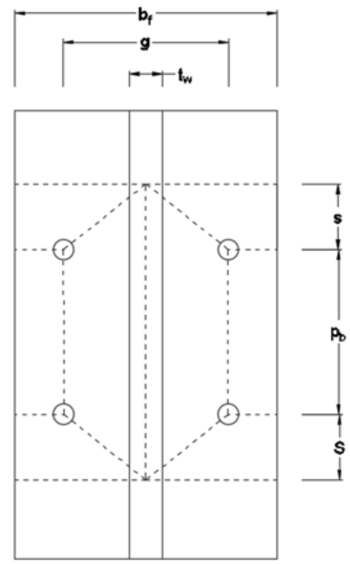
Nominal Strength: $V_n := 0.6 \cdot F_y \cdot A_v \cdot C_v = 396$ kip

Design Strength: $\phi_v \cdot V_n = 396$ kip

$$\begin{cases} \text{"Ok"} & \text{if } \phi_v \cdot V_n \ge V_u \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Bearing Stiffeners: Check Concentrated Force From Actuator

Flange Local Bending: From Yield Line Analysis



$\phi_{FLB} := 0.9$

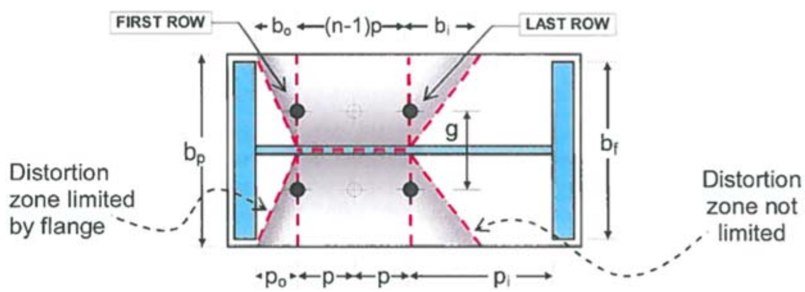
$s := \frac{1}{2} \cdot \sqrt{b_f \cdot g_{act}} = 4.265$ in

$Y := \frac{\left(4 \cdot s^2 + b_f \cdot g_{act} + 2 \cdot p_{b.act} \cdot s\right)}{s \cdot \left(g_{act}\right)} = 7.834$

Nominal Strength: $R_{n.FLB} := F_y \cdot t_f^{\ 2} \cdot Y = 430.9$ kip

Design Strength: $\phi_{FLB} \cdot R_{n.FLB} = 387.8$ kip

Flange Local Bending: BlueScope Buildings Yield Line Analysis



$b := \frac{b_f}{\sqrt{2}} = 8.485$ in

$b_i := b = 8.485$ in

$b_o := b = 8.485$ in

Number of Bolt Rows: $n := 2$

$$Y := \begin{cases} \frac{1}{g_{act}} \cdot \left[\frac{b_f^{\ 2}}{2} \cdot \left(\frac{1}{b_o} + \frac{1}{b_i}\right) + b_o + b_i + (n-1) \cdot p_{b.act}\right] & \text{if } p_{b.act} \le \frac{b_f}{\sqrt{2}} \\ \frac{1}{g_{act}} \cdot \left(2 \cdot \frac{b_f^{\ 2}}{b} + 4 \cdot b\right) & \text{otherwise} \end{cases} = 6.702$$

Nominal Strength: $R_{n.BS} := F_y \cdot t_f^{\ 2} \cdot Y = 368.6$ kip

Design Strength: $\phi_{FLB} \cdot R_{n.BS} = 331.7$ kip

Flange Local Bending: AISCM Chap. 9 Prying Action

$$\tan\left(\frac{\pi}{4}\right) = \frac{\left(\frac{p}{2}\right)}{g_{act} - \frac{t_w}{2}} \quad \text{-->} \quad p := 2\cdot\tan\left(\frac{\pi}{4}\right)\cdot\left(\frac{g_{act}}{2} - \frac{t_w}{2}\right)$$

Tributary Length: $p = 5.5625$ in

Bolt Hole Diameter: $\delta_1 := 1.25$ in

Bolt Diameter: $d_{b.act} := 1.125$ in

Bolt CL to the Face of the Web: $b := \frac{g_{act}}{2} - \frac{t_w}{2} = 2.78125$ in

Bolt Hole Edge to the Face of the Web: $b_1 := b - \frac{d_{b.act}}{2} = 2.21875$ in

Bolt Spacing: $S := p_{b.act} = 6.6875$ in

Required Strength per Bolt: $T := \frac{1}{4}\cdot V_u = 36.035$ kip 1/4 because 4 bolts

$$t_{min.FLB} := \sqrt{\frac{4\cdot T\cdot b_1}{\phi_{FLB}\cdot p\cdot F_u}} = 0.955 \text{ in}$$ AISCM (9-20a)

$$\delta := 1 - \frac{\delta_1}{p} = 0.77528$$ AISCM (9-24)

Bolt CL to Edge of Fitting: $a := \left(\frac{b_f - t_w}{2}\right) - b = 2.969$ in

$$a_1 := \min\left(a + \frac{d_{b.act}}{2}, 1.25\cdot b + \frac{d_{b.act}}{2}\right) = 3.5313 \text{ in}$$

$$\rho := \frac{b_1}{a_1} = 0.62832$$ AISCM (9-26)

Available Tension per Bolt: $B := 0.75\cdot\frac{\pi}{4}\cdot d_{b.act}^{\,2}\cdot F_t = 84.243$ kip

$$\beta := \frac{1}{\rho}\cdot\left(\frac{B}{T} - 1\right) = 2.129$$ AISCM (9-25)

Thickness Required to Develop Available Strength of the Bolt Without Prying:

$$t_c := \sqrt{\frac{4\cdot B\cdot b_1}{\phi_{FLB}\cdot p\cdot F_u}} = 1.461 \text{ in}$$ AISCM (9-30a)

Ratio of Moment at the Face of the Web to the Moment at the Bolt Line:

$$\alpha := \frac{1}{\delta}\cdot\left[\frac{T}{B}\cdot\left(\frac{t_c}{t_f}\right)^2 - 1\right] = -0.1127$$ AISCM (9-29)

Prying Force per Bolt: $q := B\cdot\left[\delta\cdot\alpha\cdot\rho\cdot\left(\frac{t_f}{t_c}\right)^2\right] = -2.169$ kip AISCM (9-28)

Value of α that either maximizes the bolt available tensile strength for a given thickness or minimizes the thickness required for a given bolt available tensile strength

$$\alpha_1 := \frac{1}{\delta\cdot(1+\rho)}\cdot\left[\left(\frac{t_c}{t_f}\right)^2 - 1\right] = 0.898$$ AISCM (9-35)

$$Q := \begin{cases} 1 & \text{if } \alpha_1 < 0 \\ \left(\frac{t_f}{t_c}\right)^2\cdot(\delta+1) & \text{if } \alpha_1 > 1 \\ \left(\frac{t_f}{t_c}\right)^2\cdot\left(\delta\cdot\alpha_1 + 1\right) & \text{otherwise} \end{cases}$$

AISCM (9-32)

AISCM (9-34)

AISCM (9-33)

$Q = 0.795$

Available Tensile Strength per Bolt Including Prying Action: $T_{avail} := B\cdot Q = 66.97$ kip AISCM (9-31)

Design Strength: $\phi R_{n.pry} := 4\cdot T_{avail} = 267.9$ kip

<u>Web Local Yielding</u>: $\phi_{WLY} := 1.00$ AISC 360-10 Chap. J10

Length of Bearing $l_b := 18$ in

Outside Flange Surface to Fillet Toe $k := t_f = 1$ in

Nominal Strength: $R_{n.WLY} := t_w\cdot F_y\cdot\left(5\cdot k + l_b\right) = 632.5$ kip

Design Strength: $\phi_{WLY}\cdot R_{n.WLY} = 632.5$ kip

<u>Web Crippling</u>: $\phi_{WC} := 0.75$

Nominal Strength: $R_{n.WC} := 0.8\cdot t_w^{\,2}\cdot\left[1 + 3\cdot\frac{l_b}{d}\cdot\left(\frac{t_w}{t_f}\right)^{1.5}\right]\cdot\sqrt{\frac{E\cdot F_y\cdot t_f}{t_w}} = 641$ kip

Design Strength: $\phi_{WC}\cdot R_{n.WC} = 481$ kip

Web Sidesway Buckling: $\phi_{WSB} := 0.85$

Compression Flange Not Restrained Against Rotation $L_b := 15 \cdot 12 = 180$ in

$C_r := 480000$ ksi

Nominal Strength: $R_{n.WSB} := \frac{C_r \cdot t_w^{\ 3} \cdot t_f}{h_w^{\ 2}} \cdot \left[0.4 \cdot \left[\frac{\left(\frac{h_w}{t_w}\right)}{\left(\frac{L_b}{b_f}\right)} \right]^3 \right] = 1251.6$ kip

Design Strength: $\phi_{WSB} \cdot R_{n.WSB} = 1063.8$ kip

$$\begin{array}{|ll} \text{"WSB applies"} & \text{if } \frac{\left(\frac{h_w}{t_w}\right)}{\left(\frac{L_b}{b_f}\right)} \le 1.7 \\ \text{"WSB does not apply"} & \text{otherwise} \end{array} = \text{"WSB does not apply"}$$

Web Compression Buckling: $\phi_{WCB} := 0.9$

Nominal Strength: $R_{n.WCB} := \frac{24 \cdot t_w^{\ 3} \cdot \sqrt{E \cdot F_y}}{h_w} = 172.2$ kip

Design Strength: $\phi_{WCB} \cdot R_{n.WCB} = 155$ kip

Concentrated Force Strength:

$\phi R_{n.min} := \min(\phi_{FLB} \cdot R_{n.FLB}, \phi_{FLB} \cdot R_{n.BS}, \phi R_{n.pry}, \phi_{WLY} \cdot R_{n.WLY}, \phi_{WC} \cdot R_{n.WC}, \phi_{WSB} \cdot R_{n.WSB}, \phi_{WCB} \cdot R_{n.WC}$

Design Strength: $\phi R_{n.min} = 155$ kip

Required Strength: $F_{ul} := V_u = 144.1$ kip

$$\begin{array}{|ll} \text{"Need bearing stiffeners"} & \text{if } \phi R_{n.min} < F_{ul} \\ \text{"Ok"} & \text{otherwise} \end{array} = \text{"Ok"}$$

44 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

End Plate Input:

Yield Strength:

$F_{py} := 55$ ksi

Weld Input:

Filler Metal Classification Strength:

$F_{exx} := 70$ ksi

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12$ in

Flange Thickness: $t_f := 1$ in

Web Thickness: $t_w := 0.75$ in

Total Beam Depth: $d := 44$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 42$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 31.5$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 22$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 43.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 21.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 21.5$ in

Moment of Inertia: $I_x := \frac{1}{12} \cdot t_w \cdot h_w^3 + 2\left(\frac{1}{12} \cdot b_f \cdot t_f^3 + b_f \cdot t_f \cdot CtC_{top.f}^2\right) = 15727$ in^4

Elastic Section Modulus: $S_x := \frac{I_x}{C_w} = 714.84$ in^3

Actuator Input:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Beam Web to Flange Weld

Within at Least the Depth of the Beam or 3 Times the Width of Flange: AISC 358-10 Sec. 6.4 (1)

Length of Weld: $L_{weld} := \min(d, 3 \cdot b_f) = 36$ in

$t_{weld} := \max(0.75 \cdot t_w, 0.25) = 0.563$ in

Weld Thickness: $t_{weld} = \text{ceil}(16 \cdot t_{weld}) = 9$ 16[th]s of an inch

Use 9/16" fillet weld on both sides of web or CJP

Remaining Portion of Beam:

Maximum Vertical Shear: $V_u := 366$ kip

First Moment of Area: $Q := b_f \cdot t_f \cdot \left(\frac{h_w}{2} + \frac{t_f}{2} \right) = 258 \quad in^3$

Shear Flow: $q := \frac{V_u \cdot Q}{I_x} = 6.004 \ \frac{kip}{in}$

$\phi_v := 0.75$

For 1 side:

$\phi_v \cdot \frac{a}{16} \cdot (0.707) \cdot (0.6) \cdot F_{exx} = q$ --> $a := \frac{16 \cdot q}{\phi_v \cdot (0.707) \cdot (0.6) \cdot F_{exx}} = 4.314$ 16[th]s of an inch

$a := \text{ceil}(a) = 5$ 16[th]s of an inch

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Use 5/16" fillet weld on one side of the web

Beam 1 - End Plate 12ES-0.75-1.00-44

Limit State: Rupture of Bolts without Prying Action

End Plate:

End Plate Thickness: $t_p := 1.00$ in

Flange to Inner Bolt CL: $p_{fi} := 1.50$ in

Bolt Spacing: $p_b := 2.25$ in

Top of Flange to Edge: $p_{ext} := 4.75$ in

Bolts:

Bolt Diameter: $d_b := 0.75$ in

Bolt Pretension: $T_b := 35$ kip

Stiffeners:

Length of Stiffener: $L_{st} := 8.25$ in

Location of Plastic Hinge: $S_h := L_{st} + t_p = 9.25$ in

Actuator Forces:

$M_{np} = M_u := 1.25 \cdot (25760) = 32200 \text{ kip} \cdot \text{in}$ Increased for 25% bolt over strength

$$F_{total} := \frac{M_u}{L_h} = 185.9 \text{ kip}$$

Stiffener to Beam & Stiffener to End Plate Welds

AISC Design Guide 4 (pg. 18)
End-Plate Stiffener Welds
The connection of the end-plate stiffener to the outside face of the beam flange and to the face of the end plate may be made using complete joint penetration groove welds or fillet welds. The CJP welds can be single or double bevel groove welds. Fillet welds should be used only if the stiffener plate is 3/8 in. or less in thickness.

AISC 358 Sec. 6.9-7 (5)
When used, all end-plate-to-stiffener joints shall be made using CJP groove welds.
Exception: When the stiffener is 3/8 in. (10 mm) thick or less, it shall be permitted to use fillet welds that develop the strength of the stiffener.

AISC 358 Sec. 6.10 Step 10
The stiffener-to-beam-flange and stiffener-to-end-plate welds shall be designed to develop the stiffener plate in shear at the beam flange and in tension at the end-plate. Either fillet or complete-joint-penetration (CJP) groove welds are suitable for the weld of the stiffener plate to the beam flange. CJP groove welds shall be used for the stiffener-to-end-plate weld. If the end-plate is 3/8 in. (10 mm) thick or less, doublesided fillet welds are permitted.

Fillet weld is sized to develop shear strength of stiffener

Thickness of Stiffener: $t_{st} := t_w = 0.75$ in

Area of Stiffener on Flange: $A_{st} := (L_{st} - 1) \cdot t_{st} = 5.438$ in^2

Length of Stiffener: $L_{st} = 8.25$ in

$\phi_r := 0.75$

Size of Fillet Weld: $t_{fillet} := \frac{9}{16} = 0.563$ in — Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Length of Weld: $L_{weld} := L_{st} - 1 = 7.25$ in — Factoring in corner clip

Web Plate Shear Buckling Coefficient: $k_v := 1.2$ — AISC 360-10 Chap. G

Web Shear Coefficient:

$$C_v(h_w, t_w) := \begin{cases} 1.0 & \text{if } \frac{h_w}{t_w} \le 1.1\cdot\sqrt{k_v\cdot\frac{E}{F_y}} \quad \text{(G2-3)} \\ \dfrac{1.1\cdot\sqrt{k_v\cdot\frac{E}{F_y}}}{\frac{h_w}{t_w}} & \text{if } 1.1\cdot\sqrt{k_v\cdot\frac{E}{F_y}} < \frac{h_w}{t_w} \le 1.37\cdot\sqrt{k_v\cdot\frac{E}{F_y}} \quad \text{(G2-4)} \\ \dfrac{1.51\cdot k_v\cdot E}{\left(\frac{h_w}{t_w}\right)^2\cdot F_y} & \text{if } \frac{h_w}{t_w} > 1.37\cdot\sqrt{k_v\cdot\frac{E}{F_y}} \quad \text{(G2-5)} \end{cases}$$

Web Shear Coefficient: $c_v := C_v(L_{st}, t_{st}) = 1$

Nominal Shear Strength: $V_n := 0.6\cdot F_y\cdot A_{st}\cdot c_v = 179.4$ kip

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

Nominal Weld Strength: $R_{n.fillet} := 2\cdot 0.707\cdot t_{fillet}\cdot 0.6\cdot F_{exx}\cdot L_{weld} = 242.2$ kip — for 2 sides

Design Weld Strength: $\phi_r\cdot R_{n.fillet} = 181.6$ kip

Required Strength: $V_u := V_n = 179.4$ kip

$$\begin{cases} \text{"Ok"} & \text{if } \phi_r\cdot R_{n.fillet} \ge V_u \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Use 9/16" double sided fillet weld for stiffener to beam flange

Use CJP weld for stiffener to end plate

Beam to End Plate Welds

Web to End Plate:

AISC 358 Sec. 6.9-7 (3):
The beam web to end plate joint shall be made using either fillet welds or CJP groove welds. When used, the fillet welds shall be sized to develop the full strength of the beam web in tension from the inside face of the flange to 6" beyond the bolt row farthest from the beam flange.

$\phi_r := 0.75$

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

Minimum Weld for Shear Capacity:

Length of Weld: $L_{weld} := h_w = 42$ in

Size of Fillet Weld: $t_{fillet} := \frac{4}{16} = 0.25$ in

Minimum Fillet Weld = 1/4" (AISC 360-10 Table J2.4)

Nominal Strength: $R_{n.fillet} := 2 \cdot 0.707 \cdot t_{fillet} \cdot 0.6 \cdot F_{exx} \cdot L_{weld} = 623.6\ \text{kip}$ for 2 sides

Design Strength: $\phi_r \cdot R_{n.fillet} = 467.7$ kip

Required Strength: $V_u := F_{total} = 185.9$ kip

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \phi_r \cdot R_{n.fillet} \geq V_u \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Use 1/4" fillet weld on each side of the web

AISC 358:

Angle of Loading: $\theta_{358} := 0.5 \cdot \pi$ rad

Nominal Stress of Weld Metal: $F_{nw.fillet} := 0.6 \cdot F_{exx} \cdot \left(1 + 0.5 \cdot \sin\left(\theta_{358}\right)^{1.5}\right) = 63$ ksi

Length of Weld: $L_{weld.358} := p_{fi} + p_b + 6 = 9.75$ in

Size of Fillet Weld: $t_{fillet.358} := \frac{10}{16}$ in

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Nominal Strength: $R_{n.fillet.358} := 2 \cdot 0.707 \cdot t_{fillet.358} \cdot F_{nw.fillet} \cdot L_{weld.358} = 542.8\ \text{kip}$ for 2 sides

Design Strength: $\phi_r \cdot R_{n.fillet.358} = 407.1$ kip

Required Strength: $F_{web} := F_y \cdot t_w \cdot L_{weld.358} = 402.2$ kip

$$\begin{array}{l} \text{"Ok"} \quad \text{if } \phi_r \cdot R_{n.fillet.358} \geq F_{web} \\ \text{"No Good"} \quad \text{otherwise} \end{array} = \text{"Ok"}$$

Use 5/8" fillet weld on each side of the web

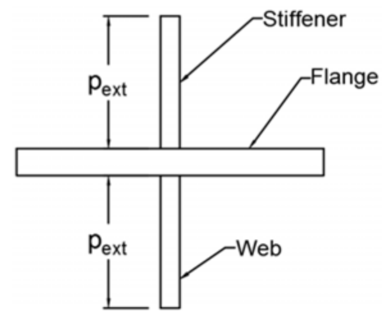
Flange to End Plate:

Cruciform Force: $F_{cru} := \dfrac{M_u}{(d - t_f)} = 748.8$ kip

Stiffener Force: $F_{st} := (p_{ext}) \cdot (t_{st}) \cdot F_y = 195.9$ kip

Web Force: $F_w := (p_{ext}) \cdot (t_w) \cdot F_y = 195.9$ kip

Flange Force: $F_{uF} := F_{cru} - F_{st} - F_w = 357$ kip



Cruciform Shape

Outside Flange to End Plate: PJP + Fillet

Size of Fillet Weld: $t_{f.PJP} := \dfrac{0}{16} = 0$ in

Size of PJP: $t_{PJP} := 0.75$ in

Effective Throat: $t_{eff} := \sqrt{t_{f.PJP}^{\ 2} + t_{PJP}^{\ 2}} = 0.75$ in

$\phi_{PJP} := 0.8$

Nominal Strength: $R_{n.PJP} := t_{eff} \cdot (0.6) \cdot F_{exx} \cdot b_f = 378$ kip

Design Strength: $\phi_{PJP} \cdot R_{n.PJP} = 302$ kip

Inside Flange to End Plate: Fillet

Web Thickness: $t_w = 0.75$ in

Length of Weld: $L_{fillet} := b_f - t_w - 2 \cdot (t_{fillet.358}) = 10$ in

Factoring in the web thickness and the web welds

Size of Fillet Weld: $t_{fillet} := \dfrac{8}{16} = 0.5$ in

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

Angle of Loading: $\theta := \dfrac{\pi}{2}$ rad

Nominal Stress of Weld Metal: $F_{nw.fillet} := 0.6 \cdot F_{exx} \cdot \left(1 + 0.5 \cdot \sin(\theta)^{1.5}\right) = 63$ ksi

$\phi_r := 0.75$

Nominal Strength: $R_{n.fillet} := 0.707 \cdot t_{fillet} \cdot F_{nw.fillet} \cdot L_{fillet} = 223$ kip

Design Strength: $\phi_r \cdot R_{n.fillet} = 167$ kip

Total Strength of Weld: $\phi R_{n.total} := \phi_{PJP} \cdot R_{n.PJP} + \phi_r \cdot R_{n.fillet} = 469.4$ kip

Required Strength: $F_{uF} = 357$ kip

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \phi R_{n.total} \geq F_{uF} \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Use 1/2" fillet weld on inside and outside edge of the flange and 3/4" PJP on the outside edge of the flange

Beam 1 - End Plate 12ES-1.25-1.50-44

Limit States: Plastic Hinging of the Beam

End Plate:

End Plate Thickness: $t_p := 1.50$ in

Flange to Inner Bolt CL: $p_{fi} := 1.875$ in

Bolt Spacing: $p_b := 3.75$ in

Top of Flange to Edge: $p_{ext} := 7.25$ in

Bolts:

Bolt Diameter: $d_b := 1.25$ in

Bolt Pretension: $T_b := 102$ kip

Stiffeners:

Length of Stiffener: $L_{st} := 12.625$ in

Location of Plastic Hinge: $S_h := L_{st} + t_p = 14.125$ in

Actuator Forces:

M_f = $M_u := 63382$ kip·in

$$F_{total} := \frac{M_u}{L_h} = 365.8 \quad \text{kip}$$

Stiffener to Beam & Stiffener to End Plate Welds

AISC Design Guide 4 (pg. 18)
End-Plate Stiffener Welds
The connection of the end-plate stiffener to the outside face of the beam flange and to the face of the end plate may be made using complete joint penetration groove welds or fillet welds. The CJP welds can be single or double bevel groove welds. Fillet welds should be used only if the stiffener plate is 3/8 in. or less in thickness.

AISC 358 Sec. 6.9-7 (5)
When used, all end-plate-to-stiffener joints shall be made using CJP groove welds.
Exception: When the stiffener is 3/8 in. (10 mm) thick or less, it shall be permitted to use fillet welds that develop the strength of the stiffener.

AISC 358 Sec. 6.10 Step 10
The stiffener-to-beam-flange and stiffener-to-end-plate welds shall be designed to develop the stiffener plate in shear at the beam flange and in tension at the end-plate. Either fillet or complete-joint-penetration (CJP) groove welds are suitable for the weld of the stiffener plate to the beam flange. CJP groove welds shall be used for the stiffener-to-end-plate weld. If the end-plate is 3/8 in. (10 mm) thick or less, doublesided fillet welds are permitted.

Fillet weld is sized to develop shear strength of stiffener

Thickness of Stiffener: $t_{st} := t_w = 0.75$ in

Area of Stiffener on Flange: $A_{st} := (L_{st} - 1) \cdot t_{st} = 8.719$ in^2

Length of Stiffener: $L_{st} = 12.625$ in

$\phi_r := 0.75$

Size of Fillet Weld: $t_{fillet} := \frac{9}{16} = 0.563$ in

Minimum Fillet Weld = 5/16"
(AISC 360-10 Table J2.4)

Length of Weld: $L_{weld} := L_{st} - 1 = 11.625$ in

Factoring in corner clip

Web Plate Shear Buckling Coefficient: $k_v := 1.2$

AISC 360-10 Chap. G

Web Shear Coefficient:

$$C_v(h_w, t_w) := \begin{cases} 1.0 & \text{if } \frac{h_w}{t_w} \le 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} \\ \frac{1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}}}{\frac{h_w}{t_w}} & \text{if } 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} < \frac{h_w}{t_w} \le 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} \\ \frac{1.51 \cdot k_v \cdot E}{\left(\frac{h_w}{t_w}\right)^2 \cdot F_y} & \text{if } \frac{h_w}{t_w} > 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} \end{cases}$$

(G2-3)
(G2-4)
(G2-5)

Web Shear Coefficient: $c_v := C_v(L_{st}, t_{st}) = 1$

Nominal Shear Strength: $V_n := 0.6 \cdot F_y \cdot A_{st} \cdot c_v = 287.7$ kip

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

Nominal Weld Strength: $R_{n.fillet} := 2 \cdot 0.707 \cdot t_{fillet} \cdot 0.6 \cdot F_{exx} \cdot L_{weld} = 388.3$ kip   for 2 sides

Design Weld Strength: $\phi_r \cdot R_{n.fillet} = 291.3$ kip

Required Strength: $V_u := V_n = 287.7$ kip

$$\begin{cases} \text{"Ok"} & \text{if } \phi_r \cdot R_{n.fillet} \ge V_u \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Use 9/16" double sided fillet weld for stiffener to beam flange

Use CJP weld for stiffener to end plate

Beam to End Plate Welds

Outside Flange to End Plate: CJP

AISC 360 Table J2.5
For loading in tension normal to weld axis, the strength of the the joint is controlled by the base metal.

Inside Flange to End Plate: Fillet

AISC 358 Sec. 6.9-7 (2)
The inside face of the flange shall have a 5/16" fillet weld.

No backing

Demand critical

Use 5/16" fillet weld on inside edge of the flange and CJP on the outside edge of the flange

Web to End Plate:

AISC 358 Sec. 6.9-7 (3):
The beam web to end plate joint shall be made using either fillet welds or CJP groove welds. When used, the fillet welds shall be sized to develop the full strength of the beam web in tension from the inside face of the flange to 6" beyond the bolt row farthest from the beam flange.

$\phi_r := 0.75$

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

Minimum Weld for Shear Capacity:

Length of Weld: $L_{weld} := h_w = 42$ in

Size of Fillet Weld: $t_{fillet} := \frac{5}{16} = 0.3125$ in

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Nominal Strength: $R_{n.fillet} := 2 \cdot 0.707 \cdot t_{fillet} \cdot 0.6 \cdot F_{exx} \cdot L_{weld} = 779.5$ kip for 2 sides

Design Strength: $\phi_r \cdot R_{n.fillet} = 584.6$ kip

Required Strength: $V_u := F_{total} = 365.8$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi_r \cdot R_{n.fillet} \ge V_u \\ \text{"No Good"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Use 5/16" fillet weld on each side of the web

AISC 358:

Angle of Loading: $\theta_{358} := 0.5 \cdot \pi \quad \text{rad}$

Nominal Stress of Weld Metal: $F_{nw.fillet} := 0.6 \cdot F_{exx} \cdot \left(1 + 0.5 \cdot \sin\left(\theta_{358}\right)^{1.5}\right) = 63 \quad \text{ksi}$

Length of Weld: $L_{weld.358} := p_{fi} + p_b + 6 = 11.625 \ \text{in}$

Size of Fillet Weld: $t_{fillet.358} := \dfrac{10}{16} \quad \text{in}$

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Nominal Strength: $R_{n.fillet.358} := 2 \cdot 0.707 \cdot t_{fillet.358} \cdot F_{nw.fillet} \cdot L_{weld.358} = 647.2 \ \text{kip}$ for 2 sides

Design Strength: $\phi_r \cdot R_{n.fillet.358} = 485.4 \quad \text{kip}$

Required Strength: $F_{web} := F_y \cdot t_w \cdot L_{weld.358} = 479.5 \quad \text{kip}$

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \phi_r \cdot R_{n.fillet.358} \geq F_{web} \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Use 5/8" fillet weld on each side of the web

24 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

End Plate Input:

Yield Strength:

$F_{py} := 55$ ksi

Weld Input:

Filler Metal Classification Strength:

$F_{exx} := 70$ ksi

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12$ in

Flange Thickness: $t_f := 1$ in

Web Thickness: $t_w := 0.5$ in

Total Beam Depth: $d := 24$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 22$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 11$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 12$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 23.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 11.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 11.5$ in

Moment of Inertia: $I_x := \frac{1}{12} \cdot t_w \cdot h_w^3 + 2\left(\frac{1}{12} \cdot b_f \cdot t_f^3 + b_f \cdot t_f \cdot CtC_{top.f}^2\right) = 3620$ in^4

Elastic Section Modulus: $S_x := \frac{I_x}{C_w} = 301.64$ in^3

Actuator Input:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Beam Web to Flange Weld

Within at Least the Depth of the Beam or 3 Times the Width of Flange: AISC 358-10 Sec. 6.4 (1)

Length of Weld: $L_{weld} := \min(d, 3 \cdot b_f) = 24$ in

$t_{weld} := \max(0.75 \cdot t_w, 0.25) = 0.375$ in

Weld Thickness: $t_{weld} = \text{ceil}(16 \cdot t_{weld}) = 6$ 16^ths of an inch

Use 3/8" fillet weld on both sides of web or CJP

Remaining Portion of Beam:

Maximum Vertical Shear: $V_u := 144.1$ kip

First Moment of Area: $Q := b_f \cdot t_f \cdot \left(\frac{h_w}{2} + \frac{t_f}{2}\right) = 138$ in^3

Shear Flow: $q := \frac{V_u \cdot Q}{I_x} = 5.494$ $\frac{kip}{in}$

$\phi_v := 0.75$

For 1 side:

$\phi_v \cdot \frac{a}{16} \cdot (0.707) \cdot (0.6) \cdot F_{exx} = q$ --> $a := \frac{16 \cdot q}{\phi_v \cdot (0.707) \cdot (0.6) \cdot F_{exx}} = 3.947$ 16^ths of an inch

$a := \text{ceil}(a) = 4$ 16^ths of an inch

Minimum Fillet Weld = 3/16" (AISC 360-10 Table J2.4)

Use 1/4" fillet weld on one side of the web

Beam 2 - End Plate 12ES-0.875-0.75-24

Limit States: End Plate Yielding & Rupture of Bolts With Prying Action

End Plate:

End Plate Thickness: $t_p := 0.75$ in

Flange to Inner Bolt CL: $p_{fi} := 1.50$ in

Bolt Spacing: $p_b := 2.625$ in

Top of Flange to Edge: $p_{ext} := 5.25$ in

Bolts:

Bolt Diameter: $d_b := 0.875$ in

Bolt Pretension: $T_b := 49$ kip

Stiffeners:

Length of Stiffener: $L_{st} := 9.125$ in

Location of Plastic Hinge: $S_h := L_{st} + t_p = 9.875$ in

Actuator Forces:

M_q = $M_u := 1.25 \cdot (14872) = 18590$ kip·in   Increased for 25% bolt over strength

$$F_{total} := \frac{M_u}{L_h} = 107.3 \text{ kip}$$

Stiffener to Beam & Stiffener to End Plate Welds

AISC Design Guide 4 (pg. 18)
End-Plate Stiffener Welds
The connection of the end-plate stiffener to the outside face of the beam flange and to the face of the end plate may be made using complete joint penetration groove welds or fillet welds. The CJP welds can be single or double bevel groove welds. Fillet welds should be used only if the stiffener plate is 3/8 in. or less in thickness.

AISC 358 Sec. 6.9-7 (5)
When used, all end-plate-to-stiffener joints shall be made using CJP groove welds.
Exception: When the stiffener is 3/8 in. (10 mm) thick or less, it shall be permitted to use fillet welds that develop the strength of the stiffener.

AISC 358 Sec. 6.10 Step 10
The stiffener-to-beam-flange and stiffener-to-end-plate welds shall be designed to develop the stiffener plate in shear at the beam flange and in tension at the end-plate. Either fillet or complete-joint-penetration (CJP) groove welds are suitable for the weld of the stiffener plate to the beam flange. CJP groove welds shall be used for the stiffener-to-end-plate weld. If the end-plate is 3/8 in. (10 mm) thick or less, doublesided fillet welds are permitted.

Fillet weld is sized to develop shear strength of stiffener

Thickness of Stiffener: $t_{st} := t_w = 0.5$ in

Area of Stiffener on Flange: $A_{st} := (L_{st} - 1) \cdot t_{st} = 4.063$ in^2

Length of Stiffener: $L_{st} = 9.125$ in

$\phi_r := 0.75$

Size of Fillet Weld: $t_{fillet} := \frac{6}{16} = 0.375$ in — Minimum Fillet Weld = 3/16" (AISC 360-10 Table J2.4)

Length of Weld: $L_{weld} := L_{st} - 1 = 8.125$ in — Factoring in corner clip

Web Plate Shear Buckling Coefficient: $k_v := 1.2$ — AISC 360-10 Chap. G

Web Shear Coefficient:

$$C_v(h_w, t_w) := \begin{cases} 1.0 & \text{if } \frac{h_w}{t_w} \le 1.1\cdot\sqrt{k_v\cdot\frac{E}{F_y}} & \text{(G2-3)} \\ \dfrac{1.1\cdot\sqrt{k_v\cdot\frac{E}{F_y}}}{\frac{h_w}{t_w}} & \text{if } 1.1\cdot\sqrt{k_v\cdot\frac{E}{F_y}} < \frac{h_w}{t_w} \le 1.37\cdot\sqrt{k_v\cdot\frac{E}{F_y}} & \text{(G2-4)} \\ \dfrac{1.51\cdot k_v\cdot E}{\left(\frac{h_w}{t_w}\right)^2\cdot F_y} & \text{if } \frac{h_w}{t_w} > 1.37\cdot\sqrt{k_v\cdot\frac{E}{F_y}} & \text{(G2-5)} \end{cases}$$

Web Shear Coefficient: $c_v := C_v(L_{st}, t_{st}) = 1$

Nominal Shear Strength: $V_n := 0.6\cdot F_y\cdot A_{st}\cdot c_v = 134.1$ kip

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

Nominal Weld Strength: $R_{n.fillet} := 2\cdot 0.707\cdot t_{fillet}\cdot 0.6\cdot F_{exx}\cdot L_{weld} = 180.9$ kip — for 2 sides

Design Weld Strength: $\phi_r\cdot R_{n.fillet} = 135.7$ kip

Required Strength: $V_u := V_n = 134.1$ kip

$$\begin{cases} \text{"Ok"} & \text{if } \phi_r\cdot R_{n.fillet} \ge V_u \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Use 3/8" double sided fillet weld for stiffener to beam flange

Use CJP weld for stiffener to end plate

Beam to End Plate Welds

Web to End Plate:

AISC 358 Sec. 6.9-7 (3):
The beam web to end plate joint shall be made using either fillet welds or CJP groove welds. When used, the fillet welds shall be sized to develop the full strength of the beam web in tension from the inside face of the flange to 6" beyond the bolt row farthest from the beam flange.

$\phi_r := 0.75$

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

Minimum Weld for Shear Capacity:

Length of Weld: $L_{weld} := h_w = 22$ in

Size of Fillet Weld: $t_{fillet} := \frac{4}{16} = 0.25$ in

Minimum Fillet Weld = 3/16" (AISC 360-10 Table J2.4)

Nominal Strength: $R_{n.fillet} := 2 \cdot 0.707 \cdot t_{fillet} \cdot 0.6 \cdot F_{exx} \cdot L_{weld} = 326.6$ kip for 2 sides

Design Strength: $\phi_r \cdot R_{n.fillet} = 245$ kip

Required Strength: $V_u := F_{total} = 107.3$ kip

$$\left| \begin{array}{l} \text{"Ok"} \ \ \text{if} \ \ \phi_r \cdot R_{n.fillet} \geq V_u \\ \text{"No Good"} \ \ \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Use 1/4" fillet weld on each side of the web

AISC 358:

Angle of Loading: $\theta_{358} := 0.5 \cdot \pi$ rad

Nominal Stress of Weld Metal: $F_{nw.fillet} := 0.6 \cdot F_{exx} \cdot \left(1 + 0.5 \cdot \sin\left(\theta_{358}\right)^{1.5}\right) = 63$ ksi

Length of Weld: $L_{weld.358} := p_{fi} + p_b + 6 = 10.125$ in

Size of Fillet Weld: $t_{fillet.358} := \frac{7}{16}$ in

Minimum Fillet Weld = 3/16" (AISC 360-10 Table J2.4)

Nominal Strength: $R_{n.fillet.358} := 2 \cdot 0.707 \cdot t_{fillet.358} \cdot F_{nw.fillet} \cdot L_{weld.358} = 394.6$ kip for 2 sides

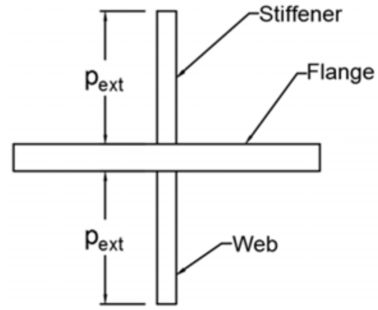
Design Strength: $\phi_r \cdot R_{n.fillet.358} = 296$ kip

Required Strength: $F_{web} := F_y \cdot t_w \cdot L_{weld.358} = 278.4$ kip

$$\begin{cases} \text{"Ok"} & \text{if } \phi_r \cdot R_{n.fillet.358} \geq F_{web} \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Use 7/16" fillet weld on each side of the web

<u>Flange to End Plate</u>:



Cruciform Shape

Cruciform Force: $F_{cru} := \dfrac{M_u}{(d - t_f)} = 808.3$ kip

Stiffener Force: $F_{st} := (p_{ext}) \cdot (t_{st}) \cdot F_y = 144.4$ kip

Web Force: $F_w := (p_{ext}) \cdot (t_w) \cdot F_y = 144.4$ kip

Flange Force: $F_{uF} := F_{cru} - F_{st} - F_w = 519.5$ kip

<u>Outside Flange to End Plate</u>: PJP + Fillet

Size of Fillet Weld: $t_{f.PJP} := \dfrac{0}{16} = 0$ in

Size of PJP: $t_{PJP} := 0.75$ in

Effective Throat: $t_{eff} := \sqrt{t_{f.PJP}^{\ 2} + t_{PJP}^{\ 2}} = 0.75$ in

$\phi_{PJP} := 0.8$

Nominal Strength: $R_{n.PJP} := t_{eff} \cdot (0.6) \cdot F_{exx} \cdot b_f = 378$ kip

Design Strength: $\phi_{PJP} \cdot R_{n.PJP} = 302$ kip

<u>Inside Flange to End Plate</u>: Fillet

Web Thickness: $t_w = 0.5$ in

Length of Weld: $L_{fillet} := b_f - t_w - 2 \cdot (t_{fillet.358}) = 10.625$ in

Factoring in the web thickness and the web welds

Size of Fillet Weld: $t_{fillet} := \dfrac{10}{16} = 0.625$ in

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

Angle of Loading: $\theta := \dfrac{\pi}{2}$ rad

Nominal Stress of Weld Metal: $F_{nw.fillet} := 0.6 \cdot F_{exx} \cdot \left(1 + 0.5 \cdot \sin(\theta)^{1.5}\right) = 63$ ksi

$\phi_r := 0.75$

Nominal Strength: $R_{n.fillet} := 0.707 \cdot t_{fillet} \cdot F_{nw.fillet} \cdot L_{fillet} = 296$ kip

Design Strength: $\phi_r \cdot R_{n.fillet} = 222$ kip

Total Strength of Weld: $\phi R_{n.total} := \phi_{PJP} \cdot R_{n.PJP} + \phi_r \cdot R_{n.fillet} = 524.2$ kip

Required Strength: $F_{uF} = 519.5$ kip

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \phi R_{n.total} \geq F_{uF} \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Use 5/8" fillet weld on inside edge of the flange and 3/4" PJP on the outside edge of the flange

Beam 2 - End Plate 12ES-1.125-1.25-24

Limit States: Plastic Hinging of the Beam

End Plate:

End Plate Thickness: $t_p := 1.25$ in

Flange to Inner Bolt CL: $p_{fi} := 1.75$ in

Bolt Spacing: $p_b := 3.375$ in

Top of Flange to Edge: $p_{ext} := 6.625$ in

Bolts:

Bolt Diameter: $d_b := 1.125$ in

Bolt Pretension: $T_b := 80$ kip

Stiffeners:

Length of Stiffener: $L_{st} := 11.5$ in

Location of Plastic Hinge: $S_h := L_{st} + t_p = 12.75$ in

Actuator Forces:

M_f = $M_u := 24972$ kip·in

$$F_{total} := \frac{M_u}{L_h} = 144.1 \quad \text{kip}$$

Stiffener to Beam & Stiffener to End Plate Welds

AISC Design Guide 4 (pg. 18)
End-Plate Stiffener Welds
The connection of the end-plate stiffener to the outside face of the beam flange and to the face of the end plate may be made using complete joint penetration groove welds or fillet welds. The CJP welds can be single or double bevel groove welds. Fillet welds should be used only if the stiffener plate is 3/8 in. or less in thickness.

AISC 358 Sec. 6.9-7 (5)
When used, all end-plate-to-stiffener joints shall be made using CJP groove welds.
Exception: When the stiffener is 3/8 in. (10 mm) thick or less, it shall be permitted to use fillet welds that develop the strength of the stiffener.

AISC 358 Sec. 6.10 Step 10
The stiffener-to-beam-flange and stiffener-to-end-plate welds shall be designed to develop the stiffener plate in shear at the beam flange and in tension at the end-plate. Either fillet or complete-joint-penetration (CJP) groove welds are suitable for the weld of the stiffener plate to the beam flange. CJP groove welds shall be used for the stiffener-to-end-plate weld. If the end-plate is 3/8 in. (10 mm) thick or less, doublesided fillet welds are permitted.

Fillet weld is sized to develop shear strength of stiffener

Thickness of Stiffener: $t_{st} := t_w = 0.5$ in

Area of Stiffener on Flange: $A_{st} := \left(L_{st} - 1\right) \cdot t_{st} = 5.25$ in^2

Length of Stiffener: $L_{st} = 11.5$ in

$\phi_r := 0.75$

Size of Fillet Weld: $t_{fillet} := \frac{6}{16} = 0.375$ in — Minimum Fillet Weld = 3/16" (AISC 360-10 Table J2.4)

Length of Weld: $L_{weld} := L_{st} - 1 = 10.5$ in — Factoring in corner clip

Web Plate Shear Buckling Coefficient: $k_v := 1.2$ — AISC 360-10 Chap. G

Web Shear Coefficient:

$$C_v(h_w, t_w) := \begin{cases} 1.0 & \text{if } \frac{h_w}{t_w} \le 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} \quad \text{(G2-3)} \\ \dfrac{1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}}}{\frac{h_w}{t_w}} & \text{if } 1.1 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} < \frac{h_w}{t_w} \le 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} \quad \text{(G2-4)} \\ \dfrac{1.51 \cdot k_v \cdot E}{\left(\frac{h_w}{t_w}\right)^2 \cdot F_y} & \text{if } \frac{h_w}{t_w} > 1.37 \cdot \sqrt{k_v \cdot \frac{E}{F_y}} \quad \text{(G2-5)} \end{cases}$$

Web Shear Coefficient: $c_v := C_v(L_{st}, t_{st}) = 1$

Nominal Shear Strength: $V_n := 0.6 \cdot F_y \cdot A_{st} \cdot c_v = 173.3$ kip

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

Nominal Weld Strength: $R_{n.fillet} := 2 \cdot 0.707 \cdot t_{fillet} \cdot 0.6 \cdot F_{exx} \cdot L_{weld} = 233.8$ kip — for 2 sides

Design Weld Strength: $\phi_r \cdot R_{n.fillet} = 175.4$ kip

Required Strength: $V_u := V_n = 173.3$ kip

$$\begin{cases} \text{"Ok"} & \text{if } \phi_r \cdot R_{n.fillet} \ge V_u \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Use 3/8" double sided fillet weld for stiffener to beam flange

Use CJP weld for stiffener to end plate

Beam to End Plate Welds

Outside Flange to End Plate: CJP

AISC 360 Table J2.5
For loading in tension normal to weld axis, the strength of the the joint is controlled by the base metal.

Inside Flange to End Plate: Fillet

AISC 358 Sec. 6.9-7 (2)
The inside face of the flange shall have a 5/16" fillet weld.

No backing

Demand critical

Use 5/16" fillet weld on inside edge of the flange and CJP on the outside edge of the flange

Web to End Plate:

AISC 358 Sec. 6.9-7 (3):
The beam web to end plate joint shall be made using either fillet welds or CJP groove welds. When used, the fillet welds shall be sized to develop the full strength of the beam web in tension from the inside face of the flange to 6" beyond the bolt row farthest from the beam flange.

$\phi_r := 0.75$

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

Minimum Weld for Shear Capacity:

Length of Weld: $L_{weld} := h_w = 22$ in

Size of Fillet Weld: $t_{fillet} := \frac{4}{16} = 0.25$ in

Minimum Fillet Weld = 3/16" (AISC 360-10 Table J2.4)

Nominal Strength: $R_{n.fillet} := 2 \cdot 0.707 \cdot t_{fillet} \cdot 0.6 \cdot F_{exx} \cdot L_{weld} = 326.6$ kip for 2 sides

Design Strength: $\phi_r \cdot R_{n.fillet} = 245$ kip

Required Strength: $V_u := F_{total} = 144.1$ kip

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \phi_r \cdot R_{n.fillet} \ge V_u \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Use 1/4" fillet weld on each side of the web

AISC 358:

Angle of Loading: $\theta_{358} := 0.5 \cdot \pi$ rad

Nominal Stress of Weld Metal: $F_{nw.fillet} := 0.6 \cdot F_{exx} \cdot \left(1 + 0.5 \cdot \sin\left(\theta_{358}\right)^{1.5}\right) = 63$ ksi

Length of Weld: $L_{weld.358} := p_{fi} + p_b + 6 = 11.125$ in

Size of Fillet Weld: $t_{fillet.358} := \frac{7}{16}$ in

Minimum Fillet Weld = 3/16" (AISC 360-10 Table J2.4)

Nominal Strength: $R_{n.fillet.358} := 2 \cdot 0.707 \cdot t_{fillet.358} \cdot F_{nw.fillet} \cdot L_{weld.358} = 433.6$ kip for 2 sides

Design Strength: $\phi_r \cdot R_{n.fillet.358} = 325.2$ kip

Required Strength: $F_{web} := F_y \cdot t_w \cdot L_{weld.358} = 305.9$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi_r \cdot R_{n.fillet.358} \geq F_{web} \\ \text{"No Good"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Use 7/16" fillet weld on each side of the web

Column

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_{yc} := 50$ ksi

Ultimate Strength: $F_{uc} := 65$ ksi

Expected Yield Ratio: $R_y := 1.1$

Column Input: Assume: Doubly Symmetric

Flange Width: $b_{fc} := 16.00$ in

Flange Thickness: $t_{fc} := 1.25$ in

Web Thickness: $t_{wc} := 1.00$ in

Total Beam Depth: $d_c := 42.00$ in

Web Plate Depth: $h_{wc} := d_c - 2 \cdot t_{fc} = 39.50$ in

Gross Cross Sectional Area: $A_{gc} := 2 \cdot b_{fc} \cdot t_{fc} + h_{wc} \cdot t_{wc} = 79.5$ in^2

Top Flange Cross-Sectional Area: $A_{top.fc} := b_{fc} \cdot t_{fc} = 20$ in^2

Web Cross-Sectional Area: $A_{wc} := h_{wc} \cdot t_{wc} = 39.5$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.fc} := b_{fc} \cdot t_{fc} = 20$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.fc} := \frac{t_{fc}}{2} = 0.625$ in

Web: $C_{wc} := t_{fc} + \frac{h_{wc}}{2} = 21$ in

Bottom Flange: $C_{bot.fc} := t_{fc} + h_{wc} + \frac{t_{fc}}{2} = 41.375$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.fc} := C_{wc} - C_{top.fc} = 20.375$ in

Web: $CtC_{wc} := 0$ in

Bottom Flange: $CtC_{bot.fc} := C_{bot.fc} - C_{wc} = 20.375$ in

Plastic Section Modulus: $Z_{xc} := 2 \cdot \left[A_{top.fc} \cdot \left(C_{wc} - C_{top.fc}\right) + \left(\frac{A_{wc}}{2}\right) \cdot \left(\frac{h_{wc}}{4}\right)\right] = 1205$ in^3

Column Height: $H_c := 20 \cdot 12 + 6 + \frac{1}{16} = 246.063$ in

Column Axial Force: $P_{uc} := 0$ kip

Stub Section Bracing:

$g_{stub} := 10.875$ in

$p_{b.stub} := 11.25$ in

Distance between Stub Sections:

$L_{bC} := 16 \cdot 12 + 4 + \frac{7}{8} = 196.875$ in

44 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12.00$ in

Flange Thickness: $t_f := 1.00$ in

Web Thickness: $t_w := 0.75$ in

Total Beam Depth: $d := 44.00$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 42.00$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 31.5$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \dfrac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \dfrac{h_w}{2} = 22$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \dfrac{t_f}{2} = 43.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 21.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 21.5$ in

Plastic Section Modulus: $Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f}\right) + \left(\dfrac{A_w}{2}\right) \cdot \left(\dfrac{h_w}{4}\right)\right] = 847$ in^3

Moment of Inertia: $I_x := \dfrac{1}{12} \cdot t_w \cdot h_w^{\,3} + 2\left(\dfrac{1}{12} \cdot b_f \cdot t_f^{\,3} + b_f \cdot t_f \cdot CtC_{top.f}^{\,2}\right) = 15727$ in^4

Elastic Section Modulus: $S_x := \dfrac{I_x}{C_w} = 714.84$ in^3

Peak Connection Strength Factor: $C_{pr} := \frac{(F_y + F_u)}{2 \cdot F_y} = 1.136$

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 46571$ kip·in

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 58214$ kip·in

Actuator Force:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Beam 1 - End Plate 12ES-0.75-1.00-44

Limit States: Rupture of Bolts without Prying Action

End Plate Input:

Yield Strength:	$F_{py} := 55$	ksi
Ultimate Strength:	$F_{pu} := 70$	ksi
End Plate Thickness:	$t_p := 1.00$	in
End Plate Width:	$b_p := 12.00$	in
Inner Bolt Gage:	$g := 4.00$	in
Outer Bolt Gage:	$g_o := 2.25$	in
Flange to Outer Bolt CL:	$p_{fo} := 1.50$	in
Flange to Inner Bolt CL:	$p_{fi} := 1.50$	in
Bolt Spacing:	$p_b := 2.25$	in
Outer Bolt to Edge:	$d_e := 1.00$	in
Top of Flange to Edge:	$p_{ext} := p_{fo} + p_b + d_e = 4.75$	in

Bolts:

$d_b := 0.75$ in

$T_b := 35$ kip

(AISC 360-10 Table J3.1)

Stiffeners:

$L_{st} := 8.25$ in

$S_h := L_{st} + t_p = 9.25$ in

Top of Compression Flange to Tension Face Bolt Rows:

Outermost Row:	$h_0 := d + p_{fo} + p_b = 47.75$	in
Outside of Flange:	$h_1 := d + p_{fo} = 45.5$	in
Inside of Flange:	$h_2 := d - t_f - p_{fi} = 41.5$	in
Innermost Row:	$h_3 := h_2 - p_b = 39.25$	in

Centroid of Compression Flange to Tension Face Bolt Rows:

Outermost Row:	$d_0 := d - \frac{t_f}{2} + p_{fo} + p_b = 47.25$	in
Outside of Flange:	$d_1 := d - \frac{t_f}{2} + p_{fo} = 45$	in
Inside of Flange:	$d_2 := d - \frac{3 \cdot t_f}{2} - p_{fi} = 41$	in
Innermost Row:	$d_3 := d_2 - p_b = 38.75$	in

Actuator Force:

M_{np} = $M_u := 1.25 \cdot (25760) = 32200 \quad kip \cdot in$ Increased for 25% bolt over strength

$F_{total} := \frac{M_u}{L_h} = 185.9 \quad kip$

Check Column

Check Column Flange Width vs Beam Flange Width:

$b_f = 12 \quad in$

$b_{fc} = 16 \quad in$

$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } b_{fc} > b_f \\ \text{"Increase Column Flange Width"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$

Check for Continuity Plates: AISC 341 Sect. E3.6f

Minimum Column Flange Thickness: $t_{fc.min1} := 0.4 \cdot \sqrt{1.8 \cdot b_f \cdot t_f \cdot \frac{F_y \cdot R_y}{F_{yc} \cdot R_y}} = 1.95 \quad in$

Minimum Column Flange Thickness: $t_{fc.min2} := \frac{b_f}{6} = 2 \quad in$

$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} = 4 \quad in$

$$Y_c := \left[\frac{b_{fc}}{2} \cdot \left(\frac{h_0}{s} + \frac{h_1}{p_{fo}} + \frac{h_2}{p_{fi}} + \frac{h_3}{s} \right) + \frac{1}{2 \cdot g} \cdot \left[\begin{array}{l} h_0 \cdot (4 \cdot s + 3 \cdot p_b) + h_1 \cdot (4 \cdot p_{fo} + p_b) \ldots \\ + h_2 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_3 \cdot (4 \cdot s + p_b) \end{array} \right] + g \right] = 980 \quad in$$

Minimum Column Flange Thickness: $t_{fc.min3} := \sqrt{\frac{1.11 \cdot M_u}{1.00 \cdot F_{yc} \cdot Y_c}} = 0.854 \quad in$ AISC 358-10 (6.10-13)

Column Flange Thickness: $t_{fc} = 1.25 \quad in$

$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \max(t_{fc.min1}, t_{fc.min2}, t_{fc.min3}) < t_{fc} \\ \text{"Continuity plates required"} \quad \text{otherwise} \end{array} \right. = \text{"Continuity plates required"}$

Panel Zone Check:

$\Sigma M_o = 0 = F_{max} \cdot \left(L_h + \frac{d_c}{2} \right) - V_c \cdot L_b$ --> $V_c := \frac{F_{total}}{L_{bC}} \cdot \left(L_h + \frac{d_c}{2} \right) = 183 \quad kip$

Distance Between Flange Centroids: $d_p := d - t_f = 43 \quad in$

Moment at Column Face: $M_u = 32200 \quad kip \cdot in$

Panel Zone Shear: $R_u := \frac{M_u}{d_p} = 749 \quad kip$

Doubler Plate Thickness: $t_{dp} := 0 \quad in$

Panel Zone Thickness: $t_z := t_{wc} + t_{dp} = 1$ in

Minumum Panel Zone: $t_{z.min} := \dfrac{(d - 2 \cdot t_f + d_c - 2 \cdot t_{fc})}{90} = 0.906$ in

$$R_v := 0.6 \cdot F_{yc} \cdot d_c \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^2}{d \cdot d_c \cdot t_{wc}}\right) = 1311 \text{ kip}$$

AISC 360-10 (J10-11)

$\phi_v := 1.00$

$t_{wc} = 1$ in | $\phi_v \cdot R_v = 1311$ kip

$t_{z.min} = 0.906$ in | $R_u - V_c = 565$ kip

$$\begin{cases} \text{"Ok"} & \text{if } (t_{wc} \ge t_{z.min}) \wedge (\phi_v \cdot R_v \ge R_u - V_c) \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Column-Beam Moment Ratio: AISC 341-10 E3.4a

$$\Sigma M_{pc} := 2 \cdot Z_{xc} \cdot \left(F_{yc} - \frac{P_{uc}}{A_{gc}}\right) \cdot \left(\frac{\frac{H_c}{2}}{\frac{H_c}{2} - \frac{d}{2}}\right) = 146747 \text{ kip} \cdot \text{in}$$

2 because column extends above and below beam

$$\Sigma M_{pb} := M_{pr} \cdot \left(\frac{L_h + \frac{d_c}{2}}{L_h}\right) = 65270 \text{ kip} \cdot \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.248$$

$$\begin{cases} \text{"Ok"} & \text{if } \dfrac{\Sigma M_{pc}}{\Sigma M_{pb}} > 1 \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Highly Ductile Section Check: AISC 341-10 Table D1.1

Compression Ratio: $C_a := \dfrac{\left(\frac{F_{total}}{2}\right)}{A_{gc} \cdot F_{yc}} = 0.023$ assuming $C_a < 0.125$

Flange Limit:

$$bt_{limit} := 0.3 \cdot \sqrt{\frac{E}{F_{yc}}} = 7.225$$

$$bf2tf := \frac{b_{fc}}{2 \cdot t_{fc}} = 6.4$$

Web Limit:

$$ht_{limit} := 2.45 \cdot \sqrt{\frac{E}{F_{yc}}} \cdot (1 - 0.93 \cdot C_a) = 57.721$$

$$htw := \frac{h_{wc}}{t_{wc}} = 39.5$$

$$\begin{cases} \text{"Ok"} & \text{if } (bf2tf \le bt_{limit}) \wedge (htw \le ht_{limit}) \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Continuity Plates:

Minimum Continuity Plate Thickness $t_{cp.min} := \frac{t_f}{2} = 0.5 \text{ in}$ AISC 341-10 E3-6f-2a

Column Flange Flexural Yielding: $\phi_{FFY} := 1.00$ AISC 358-10 Sec. 6.10

$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} = 4 \quad \text{in}$

Based on including continuity plates

$$Y_c := \left[\frac{b_{fc}}{2} \cdot \left(\frac{h_0}{s} + \frac{h_1}{p_{fo}} + \frac{h_2}{p_{fi}} + \frac{h_3}{s} \right) + \frac{1}{2 \cdot g} \cdot \left[\begin{array}{l} h_0 \cdot (4 \cdot s + 3 \cdot p_b) + h_1 \cdot (4 \cdot p_{fo} + p_b) \ldots \\ + h_2 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_3 \cdot (4 \cdot s + p_b) \end{array} \right] + g \right] = 980 \quad \text{in}$$

$M_{n.FFY} := F_{yc} \cdot t_{fc}^{\ 2} \cdot Y_c = 76593 \quad \text{kip} \cdot \text{in}$ AISC 358-10 (6.10-14)

$R_{n.FFY} := \frac{M_{n.FFY}}{(d - t_f)} = 1781 \text{ kip}$ AISC 358-10 (6.10-15)

Equivalent Column Flange Design Force: $\phi_{FFY} \cdot R_{n.FFY} = 1781 \text{ kip}$

Column Local Web Yielding: $\phi_{LWY} := 1.00$

Assume: Flange to Web Weld $t_{weld} := \frac{5}{16} = 0.3125 \quad \text{in}$

$$C_t := \left| \begin{array}{l} 0.5 \ \text{ if } \ \frac{H_c}{2} - \frac{d}{2} < d_c \\ 1.0 \ \text{ otherwise} \end{array} \right. = 1$$

Distance from outer face of column flange to web toe of fillet: $k_c := t_{fc} + t_{weld} = 1.5625 \quad \text{in}$

$R_{n.LWY} := C_t \cdot (6k_c + t_f + 2 \cdot t_p) \cdot F_{yc} \cdot t_{wc} = 618.75 \quad \text{kip}$ AISC 358-10 (6.10-17)

$\phi_{LWY} \cdot R_{n.LWY} = 618.75 \text{ kip}$

Web Compression Buckling: $\phi_{WCB} := 0.75$

$R_{n.WCB} := \frac{24 \cdot t_{wc}^{\ 3} \cdot \sqrt{E \cdot F_{yc}}}{h_{wc}} = 731.641 \quad \text{kip}$ AISC 358-10 (6.10-19)

$\phi_{WCB} \cdot R_{n.WCB} = 548.7 \quad \text{kip}$

Web Crippling: $\phi_{WC} := 0.75$

Groove Weld Leg Reinforcement: $t_{weld.f} := \frac{5}{16} = 0.3125$ in

$$R_{n.WC} := 0.8 \cdot t_{wc}^{2} \cdot \left[1 + 3 \cdot \frac{t_f + 2 \cdot t_{weld.f}}{d_c} \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}} = 1166 \text{ kip}$$

AISC 358-10 (6.10-22)

$\phi_{WC} \cdot R_{n.WC} = 874.9$ kip

Stiffener Plate Required Strength:

$$\phi R_{n.min} := \min\left(\phi_{FFY} \cdot R_{n.FFY}, \phi_{LWY} \cdot R_{n.LWY}, \phi_{WCB} \cdot R_{n.WCB}, \phi_{WC} \cdot R_{n.WC}\right) = 548.7 \text{ kip}$$

Applied Force: $F_{fu} := \frac{M_u}{\left(d - t_f\right)} = 748.8$ kip

Required Stiffener Strength: $F_{su} := F_{fu} - \phi R_{n.min} = 200.1$ kip

Minimum Stiffener Dimensions: AISC 360-10 J10-8

Stiffener Thickness: $t_{cp} := 1.00$ in

Stiffener Width: $w_{st} := \left(\frac{b_{fc} - t_{wc}}{2}\right) - 0.5 = 7$ in

$w_{st} + 0.5 \cdot t_{wc} = 7.5$ in > $\frac{1}{3} \cdot b_{fc} = 5.333$ in

$t_{cp} = 1$ in > $\max\left(0.5t_f, 0.5t_p, \frac{b_f}{16}\right) = 0.75$ in

Local Buckling Criterion: $t_{cp.min} := \frac{w_{st}}{0.56 \cdot \sqrt{\frac{E}{F_{yc}}}} = 0.519$ in

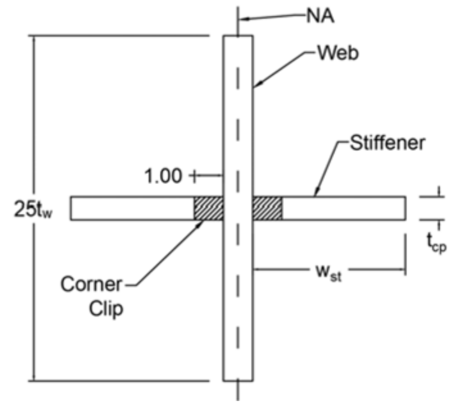
$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \left(w_{st} + 0.5 \cdot t_{wc} \geq \frac{1}{3} \cdot b_{fc}\right) \wedge \left(t_{cp} \geq \max\left(0.5 \cdot t_f, 0.5 \cdot t_p, \frac{b_f}{16}\right)\right) \wedge \left(t_{cp} \geq t_{cp.min}\right) \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Check Stiffener Strength: Tension $\phi := 1.00$

Gross Cruciform Area: $A_{g.st} := 2t_{cp} \cdot (w_{st}) + 25 \cdot t_{wc} = 39$ in^2

Net Cruciform Area: $A_{n.st} := 2t_{cp} \cdot (w_{st} - 1) + 25 \cdot t_{wc} = 37$ in^2

Factoring in corner clip



Cruciform shape used for tension and compression design

Nominal Tensile Strength: $R_{n.st} := A_{n.st} \cdot F_{yc} = 1850$ kip

Design Tensile Strength: $\phi \cdot R_{n.st} = 1850$ kip

$$\begin{array}{l} \text{"Ok"} \quad \text{if } \phi \cdot R_{n.st} > F_{su} \\ \text{"No Good"} \quad \text{otherwise} \end{array} = \text{"Ok"}$$

Check Stiffener Strength: Compression $\phi_c := 0.9$

Assume: Effective Length Factor: $k := 0.75$

Stiffener Height: $L_{st} := h_{wc} = 39.5$ in

Cruciform Moment of Inertia: $I_{st} := \frac{1}{12} \cdot t_{cp} \cdot (2 \cdot w_{st} + t_{wc})^3 + \frac{1}{12} \cdot (25 \cdot t_{wc} - w_{st}) \cdot t_{wc}^{\ 3} = 282.8$ in^4

Cruciform Radius of Gyration: $r_{st} := \sqrt{\frac{I_{st}}{A_{g.st}}} = 2.693$ in

Effective Slenderness Ratio: $\frac{k \cdot L_{st}}{r_{st}} = 11$

Effective Slenderness Limit: $4.71 \cdot \sqrt{\frac{E}{F_{yc}}} = 113.432$

Nominal Compressive Strength:
$$P_{n.st} := \left| \begin{array}{l} F_e \leftarrow \dfrac{\pi^2 \cdot E}{\left(\dfrac{k \cdot L_{st}}{r_{st}}\right)^2} \\ A_{g.st} \cdot \left(0.658^{\frac{F_{yc}}{F_e}}\right) \cdot F_{yc} \quad \text{if } \dfrac{k \cdot L_{st}}{r_{st}} \le 4.71 \cdot \sqrt{\dfrac{E}{F_{yc}}} \\ A_{g.st} \cdot (0.877) \cdot F_e \quad \text{otherwise} \end{array} \right.$$

$P_{n.st} = 1933$ kip

Design Compressive Strength: $\phi_c \cdot P_{n.st} = 1740$ kip

Required Compressive Strength: $F_{su} = 200.1$ kip

$$\begin{array}{|l} \text{"Ok"} \quad \text{if } \phi_c \cdot P_{n.st} \geq F_{su} \\ \text{"No Good"} \quad \text{otherwise} \end{array} = \text{"Ok"}$$

Check Bearing on Stiffeners: $\phi := 0.9$

Net Stiffener Pair Area: $A_{e.st} := 2 \cdot (w_{st} - 1) \cdot t_{cp} = 12$ in^2 Factoring in corner clips

Nominal Bearing Strength: $R_{n.bearing} := 1.8 \cdot F_{yc} \cdot A_{e.st} = 1080$ kip

Design Bearing Strength: $\phi \cdot R_{n.bearing} = 972$ kip

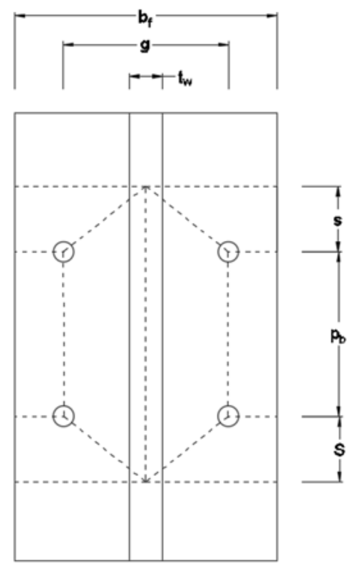
Required Bearing Strength: $F_{su} = 200.1$ kip

$$\begin{array}{|l} \text{"Ok"} \quad \text{if } \phi \cdot R_{n.bearing} \geq F_{su} \\ \text{"No Good"} \quad \text{otherwise} \end{array} = \text{"Ok"}$$

Bearing Stiffeners: Concentrated Force from W14x873 Stub Sections

AISC 360-10 Chap. J10

Maximum Force from Stub Section: $V_u := \frac{F_{total}}{L_{bC}} \cdot \left(L_h + \frac{d_c}{2}\right) = 183$ kip



Flange Local Bending: From Yield Line Analysis

$\phi_{FLB} := 0.9$

$$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g_{stub}} = 6.595 \quad \text{in}$$

$$Y := \frac{\left(4 \cdot s^2 + b_{fc} \cdot g_{stub} + 2 \cdot p_{b.stub} \cdot s\right)}{s \cdot (g_{stub})} = 6.921$$

Nominal Strength: $R_{n.FLB} := F_{yc} \cdot t_{fc}^{\,2} \cdot Y = 540.7$ kip

Design Strength: $\phi_{FLB} \cdot R_{n.FLB} = 486.6$ kip

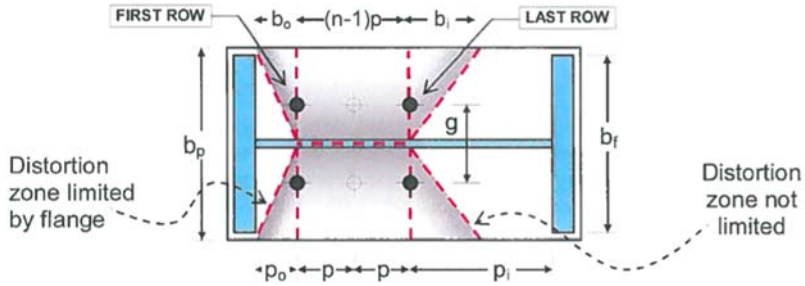
Flange Local Bending: BlueScope Buildings Yield Line Analysis

$b := \dfrac{b_{fc}}{\sqrt{2}} = 11.314$ in

$b_i := b = 11.314$ in

$b_o := b = 11.314$ in

Number of Bolt Rows: $n := 2$



$$Y := \begin{cases} \dfrac{1}{g_{stub}} \cdot \left[\dfrac{b_{fc}^{\,2}}{2} \cdot \left(\dfrac{1}{b_o} + \dfrac{1}{b_i}\right) + b_o + b_i + (n-1) \cdot p_{b.stub}\right] & \text{if } p_{b.stub} \le \dfrac{b_f}{\sqrt{2}} = 8.323 \\ \dfrac{1}{g_{stub}} \cdot \left(2 \cdot \dfrac{b_{fc}^{\,2}}{b} + 4 \cdot b\right) & \text{otherwise} \end{cases}$$

Nominal Strength: $R_{n.BS} := F_{yc} \cdot t_{fc}^{\,2} \cdot Y = 650.2$ kip

Design Strength: $\phi_{FLB} \cdot R_{n.BS} = 585.2$ kip

Web Local Yielding: $\phi_{WLY} := 1.00$

Length of Bearing $l_b := 19.625$ in

Outside Flange Surface to Fillet Toe $k := t_{fc} = 1.25$ in

Nominal Strength: $R_{n.WLY} := t_{wc} \cdot F_{yc} \cdot (5 \cdot k + l_b) = 1293.8$ kip

Design Strength: $\phi_{WLY} \cdot R_{n.WLY} = 1293.8$ kip

Web Crippling: $\phi_{WC} := 0.9$

Nominal Strength: $R_{n.WC} := 0.8 \cdot t_{wc}^{\,2} \cdot \left[1 + 3 \cdot \dfrac{l_b}{d_c} \cdot \left(\dfrac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \cdot \sqrt{\dfrac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}} = 2157.3$ kip

Design Strength: $\phi_{WC} \cdot R_{n.WC} = 1941.6$ kip

Web Sidesway Buckling: $\phi_{WSB} := 0.9$

Compression Flange Not Restrained Against Rotation

$C_r := 480000$ ksi

Nominal Strength: $$R_{n.WSB} := \frac{C_r \cdot t_{wc}^{\ 3} \cdot t_{fc}}{h_{wc}^{\ 2}} \cdot \left[0.4 \cdot \left[\frac{\left(\frac{h_{wc}}{t_{wc}}\right)}{\left(\frac{L_{bC}}{b_{fc}}\right)}\right]^3\right] = 5088.6 \quad \text{kip}$$

Design Strength: $\phi_{WSB} \cdot R_{n.WSB} = 4579.7$ kip

$$\left| \begin{array}{ll} \text{"WSB applies"} & \text{if } \frac{\left(\frac{h_w}{t_w}\right)}{\left(\frac{L_{bC}}{b_f}\right)} \le 1.7 \\ \text{"WSB does not apply"} & \text{otherwise} \end{array} \right. = \text{"WSB does not apply"}$$

Web Compression Buckling: $\phi_{WCB} := 1.00$

Nominal Strength: $$R_{n.WCB} := \frac{24 \cdot t_{wc}^{\ 3} \cdot \sqrt{E \cdot F_{yc}}}{h_{wc}} = 731.6 \quad \text{kip}$$

Design Strength: $\phi_{WCB} \cdot R_{n.WCB} = 731.6$ kip

Concentrated Force Strength:

$$\phi R_{n.min} := \min\left(\phi_{FLB} \cdot R_{n.FLB}, \phi_{FLB} \cdot R_{n.BS}, \phi_{WLY} \cdot R_{n.WLY}, \phi_{WC} \cdot R_{n.WC}, \phi_{WSB} \cdot R_{n.WSB}, \phi_{WCB} \cdot R_{n.WCB}\right)$$

Design Strength: $\phi R_{n.min} = 486.6$ kip

Required Strength: $F_u := V_u = 183.4$ kip

$$\left| \begin{array}{ll} \text{"Need bearing stiffeners"} & \text{if } \phi R_{n.min} < F_u \\ \text{"Do not need bearing stiffeners"} & \text{otherwise} \end{array} \right. = \text{"Do not need bearing stiffeners"}$$

Column Base Plates

Required Strength: $F_u := \frac{F_{total}}{2} = 92.9$ kip

One end is in tension, while other end is in compression

Base Plate Yield Strength: $F_{y.plate} := F_{yc} = 50$ ksi

Base Plate Thickness: $t_{plate} := 1.00$ in

Base Plate Width: $b_p := 21$ in

Bolt Spacing: $p_b := 9$ in

Bolt Gage: $g := 9$ in

Distance from Bolt Hole to Inside of Flange: $s_1 := \frac{h_{wc}}{2} - \frac{p_b}{2} = 15.25$ in

Distance Required to Minimize Energy: $s_2 := \frac{1}{2} \cdot \sqrt{b_p \cdot g} = 6.874$ in

$s := \min(s_1, s_2) = 6.874$ in

$\phi := 1.00$

Yield Line Parameter: $Y := \frac{\left(4 \cdot s^2 + 2 \cdot p_b \cdot s + b_p \cdot g\right)}{g \cdot s} = 8.11$

Nominal Strength: $P_{n.plate} := F_{y.plate} \cdot t_{plate}^{\ 2} \cdot Y = 405.5$ kip

Design Strength: $\phi \cdot P_{n.plate} = 405.5$ kip

"Ok" if $\phi \cdot P_{n.plate} > F_u$	= "Ok"
"Increase Base Plate Thickness" otherwise	

Column Bracing Threaded Rods

F1554 GR. 105 Threaded Rod

Diameter of Rods: $d_{rod} := 3$ in

Rod Yield Strength: $F_{yR} := 105$ ksi

Number of Rods: $n_{rods} := 4$

$\phi := 0.9$

Nominal Strength: $R_{n.rod} := n_{rods} \cdot \frac{\pi}{4} \cdot d_{rod}^{\ 2} \cdot F_{yR} = 2969$ kip

Design Strength: $\phi \cdot R_{n.rod} = 2672$ kip

Required Strength: $V_u = 183.4$ kip

Check :=	"Ok" if $\phi R_{n.min} \geq F_u$	= "Ok"
	"Increase bolt size" otherwise	

Beam 1 - End Plate 12ES-1.25-1.50-44

Limit States: Plastic Hinging of the Beam

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.50$ in

End Plate Width: $b_p := 16.00$ in

Inner Bolt Gage: $g := 4.50$ in

Outer Bolt Gage: $g_o := 3.75$ in

Flange to Outer Bolt CL: $p_{fo} := 1.875$ in

Flange to Inner Bolt CL: $p_{fi} := 1.875$ in

Bolt Spacing: $p_b := 3.75$ in

Outer Bolt to Edge: $d_e := 1.625$ in

Top of Flange to Edge: $p_{ext} := p_{fo} + p_b + d_e = 7.25$ in

Bolts:

$d_b := 1.25$ in

$T_b := 102$ kip

(AISC 360-10 Table J3.1)

Stiffeners:

$L_{st} := 12.625$ in

$S_h := L_{st} + t_p = 14.125$ in

Top of Compression Flange to Tension Face Bolt Rows:

Outermost Row: $h_0 := d + p_{fo} + p_b = 49.625$ in

Outside of Flange: $h_1 := d + p_{fo} = 45.875$ in

Inside of Flange: $h_2 := d - t_f - p_{fi} = 41.125$ in

Innermost Row: $h_3 := h_2 - p_b = 37.375$ in

Centroid of Compression Flange to Tension Face Bolt Rows:

Outermost Row: $d_0 := d - \frac{t_f}{2} + p_{fo} + p_b = 49.125$ in

Outside of Flange: $d_1 := d - \frac{t_f}{2} + p_{fo} = 45.375$ in

Inside of Flange: $d_2 := d - \frac{3 \cdot t_f}{2} - p_{fi} = 40.625$ in

Innermost Row: $d_3 := d_2 - p_b = 36.875$ in

Actuator Force:

M_f = $M_u := 63382 \quad kip \cdot in$

$$F_{total} := \frac{M_u}{L_h} = 365.8 \quad kip$$

Check Column

Check Column Flange Width vs Beam Flange Width:

$b_f = 12 \quad in$

$b_{fc} = 16 \quad in$

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } b_{fc} > b_f \\ \text{"Increase Column Flange Width"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Check for Continuity Plates: AISC 341 Sect. E3.6f

Minimum Column Flange Thickness: $t_{fc.min1} := 0.4 \cdot \sqrt{1.8 \cdot b_f \cdot t_f \cdot \frac{F_y \cdot R_y}{F_{yc} \cdot R_y}} = 1.95 \quad in$

Minimum Column Flange Thickness: $t_{fc.min2} := \frac{b_f}{6} = 2 \quad in$

$$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} = 4.243 \quad in$$

$$Y_c := \left[\frac{b_{fc}}{2} \cdot \left(\frac{h_0}{s} + \frac{h_1}{p_{fo}} + \frac{h_2}{p_{fi}} + \frac{h_3}{s} \right) + \frac{1}{2 \cdot g} \cdot \left[\begin{array}{l} h_0 \cdot (4 \cdot s + 3 \cdot p_b) + h_1 \cdot (4 \cdot p_{fo} + p_b) \ldots \\ + h_2 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_3 \cdot (4 \cdot s + p_b) \end{array} \right] + g \right] = 924 \quad in$$

Minimum Column Flange Thickness: $t_{fc.min3} := \sqrt{\frac{1.11 \cdot M_u}{1.00 \cdot F_{yc} \cdot Y_c}} = 1.234 \quad in$ AISC 358-10 (6.10-13)

Column Flange Thickness: $t_{fc} = 1.25 \quad in$

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \max(t_{fc.min1}, t_{fc.min2}, t_{fc.min3}) < t_{fc} \\ \text{"Continuity plates required"} & \text{otherwise} \end{array} \right. = \text{"Continuity plates required"}$$

Panel Zone Check:

$$\Sigma M_o = 0 = F_{max} \cdot \left(L_h + \frac{d_c}{2} \right) - V_c \cdot L_b \quad \text{-->} \quad V_c := \frac{F_{total}}{L_{bC}} \cdot \left(L_h + \frac{d_c}{2} \right) = 361 \quad kip$$

Distance Between Flange Centroids: $d_p := d - t_f = 43 \quad in$

Moment at Column Face: $M_u = 63382 \quad kip \cdot in$

Panel Zone Shear: $R_u := \frac{M_u}{d_p} = 1474 \quad kip$

Doubler Plate Thickness: $t_{dp} := 0 \quad in$

Panel Zone Thickness: $t_z := t_{wc} + t_{dp} = 1$ in

Minumum Panel Zone: $t_{z.min} := \dfrac{(d - 2 \cdot t_f + d_c - 2 \cdot t_{fc})}{90} = 0.906$ in

$$R_v := 0.6 \cdot F_{yc} \cdot d_c \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^2}{d \cdot d_c \cdot t_{wc}}\right) = 1311 \ \text{kip}$$

AISC 360-10 (J10-11)

$\phi_v := 1.00$

$t_{wc} = 1$ in $\quad \phi_v \cdot R_v = 1311$ kip

$t_{z.min} = 0.906$ in $\quad R_u - V_c = 1113$ kip

$$\left| \begin{array}{l} \text{"Ok" if } (t_{wc} \ge t_{z.min}) \wedge (\phi_v \cdot R_v \ge R_u - V_c) \\ \text{"No Good" otherwise} \end{array} \right. = \text{"Ok"}$$

Column-Beam Moment Ratio: AISC 341-10 E3.4a

$$\Sigma M_{pc} := 2 \cdot Z_{xc} \cdot \left(F_{yc} - \frac{P_{uc}}{A_{gc}}\right) \cdot \left(\frac{\frac{H_c}{2}}{\frac{H_c}{2} - \frac{d}{2}}\right) = 146747 \ \text{kip} \cdot \text{in}$$

2 because column extends above and below beam

$$\Sigma M_{pb} := M_{pr} \cdot \left(\frac{L_h + \frac{d_c}{2}}{L_h}\right) = 65270 \ \text{kip} \cdot \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.248$$

$$\left| \begin{array}{l} \text{"Ok" if } \dfrac{\Sigma M_{pc}}{\Sigma M_{pb}} > 1 \\ \text{"No Good" otherwise} \end{array} \right. = \text{"Ok"}$$

Highly Ductile Section Check: AISC 341-10 Table D1.1

Compression Ratio: $C_a := \dfrac{\left(\frac{F_{total}}{2}\right)}{A_{gc} \cdot F_{yc}} = 0.046$ assuming $C_a < 0.125$

Flange Limit:

$bt_{limit} := 0.3 \cdot \sqrt{\dfrac{E}{F_{yc}}} = 7.225$

$bf2tf := \dfrac{b_{fc}}{2 \cdot t_{fc}} = 6.4$

Web Limit:

$ht_{limit} := 2.45 \cdot \sqrt{\dfrac{E}{F_{yc}}} \cdot (1 - 0.93 \cdot C_a) = 56.479$

$htw := \dfrac{h_{wc}}{t_{wc}} = 39.5$

$$\left| \begin{array}{l} \text{"Ok" if } (bf2tf \le bt_{limit}) \wedge (htw \le ht_{limit}) \\ \text{"No Good" otherwise} \end{array} \right. = \text{"Ok"}$$

Continuity Plates:

Minimum Continuity Plate Thickness $t_{cp.min} := \frac{t_f}{2} = 0.5\ \text{in}$ AISC 341-10 E3-6f-2a

Column Flange Flexural Yielding: $\phi_{FFY} := 1.00$ AISC 358-10 Sec. 6.10

$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} = 4.243\ \text{in}$ Based on including continuity plates

$$Y_c := \left[\frac{b_{fc}}{2} \cdot \left(\frac{h_0}{s} + \frac{h_1}{p_{fo}} + \frac{h_2}{p_{fi}} + \frac{h_3}{s} \right) + \frac{1}{2 \cdot g} \cdot \left[\begin{array}{l} h_0 \cdot (4 \cdot s + 3 \cdot p_b) + h_1 \cdot (4 \cdot p_{fo} + p_b) \ldots \\ + h_2 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_3 \cdot (4 \cdot s + p_b) \end{array} \right] + g \right] = 924\ \text{in}$$

$M_{n.FFY} := F_{yc} \cdot t_{fc}{}^2 \cdot Y_c = 72221\ \text{kip} \cdot \text{in}$ AISC 358-10 (6.10-14)

$R_{n.FFY} := \frac{M_{n.FFY}}{(d - t_f)} = 1680\ \text{kip}$ AISC 358-10 (6.10-15)

Equivalent Column Flange Design Force: $\phi_{FFY} \cdot R_{n.FFY} = 1680\ \text{kip}$

Column Local Web Yielding: $\phi_{LWY} := 1.00$

Assume: Flange to Web Weld $t_{weld} := \frac{5}{16} = 0.3125\ \text{in}$

$$C_t := \begin{array}{|l} 0.5 \ \text{if} \ \frac{H_c}{2} - \frac{d}{2} < d_c \\ 1.0 \ \text{otherwise} \end{array} = 1$$

Distance from outer face of column flange to web toe of fillet: $k_c := t_{fc} + t_{weld} = 1.5625\ \text{in}$

$R_{n.LWY} := C_t \cdot (6k_c + t_f + 2 \cdot t_p) \cdot F_{yc} \cdot t_{wc} = 668.75\ \text{kip}$ AISC 358-10 (6.10-17)

$\phi_{LWY} \cdot R_{n.LWY} = 668.75\ \text{kip}$

Web Compression Buckling: $\phi_{WCB} := 0.75$

$R_{n.WCB} := \frac{24 \cdot t_{wc}{}^3 \cdot \sqrt{E \cdot F_{yc}}}{h_{wc}} = 731.641\ \text{kip}$ AISC 358-10 (6.10-19)

$\phi_{WCB} \cdot R_{n.WCB} = 548.7\ \text{kip}$

Web Crippling: $\phi_{WC} := 0.75$

Groove Weld Leg Reinforcement: $t_{weld.f} := \frac{5}{16} = 0.3125$ in

$$R_{n.WC} := 0.8 \cdot t_{wc}^{2} \cdot \left[1 + 3 \cdot \frac{t_f + 2 \cdot t_{weld.f}}{d_c} \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}} = 1166 \text{ kip}$$

AISC 358-10 (6.10-22)

$\phi_{WC} \cdot R_{n.WC} = 874.9$ kip

Stiffener Plate Required Strength:

$$\phi R_{n.min} := \min\left(\phi_{FFY} \cdot R_{n.FFY}, \phi_{LWY} \cdot R_{n.LWY}, \phi_{WCB} \cdot R_{n.WCB}, \phi_{WC} \cdot R_{n.WC}\right) = 548.7 \text{ kip}$$

Applied Force: $F_{fu} := \frac{M_u}{\left(d - t_f\right)} = 1474$ kip

Required Stiffener Strength: $F_{su} := F_{fu} - \phi R_{n.min} = 925.3$ kip

Minimum Stiffener Dimensions: AISC 360-10 J10-8

Stiffener Thickness: $t_{cp} := 1.00$ in

Stiffener Width: $w_{st} := \left(\frac{b_{fc} - t_{wc}}{2}\right) - 0.5 = 7$ in

$w_{st} + 0.5 \cdot t_{wc} = 7.5$ in > $\frac{1}{3} \cdot b_{fc} = 5.333$ in

$t_{cp} = 1$ in > $\max\left(0.5t_f, 0.5t_p, \frac{b_f}{16}\right) = 0.75$ in

Local Buckling Criterion: $t_{cp.min} := \frac{w_{st}}{0.56 \cdot \sqrt{\frac{E}{F_{yc}}}} = 0.519$ in

$$\begin{cases} \text{"Ok"} & \text{if } \left(w_{st} + 0.5 \cdot t_{wc} \ge \frac{1}{3} \cdot b_{fc}\right) \wedge \left(t_{cp} \ge \max\left(0.5 \cdot t_f, 0.5 \cdot t_p, \frac{b_f}{16}\right)\right) \wedge \left(t_{cp} \ge t_{cp.min}\right) \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Check Stiffener Strength: Tension $\phi := 1.00$

Gross Cruciform Area: $A_{g.st} := 2t_{cp} \cdot (w_{st}) + 25 \cdot t_{wc} = 39$ in^2

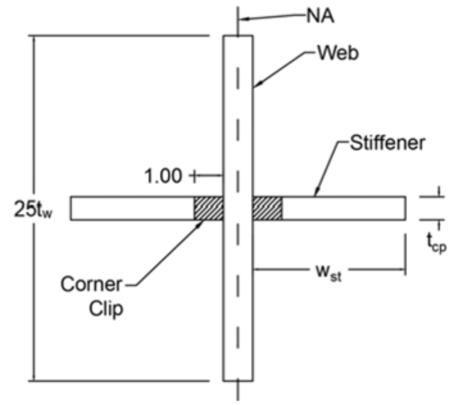
Net Cruciform Area: $A_{n.st} := 2t_{cp} \cdot (w_{st} - 1) + 25 \cdot t_{wc} = 37$ in^2

Nominal Tensile Strength: $R_{n.st} := A_{n.st} \cdot F_{yc} = 1850$ kip

Design Tensile Strength: $\phi \cdot R_{n.st} = 1850$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi \cdot R_{n.st} > F_{su} \\ \text{"No Good"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Factoring in corner clip



Cruciform shape used for tension and compression design

Check Stiffener Strength: Compression $\phi_c := 0.9$

Assume: Effective Length Factor: $k := 0.75$

Stiffener Height: $L_{st} := h_{wc} = 39.5$ in

Cruciform Moment of Inertia: $I_{st} := \frac{1}{12} \cdot t_{cp} \cdot (2 \cdot w_{st} + t_{wc})^3 + \frac{1}{12} \cdot (25 \cdot t_{wc} - w_{st}) \cdot t_{wc}^{\ 3} = 282.8$ in^4

Cruciform Radius of Gyration: $r_{st} := \sqrt{\frac{I_{st}}{A_{g.st}}} = 2.693$ in

Effective Slenderness Ratio: $\frac{k \cdot L_{st}}{r_{st}} = 11$

Effective Slenderness Limit: $4.71 \cdot \sqrt{\frac{E}{F_{yc}}} = 113.432$

Nominal Compressive Strength:

$$P_{n.st} := \left| \begin{array}{l} F_e \leftarrow \dfrac{\pi^2 \cdot E}{\left(\dfrac{k \cdot L_{st}}{r_{st}}\right)^2} \\ A_{g.st} \cdot \left(0.658^{\frac{F_{yc}}{F_e}}\right) \cdot F_{yc} \quad \text{if } \dfrac{k \cdot L_{st}}{r_{st}} \le 4.71 \cdot \sqrt{\dfrac{E}{F_{yc}}} \\ A_{g.st} \cdot (0.877) \cdot F_e \quad \text{otherwise} \end{array} \right.$$

$P_{n.st} = 1933$ kip

Design Compressive Strength: $\phi_c \cdot P_{n.st} = 1740$ kip

Required Compressive Strength: $F_{su} = 925.3$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi_c \cdot P_{n.st} \ge F_{su} \\ \text{"No Good"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Check Bearing on Stiffeners: $\phi := 0.9$

Net Stiffener Pair Area: $A_{e.st} := 2\cdot(w_{st} - 1)\cdot t_{cp} = 12 \quad in^2$ Factoring in corner clips

Nominal Bearing Strength: $R_{n.bearing} := 1.8\cdot F_{yc}\cdot A_{e.st} = 1080 \quad kip$

Design Bearing Strength: $\phi\cdot R_{n.bearing} = 972 \quad kip$

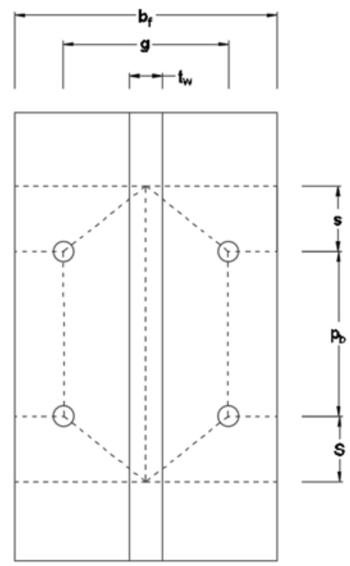
Required Bearing Strength: $F_{su} = 925.3 \quad kip$

$$\left| \begin{array}{l} \text{"Ok" if } \phi\cdot R_{n.bearing} \geq F_{su} \\ \text{"No Good" otherwise} \end{array} \right. = \text{"Ok"}$$

Bearing Stiffeners: Concentrated Force from W14x873 Stub Sections

AISC 360-10 Chap. J10

Maximum Force from Stub Section: $V_u := \frac{F_{total}}{L_{bC}}\cdot\left(L_h + \frac{d_c}{2}\right) = 361 \quad kip$



Flange Local Bending: From Yield Line Analysis

$\phi_{FLB} := 0.9$

$$s := \frac{1}{2}\cdot\sqrt{b_{fc}\cdot g_{stub}} = 6.595 \quad in$$

$$Y := \frac{\left(4\cdot s^2 + b_{fc}\cdot g_{stub} + 2\cdot p_{b.stub}\cdot s\right)}{s\cdot\left(g_{stub}\right)} = 6.921$$

Nominal Strength: $R_{n.FLB} := F_{yc}\cdot t_{fc}^{\,2}\cdot Y = 540.7 \quad kip$

Design Strength: $\phi_{FLB}\cdot R_{n.FLB} = 486.6 \quad kip$

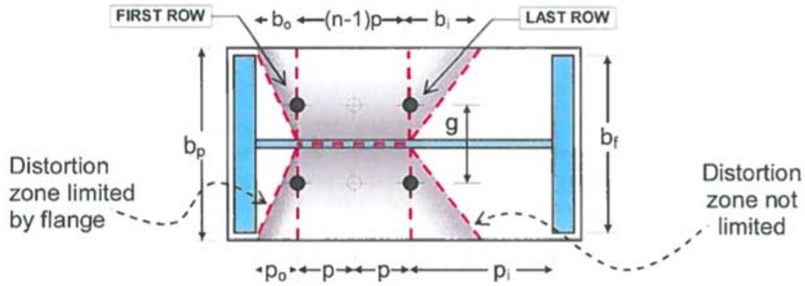
Flange Local Bending: BlueScope Buildings Yield Line Analysis

$b := \dfrac{b_{fc}}{\sqrt{2}} = 11.314$ in

$b_i := b = 11.314$ in

$b_o := b = 11.314$ in

Number of Bolt Rows: $n := 2$



$$Y := \begin{cases} \dfrac{1}{g_{stub}} \cdot \left[\dfrac{b_{fc}^{\,2}}{2} \cdot \left(\dfrac{1}{b_o} + \dfrac{1}{b_i}\right) + b_o + b_i + (n-1) \cdot p_{b.stub}\right] & \text{if } p_{b.stub} \le \dfrac{b_f}{\sqrt{2}} \\ \dfrac{1}{g_{stub}} \cdot \left(2 \cdot \dfrac{b_{fc}^{\,2}}{b} + 4 \cdot b\right) & \text{otherwise} \end{cases} = 8.323$$

Nominal Strength: $R_{n.BS} := F_{yc} \cdot t_{fc}^{\,2} \cdot Y = 650.2$ kip

Design Strength: $\phi_{FLB} \cdot R_{n.BS} = 585.2$ kip

Web Local Yielding: $\phi_{WLY} := 1.00$

Length of Bearing $l_b := 19.625$ in

Outside Flange Surface to Fillet Toe $k := t_{fc} = 1.25$ in

Nominal Strength: $R_{n.WLY} := t_{wc} \cdot F_{yc} \cdot (5 \cdot k + l_b) = 1293.8$ kip

Design Strength: $\phi_{WLY} \cdot R_{n.WLY} = 1293.8$ kip

Web Crippling: $\phi_{WC} := 0.9$

Nominal Strength: $R_{n.WC} := 0.8 \cdot t_{wc}^{\,2} \cdot \left[1 + 3 \cdot \dfrac{l_b}{d_c} \cdot \left(\dfrac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \cdot \sqrt{\dfrac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}} = 2157.3$ kip

Design Strength: $\phi_{WC} \cdot R_{n.WC} = 1941.6$ kip

Web Sidesway Buckling: $\phi_{WSB} := 0.9$

Compression Flange Not Restrained Against Rotation

$C_r := 480000$ ksi

Nominal Strength: $R_{n.WSB} := \dfrac{C_r \cdot t_{wc}^{\ 3} \cdot t_{fc}}{h_{wc}^{\ 2}} \cdot \left[0.4 \cdot \left[\dfrac{\left(\dfrac{h_{wc}}{t_{wc}}\right)}{\left(\dfrac{L_{bC}}{b_{fc}}\right)}\right]^3\right] = 5088.6$ kip

Design Strength: $\phi_{WSB} \cdot R_{n.WSB} = 4579.7$ kip

$$\begin{vmatrix} \text{"WSB applies"} & \text{if } \dfrac{\left(\dfrac{h_w}{t_w}\right)}{\left(\dfrac{L_{bC}}{b_f}\right)} \le 1.7 \\ \text{"WSB does not apply"} & \text{otherwise} \end{vmatrix} = \text{"WSB does not apply"}$$

Web Compression Buckling: $\phi_{WCB} := 1.00$

Nominal Strength: $R_{n.WCB} := \dfrac{24 \cdot t_{wc}^{\ 3} \cdot \sqrt{E \cdot F_{yc}}}{h_{wc}} = 731.6$ kip

Design Strength: $\phi_{WCB} \cdot R_{n.WCB} = 731.6$ kip

Concentrated Force Strength:

$\phi R_{n.min} := \min\left(\phi_{FLB} \cdot R_{n.FLB}, \phi_{FLB} \cdot R_{n.BS}, \phi_{WLY} \cdot R_{n.WLY}, \phi_{WC} \cdot R_{n.WC}, \phi_{WSB} \cdot R_{n.WSB}, \phi_{WCB} \cdot R_{n.WCB}\right)$

Design Strength: $\phi R_{n.min} = 486.6$ kip

Required Strength: $F_u := V_u = 361$ kip

$$\begin{vmatrix} \text{"Need bearing stiffeners"} & \text{if } \phi R_{n.min} < F_u \\ \text{"Do not need bearing stiffeners"} & \text{otherwise} \end{vmatrix} = \text{"Do not need bearing stiffeners"}$$

Column Base Plates

Required Strength: $F_u := \frac{F_{total}}{2} = 182.9$ kip

One end is in tension, while other end is in compression

Base Plate Yield Strength: $F_{y.plate} := F_{yc} = 50$ ksi

Base Plate Thickness: $t_{plate} := 1.00$ in

Base Plate Width: $b_p := 21$ in

Bolt Spacing: $p_b := 9$ in

Bolt Gage: $g := 9$ in

Distance from Bolt Hole to Inside of Flange: $s_1 := \frac{h_{wc}}{2} - \frac{p_b}{2} = 15.25$ in

Distance Required to Minimize Energy: $s_2 := \frac{1}{2} \cdot \sqrt{b_p \cdot g} = 6.874$ in

$s := \min(s_1, s_2) = 6.874$ in

$\phi := 1.00$

Yield Line Parameter: $Y := \frac{\left(4 \cdot s^2 + 2 \cdot p_b \cdot s + b_p \cdot g\right)}{g \cdot s} = 8.11$

Nominal Strength: $P_{n.plate} := F_{y.plate} \cdot t_{plate}^{\,2} \cdot Y = 405.5$ kip

Design Strength: $\phi \cdot P_{n.plate} = 405.5$ kip

$$\begin{array}{|ll} \text{"Ok"} & \text{if } \phi \cdot P_{n.plate} > F_u \\ \text{"Increase Base Plate Thickness"} & \text{otherwise} \end{array} = \text{"Ok"}$$

Column Bracing Threaded Rods

F1554 GR. 105 Threaded Rod

Diameter of Rods: $d_{rod} := 3$ in

Rod Yield Strength: $F_{yR} := 105$ ksi

Number of Rods: $n_{rods} := 4$

$\phi := 0.9$

Nominal Strength: $R_{n.rod} := n_{rods} \cdot \frac{\pi}{4} \cdot d_{rod}^{\,2} \cdot F_{yR} = 2969$ kip

Design Strength: $\phi \cdot R_{n.rod} = 2672$ kip

Required Strength: $V_u = 361$ kip

$$\text{Check} := \begin{array}{|ll} \text{"Ok"} & \text{if } \phi R_{n.min} \geq F_u \\ \text{"Increase bolt size"} & \text{otherwise} \end{array} = \text{"Ok"}$$

24 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12.00$ in

Flange Thickness: $t_f := 1.00$ in

Web Thickness: $t_w := 0.50$ in

Total Beam Depth: $d := 24.00$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 22.00$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 11$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 12$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 23.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 11.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 11.5$ in

Plastic Section Modulus: $Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f}\right) + \left(\frac{A_w}{2}\right) \cdot \left(\frac{h_w}{4}\right)\right] = 337$ in^3

Moment of Inertia: $I_x := \frac{1}{12} \cdot t_w \cdot h_w^3 + 2\left(\frac{1}{12} \cdot b_f \cdot t_f^3 + b_f \cdot t_f \cdot CtC_{top.f}^2\right) = 3620$ in^4

Elastic Section Modulus: $S_x := \frac{I_x}{C_w} = 301.64$ in^3

Peak Connection Strength Factor: $C_{pr} := \frac{\left(F_y + F_u\right)}{2 \cdot F_y} = 1.136$

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 18508$ kip·in

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 23134$ kip·in

Actuator Force:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Beam 2 - End Plate 12ES-0.875-0.75-24

Limit States: End Plate Yielding & Rupture of Bolts With Prying Action

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 0.75$ in

End Plate Width: $b_p := 12.00$ in

Inner Bolt Gage: $g := 3.50$ in

Outer Bolt Gage: $g_o := 2.625$ in

Flange to Outer Bolt CL: $p_{fo} := 1.50$ in

Flange to Inner Bolt CL: $p_{fi} := 1.50$ in

Bolt Spacing: $p_b := 2.625$ in

Outer Bolt to Edge: $d_e := 1.125$ in

Top of Flange to Edge: $p_{ext} := p_{fo} + p_b + d_e = 5.25$ in

Bolts:

$d_b := 0.875$ in

$T_b := 49$ kip

(AISC 360-10 Table J3.1)

Stiffeners:

$L_{st} := 9.125$ in

$S_h := L_{st} + t_p = 9.875$ in

Top of Compression Flange to Tension Face Bolt Rows:

Outermost Row: $h_0 := d + p_{fo} + p_b = 28.125$ in

Outside of Flange: $h_1 := d + p_{fo} = 25.5$ in

Inside of Flange: $h_2 := d - t_f - p_{fi} = 21.5$ in

Innermost Row: $h_3 := h_2 - p_b = 18.875$ in

Centroid of Compression Flange to Tension Face Bolt Rows:

Outermost Row: $d_0 := d - \frac{t_f}{2} + p_{fo} + p_b = 27.625$ in

Outside of Flange: $d_1 := d - \frac{t_f}{2} + p_{fo} = 25$ in

Inside of Flange: $d_2 := d - \frac{3 \cdot t_f}{2} - p_{fi} = 21$ in

Innermost Row: $d_3 := d_2 - p_b = 18.375$ in

Actuator Force:

M_q = $M_u := 1.25 \cdot (14872) = 18590$ kip·in Increased for 25% bolt over strength

$$F_{total} := \frac{M_u}{L_h} = 107.3 \quad \text{kip}$$

Check Column

Check Column Flange Width vs Beam Flange Width:

$b_f = 12$ in

$b_{fc} = 16$ in

$$\left| \begin{array}{l} \text{"Ok" if } b_{fc} > b_f \\ \text{"Increase Column Flange Width" otherwise} \end{array} \right. = \text{"Ok"}$$

Check for Continuity Plates: AISC 341 Sect. E3.6f

Minimum Column Flange Thickness: $t_{fc.min1} := 0.4 \cdot \sqrt{1.8 \cdot b_f \cdot t_f \cdot \dfrac{F_y \cdot R_y}{F_{yc} \cdot R_y}} = 1.95$ in

Minimum Column Flange Thickness: $t_{fc.min2} := \dfrac{b_f}{6} = 2$ in

$$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} = 3.742 \quad \text{in}$$

$$Y_c := \left[\frac{b_{fc}}{2} \cdot \left(\frac{h_0}{s} + \frac{h_1}{p_{fo}} + \frac{h_2}{p_{fi}} + \frac{h_3}{s} \right) + \frac{1}{2 \cdot g} \cdot \left[\begin{array}{l} h_0 \cdot (4 \cdot s + 3 \cdot p_b) + h_1 \cdot (4 \cdot p_{fo} + p_b) \ldots \\ + h_2 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_3 \cdot (4 \cdot s + p_b) \end{array} \right] + g \right] = 568 \quad \text{in}$$

Minimum Column Flange Thickness: $t_{fc.min3} := \sqrt{\dfrac{1.11 \cdot M_u}{1.00 \cdot F_{yc} \cdot Y_c}} = 0.852$ in AISC 358-10 (6.10-13)

Column Flange Thickness: $t_{fc} = 1.25$ in

$$\left| \begin{array}{l} \text{"Ok" if } \max(t_{fc.min1}, t_{fc.min2}, t_{fc.min3}) < t_{fc} \\ \text{"Continuity plates required" otherwise} \end{array} \right. = \text{"Continuity plates required"}$$

Panel Zone Check:

$$\Sigma M_o = 0 = F_{max} \cdot \left(L_h + \frac{d_c}{2} \right) - V_c \cdot L_b \quad \text{-->} \quad V_c := \frac{F_{total}}{L_{bC}} \cdot \left(L_h + \frac{d_c}{2} \right) = 106 \quad \text{kip}$$

Distance Between Flange Centroids: $d_p := d - t_f = 23$ in

Moment at Column Face: $M_u = 18590$ kip·in

Panel Zone Shear: $R_u := \dfrac{M_u}{d_p} = 808$ kip

Doubler Plate Thickness: $t_{dp} := 0$ in

Panel Zone Thickness: $t_z := t_{wc} + t_{dp} = 1$ in

Minumum Panel Zone: $t_{z.min} := \frac{(d - 2 \cdot t_f + d_c - 2 \cdot t_{fc})}{90} = 0.683$ in

$$R_v := 0.6 \cdot F_{yc} \cdot d_c \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d \cdot d_c \cdot t_{wc}}\right) = 1354 \text{ kip}$$

AISC 360-10 (J10-11)

$\phi_v := 1.00$

$t_{wc} = 1$ in $\phi_v \cdot R_v = 1354$ kip

$t_{z.min} = 0.683$ in $R_u - V_c = 702$ kip

$$\begin{cases} \text{"Ok"} & \text{if } (t_{wc} \geq t_{z.min}) \wedge (\phi_v \cdot R_v \geq R_u - V_c) \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Column-Beam Moment Ratio: AISC 341-10 E3.4a

$$\Sigma M_{pc} := 2 \cdot Z_{xc} \cdot \left(F_{yc} - \frac{P_{uc}}{A_{gc}}\right) \cdot \left(\frac{\frac{H_c}{2}}{\frac{H_c}{2} - \frac{d}{2}}\right) = 133530 \text{ kip} \cdot \text{in}$$

2 because column extends above and below beam

$$\Sigma M_{pb} := M_{pr} \cdot \left(\frac{L_h + \frac{d_c}{2}}{L_h}\right) = 25939 \text{ kip} \cdot \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 5.148$$

$$\begin{cases} \text{"Ok"} & \text{if } \frac{\Sigma M_{pc}}{\Sigma M_{pb}} > 1 \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Highly Ductile Section Check: AISC 341-10 Table D1.1

Compression Ratio: $C_a := \frac{\left(\frac{F_{total}}{2}\right)}{A_{gc} \cdot F_{yc}} = 0.013$ assuming $C_a < 0.125$

Flange Limit:

$$bt_{limit} := 0.3 \cdot \sqrt{\frac{E}{F_{yc}}} = 7.225$$

$$bf2tf := \frac{b_{fc}}{2 \cdot t_{fc}} = 6.4$$

Web Limit:

$$ht_{limit} := 2.45 \cdot \sqrt{\frac{E}{F_{yc}}} \cdot (1 - 0.93 \cdot C_a) = 58.263$$

$$htw := \frac{h_{wc}}{t_{wc}} = 39.5$$

$$\begin{cases} \text{"Ok"} & \text{if } (bf2tf \leq bt_{limit}) \wedge (htw \leq ht_{limit}) \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Continuity Plates:

Minimum Continuity Plate Thickness $t_{cp.min} := \frac{t_f}{2} = 0.5$ in AISC 341-10 E3-6f-2a

Column Flange Flexural Yielding: $\phi_{FFY} := 1.00$ AISC 358-10 Sec. 6.10

$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} = 3.742$ in

Based on including continuity plates

$$Y_c := \left[\frac{b_{fc}}{2} \cdot \left(\frac{h_0}{s} + \frac{h_1}{p_{fo}} + \frac{h_2}{p_{fi}} + \frac{h_3}{s} \right) + \frac{1}{2 \cdot g} \cdot \left[\begin{array}{l} h_0 \cdot (4 \cdot s + 3 \cdot p_b) + h_1 \cdot (4 \cdot p_{fo} + p_b) \ldots \\ + h_2 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_3 \cdot (4 \cdot s + p_b) \end{array} \right] + g \right] = 568 \quad \text{in}$$

$M_{n.FFY} := F_{yc} \cdot t_{fc}^{\ 2} \cdot Y_c = 44367$ kip·in AISC 358-10 (6.10-14)

$R_{n.FFY} := \frac{M_{n.FFY}}{(d - t_f)} = 1929$ kip AISC 358-10 (6.10-15)

Equivalent Column Flange Design Force: $\phi_{FFY} \cdot R_{n.FFY} = 1929$ kip

Column Local Web Yielding: $\phi_{LWY} := 1.00$

Assume: Flange to Web Weld $t_{weld} := \frac{5}{16} = 0.3125$ in

$$C_t := \left| \begin{array}{l} 0.5 \ \text{ if } \ \frac{H_c}{2} - \frac{d}{2} < d_c \\ 1.0 \ \text{ otherwise} \end{array} \right. = 1$$

Distance from outer face of column flange to web toe of fillet: $k_c := t_{fc} + t_{weld} = 1.5625$ in

$R_{n.LWY} := C_t \cdot (6k_c + t_f + 2 \cdot t_p) \cdot F_{yc} \cdot t_{wc} = 593.75$ kip AISC 358-10 (6.10-17)

$\phi_{LWY} \cdot R_{n.LWY} = 593.75$ kip

Web Compression Buckling: $\phi_{WCB} := 0.75$

$R_{n.WCB} := \frac{24 \cdot t_{wc}^{\ 3} \cdot \sqrt{E \cdot F_{yc}}}{h_{wc}} = 731.641$ kip AISC 358-10 (6.10-19)

$\phi_{WCB} \cdot R_{n.WCB} = 548.7$ kip

Web Crippling: $\phi_{WC} := 0.75$

Groove Weld Leg Reinforcement: $t_{weld.f} := \frac{5}{16} = 0.3125$ in

$$R_{n.WC} := 0.8 \cdot t_{wc}^{2} \cdot \left[1 + 3 \cdot \frac{t_f + 2 \cdot t_{weld.f}}{d_c} \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}} = 1166 \text{ kip}$$

AISC 358-10 (6.10-22)

$\phi_{WC} \cdot R_{n.WC} = 874.9$ kip

Stiffener Plate Required Strength:

$$\phi R_{n.min} := \min\left(\phi_{FFY} \cdot R_{n.FFY}, \phi_{LWY} \cdot R_{n.LWY}, \phi_{WCB} \cdot R_{n.WCB}, \phi_{WC} \cdot R_{n.WC}\right) = 548.7 \text{ kip}$$

Applied Force: $F_{fu} := \frac{M_u}{\left(d - t_f\right)} = 808.3$ kip

Required Stiffener Strength: $F_{su} := F_{fu} - \phi R_{n.min} = 259.5$ kip

Minimum Stiffener Dimensions: AISC 360-10 J10-8

Stiffener Thickness: $t_{cp} := 1.00$ in

Stiffener Width: $w_{st} := \left(\frac{b_{fc} - t_{wc}}{2}\right) - 0.5 = 7$ in

$w_{st} + 0.5 \cdot t_{wc} = 7.5$ in > $\frac{1}{3} \cdot b_{fc} = 5.333$ in

$t_{cp} = 1$ in > $\max\left(0.5t_f, 0.5t_p, \frac{b_f}{16}\right) = 0.75$ in

Local Buckling Criterion: $t_{cp.min} := \frac{w_{st}}{0.56 \cdot \sqrt{\frac{E}{F_{yc}}}} = 0.519$ in

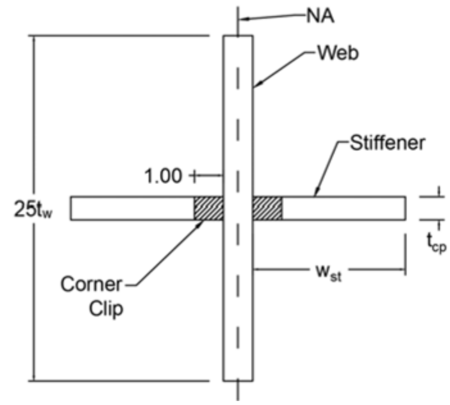
$$\begin{array}{l} \text{"Ok" if } \left(w_{st} + 0.5 \cdot t_{wc} \geq \frac{1}{3} \cdot b_{fc}\right) \wedge \left(t_{cp} \geq \max\left(0.5 \cdot t_f, 0.5 \cdot t_p, \frac{b_f}{16}\right)\right) \wedge \left(t_{cp} \geq t_{cp.min}\right) \\ \text{"No Good" otherwise} \end{array} = \text{"Ok"}$$

Check Stiffener Strength: Tension $\phi := 1.00$

Gross Cruciform Area: $A_{g.st} := 2t_{cp} \cdot (w_{st}) + 25 \cdot t_{wc} = 39$ in^2

Net Cruciform Area: $A_{n.st} := 2t_{cp} \cdot (w_{st} - 1) + 25 \cdot t_{wc} = 37$ in^2

Factoring in corner clip



Cruciform shape used for tension and compression design

Nominal Tensile Strength: $R_{n.st} := A_{n.st} \cdot F_{yc} = 1850$ kip

Design Tensile Strength: $\phi \cdot R_{n.st} = 1850$ kip

$$\begin{array}{l} \text{"Ok"} \quad \text{if } \phi \cdot R_{n.st} > F_{su} \\ \text{"No Good"} \quad \text{otherwise} \end{array} = \text{"Ok"}$$

Check Stiffener Strength: Compression $\phi_c := 0.9$

Assume: Effective Length Factor: $k := 0.75$

Stiffener Height: $L_{st} := h_{wc} = 39.5$ in

Cruciform Moment of Inertia: $I_{st} := \frac{1}{12} \cdot t_{cp} \cdot (2 \cdot w_{st} + t_{wc})^3 + \frac{1}{12} \cdot (25 \cdot t_{wc} - w_{st}) \cdot t_{wc}^{\;3} = 282.8$ in^4

Cruciform Radius of Gyration: $r_{st} := \sqrt{\frac{I_{st}}{A_{g.st}}} = 2.693$ in

Effective Slenderness Ratio: $\frac{k \cdot L_{st}}{r_{st}} = 11$

Effective Slenderness Limit: $4.71 \cdot \sqrt{\frac{E}{F_{yc}}} = 113.432$

Nominal Compressive Strength:

$$P_{n.st} := \left| \begin{array}{l} F_e \leftarrow \dfrac{\pi^2 \cdot E}{\left(\dfrac{k \cdot L_{st}}{r_{st}}\right)^2} \\ A_{g.st} \cdot \left(0.658^{\frac{F_{yc}}{F_e}}\right) \cdot F_{yc} \quad \text{if } \dfrac{k \cdot L_{st}}{r_{st}} \le 4.71 \cdot \sqrt{\dfrac{E}{F_{yc}}} \\ A_{g.st} \cdot (0.877) \cdot F_e \quad \text{otherwise} \end{array} \right.$$

$P_{n.st} = 1933$ kip

Design Compressive Strength: $\phi_c \cdot P_{n.st} = 1740$ kip

Required Compressive Strength: $F_{su} = 259.5$ kip

$$\begin{array}{l} \text{"Ok"} \quad \text{if } \phi_c \cdot P_{n.st} \ge F_{su} \\ \text{"No Good"} \quad \text{otherwise} \end{array} = \text{"Ok"}$$

Check Bearing on Stiffeners: $\phi := 0.9$

Net Stiffener Pair Area: $A_{e.st} := 2 \cdot (w_{st} - 1) \cdot t_{cp} = 12 \quad in^2$ Factoring in corner clips

Nominal Bearing Strength: $R_{n.bearing} := 1.8 \cdot F_{yc} \cdot A_{e.st} = 1080 \quad kip$

Design Bearing Strength: $\phi \cdot R_{n.bearing} = 972 \quad kip$

Required Bearing Strength: $F_{su} = 259.5 \ \ kip$

$$\left| \begin{array}{l} \text{"Ok"} \ \text{ if } \ \phi \cdot R_{n.bearing} \geq F_{su} \\ \text{"No Good"} \ \text{ otherwise} \end{array} \right. = \text{"Ok"}$$

Bearing Stiffeners: Concentrated Force from W14x873 Stub Sections

AISC 360-10 Chap. J10

Maximum Force from Stub Section: $V_u := \frac{F_{total}}{L_{bC}} \cdot \left(L_h + \frac{d_c}{2} \right) = 106 \quad kip$

Flange Local Bending: From Yield Line Analysis

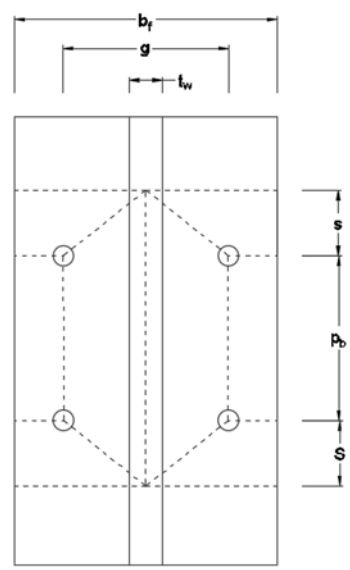
$\phi_{FLB} := 0.9$

$$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g_{stub}} = 6.595 \quad in$$

$$Y := \frac{\left(4 \cdot s^2 + b_{fc} \cdot g_{stub} + 2 \cdot p_{b.stub} \cdot s\right)}{s \cdot \left(g_{stub}\right)} = 6.921$$

Nominal Strength: $R_{n.FLB} := F_{yc} \cdot t_{fc}^{\ 2} \cdot Y = 540.7 \quad kip$

Design Strength: $\phi_{FLB} \cdot R_{n.FLB} = 486.6 \quad kip$



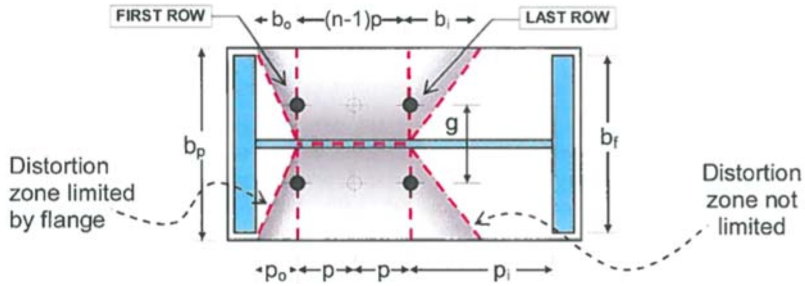
Flange Local Bending: BlueScope Buildings Yield Line Analysis

$b := \frac{b_{fc}}{\sqrt{2}} = 11.314$ in

$b_i := b = 11.314$ in

$b_o := b = 11.314$ in

Number of Bolt Rows: $n := 2$



$$Y := \begin{cases} \frac{1}{g_{stub}} \cdot \left[\frac{b_{fc}^2}{2} \cdot \left(\frac{1}{b_o} + \frac{1}{b_i}\right) + b_o + b_i + (n-1) \cdot p_{b.stub}\right] & \text{if } p_{b.stub} \le \frac{b_f}{\sqrt{2}} \\ \frac{1}{g_{stub}} \cdot \left(2 \cdot \frac{b_{fc}^2}{b} + 4 \cdot b\right) & \text{otherwise} \end{cases} = 8.323$$

Nominal Strength: $R_{n.BS} := F_{yc} \cdot t_{fc}^2 \cdot Y = 650.2$ kip

Design Strength: $\phi_{FLB} \cdot R_{n.BS} = 585.2$ kip

Web Local Yielding: $\phi_{WLY} := 1.00$

Length of Bearing $l_b := 19.625$ in

Outside Flange Surface to Fillet Toe $k := t_{fc} = 1.25$ in

Nominal Strength: $R_{n.WLY} := t_{wc} \cdot F_{yc} \cdot (5 \cdot k + l_b) = 1293.8$ kip

Design Strength: $\phi_{WLY} \cdot R_{n.WLY} = 1293.8$ kip

Web Crippling: $\phi_{WC} := 0.9$

Nominal Strength: $R_{n.WC} := 0.8 \cdot t_{wc}^2 \cdot \left[1 + 3 \cdot \frac{l_b}{d_c} \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}} = 2157.3$ kip

Design Strength: $\phi_{WC} \cdot R_{n.WC} = 1941.6$ kip

Web Sidesway Buckling: $\phi_{WSB} := 0.9$

Compression Flange Not Restrained Against Rotation

$C_r := 480000$ ksi

Nominal Strength: $$R_{n.WSB} := \frac{C_r \cdot t_{wc}^{\ 3} \cdot t_{fc}}{h_{wc}^{\ 2}} \cdot \left[0.4 \cdot \left[\frac{\left(\frac{h_{wc}}{t_{wc}} \right)}{\left(\frac{L_{bC}}{b_{fc}} \right)} \right]^3 \right] = 5088.6 \quad \text{kip}$$

Design Strength: $\phi_{WSB} \cdot R_{n.WSB} = 4579.7$ kip

$$\left| \begin{array}{ll} \text{"WSB applies"} & \text{if } \frac{\left(\frac{h_w}{t_w} \right)}{\left(\frac{L_{bC}}{b_f} \right)} \le 1.7 \\ \text{"WSB does not apply"} & \text{otherwise} \end{array} \right. = \text{"WSB does not apply"}$$

Web Compression Buckling: $\phi_{WCB} := 1.00$

Nominal Strength: $$R_{n.WCB} := \frac{24 \cdot t_{wc}^{\ 3} \cdot \sqrt{E \cdot F_{yc}}}{h_{wc}} = 731.6 \quad \text{kip}$$

Design Strength: $\phi_{WCB} \cdot R_{n.WCB} = 731.6$ kip

Concentrated Force Strength:

$$\phi R_{n.min} := \min\left(\phi_{FLB} \cdot R_{n.FLB}, \phi_{FLB} \cdot R_{n.BS}, \phi_{WLY} \cdot R_{n.WLY}, \phi_{WC} \cdot R_{n.WC}, \phi_{WSB} \cdot R_{n.WSB}, \phi_{WCB} \cdot R_{n.WCB}\right)$$

Design Strength: $\phi R_{n.min} = 486.6$ kip

Required Strength: $F_u := V_u = 105.9$ kip

$$\left| \begin{array}{ll} \text{"Need bearing stiffeners"} & \text{if } \phi R_{n.min} < F_u \\ \text{"Do not need bearing stiffeners"} & \text{otherwise} \end{array} \right. = \text{"Do not need bearing stiffeners"}$$

Column Base Plates

Required Strength: $F_u := \frac{F_{total}}{2} = 53.7$ kip

One end is in tension, while other end is in compression

Base Plate Yield Strength: $F_{y.plate} := F_{yc} = 50$ ksi

Base Plate Thickness: $t_{plate} := 1.00$ in

Base Plate Width: $b_p := 21$ in

Bolt Spacing: $p_b := 9$ in

Bolt Gage: $g := 9$ in

Distance from Bolt Hole to Inside of Flange: $s_1 := \frac{h_{wc}}{2} - \frac{p_b}{2} = 15.25$ in

Distance Required to Minimize Energy: $s_2 := \frac{1}{2} \cdot \sqrt{b_p \cdot g} = 6.874$ in

$s := \min(s_1, s_2) = 6.874$ in

$\phi := 1.00$

Yield Line Parameter: $Y := \frac{\left(4 \cdot s^2 + 2 \cdot p_b \cdot s + b_p \cdot g\right)}{g \cdot s} = 8.11$

Nominal Strength: $P_{n.plate} := F_{y.plate} \cdot t_{plate}^{2} \cdot Y = 405.5$ kip

Design Strength: $\phi \cdot P_{n.plate} = 405.5$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi \cdot P_{n.plate} > F_u \\ \text{"Increase Base Plate Thickness"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Column Bracing Threaded Rods

F1554 GR. 105 Threaded Rod

Diameter of Rods: $d_{rod} := 3$ in

Rod Yield Strength: $F_{yR} := 105$ ksi

Number of Rods: $n_{rods} := 4$

$\phi := 0.9$

Nominal Strength: $R_{n.rod} := n_{rods} \cdot \frac{\pi}{4} \cdot d_{rod}^{2} \cdot F_{yR} = 2969$ kip

Design Strength: $\phi \cdot R_{n.rod} = 2672$ kip

Required Strength: $V_u = 105.9$ kip

$$Check := \left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi R_{n.min} \geq F_u \\ \text{"Increase bolt size"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Beam 2 - End Plate 12ES-1.125-1.25-24

Limit States: Plastic Hinging of the Beam

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.25$ in

End Plate Width: $b_p := 14.00$ in

Inner Bolt Gage: $g := 4.00$ in

Outer Bolt Gage: $g_o := 3.375$ in

Flange to Outer Bolt CL: $p_{fo} := 1.75$ in

Flange to Inner Bolt CL: $p_{fi} := 1.75$ in

Bolt Spacing: $p_b := 3.375$ in

Outer Bolt to Edge: $d_e := 1.50$ in

Top of Flange to Edge: $p_{ext} := p_{fo} + p_b + d_e = 6.625$ in

Bolts:

$d_b := 1.125$ in

$T_b := 80$ kip

(AISC 360-10 Table J3.1)

Stiffeners:

$L_{st} := 11.5$ in

$S_h := L_{st} + t_p = 12.75$ in

Top of Compression Flange to Tension Face Bolt Rows:

Outermost Row: $h_0 := d + p_{fo} + p_b = 29.125$ in

Outside of Flange: $h_1 := d + p_{fo} = 25.75$ in

Inside of Flange: $h_2 := d - t_f - p_{fi} = 21.25$ in

Innermost Row: $h_3 := h_2 - p_b = 17.87$ in

Centroid of Compression Flange to Tension Face Bolt Rows:

Outermost Row: $d_0 := d - \frac{t_f}{2} + p_{fo} + p_b = 28.625$ in

Outside of Flange: $d_1 := d - \frac{t_f}{2} + p_{fo} = 25.25$ in

Inside of Flange: $d_2 := d - \frac{3 \cdot t_f}{2} - p_{fi} = 20.75$ in

Innermost Row: $d_3 := d_2 - p_b = 17.375$ in

Actuator Force:

M_f = $M_u := 24972$ kip·in

$$F_{total} := \frac{M_u}{L_h} = 144.1 \quad \text{kip}$$

Check Column

Check Column Flange Width vs Beam Flange Width:

$b_f = 12$ in

$b_{fc} = 16$ in

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } b_{fc} > b_f \\ \text{"Increase Column Flange Width"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Check for Continuity Plates: AISC 341 Sect. E3.6f

Minimum Column Flange Thickness: $t_{fc.min1} := 0.4 \cdot \sqrt{1.8 \cdot b_f \cdot t_f \cdot \frac{F_y \cdot R_y}{F_{yc} \cdot R_y}} = 1.95$ in

Minimum Column Flange Thickness: $t_{fc.min2} := \frac{b_f}{6} = 2$ in

$$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} = 4 \quad \text{in}$$

$$Y_c := \left[\frac{b_{fc}}{2} \cdot \left(\frac{h_0}{s} + \frac{h_1}{p_{fo}} + \frac{h_2}{p_{fi}} + \frac{h_3}{s} \right) + \frac{1}{2 \cdot g} \cdot \left[\begin{array}{l} h_0 \cdot (4 \cdot s + 3 \cdot p_b) + h_1 \cdot (4 \cdot p_{fo} + p_b) \ldots \\ + h_2 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_3 \cdot (4 \cdot s + p_b) \end{array} \right] + g \right] = 530 \quad \text{in}$$

Minimum Column Flange Thickness: $t_{fc.min3} := \sqrt{\frac{1.11 \cdot M_u}{1.00 \cdot F_{yc} \cdot Y_c}} = 1.023$ in AISC 358-10 (6.10-13)

Column Flange Thickness: $t_{fc} = 1.25$ in

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \max(t_{fc.min1}, t_{fc.min2}, t_{fc.min3}) < t_{fc} \\ \text{"Continuity plates required"} & \text{otherwise} \end{array} \right. = \text{"Continuity plates required"}$$

Panel Zone Check:

$$\Sigma M_o = 0 = F_{max} \cdot \left(L_h + \frac{d_c}{2} \right) - V_c \cdot L_b \quad \text{-->} \quad V_c := \frac{F_{total}}{L_{bC}} \cdot \left(L_h + \frac{d_c}{2} \right) = 142 \quad \text{kip}$$

Distance Between Flange Centroids: $d_p := d - t_f = 23$ in

Moment at Column Face: $M_u = 24972$ kip·in

Panel Zone Shear: $R_u := \frac{M_u}{d_p} = 1086$ kip

Doubler Plate Thickness: $t_{dp} := 0$ in

Panel Zone Thickness: $t_z := t_{wc} + t_{dp} = 1$ in

Minumum Panel Zone: $t_{z.min} := \dfrac{(d - 2 \cdot t_f + d_c - 2 \cdot t_{fc})}{90} = 0.683$ in

$$R_v := 0.6 \cdot F_{yc} \cdot d_c \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^2}{d \cdot d_c \cdot t_{wc}}\right) = 1354 \text{ kip}$$

AISC 360-10 (J10-11)

$\phi_v := 1.00$

$t_{wc} = 1$ in

$t_{z.min} = 0.683$ in

$\phi_v \cdot R_v = 1354$ kip

$R_u - V_c = 944$ kip

$$\begin{array}{|l} \text{"Ok"} \ \text{ if } (t_{wc} \ge t_{z.min}) \wedge (\phi_v \cdot R_v \ge R_u - V_c) \\ \text{"No Good"} \ \text{ otherwise} \end{array} = \text{"Ok"}$$

Column-Beam Moment Ratio: AISC 341-10 E3.4a

$$\Sigma M_{pc} := 2 \cdot Z_{xc} \cdot \left(F_{yc} - \frac{P_{uc}}{A_{gc}}\right) \cdot \left(\frac{\frac{H_c}{2}}{\frac{H_c}{2} - \frac{d}{2}}\right) = 133530 \text{ kip} \cdot \text{in}$$

2 because column extends above and below beam

$$\Sigma M_{pb} := M_{pr} \cdot \left(\frac{L_h + \frac{d_c}{2}}{L_h}\right) = 25939 \text{ kip} \cdot \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 5.148$$

$$\begin{array}{|l} \text{"Ok"} \ \text{ if } \dfrac{\Sigma M_{pc}}{\Sigma M_{pb}} > 1 \\ \text{"No Good"} \ \text{ otherwise} \end{array} = \text{"Ok"}$$

Highly Ductile Section Check: AISC 341-10 Table D1.1

Compression Ratio: $C_a := \dfrac{\left(\frac{F_{total}}{2}\right)}{A_{gc} \cdot F_{yc}} = 0.018$ assuming $C_a < 0.125$

Flange Limit:

$$bt_{limit} := 0.3 \cdot \sqrt{\frac{E}{F_{yc}}} = 7.225$$

$$bf2tf := \frac{b_{fc}}{2 \cdot t_{fc}} = 6.4$$

Web Limit:

$$ht_{limit} := 2.45 \cdot \sqrt{\frac{E}{F_{yc}}} \cdot (1 - 0.93 \cdot C_a) = 58.009$$

$$htw := \frac{h_{wc}}{t_{wc}} = 39.5$$

$$\begin{array}{|l} \text{"Ok"} \ \text{ if } (bf2tf \le bt_{limit}) \wedge (htw \le ht_{limit}) \\ \text{"No Good"} \ \text{ otherwise} \end{array} = \text{"Ok"}$$

Continuity Plates:

Minimum Continuity Plate Thickness $t_{cp.min} := \frac{t_f}{2} = 0.5$ in AISC 341-10 E3-6f-2a

Column Flange Flexural Yielding: $\phi_{FFY} := 1.00$ AISC 358-10 Sec. 6.10

$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} = 4$ in

Based on including continuity plates

$$Y_c := \left[\frac{b_{fc}}{2} \cdot \left(\frac{h_0}{s} + \frac{h_1}{p_{fo}} + \frac{h_2}{p_{fi}} + \frac{h_3}{s} \right) + \frac{1}{2 \cdot g} \cdot \left[\begin{array}{l} h_0 \cdot (4 \cdot s + 3 \cdot p_b) + h_1 \cdot (4 \cdot p_{fo} + p_b) \ldots \\ + h_2 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_3 \cdot (4 \cdot s + p_b) \end{array} \right] + g \right] = 530 \quad \text{in}$$

$M_{n.FFY} := F_{yc} \cdot t_{fc}^{\,2} \cdot Y_c = 41417$ kip·in AISC 358-10 (6.10-14)

$R_{n.FFY} := \frac{M_{n.FFY}}{(d - t_f)} = 1801$ kip AISC 358-10 (6.10-15)

Equivalent Column Flange Design Force: $\phi_{FFY} \cdot R_{n.FFY} = 1801$ kip

Column Local Web Yielding: $\phi_{LWY} := 1.00$

Assume: Flange to Web Weld $t_{weld} := \frac{5}{16} = 0.3125$ in

$$C_t := \begin{cases} 0.5 & \text{if } \frac{H_c}{2} - \frac{d}{2} < d_c \\ 1.0 & \text{otherwise} \end{cases} = 1$$

Distance from outer face of column flange to web toe of fillet: $k_c := t_{fc} + t_{weld} = 1.5625$ in

$R_{n.LWY} := C_t \cdot (6k_c + t_f + 2 \cdot t_p) \cdot F_{yc} \cdot t_{wc} = 643.75$ kip AISC 358-10 (6.10-17)

$\phi_{LWY} \cdot R_{n.LWY} = 643.75$ kip

Web Compression Buckling: $\phi_{WCB} := 0.75$

$R_{n.WCB} := \frac{24 \cdot t_{wc}^{\,3} \cdot \sqrt{E \cdot F_{yc}}}{h_{wc}} = 731.641$ kip AISC 358-10 (6.10-19)

$\phi_{WCB} \cdot R_{n.WCB} = 548.7$ kip

Web Crippling: $\phi_{WC} := 0.75$

Groove Weld Leg Reinforcement: $t_{weld.f} := \frac{5}{16} = 0.3125$ in

$$R_{n.WC} := 0.8 \cdot t_{wc}^{2} \cdot \left[1 + 3 \cdot \frac{t_f + 2 \cdot t_{weld.f}}{d_c} \cdot \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right] \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}} = 1166 \text{ kip}$$

AISC 358-10 (6.10-22)

$\phi_{WC} \cdot R_{n.WC} = 874.9$ kip

Stiffener Plate Required Strength:

$$\phi R_{n.min} := \min\left(\phi_{FFY} \cdot R_{n.FFY}, \phi_{LWY} \cdot R_{n.LWY}, \phi_{WCB} \cdot R_{n.WCB}, \phi_{WC} \cdot R_{n.WC}\right) = 548.7 \text{ kip}$$

Applied Force: $F_{fu} := \frac{M_u}{\left(d - t_f\right)} = 1085.7$ kip

Required Stiffener Strength: $F_{su} := F_{fu} - \phi R_{n.min} = 537$ kip

Minimum Stiffener Dimensions: AISC 360-10 J10-8

Stiffener Thickness: $t_{cp} := 1.00$ in

Stiffener Width: $w_{st} := \left(\frac{b_{fc} - t_{wc}}{2} \right) - 0.5 = 7$ in

$w_{st} + 0.5 \cdot t_{wc} = 7.5$ in > $\frac{1}{3} \cdot b_{fc} = 5.333$ in

$t_{cp} = 1$ in > $\max\left(0.5t_f, 0.5t_p, \frac{b_f}{16}\right) = 0.75$ in

Local Buckling Criterion: $t_{cp.min} := \frac{w_{st}}{0.56 \cdot \sqrt{\frac{E}{F_{yc}}}} = 0.519$ in

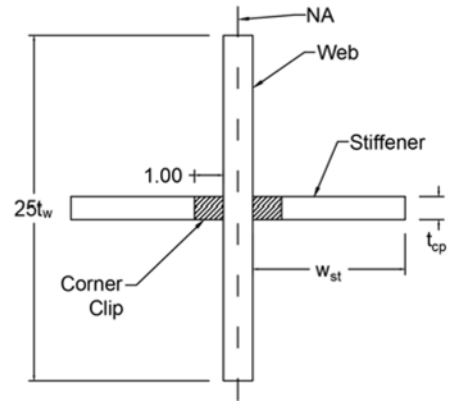
$$\begin{array}{l} \text{"Ok"} \quad \text{if } \left(w_{st} + 0.5 \cdot t_{wc} \geq \frac{1}{3} \cdot b_{fc} \right) \wedge \left(t_{cp} \geq \max\left(0.5 \cdot t_f, 0.5 \cdot t_p, \frac{b_f}{16}\right)\right) \wedge \left(t_{cp} \geq t_{cp.min}\right) \\ \text{"No Good"} \quad \text{otherwise} \end{array} = \text{"Ok"}$$

Check Stiffener Strength: Tension $\phi := 1.00$

Gross Cruciform Area: $A_{g.st} := 2t_{cp}\cdot(w_{st}) + 25\cdot t_{wc} = 39$ in^2

Net Cruciform Area: $A_{n.st} := 2t_{cp}\cdot(w_{st} - 1) + 25\cdot t_{wc} = 37$ in^2

Factoring in corner clip



Cruciform shape used for tension and compression design

Nominal Tensile Strength: $R_{n.st} := A_{n.st}\cdot F_{yc} = 1850$ kip

Design Tensile Strength: $\phi\cdot R_{n.st} = 1850$ kip

$$\begin{cases} \text{"Ok"} & \text{if } \phi\cdot R_{n.st} > F_{su} \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Check Stiffener Strength: Compression $\phi_c := 0.9$

Assume: Effective Length Factor: $k := 0.75$

Stiffener Height: $L_{st} := h_{wc} = 39.5$ in

Cruciform Moment of Inertia: $I_{st} := \frac{1}{12}\cdot t_{cp}\cdot(2\cdot w_{st} + t_{wc})^3 + \frac{1}{12}\cdot(25\cdot t_{wc} - w_{st})\cdot t_{wc}^{\ 3} = 282.8$ in^4

Cruciform Radius of Gyration: $r_{st} := \sqrt{\frac{I_{st}}{A_{g.st}}} = 2.693$ in

Effective Slenderness Ratio: $\frac{k\cdot L_{st}}{r_{st}} = 11$

Effective Slenderness Limit: $4.71\cdot\sqrt{\frac{E}{F_{yc}}} = 113.432$

Nominal Compressive Strength:

$$P_{n.st} := \left| \begin{array}{l} F_e \leftarrow \dfrac{\pi^2\cdot E}{\left(\dfrac{k\cdot L_{st}}{r_{st}}\right)^2} \\ A_{g.st}\cdot\left(0.658^{\frac{F_{yc}}{F_e}}\right)\cdot F_{yc} \quad \text{if } \dfrac{k\cdot L_{st}}{r_{st}} \le 4.71\cdot\sqrt{\dfrac{E}{F_{yc}}} \\ A_{g.st}\cdot(0.877)\cdot F_e \quad \text{otherwise} \end{array} \right.$$

$P_{n.st} = 1933$ kip

Design Compressive Strength: $\phi_c\cdot P_{n.st} = 1740$ kip

Required Compressive Strength: $F_{su} = 537$ kip

$$\begin{cases} \text{"Ok"} & \text{if } \phi_c\cdot P_{n.st} \ge F_{su} \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Check Bearing on Stiffeners: $\phi := 0.9$

Net Stiffener Pair Area: $A_{e.st} := 2 \cdot (w_{st} - 1) \cdot t_{cp} = 12 \quad in^2$ Factoring in corner clips

Nominal Bearing Strength: $R_{n.bearing} := 1.8 \cdot F_{yc} \cdot A_{e.st} = 1080 \quad kip$

Design Bearing Strength: $\phi \cdot R_{n.bearing} = 972 \quad kip$

Required Bearing Strength: $F_{su} = 537 \quad kip$

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \phi \cdot R_{n.bearing} \geq F_{su} \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Bearing Stiffeners: Concentrated Force from W14x873 Stub Sections

AISC 360-10 Chap. J10

Maximum Force from Stub Section: $V_u := \frac{F_{total}}{L_{bC}} \cdot \left(L_h + \frac{d_c}{2} \right) = 142 \quad kip$

Flange Local Bending: From Yield Line Analysis

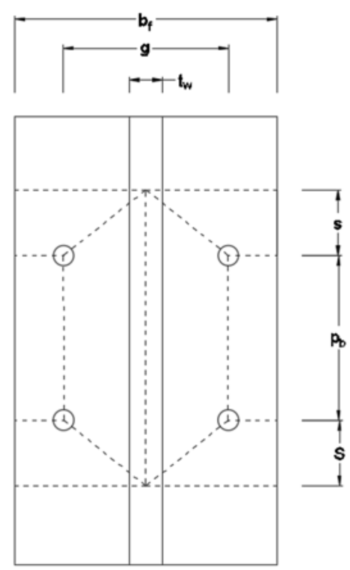
$\phi_{FLB} := 0.9$

$$s := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g_{stub}} = 6.595 \quad in$$

$$Y := \frac{\left(4 \cdot s^2 + b_{fc} \cdot g_{stub} + 2 \cdot p_{b.stub} \cdot s \right)}{s \cdot (g_{stub})} = 6.921$$

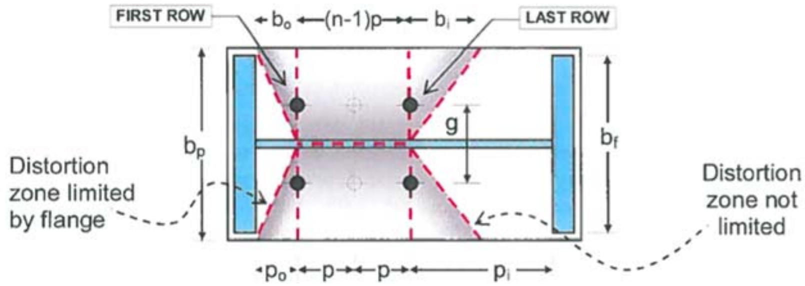
Nominal Strength: $R_{n.FLB} := F_{yc} \cdot t_{fc}^{\,2} \cdot Y = 540.7 \quad kip$

Design Strength: $\phi_{FLB} \cdot R_{n.FLB} = 486.6 \quad kip$



<u>Flange Local Bending</u>: BlueScope Buildings Yield Line Analysis

$b := \dfrac{b_{fc}}{\sqrt{2}} = 11.314$ in



$b_i := b = 11.314$ in

$b_o := b = 11.314$ in

Number of Bolt Rows: $n := 2$

$$Y := \begin{cases} \dfrac{1}{g_{stub}}\cdot\left[\dfrac{b_{fc}^{\ 2}}{2}\cdot\left(\dfrac{1}{b_o}+\dfrac{1}{b_i}\right)+b_o+b_i+(n-1)\cdot p_{b.stub}\right] & \text{if } p_{b.stub} \le \dfrac{b_f}{\sqrt{2}} = 8.323 \\ \dfrac{1}{g_{stub}}\cdot\left(2\cdot\dfrac{b_{fc}^{\ 2}}{b}+4\cdot b\right) & \text{otherwise} \end{cases}$$

Nominal Strength: $R_{n.BS} := F_{yc}\cdot t_{fc}^{\ 2}\cdot Y = 650.2$ kip

Design Strength: $\phi_{FLB}\cdot R_{n.BS} = 585.2$ kip

<u>Web Local Yielding</u>: $\phi_{WLY} := 1.00$

Length of Bearing $l_b := 19.625$ in

Outside Flange Surface to Fillet Toe $k := t_{fc} = 1.25$ in

Nominal Strength: $R_{n.WLY} := t_{wc}\cdot F_{yc}\cdot(5\cdot k + l_b) = 1293.8$ kip

Design Strength: $\phi_{WLY}\cdot R_{n.WLY} = 1293.8$ kip

<u>Web Crippling</u>: $\phi_{WC} := 0.9$

Nominal Strength: $R_{n.WC} := 0.8\cdot t_{wc}^{\ 2}\cdot\left[1+3\cdot\dfrac{l_b}{d_c}\cdot\left(\dfrac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\cdot\sqrt{\dfrac{E\cdot F_{yc}\cdot t_{fc}}{t_{wc}}} = 2157.3$ kip

Design Strength: $\phi_{WC}\cdot R_{n.WC} = 1941.6$ kip

Web Sidesway Buckling: $\phi_{WSB} := 0.9$

Compression Flange Not Restrained Against Rotation

$C_r := 480000$ ksi

Nominal Strength: $$R_{n.WSB} := \frac{C_r \cdot t_{wc}^{3} \cdot t_{fc}}{h_{wc}^{2}} \cdot \left[0.4 \cdot \left[\frac{\left(\frac{h_{wc}}{t_{wc}}\right)}{\left(\frac{L_{bC}}{b_{fc}}\right)}\right]^{3}\right] = 5088.6 \text{ kip}$$

Design Strength: $\phi_{WSB} \cdot R_{n.WSB} = 4579.7$ kip

$$\left| \begin{array}{l} \text{"WSB applies"} \quad \text{if } \dfrac{\left(\frac{h_w}{t_w}\right)}{\left(\frac{L_{bC}}{b_f}\right)} \le 1.7 \\ \text{"WSB does not apply"} \quad \text{otherwise} \end{array} \right. = \text{"WSB does not apply"}$$

Web Compression Buckling: $\phi_{WCB} := 1.00$

Nominal Strength: $$R_{n.WCB} := \frac{24 \cdot t_{wc}^{3} \cdot \sqrt{E \cdot F_{yc}}}{h_{wc}} = 731.6 \text{ kip}$$

Design Strength: $\phi_{WCB} \cdot R_{n.WCB} = 731.6$ kip

Concentrated Force Strength:

$$\phi R_{n.min} := \min\left(\phi_{FLB} \cdot R_{n.FLB}, \phi_{FLB} \cdot R_{n.BS}, \phi_{WLY} \cdot R_{n.WLY}, \phi_{WC} \cdot R_{n.WC}, \phi_{WSB} \cdot R_{n.WSB}, \phi_{WCB} \cdot R_{n.WCB}\right)$$

Design Strength: $\phi R_{n.min} = 486.6$ kip

Required Strength: $F_u := V_u = 142.2$ kip

$$\left| \begin{array}{l} \text{"Need bearing stiffeners"} \quad \text{if } \phi R_{n.min} < F_u \\ \text{"Do not need bearing stiffeners"} \quad \text{otherwise} \end{array} \right. = \text{"Do not need bearing stiffeners"}$$

Column Base Plates

Required Strength: $F_u := \frac{F_{total}}{2} = 72.1$ kip

One end is in tension, while other end is in compression

Base Plate Yield Strength: $F_{y.plate} := F_{yc} = 50$ ksi

Base Plate Thickness: $t_{plate} := 1.00$ in

Base Plate Width: $b_p := 21$ in

Bolt Spacing: $p_b := 9$ in

Bolt Gage: $g := 9$ in

Distance from Bolt Hole to Inside of Flange: $s_1 := \frac{h_{wc}}{2} - \frac{p_b}{2} = 15.25$ in

Distance Required to Minimize Energy: $s_2 := \frac{1}{2} \cdot \sqrt{b_p \cdot g} = 6.874$ in

$s := \min(s_1, s_2) = 6.874$ in

$\phi := 1.00$

Yield Line Parameter: $Y := \frac{\left(4 \cdot s^2 + 2 \cdot p_b \cdot s + b_p \cdot g\right)}{g \cdot s} = 8.11$

Nominal Strength: $P_{n.plate} := F_{y.plate} \cdot t_{plate}^{\,2} \cdot Y = 405.5$ kip

Design Strength: $\phi \cdot P_{n.plate} = 405.5$ kip

$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi \cdot P_{n.plate} > F_u \\ \text{"Increase Base Plate Thickness"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$

Column Bracing Threaded Rods

F1554 GR. 105 Threaded Rod

Diameter of Rods: $d_{rod} := 3$ in

Rod Yield Strength: $F_{yR} := 105$ ksi

Number of Rods: $n_{rods} := 4$

$\phi := 0.9$

Nominal Strength: $R_{n.rod} := n_{rods} \cdot \frac{\pi}{4} \cdot d_{rod}^{\,2} \cdot F_{yR} = 2969$ kip

Design Strength: $\phi \cdot R_{n.rod} = 2672$ kip

Required Strength: $V_u = 142.2$ kip

$\text{Check} := \left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi R_{n.min} \ge F_u \\ \text{"Increase bolt size"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$

Column

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_{yc} := 50$ ksi

Ultimate Strength: $F_{uc} := 65$ ksi

Expected Yield Ratio: $R_y := 1.1$

Weld:

Filler Metal Classification Strength:

$F_{exx} := 70$ ksi

Column Input: Assume: Doubly Symmetric

Flange Width: $b_{fc} := 16.00$ in

Flange Thickness: $t_{fc} := 1.25$ in

Web Thickness: $t_{wc} := 1.00$ in

Total Beam Depth: $d_c := 42.00$ in

Web Plate Depth: $h_{wc} := d_c - 2 \cdot t_{fc} = 39.50$ in

Distance between Stub Sections:

$$L_b := 16 \cdot 12 + 4 + \frac{7}{8} = 196.875 \text{ in}$$

Gross Cross Sectional Area: $A_{gc} := 2 \cdot b_{fc} \cdot t_{fc} + h_{wc} \cdot t_{wc} = 79.5$ in^2

Top Flange Cross-Sectional Area: $A_{top.fc} := b_{fc} \cdot t_{fc} = 20$ in^2

Web Cross-Sectional Area: $A_{wc} := h_{wc} \cdot t_{wc} = 39.5$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.fc} := b_{fc} \cdot t_{fc} = 20$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.fc} := \dfrac{t_{fc}}{2} = 0.625$ in

Web: $C_{wc} := t_{fc} + \dfrac{h_{wc}}{2} = 21$ in

Bottom Flange: $C_{bot.fc} := t_{fc} + h_{wc} + \dfrac{t_{fc}}{2} = 41.375$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.fc} := C_{wc} - C_{top.fc} = 20.375$ in

Web: $CtC_{wc} := 0$ in

Bottom Flange: $CtC_{bot.fc} := C_{bot.fc} - C_{wc} = 20.375$ in

Plastic Section Modulus:

$$Z_{xc} := 2 \cdot \left[A_{top.fc} \cdot \left(C_{wc} - C_{top.fc} \right) + \left(\frac{A_{wc}}{2} \right) \cdot \left(\frac{h_{wc}}{4} \right) \right] = 1205 \ \text{in}^3$$

Moment of Inertia:

$$I_{xc} := \frac{1}{12} \cdot t_{wc} \cdot h_{wc}^{\,3} + 2 \left(\frac{1}{12} \cdot b_{fc} \cdot t_{fc}^{\,3} + b_{fc} \cdot t_{fc} \cdot CtC_{top.fc}^{\,2} \right) = 21747 \ \text{in}^4$$

Column Height:

$$H_c := 20 \cdot 12 + 6 + \frac{1}{16} = 246.063 \ \text{in}$$

44 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12.00$ in

Flange Thickness: $t_f := 1.00$ in

Web Thickness: $t_w := 0.75$ in

Total Beam Depth: $d := 44.00$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 42.00$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 31.5$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 22$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 43.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 21.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 21.5$ in

Plastic Section Modulus: $$Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f}\right) + \left(\frac{A_w}{2}\right) \cdot \left(\frac{h_w}{4}\right)\right] = 847 \quad in^3$$

Moment of Inertia: $$I_x := \frac{1}{12} \cdot t_w \cdot h_w^3 + 2\left(\frac{1}{12} \cdot b_f \cdot t_f^3 + b_f \cdot t_f \cdot CtC_{top.f}^2\right) = 15727 \quad in^4$$

Elastic Section Modulus: $$S_x := \frac{I_x}{C_w} = 714.84 \quad in^3$$

Peak Connection Strength Factor: $C_{pr} := \dfrac{(F_y + F_u)}{2 \cdot F_y} = 1.136$

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 46571$ kip·in

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 58214$ kip·in

Actuator Force:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \dfrac{1}{4}\right) = 173.25$ in

Beam 1 - End Plate 12ES-0.75-1.00-44

Limit State: Rupture of Bolts without Prying Action

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.00$ in

End Plate Width: $b_p := 12.00$ in

Inner Bolt Gage: $g := 4.00$ in

Outer Bolt Gage: $g_o := 2.25$ in

Flange to Outer Bolt CL: $p_{fo} := 1.50$ in

Flange to Inner Bolt CL: $p_{fi} := 1.50$ in

Bolt Spacing: $p_b := 2.25$ in

Outer Bolt to Edge: $d_e := 1.00$ in

Top of Flange to Edge: $p_{ext} := d_e + p_b + p_{fo} = 4.75$ in

Bolts:

$d_b := 0.75$ in

$T_b := 35$ kip

(AISC 360-10 Table J3.1)

Stiffeners:

$L_{st} := 8.25$ in

$S_h := L_{st} + t_p = 9.25$ in

Actuator Force:

$M_{np} = M_u := 1.25 \cdot (25760) = 32200$ kip·in Increased for 25% bolt over strength

$$F_{total} := \frac{M_u}{L_h} = 185.9 \quad \text{kip}$$

Beam Web to Flange Weld

Panel Zone:

AISC 358-10 Sect. 2.3-2b.(1)
The elements of built-up I-shaped columns shall conform to the requirements of the AISC *Seismic Provisions*.
Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, unless specifically indicated in this Standard, the column webs and flanges shall be connected using CJP groove welds with a pair of reinforcing fillet welds. The minimum size of the fillet welds shall be the lesser of 5/16 in. (8 mm) and the thickness of the column web.

Length of Weld: $L_{weld} := d + 2 \cdot (12) = 68$ in

Size of Fillet Weld: $t_{fillet} := \min\left(\frac{5}{16}, t_{wc}\right) = 0.3125$ in

The web to flange shall be CJP groove welded for the middle 68 inches with 5/16" double sided reinforcing fillet welds.

Remaining Portion of Beam:

Shear in Column Outside Panel Zone: $V_c := \frac{F_{total}}{L_b} \cdot \left(L_h + \frac{d_c}{2} \right) = 183.4$ kip

First Moment of Area: $Q := b_{fc} \cdot t_{fc} \cdot \left(\frac{h_{wc}}{2} + \frac{t_{fc}}{2} \right) = 407.5$ in^3

Shear Flow: $q := \frac{V_c \cdot Q}{I_{xc}} = 3.436$ $\frac{kip}{in}$

$\phi_v := 0.75$

For 1 side:

$\phi_v \cdot \frac{a}{16} \cdot (0.707) \cdot (0.6) \cdot F_{exx} = q$ --> $a := \frac{16 \cdot q}{\phi_v \cdot (0.707) \cdot (0.6) \cdot F_{exx}} = 2.469$ 16ths of an inch

$a := ceil(a) = 3$ 16ths of an inch

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Use 5/16" fillet weld on one side of the web

Continuity Plate Welds

Stiffener to Flange:

AISC 341-10 E3-6f.(3)
Continuity plates shall be welded to column flanges using CJP groove welds.

Stiffener to Web: AISC 341-10 E3-6f.(3)

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

$\phi_r := 0.75$

Length of Weld: $L_{weld} := h_{wc} - 2 \cdot (1) = 37.5$ in subracting out corner clips

Size of Fillet Weld: $t_{fillet} := \frac{5}{16} = 0.3125$ in

Nominal Strength: $R_{n.fillet} := 4 \cdot 0.707 \cdot t_{fillet} \cdot 0.6 \cdot F_{exx} \cdot L_{weld} = 1391.9$ kip

for 2 sides of 2 stiffeners on each = 4 welds

Design Strength: $\phi_r \cdot R_{n.fillet} = 1044$ kip

Required Strength: $F_{su} := 68.2$ kip

$$\left| \begin{array}{ll} \text{"Ok"} & \text{if } \phi_r \cdot R_{n.fillet} \ge F_{su} \\ \text{"No Good"} & \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Use 5/16" fillet weld on each side of the stiffeners

Column Base Plate Welds

Filler Metal Classification Strength: $F_{exx} = 70 \ \ \text{ksi}$

Length of Weld: $\text{Perimeter}_c := 2 \cdot b_{fc} + 2 \cdot (b_{fc} - t_{wc}) + 2 \cdot (d_c - t_{fc}) = 143.5$ in

Size of Fillet Weld: $t_{fillet} := \dfrac{5}{16} = 0.3125$ in

$\phi_r := 0.75$

Angle of Loading: $\theta := \dfrac{\pi}{2}$ rad

Nominal Stress of Weld Metal: $F_{nw.fillet} := 0.6 \cdot F_{exx} \cdot \left(1 + 0.5 \cdot \sin(\theta)^{1.5}\right) = 63 \ \ \text{ksi}$

Nominal Strength: $R_{n.fillet} := 0.707 \cdot t_{fillet} \cdot F_{nw.fillet} \cdot \text{Perimeter}_c = 1997.4$ kip

Design Strength: $\phi_r \cdot R_{n.fillet} = 1498$ kip

Required Strength: $F_u := \dfrac{F_{total}}{2} = 92.9$ kip

$$\left| \begin{array}{l} \text{"Ok" if } \phi_r \cdot R_{n.fillet} \geq F_u \\ \text{"No Good" otherwise} \end{array} \right. = \text{"Ok"}$$

Minimum fillet weld = 5/16" (AISC 360 Table J2.4)

Use 5/16" fillet weld on each side of the web

Beam 1 - End Plate 12ES-1.25-1.50-44

Limit State: Plastic Hinging of the Beam

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.50$ in

End Plate Width: $b_p := 16.00$ in

Inner Bolt Gage: $g := 4.50$ in

Outer Bolt Gage: $g_o := 3.75$ in

Flange to Outer Bolt CL: $p_{fo} := 1.875$ in

Flange to Inner Bolt CL: $p_{fi} := 1.875$ in

Bolt Spacing: $p_b := 3.75$ in

Outer Bolt to Edge: $d_e := 1.625$ in

Top of Flange to Edge: $p_{ext} := d_e + p_b + p_{fo} = 7.25$ in

Bolts:

$d_b := 1.25$ in

$T_b := 102$ kip

(AISC 360-10 Table J3.1)

Stiffeners:

$L_{st} := 12.625$ in

$S_h := L_{st} + t_p = 14.125$ in

Actuator Force:

M_f = $M_u := 63382$ kip·in

$$F_{total} := \frac{M_u}{L_h} = 365.8 \quad \text{kip}$$

Beam Web to Flange Weld

Panel Zone:

AISC 358-10 Sect. 2.3-2b.(1)
The elements of built-up I-shaped columns shall conform to the requirements of the AISC *Seismic Provisions*.
Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, unless specifically indicated in this Standard, the column webs and flanges shall be connected using CJP groove welds with a pair of reinforcing fillet welds. The minimum size of the fillet welds shall be the lesser of 5/16 in. (8 mm) and the thickness of the column web.

Length of Weld: $L_{weld} := d + 2\cdot(12) = 68$ in

Size of Fillet Weld: $t_{fillet} := \min\left(\frac{5}{16}, t_{wc}\right) = 0.3125$ in

The web to flange shall be CJP groove welded for the middle 68 inches with 5/16" double sided reinforcing fillet welds.

Remaining Portion of Beam:

Shear in Column Outside Panel Zone: $V_c := \frac{F_{total}}{L_b} \cdot \left(L_h + \frac{d_c}{2}\right) = 361$ kip

First Moment of Area: $Q := b_{fc} \cdot t_{fc} \cdot \left(\frac{h_{wc}}{2} + \frac{t_{fc}}{2}\right) = 407.5 \ \ in^3$

Shear Flow: $q := \frac{V_c \cdot Q}{I_{xc}} = 6.764 \ \ \frac{kip}{in}$

$\phi_v := 0.75$

For 1 side:

$\phi_v \cdot \frac{a}{16} \cdot (0.707) \cdot (0.6) \cdot F_{exx} = q$ --> $a := \frac{16 \cdot q}{\phi_v \cdot (0.707) \cdot (0.6) \cdot F_{exx}} = 4.859$ 16ths of an inch

$a := \text{ceil}(a) = 5$ 16ths of an inch

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Use 5/16" fillet weld on one side of the web

Continuity Plate Welds

Stiffener to Flange:

AISC 341-10 E3-6f.(3)
Continuity plates shall be welded to column flanges using CJP groove welds.

Stiffener to Web: AISC 341-10 E3-6f.(3)

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

$\phi_r := 0.75$

Length of Weld: $L_{weld} := h_{wc} - 2 \cdot (1) = 37.5$ in subracting out corner clips

Size of Fillet Weld: $t_{fillet} := \frac{5}{16} = 0.3125$ in

Nominal Strength: $R_{n.fillet} := 4 \cdot 0.707 \cdot t_{fillet} \cdot 0.6 \cdot F_{exx} \cdot L_{weld} = 1391.9$ kip

for 2 sides of 2 stiffeners on each = 4 welds

Design Strength: $\phi_r \cdot R_{n.fillet} = 1044$ kip

Required Strength: $F_{su} := 68.2$ kip

$$\left| \begin{array}{l} \text{"Ok"} \ \ \text{if} \ \ \phi_r \cdot R_{n.fillet} \geq F_{su} \\ \text{"No Good"} \ \ \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Use 5/16" fillet weld on each side of the stiffeners

Column Base Plate Welds

Filler Metal Classification Strength: $F_{exx} = 70 \ \ ksi$

Length of Weld: $Perimeter_c := 2 \cdot b_{fc} + 2 \cdot (b_{fc} - t_{wc}) + 2 \cdot (d_c - t_{fc}) = 143.5$ in

Size of Fillet Weld: $t_{fillet} := \frac{5}{16} = 0.3125$ in

$\phi_r := 0.75$

Angle of Loading: $\theta := \frac{\pi}{2}$ rad

Nominal Stress of Weld Metal: $F_{nw.fillet} := 0.6 \cdot F_{exx} \cdot \left(1 + 0.5 \cdot \sin(\theta)^{1.5}\right) = 63 \ \ ksi$

Nominal Strength: $R_{n.fillet} := 0.707 \cdot t_{fillet} \cdot F_{nw.fillet} \cdot Perimeter_c = 1997.4 \ \ kip$

Design Strength: $\phi_r \cdot R_{n.fillet} = 1498 \ \ kip$

Required Strength: $F_u := \frac{F_{total}}{2} = 182.9 \ \ kip$

$$\left| \begin{array}{l} \text{"Ok"} \ \ \text{if } \phi_r \cdot R_{n.fillet} \geq F_u \\ \text{"No Good"} \ \ \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Minimum fillet weld = 5/16" (AISC 360 Table J2.4)

Use 5/16" fillet weld on each side of the web

24 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12.00$ in

Flange Thickness: $t_f := 1.00$ in

Web Thickness: $t_w := 0.50$ in

Total Beam Depth: $d := 24.00$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 22.00$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 11$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 12$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 23.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 11.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 11.5$ in

Plastic Section Modulus: $Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f}\right) + \left(\frac{A_w}{2}\right) \cdot \left(\frac{h_w}{4}\right)\right] = 337$ in^3

Moment of Inertia: $I_x := \frac{1}{12} \cdot t_w \cdot h_w^3 + 2\left(\frac{1}{12} \cdot b_f \cdot t_f^3 + b_f \cdot t_f \cdot CtC_{top.f}^2\right) = 3620$ in^4

Elastic Section Modulus: $S_x := \frac{I_x}{C_w} = 301.64$ in^3

Peak Connection Strength Factor: $C_{pr} := \frac{(F_y + F_u)}{2 \cdot F_y} = 1.136$

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 18508$ kip·in

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 23134$ kip·in

Actuator Force:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Beam 2 - End Plate 12ES-0.875-0.75-24

Limit States: End Plate Yielding & Rupture of Bolts With Prying Action

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 0.75$ in

End Plate Width: $b_p := 12.00$ in

Inner Bolt Gage: $g := 3.50$ in

Outer Bolt Gage: $g_o := 2.625$ in

Flange to Outer Bolt CL: $p_{fo} := 1.50$ in

Flange to Inner Bolt CL: $p_{fi} := 1.50$ in

Bolt Spacing: $p_b := 2.625$ in

Outer Bolt to Edge: $d_e := 1.125$ in

Top of Flange to Edge: $p_{ext} := d_e + p_b + p_{fo} = 5.25$ in

Bolts:

$d_b := 0.875$ in

$T_b := 49$ kip

(AISC 360-10 Table J3.1)

Stiffeners:

$L_{st} := 9.125$ in

$S_h := L_{st} + t_p = 9.875$ in

Actuator Force:

$M_q = M_u := 1.25 \cdot (14872) = 18590$ kip·in   Increased for 25% bolt over strength

$$F_{total} := \frac{M_u}{L_h} = 107.3 \text{ kip}$$

Beam Web to Flange Weld

Panel Zone:

AISC 358-10 Sect. 2.3-2b.(1)
The elements of built-up I-shaped columns shall conform to the requirements of the AISC *Seismic Provisions*.
Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, unless specifically indicated in this Standard, the column webs and flanges shall be connected using CJP groove welds with a pair of reinforcing fillet welds. The minimum size of the fillet welds shall be the lesser of 5/16 in. (8 mm) and the thickness of the column web.

Length of Weld: $L_{weld} := d + 2 \cdot (12) = 48$ in

Size of Fillet Weld: $t_{fillet} := \min\left(\frac{5}{16}, t_{wc}\right) = 0.3125$ in

The web to flange shall be CJP groove welded for the middle 48 inches with 5/16" double sided reinforcing fillet welds.

Remaining Portion of Beam:

Shear in Column Outside Panel Zone: $V_c := \frac{F_{total}}{L_b} \cdot \left(L_h + \frac{d_c}{2}\right) = 105.9$ kip

First Moment of Area: $Q := b_{fc} \cdot t_{fc} \cdot \left(\frac{h_{wc}}{2} + \frac{t_{fc}}{2}\right) = 407.5$ in^3

Shear Flow: $q := \frac{V_c \cdot Q}{I_{xc}} = 1.984$ $\frac{kip}{in}$

$\phi_v := 0.75$

For 1 side:

$\phi_v \cdot \frac{a}{16} \cdot (0.707) \cdot (0.6) \cdot F_{exx} = q$ --> $a := \frac{16 \cdot q}{\phi_v \cdot (0.707) \cdot (0.6) \cdot F_{exx}} = 1.425$ 16ths of an inch

$a := ceil(a) = 2$ 16ths of an inch

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Use 5/16" fillet weld on one side of the web

Continuity Plate Welds

Stiffener to Flange:

AISC 341-10 E3-6f.(3)
Continuity plates shall be welded to column flanges using CJP groove welds.

Stiffener to Web: AISC 341-10 E3-6f.(3)

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

$\phi_r := 0.75$

Length of Weld: $L_{weld} := h_{wc} - 2 \cdot (1) = 37.5$ in subracting out corner clips

Size of Fillet Weld: $t_{fillet} := \frac{5}{16} = 0.3125$ in

Nominal Strength: $R_{n.fillet} := 4 \cdot 0.707 \cdot t_{fillet} \cdot 0.6 \cdot F_{exx} \cdot L_{weld} = 1391.9$ kip

for 2 sides of 2 stiffeners on each = 4 welds

Design Strength: $\phi_r \cdot R_{n.fillet} = 1044$ kip

Required Strength: $F_{su} := 68.2$ kip

$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \phi_r \cdot R_{n.fillet} \geq F_{su} \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Use 5/16" fillet weld on each side of the stiffeners

Column Base Plate Welds

Filler Metal Classification Strength: $F_{exx} = 70 \ \ \text{ksi}$

Length of Weld: $\text{Perimeter}_c := 2 \cdot b_{fc} + 2 \cdot \left(b_{fc} - t_{wc}\right) + 2 \cdot \left(d_c - t_{fc}\right) = 143.5 \quad \text{in}$

Size of Fillet Weld: $t_{fillet} := \dfrac{5}{16} = 0.3125 \quad \text{in}$

$\phi_r := 0.75$

Angle of Loading: $\theta := \dfrac{\pi}{2} \quad \text{rad}$

Nominal Stress of Weld Metal: $F_{nw.fillet} := 0.6 \cdot F_{exx} \cdot \left(1 + 0.5 \cdot \sin(\theta)^{1.5}\right) = 63 \ \ \text{ksi}$

Nominal Strength: $R_{n.fillet} := 0.707 \cdot t_{fillet} \cdot F_{nw.fillet} \cdot \text{Perimeter}_c = 1997.4 \quad \text{kip}$

Design Strength: $\phi_r \cdot R_{n.fillet} = 1498 \quad \text{kip}$

Required Strength: $F_u := \dfrac{F_{total}}{2} = 53.7 \quad \text{kip}$

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \phi_r \cdot R_{n.fillet} \geq F_u \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Minimum fillet weld = 5/16" (AISC 360 Table J2.4)

Use 5/16" fillet weld on each side of the web

Beam 2 - End Plate 12ES-1.125-1.25-24

Limit State: Plastic Hinging of the Beam

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.25$ in

End Plate Width: $b_p := 14.00$ in

Inner Bolt Gage: $g := 4.00$ in

Outer Bolt Gage: $g_o := 3.375$ in

Flange to Outer Bolt CL: $p_{fo} := 1.75$ in

Flange to Inner Bolt CL: $p_{fi} := 1.75$ in

Bolt Spacing: $p_b := 3.375$ in

Outer Bolt to Edge: $d_e := 1.50$ in

Top of Flange to Edge: $p_{ext} := d_e + p_b + p_{fo} = 6.625$ in

Bolts:

$d_b := 1.125$ in

$T_b := 80$ kip

(AISC 360-10 Table J3.1)

Stiffeners:

$L_{st} := 11.5$ in

$S_h := L_{st} + t_p = 12.75$ in

Actuator Force:

M_f = $M_u := 24972$ kip·in

$$F_{total} := \frac{M_u}{L_h} = 144.1 \text{ kip}$$

Beam Web to Flange Weld

Panel Zone:

AISC 358-10 Sect. 2.3-2b.(1)
The elements of built-up I-shaped columns shall conform to the requirements of the AISC *Seismic Provisions*.
Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, unless specifically indicated in this Standard, the column webs and flanges shall be connected using CJP groove welds with a pair of reinforcing fillet welds. The minimum size of the fillet welds shall be the lesser of 5/16 in. (8 mm) and the thickness of the column web.

Length of Weld: $L_{weld} := d + 2\cdot(12) = 48$ in

Size of Fillet Weld: $t_{fillet} := \min\left(\frac{5}{16}, t_{wc}\right) = 0.3125$ in

The web to flange shall be CJP groove welded for the middle 48 inches with 5/16" double sided reinforcing fillet welds.

Remaining Portion of Beam:

Shear in Column Outside Panel Zone: $V_c := \frac{F_{total}}{L_b} \cdot \left(L_h + \frac{d_c}{2}\right) = 142.2$ kip

First Moment of Area: $Q := b_{fc} \cdot t_{fc} \cdot \left(\frac{h_{wc}}{2} + \frac{t_{fc}}{2}\right) = 407.5$ in^3

Shear Flow: $q := \frac{V_c \cdot Q}{I_{xc}} = 2.665$ $\frac{kip}{in}$

$\phi_v := 0.75$

For 1 side:

$\phi_v \cdot \frac{a}{16} \cdot (0.707) \cdot (0.6) \cdot F_{exx} = q$ --> $a := \frac{16 \cdot q}{\phi_v \cdot (0.707) \cdot (0.6) \cdot F_{exx}} = 1.915$ 16ths of an inch

$a := \text{ceil}(a) = 2$ 16ths of an inch

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Use 5/16" fillet weld on one side of the web

Continuity Plate Welds

Stiffener to Flange:

AISC 341-10 E3-6f.(3)
Continuity plates shall be welded to column flanges using CJP groove welds.

Stiffener to Web: AISC 341-10 E3-6f.(3)

Filler Metal Classification Strength: $F_{exx} = 70$ ksi

$\phi_r := 0.75$

Length of Weld: $L_{weld} := h_{wc} - 2 \cdot (1) = 37.5$ in subracting out corner clips

Size of Fillet Weld: $t_{fillet} := \frac{5}{16} = 0.3125$ in

Nominal Strength: $R_{n.fillet} := 4 \cdot 0.707 \cdot t_{fillet} \cdot 0.6 \cdot F_{exx} \cdot L_{weld} = 1391.9$ kip

for 2 sides of 2 stiffeners on each = 4 welds

Design Strength: $\phi_r \cdot R_{n.fillet} = 1044$ kip

Required Strength: $F_{su} := 68.2$ kip

$$\left| \begin{array}{l} \text{"Ok" if } \phi_r \cdot R_{n.fillet} \geq F_{su} \\ \text{"No Good" otherwise} \end{array} \right. = \text{"Ok"}$$

Minimum Fillet Weld = 5/16" (AISC 360-10 Table J2.4)

Use 5/16" fillet weld on each side of the stiffeners

Column Base Plate Welds

Filler Metal Classification Strength: $F_{exx} = 70 \ \ ksi$

Length of Weld: $Perimeter_c := 2 \cdot b_{fc} + 2 \cdot \left(b_{fc} - t_{wc}\right) + 2 \cdot \left(d_c - t_{fc}\right) = 143.5$ in

Size of Fillet Weld: $t_{fillet} := \dfrac{5}{16} = 0.3125$ in

$\phi_r := 0.75$

Angle of Loading: $\theta := \dfrac{\pi}{2}$ rad

Nominal Stress of Weld Metal: $F_{nw.fillet} := 0.6 \cdot F_{exx} \cdot \left(1 + 0.5 \cdot \sin(\theta)^{1.5}\right) = 63 \ \ ksi$

Nominal Strength: $R_{n.fillet} := 0.707 \cdot t_{fillet} \cdot F_{nw.fillet} \cdot Perimeter_c = 1997.4 \ \ kip$

Design Strength: $\phi_r \cdot R_{n.fillet} = 1498 \ \ kip$

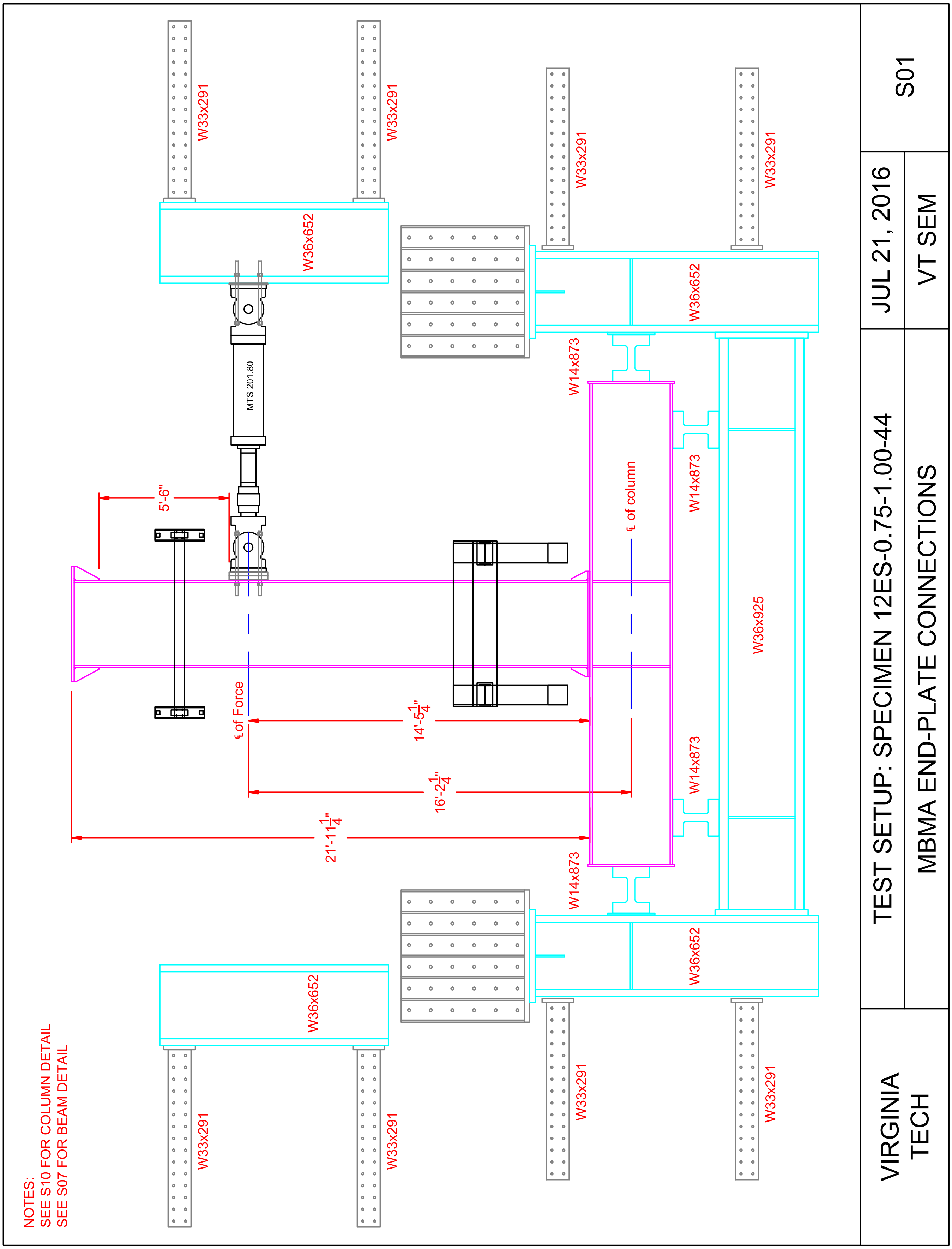
Required Strength: $F_u := \dfrac{F_{total}}{2} = 72.1 \ \ kip$

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } \phi_r \cdot R_{n.fillet} \geq F_u \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

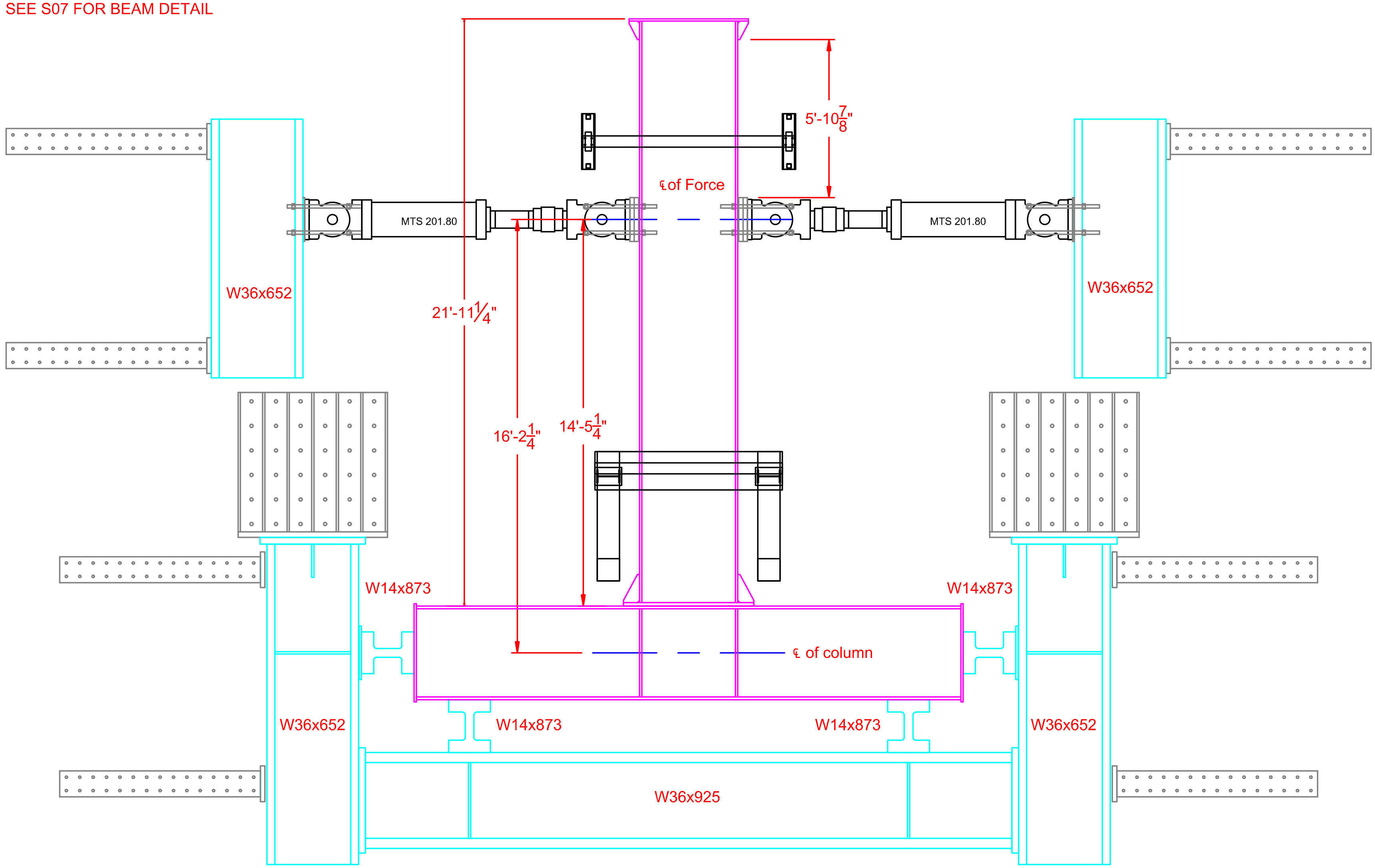
Minimum fillet weld = 5/16" (AISC 360 Table J2.4)

Use 5/16" fillet weld on each side of the web

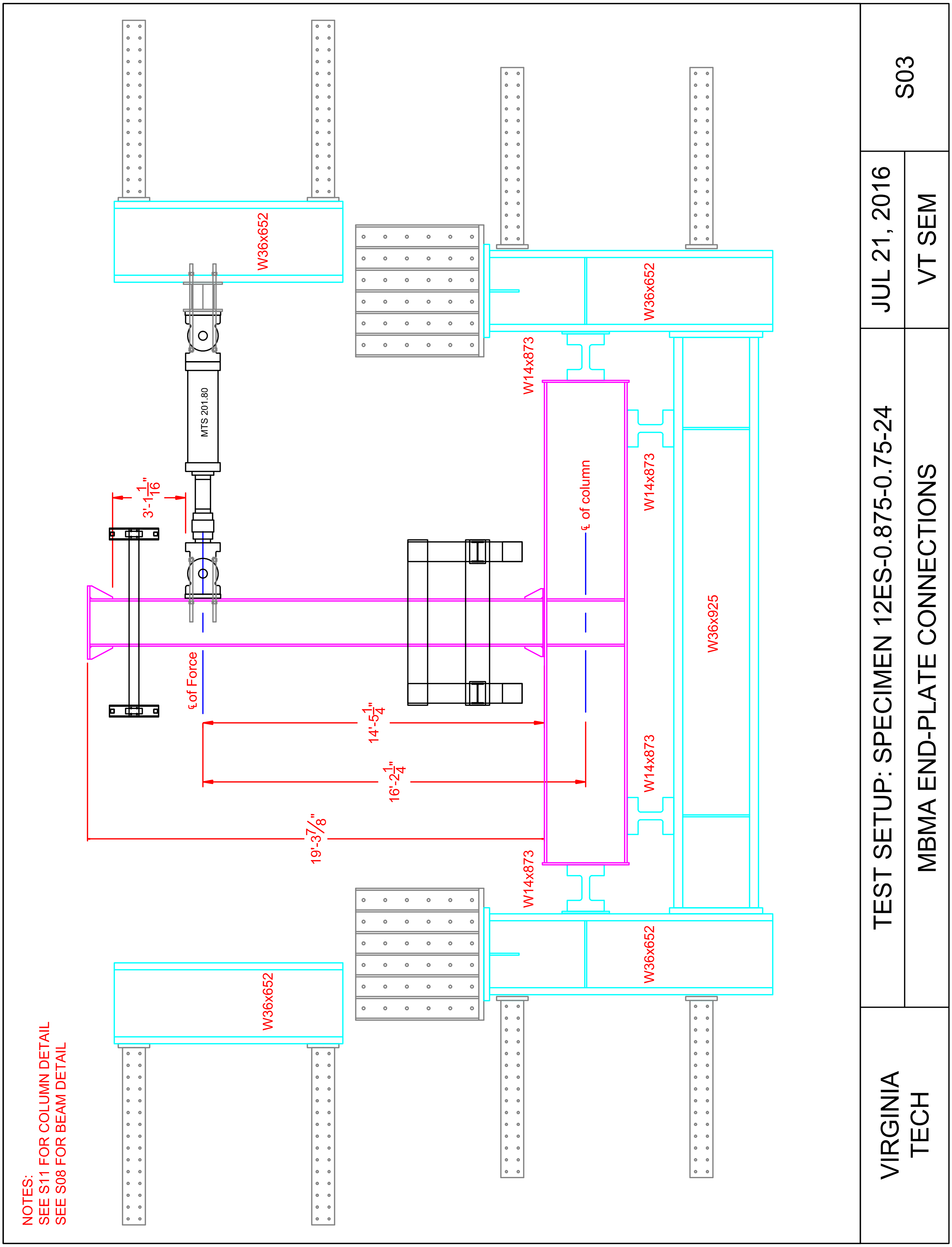
APPENDIX H TEST SETUP DRAWINGS


NOTES:
SEE S10 FOR COLUMN DETAIL
SEE S07 FOR BEAM DETAIL
W33x291
W36x652
W14x873
W36x925
MTS 201.80
5'-6"
14'-5 1/4"
16'-2 1/4"
21'-11 1/4"
℄ of Force
℄ of column
VIRGINIA TECH
TEST SETUP: SPECIMEN 12ES-0.75-1.00-44
MBMA END-PLATE CONNECTIONS
JUL 21, 2016
VT SEM
S01

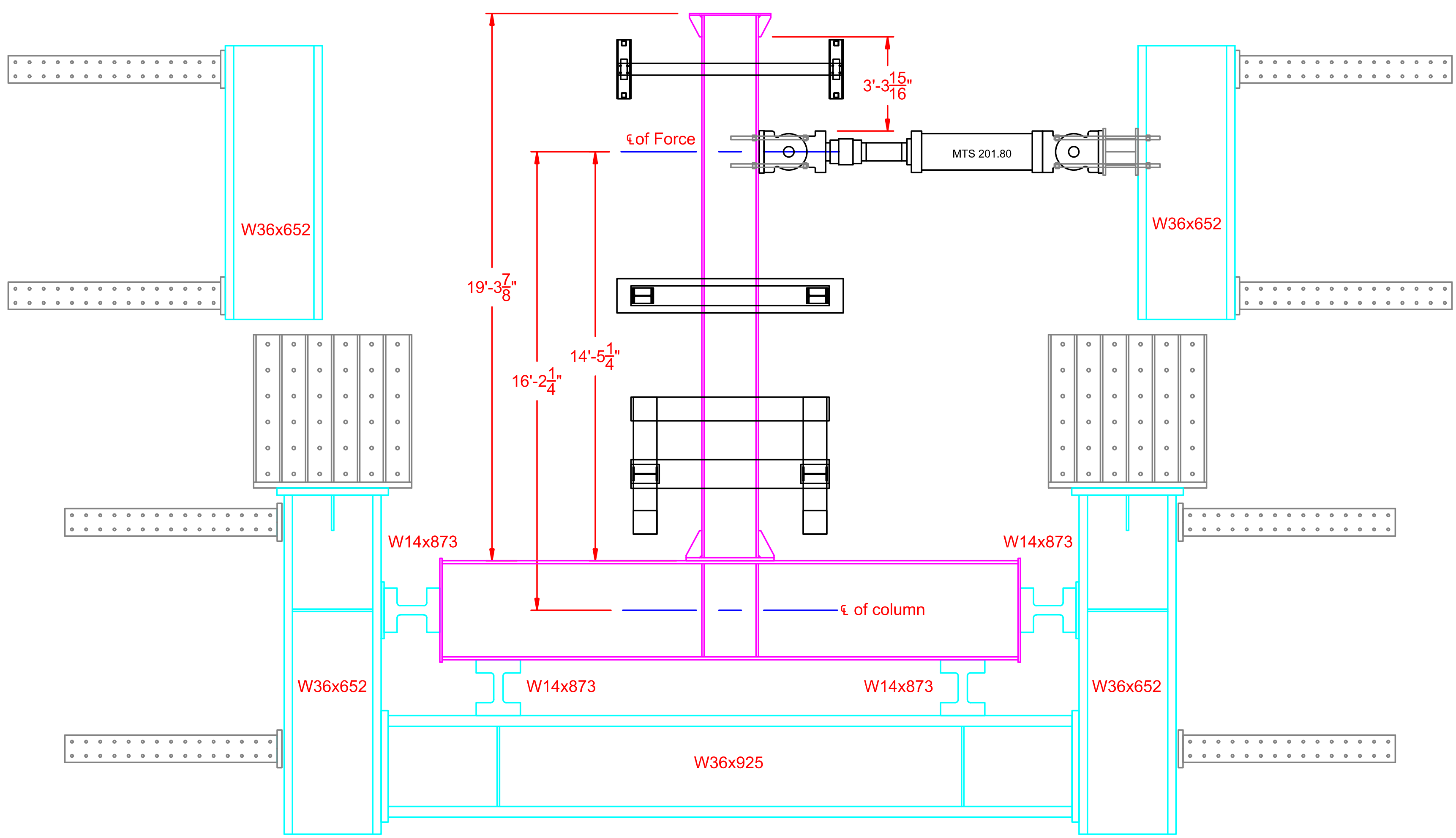
NOTES:
SEE S10 FOR COLUMN DETAIL
SEE S07 FOR BEAM DETAIL



VIRGINIA TECH	TEST SETUP: SPECIMEN 12ES-1.25-1.50-44	JUL 21, 2016	S02
	MBMA END-PLATE CONNECTIONS	VT SEM	


NOTES:
SEE S11 FOR COLUMN DETAIL
SEE S08 FOR BEAM DETAIL
W36x652
W36x652
W36x652
W36x652
W14x873
W14x873
W14x873
W14x873
W36x925
MTS 201.80
℄ of Force
℄ of column
3'-1 1/16"
14'-5 1/4"
16'-2 1/4"
19'-3 7/8"
VIRGINIA TECH
TEST SETUP: SPECIMEN 12ES-0.875-0.75-24
MBMA END-PLATE CONNECTIONS
JUL 21, 2016
VT SEM
S03

NOTES:
SEE S11 FOR COLUMN DETAIL
SEE S08 FOR BEAM DETAIL



VIRGINIA TECH	TEST SETUP: SPECIMEN 12ES-1.125-1.25-24	JUL 21, 2016	S04
	MBMA END-PLATE CONNECTIONS	VT SEM	

APPENDIX I CALCULATIONS FOR LATERAL BRACING LOCATIONS

44 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12$ in

Flange Thickness: $t_f := 1$ in

Web Thickness: $t_w := 0.75$ in

Total Beam Depth: $d := 44$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 42$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 31.5$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Total Cross-Sectional Area $A := A_{top.f} + A_w + A_{bot.f} = 55.5$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 22$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 43.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 21.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 21.5$ in

Plastic Section Modulus: $Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f} \right) + \left(\frac{A_w}{2} \right) \cdot \left(\frac{h_w}{4} \right) \right] = 846.8$ in^3

Moment of Inertia: $I_x := \frac{1}{12} \cdot t_w \cdot h_w^{\ 3} + 2 \left(\frac{1}{12} \cdot b_f \cdot t_f^{\ 3} + b_f \cdot t_f \cdot CtC_{top.f}^{\ 2} \right) = 15727$ in^4

Moment of Inertia: $I_y := 2 \cdot \left(\frac{1}{12} \cdot t_f \cdot b_f^3\right) + \frac{1}{12} \cdot h_w \cdot t_w^3 = 289.5$ in^4

Radius of Gyration $r_y := \sqrt{\frac{I_y}{A}} = 2.284$ in

Elastic Section Modulus: $S_x := \frac{I_x}{C_w} = 714.84$ in^3

$$r_{ts} := \sqrt{\frac{I_y \cdot (d - t_f)}{2 \cdot S_x}} = 2.951$$

Torsional Constant: $J := \frac{1}{3} \cdot \left[2 \cdot b_f \cdot t_f^3 + (d - t_f) \cdot t_w^3\right] = 14.047$ in^4

For Doubly Symmetric I-Shapes: $c := 1$

Peak Connection Strength Factor: $C_{pr} := \frac{(F_y + F_u)}{2 \cdot F_y} = 1.136$

Section Plastic Moment Capacity: $M_p := F_y \cdot Z_x = 46571$ kip·in

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_x = 58214$ kip·in

Actuator Input:

Distance from Column Face to CL of Actuator Forces: $L_h := 14 \cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25$ in

Slenderness:

$\lambda_f := \frac{b_f}{2 \cdot t_f} = 6$ $\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 8.726$ $\lambda_{rf} := 0.95 \cdot \sqrt{\frac{E}{F_y}} = 21.814$

$\lambda_w := \frac{h_w}{t_w} = 56$ $\lambda_{pw} := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 86.339$ $\lambda_{rw} := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 130.886$

Lateral Torsional Buckling:

$L_p := 92.3$ in

$L_r := 272$ in

Maximum Spacing of Bracing $L_{b.max} := 0.086 \cdot r_y \cdot \frac{E}{F_y} = 103.56$ in AISC 341-10 D1.2b

Beam 1 - End Plate 12ES-0.75-1.00-44

Limit State: Rupture of Bolts without Prying Action

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.00$ in

Stiffeners:

$L_{st} := 8.25$ in

$S_h := L_{st} + t_p = 9.25$ in

Actuator Force:

M_{np} = $M_u := 1.25 \cdot (25760) = 32200$ kip·in   Increased for 25% bolt over strength

$$V_u := \frac{M_u}{L_h} = 185.9 \quad \text{kip}$$

First Unbraced Length: $L_{b1} := S_h + d = 53.25$ in

Moment at Location of Plastic Hinge: $M_{PH} := V_u \cdot (L_h - S_h) = 30480.8$ kip·in

Moment at d from Location of Plastic Hinge: $M_{PH.LB1} := V_u \cdot (L_h - L_{b1}) = 22303$ kip·in

Second Unbraced Length: $L_{b2} := 12 \cdot (12) + 11 = 155$ in

Determine Lateral Torsional Buckling Modification Factor:

Maximum Moment in Unbraced Region: $M_{max} := M_{PH.LB1} = 22303$ kip·in

Moment at 1/4 Point in Unbraced Region: $$M_A := \left| V_u \cdot \left(L_h - L_{b1} - \frac{L_{b2}}{4} \right) \right| = 15101 \quad \text{kip·in}$$

Moment at 1/2 Point in Unbraced Region: $$M_B := \left| V_u \cdot \left(L_h - L_{b1} - \frac{L_{b2}}{2} \right) \right| = 7899 \quad \text{kip·in}$$

Moment at 3/4 Point in Unbraced Region: $$M_C := \left| V_u \cdot \left(L_h - L_{b1} - \frac{3 \cdot L_{b2}}{4} \right) \right| = 697 \quad \text{kip·in}$$

Lateral Torsional Buckling Modification Factor: $C_b := \dfrac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C} = 2.069$

Nominal Moment Strength: Lateral Torsional Buckling

$$
M_{n.LTB1}(L_b, C_b) := \left|
\begin{array}{l}
\text{if } \lambda_w \le \lambda_{pw} \\
\quad \left|
\begin{array}{l}
M_p \quad \text{if } L_b \le L_p \\
\min\left[M_p, C_b \cdot \left[M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right]\right] \quad \text{if } L_p < L_b \le L_r \\
\text{otherwise} \\
\quad \left|
\begin{array}{l}
F_{cr} \leftarrow \dfrac{C_b \cdot \pi^2 \cdot E}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.0078 \cdot \dfrac{J \cdot c}{S_x \cdot (d - t_f)} \cdot \left(\dfrac{L_b}{r_{ts}}\right)^2} \\
\min\left(M_p, F_{cr} \cdot S_x\right)
\end{array}
\right.
\end{array}
\right. \\
\text{otherwise} \\
\quad \left|
\begin{array}{l}
h_c \leftarrow h_w \\
a_w \leftarrow \min\left(10, \dfrac{h_c \cdot t_w}{b_f \cdot t_f}\right) \\
r_t \leftarrow \dfrac{b_f}{\sqrt{12 \cdot \left[\dfrac{(d - t_f)}{d} + \dfrac{1}{6} \cdot a_w \cdot \dfrac{h_w^{\,2}}{(d - t_f) \cdot d}\right]}} \\
R_{pg} \leftarrow \min\left[1, 1 - \dfrac{a_w}{1200 - 300 \cdot a_w} \cdot \left(\dfrac{h_c}{t_w} - 5.7 \cdot \sqrt{\dfrac{E}{F_y}}\right)\right] \\
M_p \quad \text{if } L_b \le L_p \\
\text{if } L_p < L_b \le L_r \\
\quad \left|
\begin{array}{l}
F_{cr} \leftarrow \min\left[F_y, C_b \cdot \left[F_y - 0.3 \cdot F_y \cdot \left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right]\right] \\
R_{pg} \cdot F_{cr} \cdot S_x
\end{array}
\right. \\
\text{otherwise} \\
\quad \left|
\begin{array}{l}
F_{cr} \leftarrow \min\left[F_y, \dfrac{C_b \cdot \pi^2 \cdot E}{\left(\dfrac{L_b}{r_t}\right)^2}\right] \\
R_{pg} \cdot F_{cr} \cdot S_x
\end{array}
\right.
\end{array}
\right.
\end{array}
\right.
$$

Nominal Moment Capacity: $M_{n.LTB} := M_{n.LTB1}\left(L_{b2}, C_b\right) = 46571.3 \quad kip \cdot in$

$$\begin{cases} \text{"Ok"} & \text{if } \left(M_{n.LTB} \geq M_{PH.LB1}\right) \vee \left(L_{b2} \leq L_p\right) \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Place lateral bracing at d from end of stiffener and along strong floor beam past actuator

Beam 1 - End Plate 12ES-1.25-1.50-44

Limit State: Plastic Hinging of the Beam

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.50$ in

Stiffeners:

$L_{st} := 12.625$ in

$S_h := L_{st} + t_p = 14.125$ in

Actuator Force:

M_f = $M_u := 63382$ kip·in

$$V_u := \frac{M_u}{L_h} = 365.8 \quad \text{kip}$$

First Unbraced Length: $L_{b1} := S_h + d = 58.125$ in

Moment at Location of Plastic Hinge: $M_{PH} := V_u \cdot (L_h - S_h) = 58214.5$ kip·in

Moment at d from Location of Plastic Hinge: $M_{PH.LB1} := V_u \cdot (L_h - L_{b1}) = 42117.5$ kip·in

Second Unbraced Length: $L_{b2} := 12 \cdot (17) + 4.25 - L_{b1} = 150.125$ in

Determine Lateral Torsional Buckling Modification Factor:

Maximum Moment in Unbraced Region: $M_{max} := M_{PH.LB1} = 42117.5$ kip·in

Moment at 1/4 Point in Unbraced Region: $$M_A := \left| V_u \cdot \left(L_h - L_{b1} - \frac{L_{b2}}{4} \right) \right| = 28387 \quad \text{kip·in}$$

Moment at 1/2 Point in Unbraced Region: $$M_B := \left| V_u \cdot \left(L_h - L_{b1} - \frac{L_{b2}}{2} \right) \right| = 14656.5 \quad \text{kip·in}$$

Moment at 3/4 Point in Unbraced Region: $$M_C := \left| V_u \cdot \left(L_h - L_{b1} - \frac{3 \cdot L_{b2}}{4} \right) \right| = 926 \quad \text{kip·in}$$

Lateral Torsional Buckling Modification Factor: $C_b := \dfrac{12.5\cdot M_{max}}{2.5\cdot M_{max} + 3\cdot M_A + 4\cdot M_B + 3\cdot M_C} = 2.09$

<u>Nominal Moment Strength</u>: Lateral Torsional Buckling

$$
M_{n.LTB1}(L_b, C_b) := \left|
\begin{array}{l}
\text{if } \lambda_w \le \lambda_{pw} \\
\quad \left|
\begin{array}{l}
M_p \quad \text{if } L_b \le L_p \\
\min\left[M_p, C_b\cdot\left[M_p - \left(M_p - 0.7\cdot F_y\cdot S_x\right)\cdot\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right]\right] \quad \text{if } L_p < L_b \le L_r \\
\text{otherwise} \\
\quad \left|
\begin{array}{l}
F_{cr} \leftarrow \dfrac{C_b\cdot\pi^2\cdot E}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\cdot\sqrt{1 + 0.0078\cdot\dfrac{J\cdot c}{S_x\cdot\left(d - t_f\right)}\cdot\left(\dfrac{L_b}{r_{ts}}\right)^2} \\
\min\left(M_p, F_{cr}\cdot S_x\right)
\end{array}
\right.
\end{array}
\right. \\
\text{otherwise} \\
\quad \left|
\begin{array}{l}
h_c \leftarrow h_w \\
a_w \leftarrow \min\left(10, \dfrac{h_c\cdot t_w}{b_f\cdot t_f}\right) \\
r_t \leftarrow \dfrac{b_f}{\sqrt{12\cdot\left[\dfrac{\left(d - t_f\right)}{d} + \dfrac{1}{6}\cdot a_w\cdot\dfrac{h_w^{\,2}}{\left(d - t_f\right)\cdot d}\right]}} \\
R_{pg} \leftarrow \min\left[1, 1 - \dfrac{a_w}{1200 - 300\cdot a_w}\cdot\left(\dfrac{h_c}{t_w} - 5.7\cdot\sqrt{\dfrac{E}{F_y}}\right)\right] \\
M_p \quad \text{if } L_b \le L_p \\
\text{if } L_p < L_b \le L_r \\
\quad \left|
\begin{array}{l}
F_{cr} \leftarrow \min\left[F_y, C_b\cdot\left[F_y - 0.3\cdot F_y\cdot\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right]\right] \\
R_{pg}\cdot F_{cr}\cdot S_x
\end{array}
\right. \\
\text{otherwise} \\
\quad \left|
\begin{array}{l}
F_{cr} \leftarrow \min\left[F_y, \dfrac{C_b\cdot\pi^2\cdot E}{\left(\dfrac{L_b}{r_t}\right)^2}\right] \\
R_{pg}\cdot F_{cr}\cdot S_x
\end{array}
\right.
\end{array}
\right.
\end{array}
\right.
$$

Nominal Moment Capacity: $M_{n.LTB} := M_{n.LTB1}(L_{b2}, C_b) = 46571.3 \quad kip \cdot in$

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } (M_{n.LTB} \geq M_{PH.LB1}) \vee (L_{b2} \leq L_p) \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Place lateral bracing at d from end of stiffener and along strong floor beam past actuator

24 Inch Deep Beam

Material Properties:

Modulus of Elasticity: $E := 29000$ ksi

Yield Strength: $F_y := 55$ ksi

Ultimate Strength: $F_u := 70$ ksi

Expected Yield Ratio: $R_y := 1.1$

Rafter Input: Assume: Doubly Symmetric Girder

Flange Width: $b_f := 12.00$ in

Flange Thickness: $t_f := 1.00$ in

Web Thickness: $t_w := 0.50$ in

Total Beam Depth: $d := 24.00$ in

Web Plate Depth: $h_w := d - 2 \cdot t_f = 22.00$ in

Top Flange Cross-Sectional Area: $A_{top.f} := b_f \cdot t_f = 12$ in^2

Web Cross-Sectional Area: $A_w := h_w \cdot t_w = 11$ in^2

Bottom Flange Cross-Sectional Area: $A_{bot.f} := b_f \cdot t_f = 12$ in^2

Total Cross-Sectional Area $A := A_{top.f} + A_w + A_{bot.f} = 35$ in^2

Distance From Centroid of Shape to Top of Plate Girder:

Top Flange: $C_{top.f} := \frac{t_f}{2} = 0.5$ in

Web: $C_w := t_f + \frac{h_w}{2} = 12$ in

Bottom Flange: $C_{bot.f} := t_f + h_w + \frac{t_f}{2} = 23.5$ in

Distance From Centroid of Shape to Centroid of Plate Girder:

Top Flange: $CtC_{top.f} := C_w - C_{top.f} = 11.5$ in

Web: $CtC_w := 0$ in

Bottom Flange: $CtC_{bot.f} := C_{bot.f} - C_w = 11.5$ in

Plastic Section Modulus: $Z_x := 2 \cdot \left[A_{top.f} \cdot \left(C_w - C_{top.f}\right) + \left(\frac{A_w}{2}\right) \cdot \left(\frac{h_w}{4}\right)\right] = 336.5$ in^3

Moment of Inertia: $I_x := \frac{1}{12} \cdot t_w \cdot h_w^{\ 3} + 2\left(\frac{1}{12} \cdot b_f \cdot t_f^{\ 3} + b_f \cdot t_f \cdot CtC_{top.f}^{\ 2}\right) = 3620$ in^4

Moment of Inertia: $I_y := 2\cdot\left(\frac{1}{12}\cdot t_f\cdot b_f^3\right) + \frac{1}{12}\cdot h_w\cdot t_w^3 = 288.2 \quad in^4$

Radius of Gyration $r_y := \sqrt{\frac{I_y}{A}} = 2.87 \quad in$

Elastic Section Modulus: $S_x := \frac{I_x}{C_w} = 301.64 \quad in^3$

$$r_{ts} := \sqrt{\frac{I_y\cdot(d - t_f)}{2\cdot S_x}} = 3.315$$

Torsional Constant: $J := \frac{1}{3}\cdot\left[2\cdot b_f\cdot t_f^3 + (d - t_f)\cdot t_w^3\right] = 8.958 \quad in^4$

For Doubly Symmetric I-Shapes: $c := 1$

Peak Connection Strength Factor: $C_{pr} := \frac{(F_y + F_u)}{2\cdot F_y} = 1.136$

Section Plastic Moment Capacity: $M_p := F_y\cdot Z_x = 18508 \quad kip\cdot in$

Maximum Strain Hardening Plastic Moment: $M_{pr} := C_{pr}\cdot R_y\cdot F_y\cdot Z_x = 23134 \quad kip\cdot in$

Actuator Input:

Distance from Column Face to CL of Actuator Forces: $L_h := 14\cdot 12 + \left(5 + \frac{1}{4}\right) = 173.25 \quad in$

Slenderness:

$\lambda_f := \frac{b_f}{2\cdot t_f} = 6$ $\quad \lambda_{pf} := 0.38\cdot\sqrt{\frac{E}{F_y}} = 8.726$ $\quad \lambda_{rf} := 0.95\cdot\sqrt{\frac{E}{F_y}} = 21.814$

$\lambda_w := \frac{h_w}{t_w} = 44$ $\quad \lambda_{pw} := 3.76\cdot\sqrt{\frac{E}{F_y}} = 86.339$ $\quad \lambda_{rw} := 5.70\cdot\sqrt{\frac{E}{F_y}} = 130.886$

Lateral Torsional Buckling:

$L_p := 115.98 \quad in$

$L_r := 343.49 \quad in$

Maximum Spacing of Bracing $L_{b.max} := 0.086\cdot r_y\cdot\frac{E}{F_y} = 130.127 \quad in$ AISC 341-10 D1.2b

Beam 2 - End Plate 12ES-0.875-0.75-24

Limit States: End Plate Yielding & Rupture of Bolts With Prying Action

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 0.75$ in

Stiffeners:

$L_{st} := 9.125$ in

$S_h := L_{st} + t_p = 9.875$ in

Actuator Force:

M_q = $M_u := 1.25 \cdot (14872) = 18590$ kip·in   Increased for 25% bolt over strength

$$V_u := \frac{M_u}{L_h} = 107.3 \quad \text{kip}$$

First Unbraced Length: $L_{b1} := S_h + d = 33.875$ in

Moment at Location of Plastic Hinge: $M_{PH} := V_u \cdot (L_h - S_h) = 17530.4$ kip·in

Moment at d from Location of Plastic Hinge: $M_{PH.LB1} := V_u \cdot (L_h - L_{b1}) = 14955.2$ kip·in

Second Unbraced Length: $L_{b2} := 12 \cdot (17) + 4.25 - L_{b1} = 174.375$ in

Determine Lateral Torsional Buckling Modification Factor:

Maximum Moment in Unbraced Region: $M_{max} := M_{PH.LB1} = 14955.2$ kip·in

Moment at 1/4 Point in Unbraced Region: $M_A := \left| V_u \cdot \left(L_h - L_{b1} - \frac{L_{b2}}{4} \right) \right| = 10277.5$ kip·in

Moment at 1/2 Point in Unbraced Region: $M_B := \left| V_u \cdot \left(L_h - L_{b1} - \frac{L_{b2}}{2} \right) \right| = 5599.8$ kip·in

Moment at 3/4 Point in Unbraced Region: $M_C := \left| V_u \cdot \left(L_h - L_{b1} - \frac{3 \cdot L_{b2}}{4} \right) \right| = 922.1$ kip·in

Lateral Torsional Buckling Modification Factor: $C_b := \dfrac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C} = 2.002$

<u>Nominal Moment Strength</u>: Lateral Torsional Buckling

$$
M_{n.LTB1}(L_b, C_b) := \left|
\begin{array}{l}
\text{if } \lambda_w \le \lambda_{pw} \\
\quad \left|
\begin{array}{l}
M_p \quad \text{if } L_b \le L_p \\
\min\left[M_p, C_b \cdot \left[M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right]\right] \quad \text{if } L_p < L_b \le L_r \\
\text{otherwise} \\
\quad \left|
\begin{array}{l}
F_{cr} \leftarrow \dfrac{C_b \cdot \pi^2 \cdot E}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.0078 \cdot \dfrac{J \cdot c}{S_x \cdot (d - t_f)} \cdot \left(\dfrac{L_b}{r_{ts}}\right)^2} \\
\min\left(M_p, F_{cr} \cdot S_x\right)
\end{array}
\right.
\end{array}
\right. \\
\text{otherwise} \\
\quad \left|
\begin{array}{l}
h_c \leftarrow h_w \\
a_w \leftarrow \min\left(10, \dfrac{h_c \cdot t_w}{b_f \cdot t_f}\right) \\
r_t \leftarrow \dfrac{b_f}{\sqrt{12 \cdot \left[\dfrac{(d - t_f)}{d} + \dfrac{1}{6} \cdot a_w \cdot \dfrac{h_w^{\,2}}{(d - t_f) \cdot d}\right]}} \\
R_{pg} \leftarrow \min\left[1, 1 - \dfrac{a_w}{1200 - 300 \cdot a_w} \cdot \left(\dfrac{h_c}{t_w} - 5.7 \cdot \sqrt{\dfrac{E}{F_y}}\right)\right] \\
M_p \quad \text{if } L_b \le L_p \\
\text{if } L_p < L_b \le L_r \\
\quad \left|
\begin{array}{l}
F_{cr} \leftarrow \min\left[F_y, C_b \cdot \left[F_y - 0.3 \cdot F_y \cdot \left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right]\right] \\
R_{pg} \cdot F_{cr} \cdot S_x
\end{array}
\right. \\
\text{otherwise} \\
\quad \left|
\begin{array}{l}
F_{cr} \leftarrow \min\left[F_y, \dfrac{C_b \cdot \pi^2 \cdot E}{\left(\dfrac{L_b}{r_t}\right)^2}\right] \\
R_{pg} \cdot F_{cr} \cdot S_x
\end{array}
\right.
\end{array}
\right.
\end{array}
\right.
$$

Nominal Moment Capacity: $M_{n.LTB} := M_{n.LTB1}(L_{b2}, C_b) = 18507.5 \quad kip \cdot in$

$$\left| \begin{array}{l} \text{"Ok"} \quad \text{if } (M_{n.LTB} \ge M_{PH.LB1}) \vee (L_{b2} \le L_p) \\ \text{"No Good"} \quad \text{otherwise} \end{array} \right. = \text{"Ok"}$$

Place lateral bracing at d from end of stiffener and along strong floor beam past actuator

Beam 2 - End Plate 12ES-1.125-1.25-24

Limit State: Plastic Hinging of the Beam

End Plate Input:

Yield Strength: $F_{py} := 55$ ksi

Ultimate Strength: $F_{pu} := 70$ ksi

End Plate Thickness: $t_p := 1.25$ in

Stiffeners:

$L_{st} := 11.5$ in

$S_h := L_{st} + t_p = 12.75$ in

Actuator Force:

M_f = $M_u := 24972$ kip·in

$$V_u := \frac{M_u}{L_h} = 144.1 \quad \text{kip}$$

First Unbraced Length: $L_{b1} := S_h + 1.5d = 48.75$ in

Moment at Location of Plastic Hinge: $M_{PH} := V_u \cdot (L_h - S_h) = 23134.2$ kip·in

Moment at 1.5d from Location of Plastic Hinge: $M_{PH.LB1} := V_u \cdot (L_h - L_{b1}) = 17945.2$ kip·in

Second Unbraced Length: $L_{b2} := 12 \cdot (17) + 4.25 - L_{b1} = 159.5$ in

Determine Lateral Torsional Buckling Modification Factor:

Maximum Moment in Unbraced Region: $M_{max} := M_{PH.LB1} = 17945.2$ kip·in

Moment at 1/4 Point in Unbraced Region: $M_A := \left| V_u \cdot \left(L_h - L_{b1} - \frac{L_{b2}}{4} \right) \right| = 12197.7$ kip·in

Moment at 1/2 Point in Unbraced Region: $M_B := \left| V_u \cdot \left(L_h - L_{b1} - \frac{L_{b2}}{2} \right) \right| = 6450.2$ kip·in

Moment at 3/4 Point in Unbraced Region: $M_C := \left| V_u \cdot \left(L_h - L_{b1} - \frac{3 \cdot L_{b2}}{4} \right) \right| = 702.7$ kip·in

Lateral Torsional Buckling Modification Factor: $C_b := \dfrac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C} = 2.051$

<u>Nominal Moment Strength</u>: Lateral Torsional Buckling

$$
\begin{aligned}
M_{n.LTB1}(L_b, C_b) := \;& \text{if } \lambda_w \le \lambda_{pw} \\
& \quad \left| \begin{aligned}
& M_p \quad \text{if } L_b \le L_p \\
& \min\left[M_p, C_b \cdot \left[M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right]\right] \quad \text{if } L_p < L_b \le L_r \\
& \text{otherwise} \\
& \quad \left| \begin{aligned}
& F_{cr} \leftarrow \frac{C_b \cdot \pi^2 \cdot E}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.0078 \cdot \frac{J \cdot c}{S_x \cdot \left(d - t_f\right)} \cdot \left(\frac{L_b}{r_{ts}}\right)^2} \\
& \min\left(M_p, F_{cr} \cdot S_x\right)
\end{aligned} \right.
\end{aligned} \right. \\
& \text{otherwise} \\
& \quad \left| \begin{aligned}
& h_c \leftarrow h_w \\
& a_w \leftarrow \min\left(10, \frac{h_c \cdot t_w}{b_f \cdot t_f}\right) \\
& r_t \leftarrow \frac{b_f}{\sqrt{12 \cdot \left[\dfrac{\left(d - t_f\right)}{d} + \dfrac{1}{6} \cdot a_w \cdot \dfrac{{h_w}^2}{\left(d - t_f\right) \cdot d}\right]}} \\
& R_{pg} \leftarrow \min\left[1, 1 - \frac{a_w}{1200 - 300 \cdot a_w} \cdot \left(\frac{h_c}{t_w} - 5.7 \cdot \sqrt{\frac{E}{F_y}}\right)\right] \\
& M_p \quad \text{if } L_b \le L_p \\
& \text{if } L_p < L_b \le L_r \\
& \quad \left| \begin{aligned}
& F_{cr} \leftarrow \min\left[F_y, C_b \cdot \left[F_y - 0.3 \cdot F_y \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right]\right] \\
& R_{pg} \cdot F_{cr} \cdot S_x
\end{aligned} \right. \\
& \text{otherwise} \\
& \quad \left| \begin{aligned}
& F_{cr} \leftarrow \min\left[F_y, \frac{C_b \cdot \pi^2 \cdot E}{\left(\dfrac{L_b}{r_t}\right)^2}\right] \\
& R_{pg} \cdot F_{cr} \cdot S_x
\end{aligned} \right.
\end{aligned} \right.
\end{aligned}
$$

Nominal Moment Capacity: $M_{n.LTB} := M_{n.LTB1}(L_{b2}, C_b) = 18507.5 \quad kip \cdot in$

$$\begin{cases} \text{"Ok"} & \text{if } (M_{n.LTB} \ge M_{PH.LB1}) \vee (L_{b2} \le L_p) \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Moment at End of Second Unbraced Length: $M_{PH.LB2} := V_u \cdot (L_h - S_h - d - L_{b2}) = -3315.2 \quad kip \cdot in$

Third Unbraced Length: $L_{b3} := 12 \cdot (8) = 96 \quad in$

Determine Lateral Torsional Buckling Modification Factor:

Maximum Moment in Unbraced Region: $M_{max} := M_{PH.LB2} = -3315.2 \quad kip \cdot in$

Moment at 1/4 Point in Unbraced Region: $M_A := \left| V_u \cdot \left(L_h - L_{b1} - L_{b2} - \frac{L_{b3}}{4} \right) \right| = 8504.2 \quad kip \cdot in$

Moment at 1/2 Point in Unbraced Region: $M_B := \left| V_u \cdot \left(L_h - L_{b1} - L_{b2} - \frac{L_{b3}}{2} \right) \right| = 11963.5 \quad kip \cdot in$

Moment at 3/4 Point in Unbraced Region: $M_C := \left| V_u \cdot \left(L_h - L_{b1} - L_{b2} - \frac{3 \cdot L_{b3}}{4} \right) \right| = 15422.8 \quad kip \cdot in$

Lateral Torsional Buckling Modification Factor: $C_b := \frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C} = -0.372$

Nominal Moment Capacity: $M_{n.LTB} := M_{n.LTB1}(L_{b3}, C_b) = 18507.5 \quad kip \cdot in$

$$\begin{cases} \text{"Ok"} & \text{if } (M_{n.LTB} \ge M_{PH.LB2}) \vee (L_{b3} \le L_p) \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Ok"}$$

Place lateral bracing at d from end of stiffener, at strong floor beam at centerline of specimen, and along strong floor beam past actuator

APPENDIX J   INSTRUMENTATION PLAN

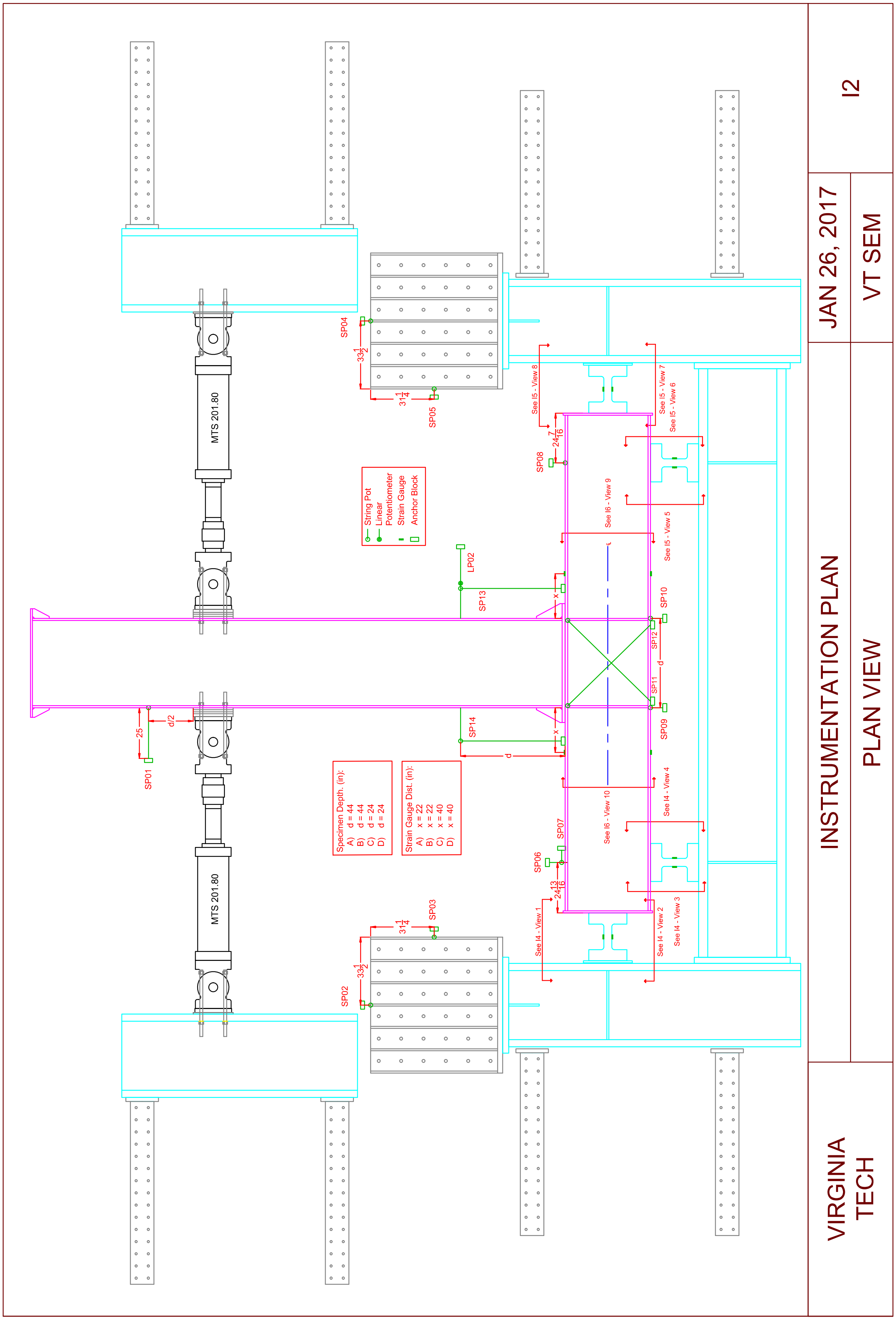
STRING POTS			
LABEL	SERIAL NUMBER	REQUIRED STROKE (in)	ACTUAL STROKE (in)
SP01	B2201247B	20.00	25.00
SP02	A36383	0.25	10.00
SP03	A36384	0.25	10.00
SP04	A36385	0.25	10.00
SP05	D2203930B	0.25	10.00
SP06	D2203932B	0.25	10.00
SP07	D2203934B	0.25	10.00
SP08	D2203939B	0.25	10.00
SP09	E1203076A	0.625	10.00
SP10	E1203080A	0.625	10.00
SP11	I2106341C	0.50	10.00
SP12	I2106344C	0.50	10.00
SP13	I2106339C	2.00	10.00
SP14	D2203933B	2.00	10.00
LINEAR POTENTIOMETERS			
LP02	HTL-14424	4.10	6.00

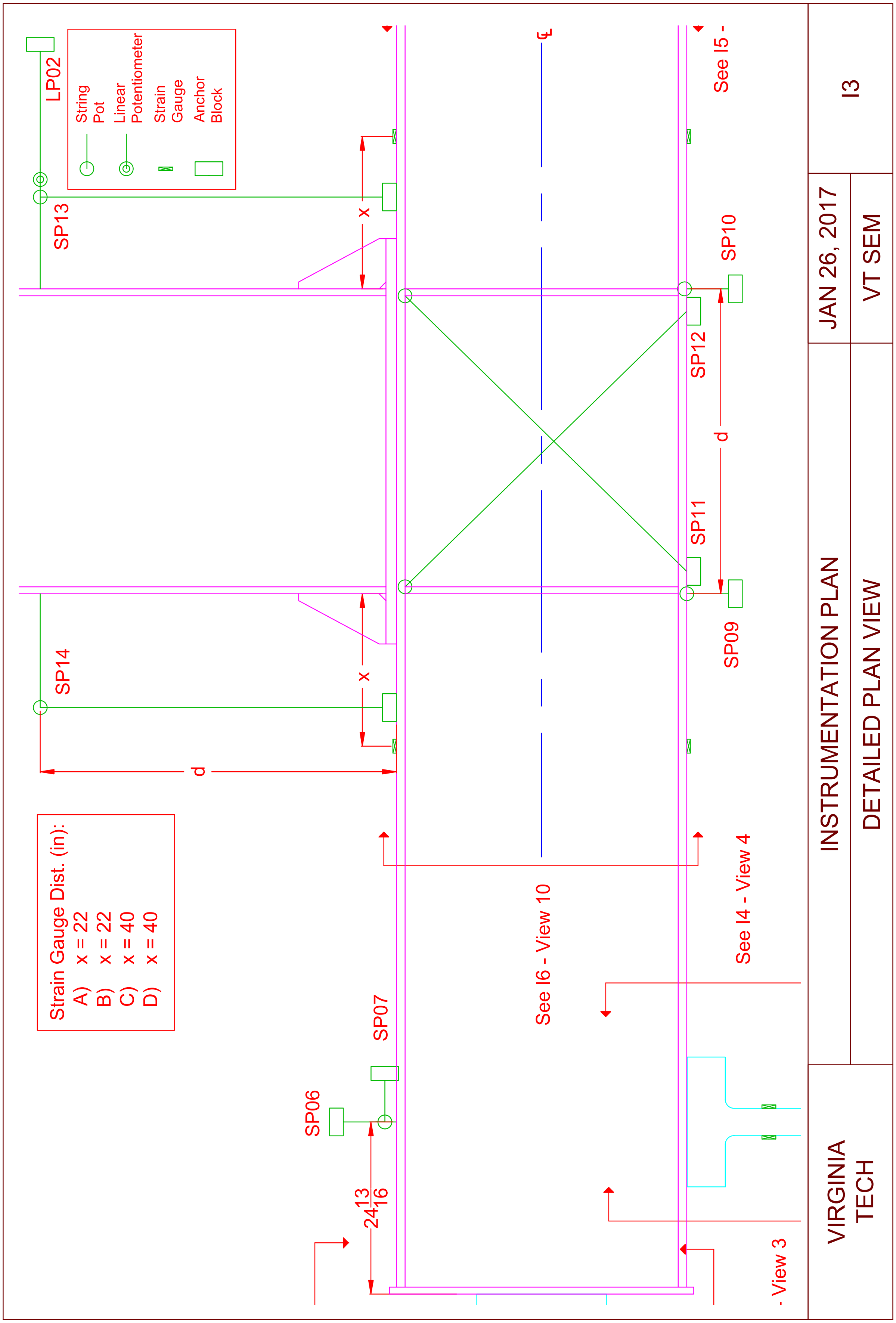
CALIPERS	
TOTAL	6

SECTION	STRAIN GAUGES		
	QTY.	SECTIONS	TOTAL
BEAM	0	5	0
COLUMN	8	2	16
STUB 1	4	1	4
STUB 2	4	1	4
STUB 3	4	1	4
STUB 4	4	1	4
TOTAL			32

SPECIMEN	BOLT STRAIN GAUGES		
	QTY.	SECTIONS	TOTAL
12ES-0.75-1.00-44	6	1	6
12ES-1.25-1.50-44	6	1	6
12ES-0.875-0.75-24	6	1	6
12ES-1.125-1.25-24	6	1	6
TOTAL			24

VIRGINIA TECH	INSTRUMENTATION PLAN	JAN 26, 2017	I1
	INSTRUMENTATION COUNT	VT SEM	

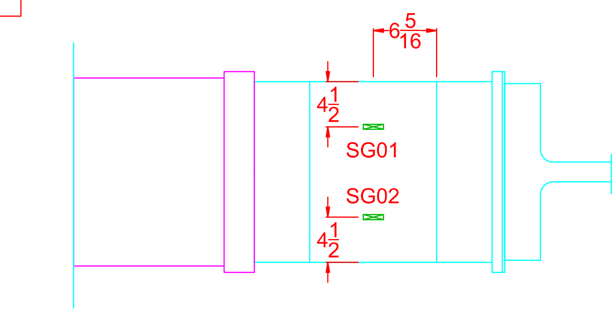

VIRGINIA TECH
INSTRUMENTATION PLAN
PLAN VIEW
JAN 26, 2017
VT SEM
I2
MTS 201.80
MTS 201.80
String Pot
Linear Potentiometer
Strain Gauge
Anchor Block
Specimen Depth, (in):
A) d = 44
B) d = 44
C) d = 24
D) d = 24
Strain Gauge Dist. (in):
A) x = 22
B) x = 22
C) x = 40
D) x = 40
SP01
SP02
SP03
SP04
SP05
SP06
SP07
SP08
SP09
SP10
SP11
SP12
SP13
SP14
LP02
See I5 - View 8
See I5 - View 7
See I5 - View 6
See I6 - View 9
See I5 - View 5
See I4 - View 4
See I6 - View 10
See I4 - View 1
See I4 - View 2
See I4 - View 3



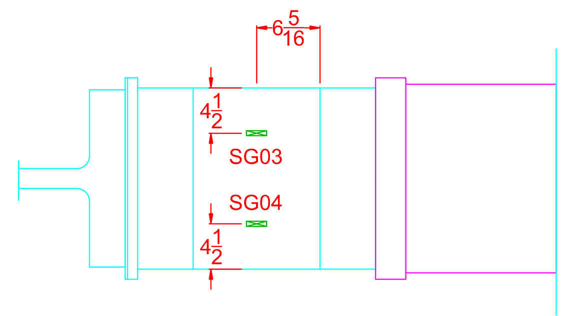
LP02
String Pot
Linear Potentiometer
Strain Gauge
Anchor Block
SP13
SP14
SP12
SP11
SP10
SP09
SP07
SP06
x
x
d
d
$24\frac{13}{16}$
Strain Gauge Dist. (in):
A) x = 22
B) x = 22
C) x = 40
D) x = 40
See I5 -
See I6 - View 10
See I4 - View 4
- View 3
℄
INSTRUMENTATION PLAN
DETAILED PLAN VIEW
JAN 26, 2017
VT SEM
I3
VIRGINIA TECH

Strain Gauge

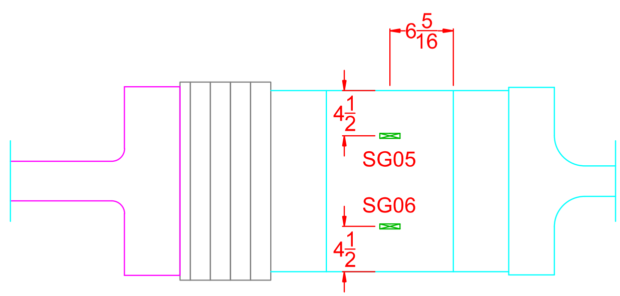
Note:
Strain gauges are placed at quarter points



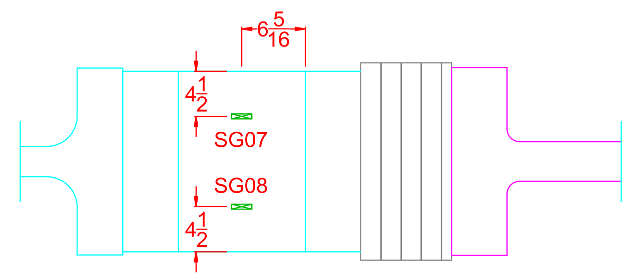
Side - View 1



Side - View 2



Side - View 3

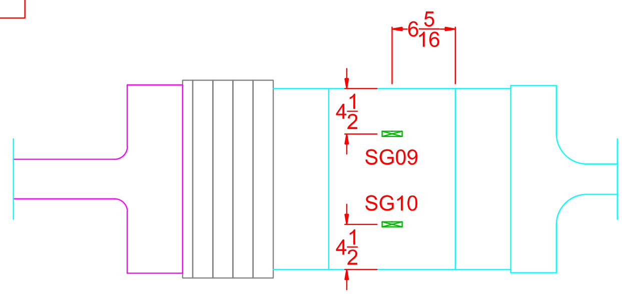


Side - View 4

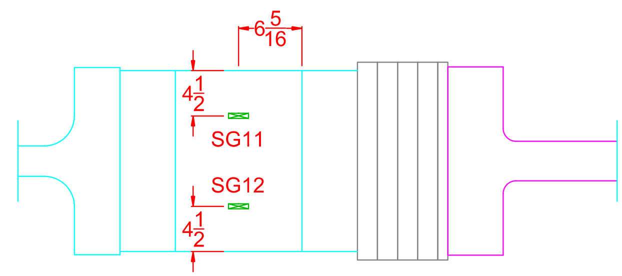
VIRGINIA TECH	INSTRUMENTATION PLAN	JAN 26, 2017	I4
	DETAILED ELEVATION VIEWS 1	VT SEM	

Strain Gauge

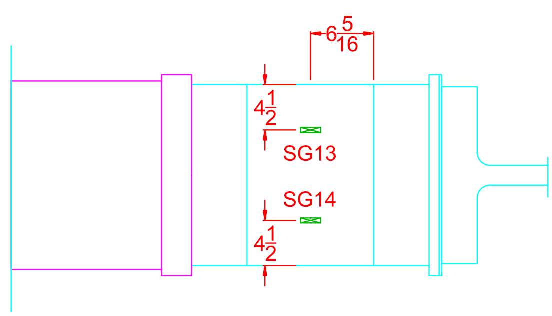
Note:
Strain gauges are placed at quarter points



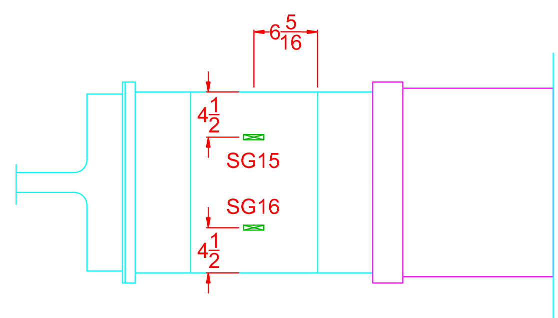
Side - View 5



Side - View 6

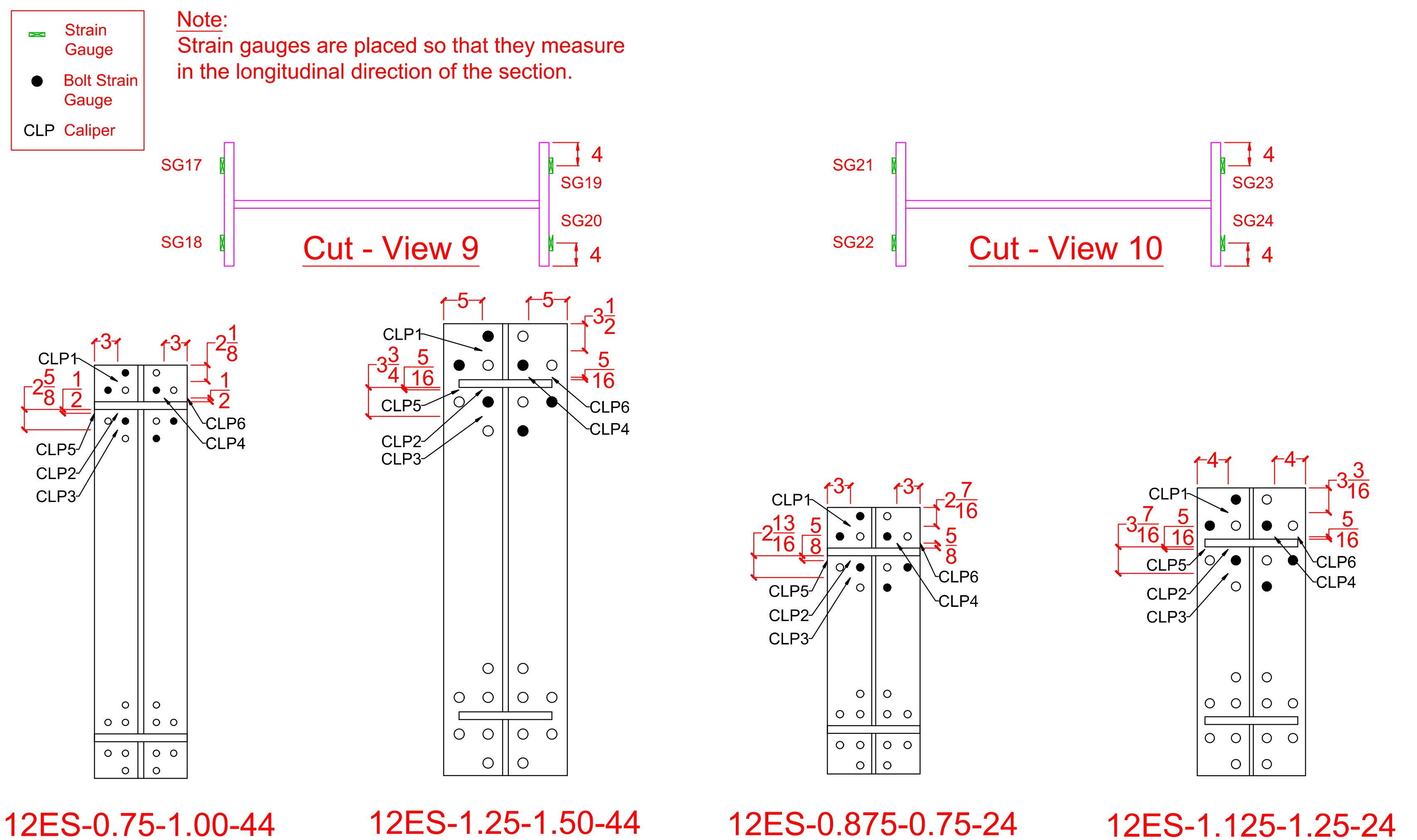


Side - View 7



Side - View 8

VIRGINIA TECH	INSTRUMENTATION PLAN	JAN 26, 2017	I5
	DETAILED ELEVATION VIEWS 2	VT SEM	



VIRGINIA TECH	INSTRUMENTATION PLAN	JAN 26, 2017	I6
	DETAILED ELEVATION VIEWS 3	VT SEM	

APPENDIX K PROCEDURE FOR THE CALIBRATION OF DATA ACQUISITION SYSTEM FOR STRAIN MEASUREMENTS

Calibration of Data Acquisition System for Strain Measurements

Software: National Instruments Measurement & Automation Explorer (NI MAX)

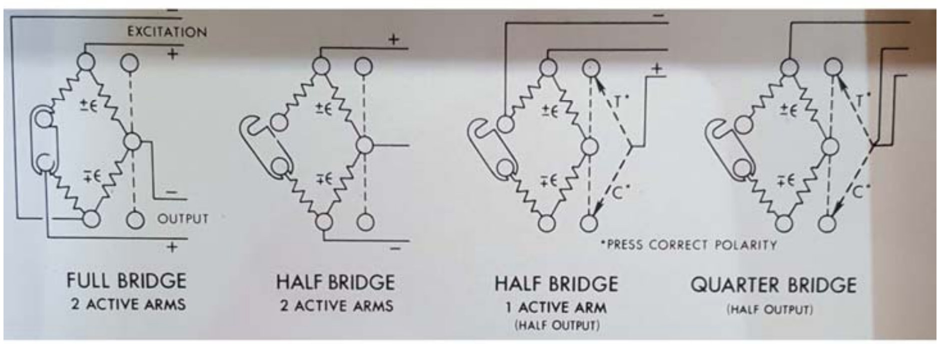
Hardware:
- National Instruments SCXI-1001 Chassis
- National Instruments SCXI-1521B Strain Gage Input Module
- National Instruments SCXI-1317 Front-Mounting Terminal Block
- Micro-Measurements 1550A Strain Indicator Calibrator

Governing Equation: $$V_o = V_{ex} \cdot \left(\frac{GF \cdot \varepsilon}{4}\right) \cdot \left(\frac{1}{1 + GF \cdot \frac{\varepsilon}{2}}\right)$$

V_o = Output Voltage
V_{ex} = Excitation Voltage
GF = Gage Factor
ε = Strain

Steps:

1) Connect the free end of the cable to the strain indicator calibrator based on the diagram for quarter bridge, as shown below



Data Acquisition System Calibration Wiring		
Strain Indicator Calibrator	**Cable Wire**	**NI SCXI-1317 Port**
P+	Red	EX+
QB (T)	Black	QTR
S-	White	AI

2) Set the Strain Indicator Calibrator to 0με

3) In NI MAX, set the following:
   - Gage Factor = 2
   - Gage Resistance = 120 Ω
   - Vex Value = 2 V
   - Strain Configuration = Quarter Bridge 1
   - Lead Resistance = 0 Ω

4) Take the initial reading in NI MAX and enter the negative of this value as the Initial Voltage to zero out the reading

5) Measure the resistance of the cable and enter the value under Lead Resistance

6) For Strain Indicator Calibrator Levels of 400με, 1600με, 3200με and 6400με, which correspond to readings of about 200με, 800με, 1600με and 3200με in NI MAX. Check that the readings match to these values respectively.
   - Based on the Gage Factor of 2 for the Strain Indicator Calibrator

7) Change the Gage Factor to the manufacturer reported value for the particular strain gauge corresponding to that channel.

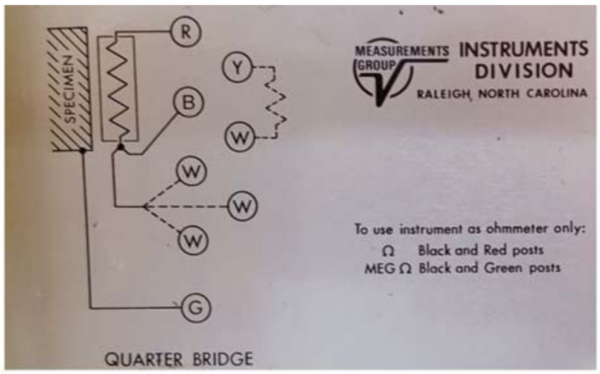
8) Repeat for every channel of strain

Strain Gage Installation Check

Hardware: Micro-Measurements 1300 Gage Installation Tester

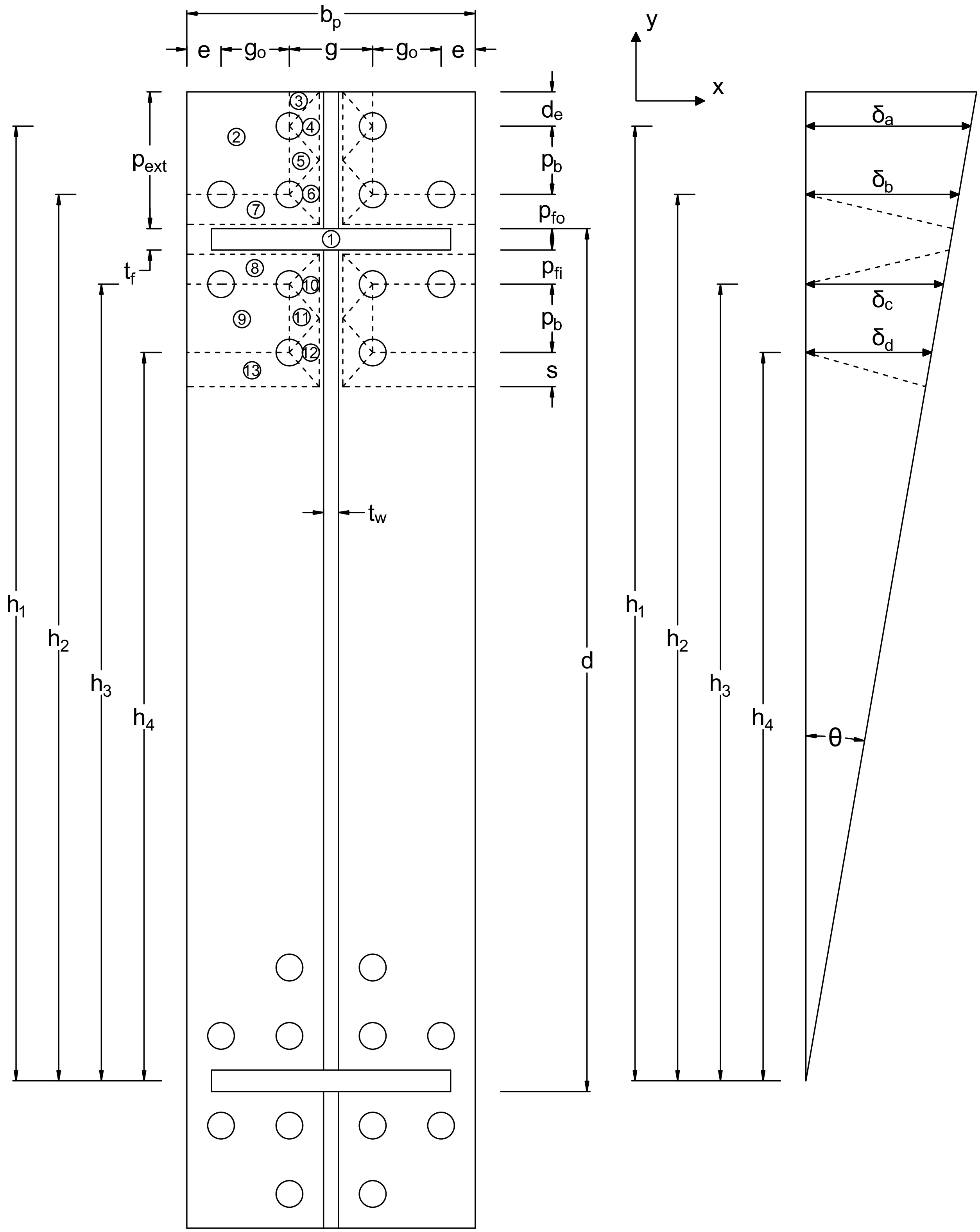
Steps:

1) Install strain gage on specimen
2) Attach strain gage lead wires to the Gage Installation Tester as shown in the diagram below



3) Check the battery level on the Gage Installation Tester by pressing "BATT"
4) Check the nominal gage resistance by pressing "Ω"
    - This should match the reported nominal resistance of the strain gage
    - The value will likely be slightly greater due to the resistance of the lead wire
5) Check the insulation resistance by pressing "MEG Ω"
    - Micro-Measurements reports that 10,000 MΩ should be considered the minimum
    - However, an insulation resistance greater than 500 MΩ will suffice based on speaking to a representative from Micro-Measurements in July 2016
    - If insulation resistance is too low, remove strain gage and replace it with a new one
6) Repeat all steps for every strain gage

APPENDIX L YIELD LINE DERIVATION FOR CASE 3 FOR 12ES



Yield Line Mechanism for Case 3 for 12ES

Panel Rotations of the Yield Line for Case 3 for 12ES

Panel	θ_{nx}	θ_{ny}
1	θ	0
2	0	0
3	0	$\frac{2[\delta_a + d_e\theta]}{g}$
4	θ	$\frac{2\delta_a}{g}$
5	0	$\frac{\delta_a + \delta_b}{g}$
6	θ	$\frac{2\delta_b}{g}$
7	$\frac{-\delta_b + p_{fo}\theta}{p_{fo}}$	0
8	$\frac{\delta_c + p_{fi}\theta}{p_{fi}}$	0
9	0	0
10	θ	$\frac{2\delta_c}{g}$
11	0	$\frac{\delta_c + \delta_d}{g}$
12	θ	$\frac{2\delta_d}{g}$
13	$\frac{-\delta_d + s\theta}{s}$	0

Energy Stored in Each Yield Line for Case 3 for 12ES

Yield Line	Energy Stored
$W_{i\text{-}2/3}$	$m_p d_e \left[\frac{2(\delta_a + d_e\theta)}{g}\right]$
$W_{i\text{-}3/4}$	$m_p \left(\frac{g}{2}\right)\theta + m_p d_e \left(\frac{2d_e\theta}{g}\right)$
$W_{i\text{-}1/4}$	$m_p \left(d_e + \frac{p_b}{2}\right)\left(\frac{2\delta_a}{g}\right)$
$W_{i\text{-}4/5}$	$m_p \left(\frac{g}{2}\right)\theta + m_p \left(\frac{p_b}{2}\right)\left(\frac{\delta_a - \delta_b}{g}\right)$
$W_{i\text{-}2/5}$	$m_p p_b \left(\frac{\delta_a + \delta_b}{g}\right)$
$W_{i\text{-}5/6}$	$m_p \left(\frac{g}{2}\right)\theta + m_p \left(\frac{p_b}{2}\right)\left(\frac{\delta_a - \delta_b}{g}\right)$
$W_{i\text{-}1/6}$	$m_p \left(p_{fo} + \frac{p_b}{2}\right)\left(\frac{2\delta_b}{g}\right)$
$W_{i\text{-}6/7}$	$m_p \left(\frac{g}{2}\right)\left(\frac{\delta_b}{p_{fo}}\right) + m_p p_{fo}\left(\frac{2\delta_b}{g}\right)$
$W_{i\text{-}2/7}$	$m_p \left(\frac{b_p - g}{2}\right)\left(\frac{\delta_b - p_{fo}\theta}{p_{fo}}\right)$
$W_{i\text{-}1/7}$	$m_p \left(\frac{b_p}{2}\right)\left(\frac{\delta_b}{p_{fo}}\right)$
$W_{i\text{-}1/8}$	$m_p \left(\frac{b_p}{2}\right)\left(\frac{\delta_c}{p_{fi}}\right)$
$W_{i\text{-}8/9}$	$m_p \left(\frac{b_p - g}{2}\right)\left(\frac{\delta_c + p_{fi}\theta}{p_{fi}}\right)$
$W_{i\text{-}8/10}$	$m_p \left(\frac{g}{2}\right)\left(\frac{\delta_c}{p_{fi}}\right) + m_p p_{fi}\left(\frac{2\delta_c}{g}\right)$
$W_{i\text{-}1/10}$	$m_p \left(p_{fi} + \frac{p_b}{2}\right)\left(\frac{2\delta_c}{g}\right)$
$W_{i\text{-}10/11}$	$m_p \left(\frac{g}{2}\right)\theta + m_p \left(\frac{p_b}{2}\right)\left(\frac{\delta_c - \delta_d}{g}\right)$

$W_{i\text{-}9/11}$	$m_p p_b \left(\frac{\delta_c + \delta_d}{g}\right)$
$W_{i\text{-}11/12}$	$m_p \left(\frac{g}{2}\right)\theta + m_p \left(\frac{p_b}{2}\right)\left(\frac{\delta_c - \delta_d}{g}\right)$
$W_{i\text{-}1/12}$	$m_p \left(s + \frac{p_b}{2}\right)\left(\frac{2\delta_d}{g}\right)$
$W_{i\text{-}12/13}$	$m_p \left(\frac{g}{2}\right)\left(\frac{\delta_d}{s}\right) + m_p s \left(\frac{2\delta_d}{g}\right)$
$W_{i\text{-}9/13}$	$m_p \left(\frac{b_p - g}{2}\right)\left(\frac{\delta_d - s\theta}{s}\right)$
$W_{i\text{-}1/13}$	$m_p \left(\frac{b_p}{2}\right)\left(\frac{\delta_d}{s}\right)$

$\Sigma W_i \quad =$

$$m_p \cdot \left[d_e \cdot \left[\frac{2\cdot(\delta_a + d_e\cdot\theta)}{g} \right] + \frac{g\cdot\theta}{2} + d_e\cdot\left(\frac{2\cdot d_e\cdot\theta}{g}\right) + \left(d_e + \frac{p_b}{2}\right)\cdot\left(\frac{2\cdot\delta_a}{g}\right) \ldots \right.$$
$$+ \frac{g\cdot\theta}{2} + \left(\frac{p_b}{2}\right)\cdot\left(\frac{\delta_a - \delta_b}{g}\right) + p_b\cdot\left(\frac{\delta_a + \delta_b}{g}\right) + \frac{g\cdot\theta}{2} + \left(\frac{p_b}{2}\right)\cdot\left(\frac{\delta_a - \delta_b}{g}\right) \ldots$$
$$+ \left(p_{fo} + \frac{p_b}{2}\right)\cdot\left(\frac{2\cdot\delta_b}{g}\right) + \left(\frac{g}{2}\right)\left(\frac{\delta_b}{p_{fo}}\right) + p_{fo}\cdot\left(\frac{2\cdot\delta_b}{g}\right) + \left(\frac{b_p - g}{2}\right)\cdot\left(\frac{\delta_b - p_{fo}\cdot\theta}{p_{fo}}\right) + \left(\frac{b_p}{2}\right)\cdot\left(\frac{\delta_b}{p_{fo}}\right) \ldots$$
$$+ \left(\frac{b_p}{2}\right)\cdot\left(\frac{\delta_c}{p_{fi}}\right) + \left(\frac{b_p - g}{2}\right)\cdot\left(\frac{\delta_c + p_{fi}\cdot\theta}{p_{fi}}\right) + \left(\frac{g}{2}\right)\cdot\left(\frac{\delta_c}{p_{fi}}\right) + p_{fi}\cdot\left(\frac{2\cdot\delta_c}{g}\right) + \left(p_{fi} + \frac{p_b}{2}\right)\cdot\left(\frac{2\cdot\delta_c}{g}\right) + \frac{g\cdot\theta}{2} \ldots$$
$$+ \left(\frac{p_b}{2}\right)\cdot\left(\frac{\delta_c - \delta_d}{g}\right) + p_b\cdot\left(\frac{\delta_c + \delta_d}{g}\right) + \frac{g\cdot\theta}{2} + \left(\frac{p_b}{2}\right)\cdot\left(\frac{\delta_c - \delta_d}{g}\right) + \left(s + \frac{p_b}{2}\right)\cdot\left(\frac{2\cdot\delta_d}{g}\right) + \left(\frac{g}{2}\right)\left(\frac{\delta_d}{s}\right) \ldots$$
$$\left. + s\cdot\left(\frac{2\cdot\delta_d}{g}\right) + \left(\frac{b_p - g}{2}\right)\left(\frac{\delta_d - s\cdot\theta}{s}\right) + \left(\frac{b_p}{2}\right)\left(\frac{\delta_d}{s}\right) \right]$$

Simplify:

$$\Sigma W_i \quad = \quad m_p\cdot\left(3\cdot\theta\cdot g - \frac{\theta\cdot b_p}{2} + \frac{4\cdot\theta\cdot d_e^2}{g} + \frac{4\cdot d_e\cdot\delta_a}{g} + \frac{b_p\cdot\delta_c}{p_{fi}} + \frac{b_p\cdot\delta_b}{p_{fo}} + \frac{b_p\cdot\delta_d}{s} + \frac{3\cdot p_b\cdot\delta_a}{g} + \frac{p_b\cdot\delta_b}{g} \ldots \right.$$
$$\left. + \frac{3\cdot p_b\cdot\delta_c}{g} + \frac{p_b\cdot\delta_d}{g} + \frac{4\cdot p_{fi}\cdot\delta_c}{g} + \frac{4\cdot p_{fo}\cdot\delta_b}{g} + \frac{4\cdot s\cdot\delta_d}{g}\right)$$

Solve for s that produces minimum stored energy:

$$\frac{d}{ds}\left[m_p\cdot\left(3\cdot\theta\cdot g - \frac{\theta\cdot b_p}{2} + \frac{4\cdot\theta\cdot d_e^2}{g} + \frac{4\cdot d_e\cdot\delta_a}{g} + \frac{b_p\cdot\delta_c}{p_{fi}} + \frac{b_p\cdot\delta_b}{p_{fo}} + \frac{b_p\cdot\delta_d}{s} + \frac{3\cdot p_b\cdot\delta_a}{g} + \frac{p_b\cdot\delta_b}{g} \ldots \right.\right.$$
$$\left.\left. + \frac{3\cdot p_b\cdot\delta_c}{g} + \frac{p_b\cdot\delta_d}{g} + \frac{4\cdot p_{fi}\cdot\delta_c}{g} + \frac{4\cdot p_{fo}\cdot\delta_b}{g} + \frac{4\cdot s\cdot\delta_d}{g}\right)\right]$$

$$= \quad \frac{4\cdot m_p\cdot\delta_d}{g} - \frac{b_p\cdot m_p\cdot\delta_d}{s^2} = 0$$

Solve for s:

$$s = \frac{1}{2}\cdot\sqrt{b_p\cdot g}$$

Simplify: Substitute the following into ΣW_i:

$\delta_a = h_1\cdot\theta \qquad \delta_b = h_2\cdot\theta \qquad \delta_c = h_3\cdot\theta \qquad \delta_d = h_4\cdot\theta$

Set $W_e = W_i$

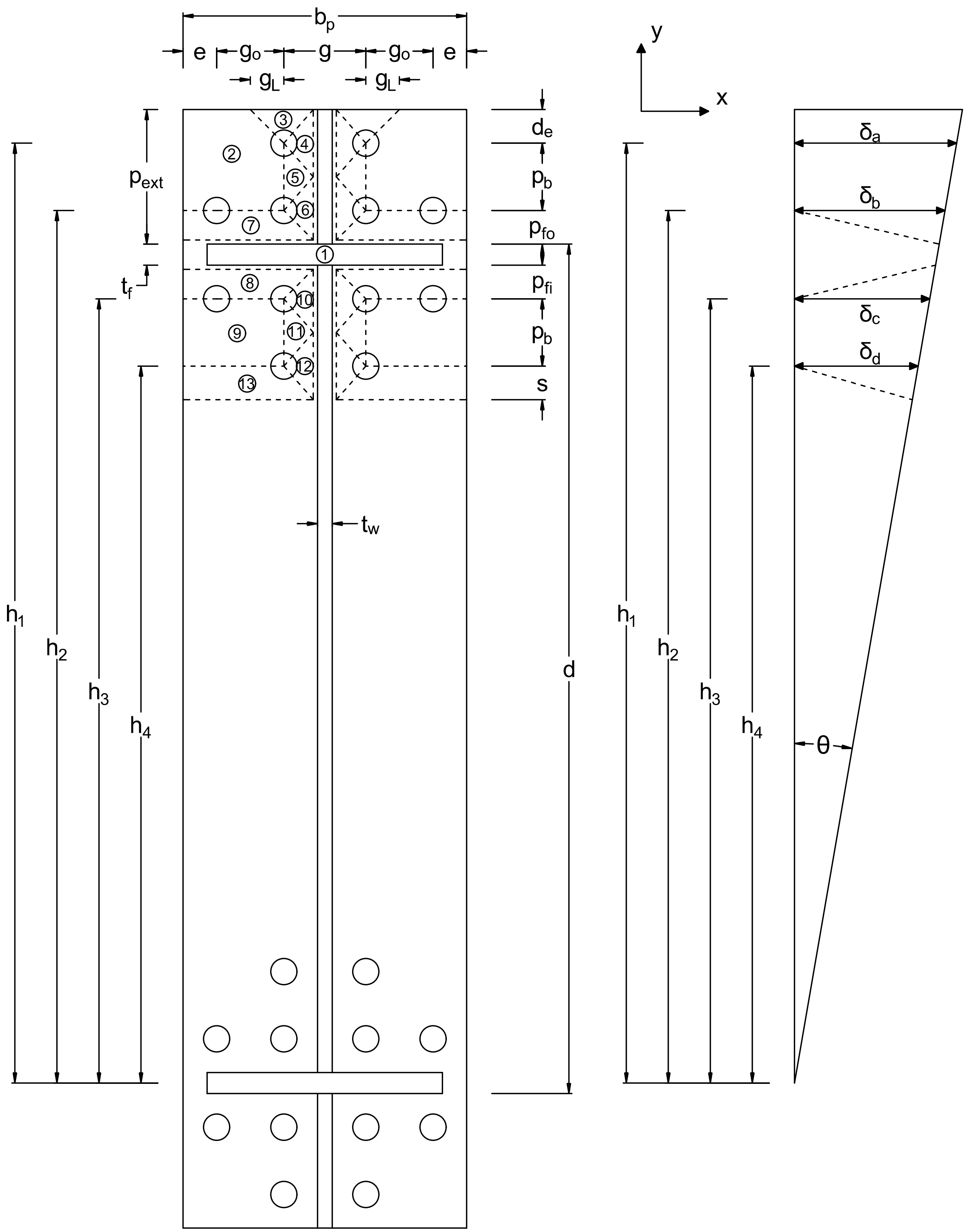
Pull θ out of ΣW_i $\qquad M_n\cdot\theta = 2\cdot F_y\cdot\frac{t_p^2}{4}\cdot\theta\cdot\Sigma W_i$

Nominal Strength: $M_{pl} = F_{py} \cdot t_p^{\ 2} \cdot Y$

Yield Line Parameter:

$$Y = \frac{b_p}{2} \cdot \left(\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2} \right) + \frac{1}{2 \cdot g} \cdot \begin{bmatrix} h_1 \cdot (4 \cdot d_e + 3 \cdot p_b) + h_2 \cdot (p_b + 4 \cdot p_{fo}) \dots \\ + h_3 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_4 \cdot (p_b + 4 \cdot s) + 4 \cdot d_e^{\ 2} \end{bmatrix} + \frac{3}{2} \cdot g$$

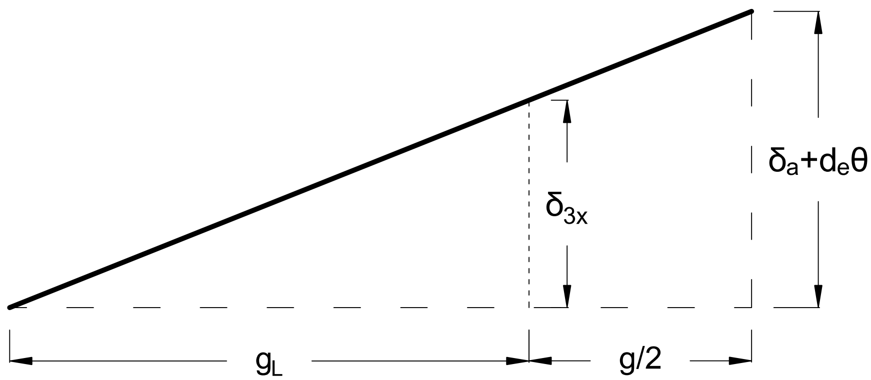
APPENDIX M YIELD LINE DERIVATION FOR CASE 5 FOR 12ES



Yield Line Mechanism for Case 5 for 12ES

Panel Rotations of the Yield Line for Case 5 for 12ES

Panel	θ_{nx}	θ_{ny}
1	θ	0
2	0	0
3	$\frac{(\delta_a + d_e\theta)}{\left(g_L + \frac{g}{2}\right)}\frac{(g_L)}{(d_e)}$	$\frac{[\delta_a + d_e\theta]}{\left(g_L + \frac{g}{2}\right)}$
4	θ	$\frac{2\delta_a}{g}$
5	0	$\frac{\delta_a + \delta_b}{g}$
6	θ	$\frac{2\delta_b}{g}$
7	$\frac{-\delta_b + p_{fo}\theta}{p_{fo}}$	0
8	$\frac{\delta_c + p_{fi}\theta}{p_{fi}}$	0
9	0	0
10	θ	$\frac{2\delta_c}{g}$
11	0	$\frac{\delta_c + \delta_d}{g}$
12	θ	$\frac{2\delta_d}{g}$
13	$\frac{-\delta_d + s\theta}{s}$	0



Elevation View of Panel 3 to Determine θ_{3x} and θ_{3y}

Using similar triangles:

$$\frac{\delta_{3x}}{g_L} = \frac{(\delta_a + d_e\theta)}{\left(g_L + \frac{g}{2}\right)}$$

$$\theta_{3x} = \frac{\delta_{3x}}{d_e}$$

Energy Stored in Each Yield Line for Case 5 for 12ES

Yield Line	Energy Stored
$W_{i\text{-}2/3}$	$m_p g_L \frac{(\delta_a + d_e\theta)}{\left(g_L + \frac{g}{2}\right)} \frac{(g_L)}{(d_e)} + m_p d_e \left[\frac{[\delta_a + d_e\theta]}{\left(g_L + \frac{g}{2}\right)}\right]$
$W_{i\text{-}3/4}$	$m_p \left(\frac{g}{2}\right) \left[\frac{(\delta_a + d_e\theta)}{\left(g_L + \frac{g}{2}\right)} \frac{(g_L)}{(d_e)} - \theta\right] + m_p d_e \left(\frac{[\delta_a + d_e\theta]}{\left(g_L + \frac{g}{2}\right)} - \frac{2\delta_a}{g}\right)$
$W_{i\text{-}1/4}$	$m_p \left(d_e + \frac{p_b}{2}\right)\left(\frac{2\delta_a}{g}\right)$
$W_{i\text{-}4/5}$	$m_p \left(\frac{g}{2}\right)\theta + m_p \left(\frac{p_b}{2}\right)\left(\frac{\delta_a - \delta_b}{g}\right)$
$W_{i\text{-}2/5}$	$m_p p_b \left(\frac{\delta_a + \delta_b}{g}\right)$
$W_{i\text{-}5/6}$	$m_p \left(\frac{g}{2}\right)\theta + m_p \left(\frac{p_b}{2}\right)\left(\frac{\delta_a - \delta_b}{g}\right)$

$W_{i-1/6}$	$m_p \left(p_{fo} + \frac{p_b}{2}\right)\left(\frac{2\delta_b}{g}\right)$
$W_{i-6/7}$	$m_p \left(\frac{g}{2}\right)\left(\frac{\delta_b}{p_{fo}}\right) + m_p p_{fo}\left(\frac{2\delta_b}{g}\right)$
$W_{i-2/7}$	$m_p \left(\frac{b_p - g}{2}\right)\left(\frac{\delta_b - p_{fo}\theta}{p_{fo}}\right)$
$W_{i-1/7}$	$m_p \left(\frac{b_p}{2}\right)\left(\frac{\delta_b}{p_{fo}}\right)$
$W_{i-1/8}$	$m_p \left(\frac{b_p}{2}\right)\left(\frac{\delta_c}{p_{fi}}\right)$
$W_{i-8/9}$	$m_p \left(\frac{b_p - g}{2}\right)\left(\frac{\delta_c + p_{fi}\theta}{p_{fi}}\right)$
$W_{i-8/10}$	$m_p \left(\frac{g}{2}\right)\left(\frac{\delta_c}{p_{fi}}\right) + m_p p_{fi}\left(\frac{2\delta_c}{g}\right)$
$W_{i-1/10}$	$m_p \left(p_{fi} + \frac{p_b}{2}\right)\left(\frac{2\delta_c}{g}\right)$
$W_{i-10/11}$	$m_p \left(\frac{g}{2}\right)\theta + m_p \left(\frac{p_b}{2}\right)\left(\frac{\delta_c - \delta_d}{g}\right)$
$W_{i-9/11}$	$m_p p_b \left(\frac{\delta_c + \delta_d}{g}\right)$
$W_{i-11/12}$	$m_p \left(\frac{g}{2}\right)\theta + m_p \left(\frac{p_b}{2}\right)\left(\frac{\delta_c - \delta_d}{g}\right)$
$W_{i-1/12}$	$m_p \left(s + \frac{p_b}{2}\right)\left(\frac{2\delta_d}{g}\right)$
$W_{i-12/13}$	$m_p \left(\frac{g}{2}\right)\left(\frac{\delta_d}{s}\right) + m_p s\left(\frac{2\delta_d}{g}\right)$
$W_{i-9/13}$	$m_p \left(\frac{b_p - g}{2}\right)\left(\frac{\delta_d - s\theta}{s}\right)$
$W_{i-1/13}$	$m_p \left(\frac{b_p}{2}\right)\left(\frac{\delta_d}{s}\right)$

$\Sigma W_i \quad =$

$$
\begin{aligned}
m_p \cdot \Bigg[& g_L \cdot \frac{\left(\delta_a + d_e \cdot \theta\right)}{\left(g_L + \frac{g}{2}\right)} \cdot \left(\frac{g_L}{d_e}\right) + d_e \cdot \frac{\left(\delta_a + d_e \cdot \theta\right)}{\left(g_L + \frac{g}{2}\right)} + \left(\frac{g}{2}\right) \cdot \left[\frac{\delta_a \cdot g_L}{d_e \cdot \left(g_L + \frac{g}{2}\right)} + \frac{g_L \cdot \theta}{\left(g_L + \frac{g}{2}\right)} - \theta\right] + d_e \cdot \left[\frac{\left(\delta_a + d_e \cdot \theta\right)}{\left(g_L + \frac{g}{2}\right)} - \frac{2 \cdot \delta_a}{g}\right] \ldots \\
& + \left(d_e + \frac{p_b}{2}\right) \cdot \frac{2 \cdot \delta_a}{g} + \frac{g \cdot \theta}{2} + \left(\frac{p_b}{2}\right) \cdot \left(\frac{\delta_a - \delta_b}{g}\right) + p_b \cdot \left(\frac{\delta_a + \delta_b}{g}\right) + \frac{g \cdot \theta}{2} + \left(\frac{p_b}{2}\right) \cdot \left(\frac{\delta_a - \delta_b}{g}\right) \ldots \\
& + \left(p_{fo} + \frac{p_b}{2}\right) \cdot \left(\frac{2 \cdot \delta_b}{g}\right) + \left(\frac{g}{2}\right)\left(\frac{\delta_b}{p_{fo}}\right) + p_{fo} \cdot \left(\frac{2 \cdot \delta_b}{g}\right) + \left(\frac{b_p - g}{2}\right) \cdot \left(\frac{\delta_b - p_{fo} \cdot \theta}{p_{fo}}\right) + \left(\frac{b_p}{2}\right) \cdot \left(\frac{\delta_b}{p_{fo}}\right) \ldots \\
& + \left(\frac{b_p}{2}\right) \cdot \left(\frac{\delta_c}{p_{fi}}\right) + \left(\frac{b_p - g}{2}\right) \cdot \left(\frac{\delta_c + p_{fi} \cdot \theta}{p_{fi}}\right) + \left(\frac{g}{2}\right) \cdot \left(\frac{\delta_c}{p_{fi}}\right) + p_{fi} \cdot \left(\frac{2 \cdot \delta_c}{g}\right) + \left(p_{fi} + \frac{p_b}{2}\right) \cdot \left(\frac{2 \cdot \delta_c}{g}\right) + \frac{g \cdot \theta}{2} \ldots \\
& + \left(\frac{p_b}{2}\right) \cdot \left(\frac{\delta_c - \delta_d}{g}\right) + p_b \cdot \left(\frac{\delta_c + \delta_d}{g}\right) + \frac{g \cdot \theta}{2} + \left(\frac{p_b}{2}\right) \cdot \left(\frac{\delta_c - \delta_d}{g}\right) + \left(s + \frac{p_b}{2}\right) \cdot \left(\frac{2 \cdot \delta_d}{g}\right) + \left(\frac{g}{2}\right)\left(\frac{\delta_d}{s}\right) \ldots \\
& + s \cdot \left(\frac{2 \cdot \delta_d}{g}\right) + \left(\frac{b_p - g}{2}\right)\left(\frac{\delta_d - s \cdot \theta}{s}\right) + \left(\frac{b_p}{2}\right)\left(\frac{\delta_d}{s}\right) \Bigg]
\end{aligned}
$$

Solve for s that produces minimum stored energy:

$$
\frac{d}{ds}\left[m_p \cdot \left(\Sigma W_i\right)\right] = 0 \qquad \text{-->} \qquad \frac{4 \cdot \delta_d}{g} - \frac{\left(\frac{b_p}{2} - \frac{g}{2}\right) \cdot \left(\delta_d - \theta \cdot s\right)}{s^2} - \frac{\theta \cdot \left(\frac{b_p}{2} - \frac{g}{2}\right)}{s} - \frac{b_p \cdot \delta_d}{2 \cdot s^2} - \frac{g \cdot \delta_d}{2 \cdot s^2} = 0
$$

Solve for s:

$$
s = \frac{1}{2} \cdot \sqrt{b_p \cdot g}
$$

Solve for g_L that produces minimum stored energy:

$$
\frac{d}{dg_L}\left[m_p \cdot \left(\Sigma W_i\right)\right] = 0 \qquad \text{-->} \qquad g_L = \sqrt{2} \cdot d_e - \frac{g}{2}
$$

Simplify: Take absolute value of terms with g_L

Substitute the following into ΣW_i:

$$
g_L = \sqrt{2} \cdot d_e - \frac{g}{2} \qquad \delta_a = h_1 \cdot \theta \qquad \delta_b = h_2 \cdot \theta \qquad \delta_c = h_3 \cdot \theta \qquad \delta_d = h_4 \cdot \theta
$$

$\Sigma W_i \quad =$

$$
\begin{aligned}
& \left| \frac{\left[\left(h_1 \cdot \theta\right) + d_e \cdot \theta\right]}{\sqrt{2} \cdot d_e} \cdot \left[\frac{\left(\sqrt{2} \cdot d_e - \frac{g}{2}\right)^2}{d_e}\right] \right| + \left| d_e \cdot \frac{\left[\left(h_1 \cdot \theta\right) + d_e \cdot \theta\right]}{\left(\sqrt{2} \cdot d_e\right)} \right| + \left| \left(\frac{g}{2}\right) \cdot \left[\frac{\left(h_1 \cdot \theta\right) \cdot \left(\sqrt{2} \cdot d_e - \frac{g}{2}\right)}{d_e \cdot \left(\sqrt{2} \cdot d_e\right)} + \frac{\left(\sqrt{2} \cdot d_e - \frac{g}{2}\right) \cdot \theta}{\left(\sqrt{2} \cdot d_e\right)} - \theta\right] \right| \\
& + \left| d_e \cdot \left[\frac{\left[\left(h_1 \cdot \theta\right) + d_e \cdot \theta\right]}{\left(\sqrt{2} \cdot d_e\right)} - \frac{2 \cdot \left(h_1 \cdot \theta\right)}{g}\right] \right| + \frac{5 \cdot \theta \cdot g}{2} - \frac{\theta \cdot b_p}{2} + \frac{2 \cdot d_e \cdot \left(h_1 \cdot \theta\right)}{g} + \frac{b_p \cdot \left(h_3 \cdot \theta\right)}{p_{fi}} + \frac{b_p \cdot \left(h_2 \cdot \theta\right)}{p_{fo}} + \frac{b_p \cdot \left(h_4 \cdot \theta\right)}{s} \ldots \\
& + \frac{3 \cdot p_b \cdot \left(h_1 \cdot \theta\right)}{g} + \frac{p_b \cdot \left(h_2 \cdot \theta\right)}{g} + \frac{3 \cdot p_b \cdot \left(h_3 \cdot \theta\right)}{g} + \frac{p_b \cdot \left(h_4 \cdot \theta\right)}{g} + \frac{4 \cdot p_{fi} \cdot \left(h_3 \cdot \theta\right)}{g} + \frac{4 \cdot p_{fo} \cdot \left(h_2 \cdot \theta\right)}{g} + \frac{4 \cdot s \cdot \left(h_4 \cdot \theta\right)}{g}
\end{aligned}
$$

Set W_e = W_i

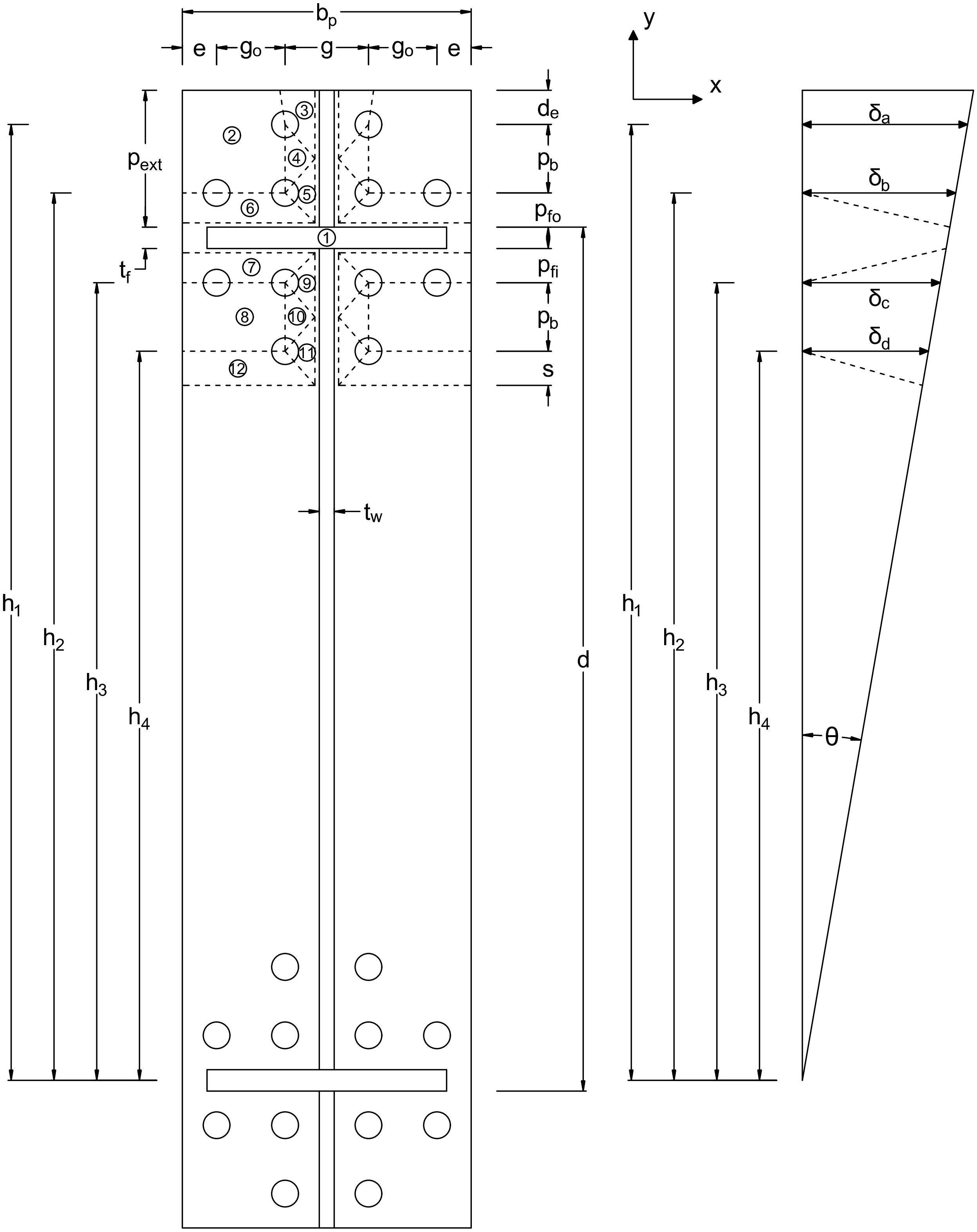
Pull θ out of ΣW_i $$M_n \cdot \theta = 2 \cdot F_y \cdot \frac{t_p^2}{4} \cdot \theta \cdot \Sigma W_i$$

Nominal Strength: $$M_{pl} = F_{py} \cdot t_p^2 \cdot Y$$

Yield Line Parameter:

$$Y = \frac{1}{2}\left[\left|\frac{(h_1 + d_e)}{\sqrt{2} \cdot d_e} \cdot \left[\frac{\left(\sqrt{2} \cdot d_e - \frac{g}{2}\right)^2}{d_e}\right]\right| + \left|d_e \cdot \frac{(h_1 + d_e)}{(\sqrt{2} \cdot d_e)}\right| + \left|\left(\frac{g}{2}\right) \cdot \left[\frac{h_1 \cdot \left(\sqrt{2} \cdot d_e - \frac{g}{2}\right)}{d_e \cdot (\sqrt{2} \cdot d_e)} + \frac{\left(\sqrt{2} \cdot d_e - \frac{g}{2}\right)}{(\sqrt{2} \cdot d_e)} - 1\right]\right| \ldots \\ + \left|d_e \cdot \left[\frac{(h_1 + d_e)}{(\sqrt{2} \cdot d_e)} - \frac{2 \cdot h_1}{g}\right]\right| + \frac{5 \cdot g}{2} - \frac{b_p}{2} + \frac{2 \cdot d_e \cdot h_1}{g} + \frac{b_p \cdot h_3}{p_{fi}} + \frac{b_p \cdot h_2}{p_{fo}} + \frac{b_p \cdot h_4}{s} + \frac{3 \cdot p_b \cdot h_1}{g} + \frac{p_b \cdot h_2}{g} \ldots \\ + \frac{3 \cdot p_b \cdot h_3}{g} + \frac{p_b \cdot h_4}{g} + \frac{4 \cdot p_{fi} \cdot h_3}{g} + \frac{4 \cdot p_{fo} \cdot h_2}{g} + \frac{4 \cdot s \cdot h_4}{g}\right]$$

APPENDIX N YIELD LINE DERIVATION FOR CONTROLLING CASE FOR 12ES



Controlling Case Yield Line Mechanism for 12ES

Panel Rotations of the Yield Line for 12ES

Panel	θ_{nx}	θ_{ny}
1	θ	0
2	0	0
3	θ	$\frac{2\delta_a}{g}$
4	0	$\frac{\delta_a + \delta_b}{g}$
5	θ	$\frac{2\delta_b}{g}$
6	$\frac{-\delta_b + p_{fo}\theta}{p_{fo}}$	0
7	$\frac{\delta_c + p_{fi}\theta}{p_{fi}}$	0
8	0	0
9	θ	$\frac{2\delta_c}{g}$
10	0	$\frac{\delta_c + \delta_d}{g}$
11	θ	$\frac{2\delta_d}{g}$
12	$\frac{-\delta_d + s\theta}{s}$	0

Energy Stored in Each Yield Line for 12ES

Yield Line	Energy Stored
$W_{i\text{-}2/3}$	$m_p(d_e)\left(\frac{2\delta_a}{g}\right)$
$W_{i\text{-}1/3}$	$m_p\left(d_e+\frac{p_b}{2}\right)\left(\frac{2\delta_a}{g}\right)$
$W_{i\text{-}3/4}$	$m_p\left(\frac{g}{2}\right)\theta+m_p\left(\frac{p_b}{2}\right)\left(\frac{\delta_a-\delta_b}{g}\right)$
$W_{i\text{-}2/4}$	$m_p(p_b)\left(\frac{\delta_a+\delta_b}{g}\right)$
$W_{i\text{-}4/5}$	$m_p\left(\frac{g}{2}\right)\theta+m_p\left(\frac{p_b}{2}\right)\left(\frac{\delta_a-\delta_b}{g}\right)$
$W_{i\text{-}1/5}$	$m_p\left(p_{fo}+\frac{p_b}{2}\right)\left(\frac{2\delta_b}{g}\right)$
$W_{i\text{-}5/6}$	$m_p\left(\frac{g}{2}\right)\left(\frac{\delta_b}{p_{fo}}\right)+m_p(p_{fo})\left(\frac{2\delta_b}{g}\right)$
$W_{i\text{-}2/6}$	$m_p\left(\frac{b_p-g}{2}\right)\left(\frac{\delta_b-p_{fo}\theta}{p_{fo}}\right)$
$W_{i\text{-}1/6}$	$m_p\left(\frac{b_p}{2}\right)\left(\frac{\delta_b}{p_{fo}}\right)$
$W_{i\text{-}1/7}$	$m_p\left(\frac{b_p}{2}\right)\left(\frac{\delta_c}{p_{fi}}\right)$
$W_{i\text{-}7/8}$	$m_p\left(\frac{b_p-g}{2}\right)\left(\frac{\delta_c+p_{fi}\theta}{p_{fi}}\right)$
$W_{i\text{-}7/9}$	$m_p\left(\frac{g}{2}\right)\left(\frac{\delta_c}{p_{fi}}\right)+m_p(p_{fi})\left(\frac{2\delta_c}{g}\right)$
$W_{i\text{-}1/9}$	$m_p\left(p_{fi}+\frac{p_b}{2}\right)\left(\frac{2\delta_c}{g}\right)$
$W_{i\text{-}9/10}$	$m_p\left(\frac{g}{2}\right)\theta+m_p\left(\frac{p_b}{2}\right)\left(\frac{\delta_c-\delta_d}{g}\right)$
$W_{i\text{-}8/10}$	$m_p(p_b)\left(\frac{\delta_c+\delta_d}{g}\right)$

$W_{i\text{-}10/11}$	$m_p\left(\frac{g}{2}\right)\theta + m_p\left(\frac{p_b}{2}\right)\left(\frac{\delta_c - \delta_d}{g}\right)$
$W_{i\text{-}1/11}$	$m_p\left(s + \frac{p_b}{2}\right)\left(\frac{2\delta_d}{g}\right)$
$W_{i\text{-}11/12}$	$m_p\left(\frac{g}{2}\right)\left(\frac{\delta_d}{s}\right) + m_p(s)\left(\frac{2\delta_d}{g}\right)$
$W_{i\text{-}8/12}$	$m_p\left(\frac{b_p - g}{2}\right)\left(\frac{\delta_d - s\theta}{s}\right)$
$W_{i\text{-}1/12}$	$m_p\left(\frac{b_p}{2}\right)\left(\frac{\delta_d}{s}\right)$

$\Sigma W_i \quad =$

$$m_p \cdot \left[d_e \cdot \left(\frac{2\cdot\delta_a}{g}\right) + \left(d_e + \frac{p_b}{2}\right)\cdot\left(\frac{2\cdot\delta_a}{g}\right) + \left(\frac{g\cdot\theta}{2}\right) + \left(\frac{p_b}{2}\right)\cdot\left(\frac{\delta_a - \delta_b}{g}\right) \ldots \right.$$
$$+ p_b\cdot\left(\frac{\delta_a + \delta_b}{g}\right) + \left(\frac{g\cdot\theta}{2}\right) + \left(\frac{p_b}{2}\right)\cdot\left(\frac{\delta_a - \delta_b}{g}\right) + \left(p_{fo} + \frac{p_b}{2}\right)\cdot\left(\frac{2\cdot\delta_b}{g}\right) + \left(\frac{g}{2}\right)\left(\frac{\delta_b}{p_{fo}}\right) \ldots$$
$$+ p_{fo}\cdot\left(\frac{2\cdot\delta_b}{g}\right) + \left(\frac{b_p - g}{2}\right)\cdot\left(\frac{\delta_b - p_{fo}\cdot\theta}{p_{fo}}\right) + \left(\frac{b_p}{2}\right)\cdot\left(\frac{\delta_b}{p_{fo}}\right) + \left(\frac{b_p}{2}\right)\cdot\left(\frac{\delta_c}{p_{fi}}\right) + \left(\frac{b_p - g}{2}\right)\cdot\left(\frac{\delta_c + p_{fi}\cdot\theta}{p_{fi}}\right) \ldots$$
$$+ \left(\frac{g}{2}\right)\cdot\left(\frac{\delta_c}{p_{fi}}\right) + p_{fi}\cdot\left(\frac{2\cdot\delta_c}{g}\right) + \left(p_{fi} + \frac{p_b}{2}\right)\cdot\left(\frac{2\cdot\delta_c}{g}\right) + \left(\frac{g\cdot\theta}{2}\right) + \left(\frac{p_b}{2}\right)\cdot\left(\frac{\delta_c - \delta_d}{g}\right) + p_b\cdot\left(\frac{\delta_c + \delta_d}{g}\right) + \left(\frac{g\cdot\theta}{2}\right) \ldots$$
$$\left. + \left(\frac{p_b}{2}\right)\cdot\left(\frac{\delta_c - \delta_d}{g}\right) + \left(s + \frac{p_b}{2}\right)\cdot\left(\frac{2\cdot\delta_d}{g}\right) + \left(\frac{g}{2}\right)\left(\frac{\delta_d}{s}\right) + s\cdot\left(\frac{2\cdot\delta_d}{g}\right) + \left(\frac{b_p - g}{2}\right)\left(\frac{\delta_d - s\cdot\theta}{s}\right) + \left(\frac{b_p}{2}\right)\left(\frac{\delta_d}{s}\right) \right]$$

Simplify:

$$\Sigma W_i \quad = \quad m_p\cdot\left(\frac{5\cdot\theta\cdot g}{2} - \frac{\theta\cdot b_p}{2} + \frac{4\cdot d_e\cdot\delta_a}{g} + \frac{b_p\cdot\delta_c}{p_{fi}} + \frac{b_p\cdot\delta_b}{p_{fo}} + \frac{b_p\cdot\delta_d}{s} + \frac{3\cdot p_b\cdot\delta_a}{g} + \frac{p_b\cdot\delta_b}{g} + \frac{3\cdot p_b\cdot\delta_c}{g} \ldots \right.$$
$$\left. + \frac{p_b\cdot\delta_d}{g} + \frac{4\cdot p_{fi}\cdot\delta_c}{g} + \frac{4\cdot p_{fo}\cdot\delta_b}{g} + \frac{4\cdot s\cdot\delta_d}{g}\right)$$

Solve for s that produces minimum stored energy:

$$\frac{d}{ds}\left[m_p\cdot\left(\frac{5\cdot\theta\cdot g}{2} - \frac{\theta\cdot b_p}{2} + \frac{4\cdot d_e\cdot\delta_a}{g} + \frac{b_p\cdot\delta_c}{p_{fi}} + \frac{b_p\cdot\delta_b}{p_{fo}} + \frac{b_p\cdot\delta_d}{s} + \frac{3\cdot p_b\cdot\delta_a}{g} + \frac{p_b\cdot\delta_b}{g} + \frac{3\cdot p_b\cdot\delta_c}{g} + \frac{p_b\cdot\delta_d}{g} \ldots \right.\right.$$
$$\left.\left. + \frac{4\cdot p_{fi}\cdot\delta_c}{g} + \frac{4\cdot p_{fo}\cdot\delta_b}{g} + \frac{4\cdot s\cdot\delta_d}{g}\right)\right]$$

$$= \quad m_p\cdot\frac{4\cdot\delta_d}{g} - m_p\cdot\frac{b_p\cdot\delta_d}{s^2} = 0$$

Solve for s:

$$s = \frac{1}{2}\cdot\sqrt{b_p\cdot g}$$

Simplify: Substitute the following into ΣW_i:

$\delta_a = h_1\cdot\theta \qquad \delta_b = h_2\cdot\theta \qquad \delta_c = h_3\cdot\theta \qquad \delta_d = h_4\cdot\theta$

Set $W_e = W_i$

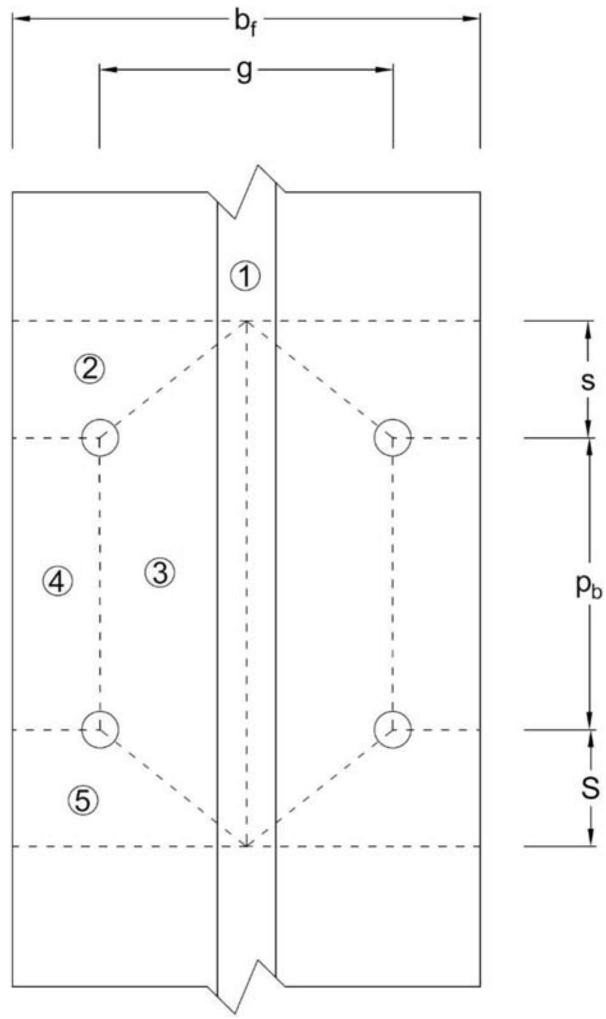
Pull θ out of ΣW_i $\qquad M_n\cdot\theta = 2\cdot F_y\cdot\frac{t_p^{\,2}}{4}\cdot\theta\cdot\Sigma W_i$

Nominal Strength: $M_{pl} = F_{py} \cdot t_p^{\,2} \cdot Y$

Yield Line Parameter:

$$Y = \frac{b_p}{2} \cdot \left(\frac{h_2}{p_{fo}} + \frac{h_3}{p_{fi}} + \frac{h_4}{s} - \frac{1}{2} \right) + \frac{1}{2g} \cdot \begin{bmatrix} h_1 \cdot (4 \cdot d_e + 3 \cdot p_b) + h_2 \cdot (p_b + 4 \cdot p_{fo}) \ldots \\ + h_3 \cdot (4 \cdot p_{fi} + 3 \cdot p_b) + h_4 \cdot (p_b + 4 \cdot s) \end{bmatrix} + \frac{5g}{4}$$

APPENDIX O YIELD LINE DERIVATION FOR 4-BOLT PATTERN



Yield Line Mechanism for 4-Bolt Flange Tension Connection

Panel Rotations of the Yield Line for 4-Bolt Flange Tension Connection

Panel	θ_{nx}	θ_{ny}
1	0	0
2	$\frac{-\delta}{s}$	0
3	0	$\frac{2\delta}{g}$
4	0	0
5	$\frac{\delta}{s}$	0

Energy Stored in Each Yield Line for 4-Bolt Flange Tension Connection

Yield Line	Energy Stored
$W_{i-1/2}$	$m_p\left(\frac{b_f}{2}\right)\left(\frac{\delta}{s}\right)$
$W_{i-2/4}$	$m_p\left(\frac{b_f - g}{2}\right)\left(\frac{\delta}{s}\right)$
$W_{i-2/3}$	$m_p\left(\frac{g}{2}\right)\left(\frac{\delta}{s}\right) + m_p(s)\left(\frac{2\delta}{g}\right)$
$W_{i-3/4}$	$m_p(p_b)\left(\frac{2\delta}{g}\right)$
$W_{i-1/3}$	$m_p(p_b + 2s)\left(\frac{2\delta}{g}\right)$
$W_{i-3/5}$	$m_p\left(\frac{g}{2}\right)\left(\frac{\delta}{s}\right) + m_p(s)\left(\frac{2\delta}{g}\right)$
$W_{i-4/5}$	$m_p\left(\frac{b_f - g}{2}\right)\left(\frac{\delta}{s}\right)$
$W_{i-1/5}$	$m_p\left(\frac{b_f}{2}\right)\left(\frac{\delta}{s}\right)$

$$\Sigma W_i = m_p\cdot\left[\left(\frac{b_f}{2}\right)\cdot\left(\frac{\delta}{s}\right)+\left(\frac{b_f-g}{2}\right)\cdot\left(\frac{\delta}{s}\right)+\left(\frac{g}{2}\right)\left(\frac{\delta}{s}\right)+(s)\cdot\left(\frac{2\cdot\delta}{g}\right)+\left(p_b\right)\cdot\left(\frac{2\cdot\delta}{g}\right)\ldots \\ +\left(p_b+2\cdot s\right)\cdot\left(\frac{2\cdot\delta}{g}\right)+\left(\frac{g}{2}\right)\left(\frac{\delta}{s}\right)+(s)\cdot\left(\frac{2\cdot\delta}{g}\right)+\left(\frac{b_f-g}{2}\right)\cdot\left(\frac{\delta}{s}\right)+\left(\frac{b_f}{2}\right)\cdot\left(\frac{\delta}{s}\right)\right]$$

Simplify:

$$\Sigma W_i = \frac{2\cdot m_p\cdot\delta\cdot\left(4\cdot s^2+2\cdot p_b\cdot s+b_f\cdot g\right)}{g\cdot s}$$

Solve for s that produces minimum stored energy:

$$\frac{d}{ds}\left[\frac{2\cdot m_p\cdot\delta\cdot\left(4\cdot s^2+2\cdot p_b\cdot s+b_f\cdot g\right)}{g\cdot s}\right]$$

$$= \frac{2\cdot\delta\cdot m_p\cdot\left(2\cdot p_b+8\cdot s\right)}{g\cdot s}-\frac{2\cdot\delta\cdot m_p\cdot\left(4\cdot s^2+2\cdot p_b\cdot s+b_f\cdot g\right)}{g\cdot s^2} = 0$$

Solve for s:

$$s = \frac{1}{2}\cdot\sqrt{b_f\cdot g}$$

Set W_e = W_i

$$W_e = P_n\cdot\delta$$

$W_e = 2\cdot\Sigma W_i$ For 2 sides

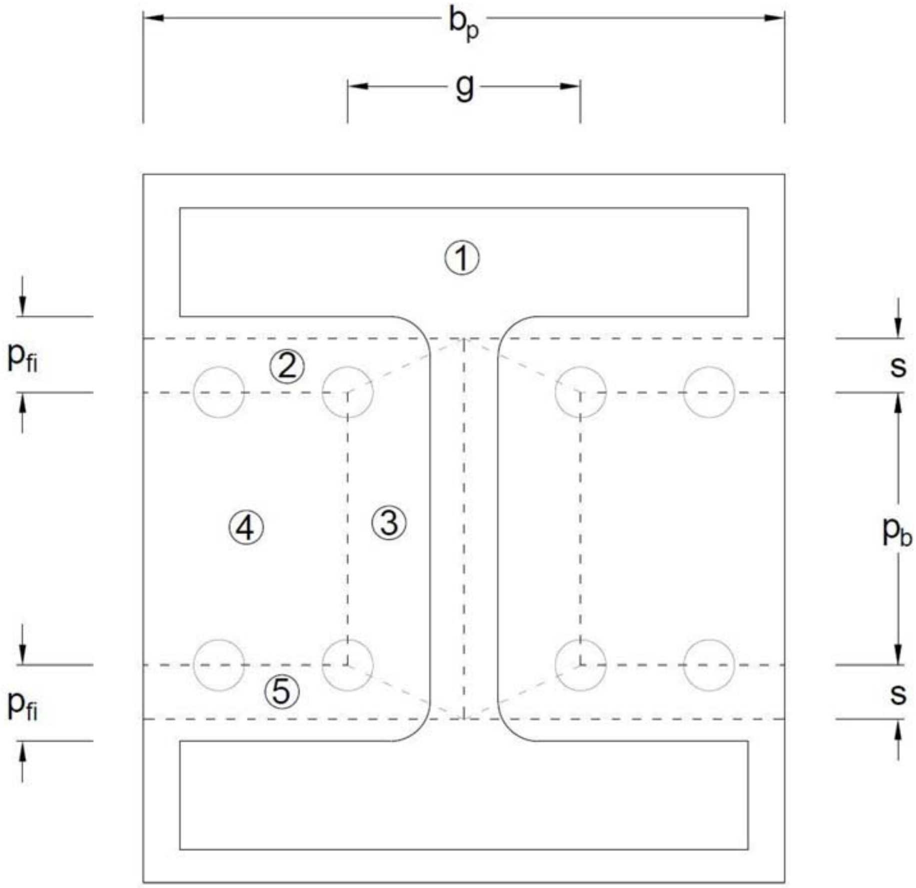
$$m_p = F_y\cdot\frac{t_f^2}{4}$$

$$P_n\cdot\delta = F_y\cdot\frac{t_f^2}{4}\cdot(2)\cdot\left[\frac{2\cdot\delta\cdot\left(4\cdot s^2+2\cdot p_b\cdot s+b_f\cdot g\right)}{g\cdot s}\right]$$

Nominal Strength: $P_n = F_y\cdot t_f^2\cdot\left(\frac{4\cdot s^2+2\cdot p_b\cdot s+b_f\cdot g}{g\cdot s}\right)$

Yield Line Parameter: $Y = \frac{4\cdot s^2+2\cdot p_b\cdot s+b_f\cdot g}{g\cdot s}$

APPENDIX P   YIELD LINE DERIVATION FOR 8-BOLT PATTERN



Yield Line Mechanism for 8-Bolt End-Plate Tension Connection

Panel Rotations of the Yield Line for 8-Bolt End-Plate Tension Connection

Panel	θ_{nx}	θ_{ny}
1	0	0
2	$\frac{-\delta}{s}$	0
3	0	$\frac{2\delta}{g}$
4	0	0
5	$\frac{\delta}{s}$	0

Energy Stored in Each Yield Line for 8-Bolt End-Plate Tension Connection

Yield Line	Energy Stored
$W_{i\text{-}1/2}$	$m_p\left(\frac{b_p}{2}\right)\left(\frac{\delta}{s}\right)$
$W_{i\text{-}2/4}$	$m_p\left(\frac{b_p - g}{2}\right)\left(\frac{\delta}{s}\right)$
$W_{i\text{-}2/3}$	$m_p\left(\frac{g}{2}\right)\left(\frac{\delta}{s}\right) + m_p(s)\left(\frac{2\delta}{g}\right)$
$W_{i\text{-}3/4}$	$m_p(p_b)\left(\frac{2\delta}{g}\right)$
$W_{i\text{-}1/3}$	$m_p(p_b + 2s)\left(\frac{2\delta}{g}\right)$
$W_{i\text{-}3/5}$	$m_p\left(\frac{g}{2}\right)\left(\frac{\delta}{s}\right) + m_p(s)\left(\frac{2\delta}{g}\right)$
$W_{i\text{-}4/5}$	$m_p\left(\frac{b_p - g}{2}\right)\left(\frac{\delta}{s}\right)$
$W_{i\text{-}1/5}$	$m_p\left(\frac{b_p}{2}\right)\left(\frac{\delta}{s}\right)$

$$\Sigma W_i = m_p \cdot \left[\left(\frac{b_p}{2}\right)\cdot\left(\frac{\delta}{s}\right) + \left(\frac{g}{2}\right)\cdot\left(\frac{\delta}{s}\right) + (s)\cdot\left(\frac{2\cdot\delta}{g}\right) + \left(\frac{b_p - g}{2}\right)\cdot\left(\frac{\delta}{s}\right) + (p_b + 2\cdot s)\cdot\frac{2\cdot\delta}{g} \ldots \right.$$
$$\left. + (p_b)\cdot\left(\frac{2\cdot\delta}{g}\right) + \left(\frac{g}{2}\right)\cdot\left(\frac{\delta}{s}\right) + (s)\cdot\left(\frac{2\cdot\delta}{g}\right) + \frac{b_p - g}{2}\cdot\frac{\delta}{s} + \left(\frac{b_p}{2}\right)\cdot\left(\frac{\delta}{s}\right) \right]$$

Simplify:

$$\Sigma W_i = \frac{m_p\cdot 2\cdot\delta\cdot\left(4\cdot s^2 + 2\cdot p_b\cdot s + b_p\cdot g\right)}{g\cdot s}$$

Solve for s that produces minimum stored energy:

$$\frac{d}{ds}\left[\frac{m_p\cdot 2\cdot\delta\cdot\left(4\cdot s^2 + 2\cdot p_b\cdot s + b_p\cdot g\right)}{g\cdot s}\right]$$

$$= \frac{2\cdot\delta\cdot m_p\cdot\left(2\cdot p_b + 8\cdot s\right)}{g\cdot s} - \frac{2\cdot\delta\cdot m_p\cdot\left(4\cdot s^2 + 2\cdot p_b\cdot s + b_p\cdot g\right)}{g\cdot s^2} = 0$$

Solve for s:

$$s = \frac{1}{2}\cdot\sqrt{b_p\cdot g}$$

Use: $S = \min\left(s, p_{fi}\right)$

Set $W_e = W_i$

$$W_e = P_n\cdot\delta$$

$W_e = 2\cdot\Sigma W_i$ For 2 sides

$$m_p = F_y\cdot\frac{t_f^{\,2}}{4}$$

$$P_n\cdot\delta = F_y\cdot\frac{t_p^{\,2}}{4}\cdot(2)\cdot\left[\frac{2\cdot\delta\cdot\left(4\cdot S^2 + 2\cdot p_b\cdot S + b_p\cdot g\right)}{g\cdot S}\right]$$

Nominal Strength: $P_n = F_y\cdot t_p^{\,2}\cdot\left(\frac{4\cdot S^2 + 2\cdot p_b\cdot S + b_p\cdot g}{g\cdot S}\right)$

Yield Line Parameter: $Y = \frac{4\cdot S^2 + 2\cdot p_b\cdot S + b_p\cdot g}{g\cdot S}$

APPENDIX Q LOADING PROTOCOL FOR BEAM-TO-COLUMN MOMENT CONNECTIONS

Loading Sequence for Beam-to-Column Moment Connections

AISC 341-10 Sect. K2-4b

Actuator CL to Column CL	194.25	in
Actuator Stroke	30	in --> Maximum Story Drift Angle = 0.0772 rad
Displacement Rate	2	in/min
Displacement Rate	30	sec/in
Photos in 1st Leg of Cycle*	4	
Photos in 1st Cycle	12	
Photo Rate	0.121	sec/photo
Total Photo Count	135841	

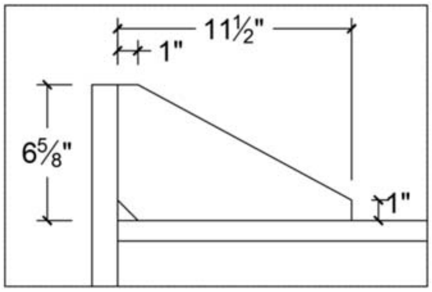
*4 legs in one cycle

Cycle Number	Story Drift Angle, θ (rad)	Furthest Act. Disp. (in)	0 to Peak Disp. Time (sec)	Cycle Time (min)
1	0.00375	0.7284	21.85	1.46
2	0.00375	0.7284	21.85	1.46
3	0.00375	0.7284	21.85	1.46
4	0.00375	0.7284	21.85	1.46
5	0.00375	0.7284	21.85	1.46
6	0.00375	0.7284	21.85	1.46
7	0.005	0.9713	29.14	1.94
8	0.005	0.9713	29.14	1.94
9	0.005	0.9713	29.14	1.94
10	0.005	0.9713	29.14	1.94
11	0.005	0.9713	29.14	1.94
12	0.005	0.9713	29.14	1.94
13	0.0075	1.4569	43.71	2.91
14	0.0075	1.4569	43.71	2.91
15	0.0075	1.4569	43.71	2.91
16	0.0075	1.4569	43.71	2.91
17	0.0075	1.4569	43.71	2.91
18	0.0075	1.4569	43.71	2.91
19	0.01	1.9425	58.28	3.89
20	0.01	1.9425	58.28	3.89
21	0.01	1.9425	58.28	3.89
22	0.01	1.9425	58.28	3.89
23	0.015	2.9138	87.41	5.83
24	0.015	2.9138	87.41	5.83
25	0.02	3.8850	116.55	7.77
26	0.02	3.8850	116.55	7.77
27	0.03	5.8275	174.83	11.66
28	0.03	5.8275	174.83	11.66
29	0.04	7.7700	233.10	15.54
30	0.04	7.7700	233.10	15.54
Continue loading at increments of θ = 0.01 rad, with two cycles of loading at each step.				
31	0.05	9.7125	291.38	19.43
32	0.05	9.7125	291.38	19.43
33	0.06	11.6550	349.65	23.31
34	0.06	11.6550	349.65	23.31
35	0.07	13.5975	407.93	27.20
36	0.07	13.5975	407.93	27.20
			Total	274.86 min

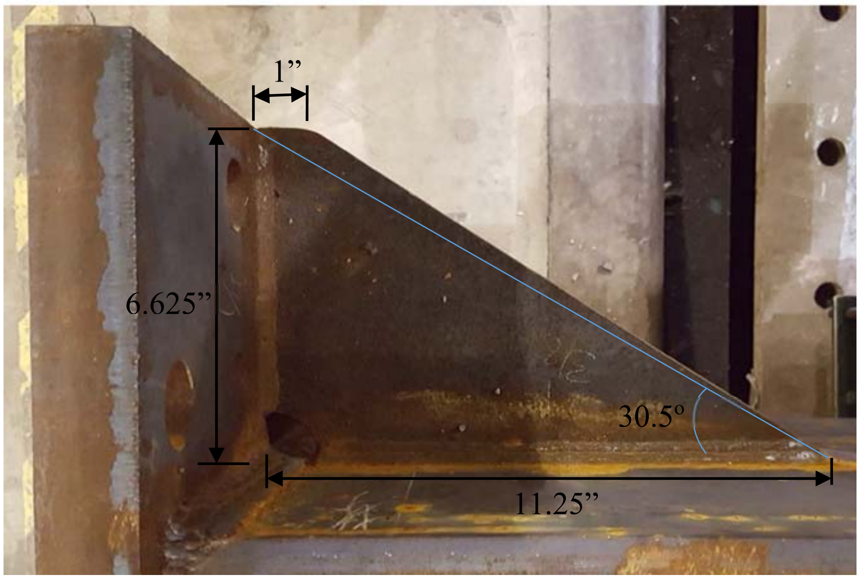
APPENDIX R SPECIMEN 12ES-1.125-1.25-24 STIFFENER CORRECTION

End-Plate Moment Connection: Specimen 12ES-1.125-1.25-24

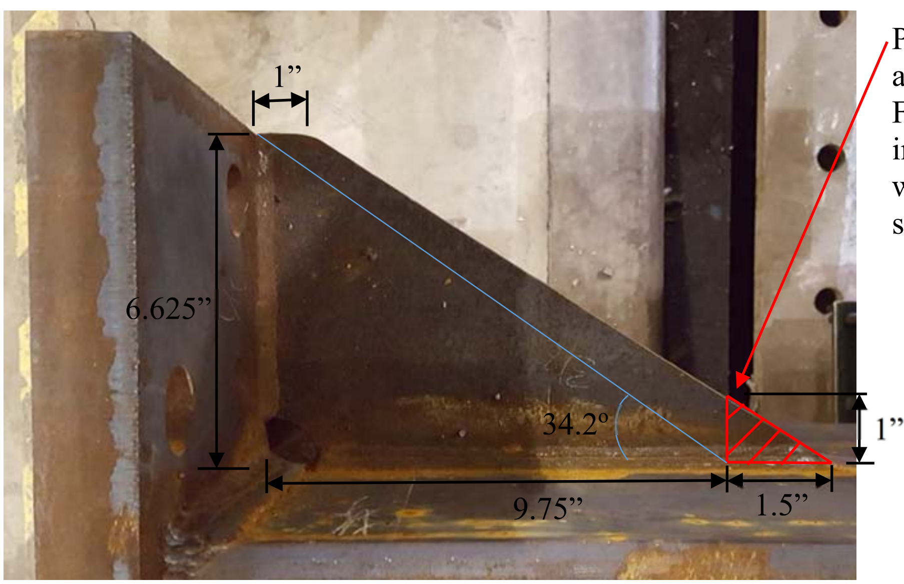
Detailed Stiffener Geometry:



Current Stiffener Geometry:

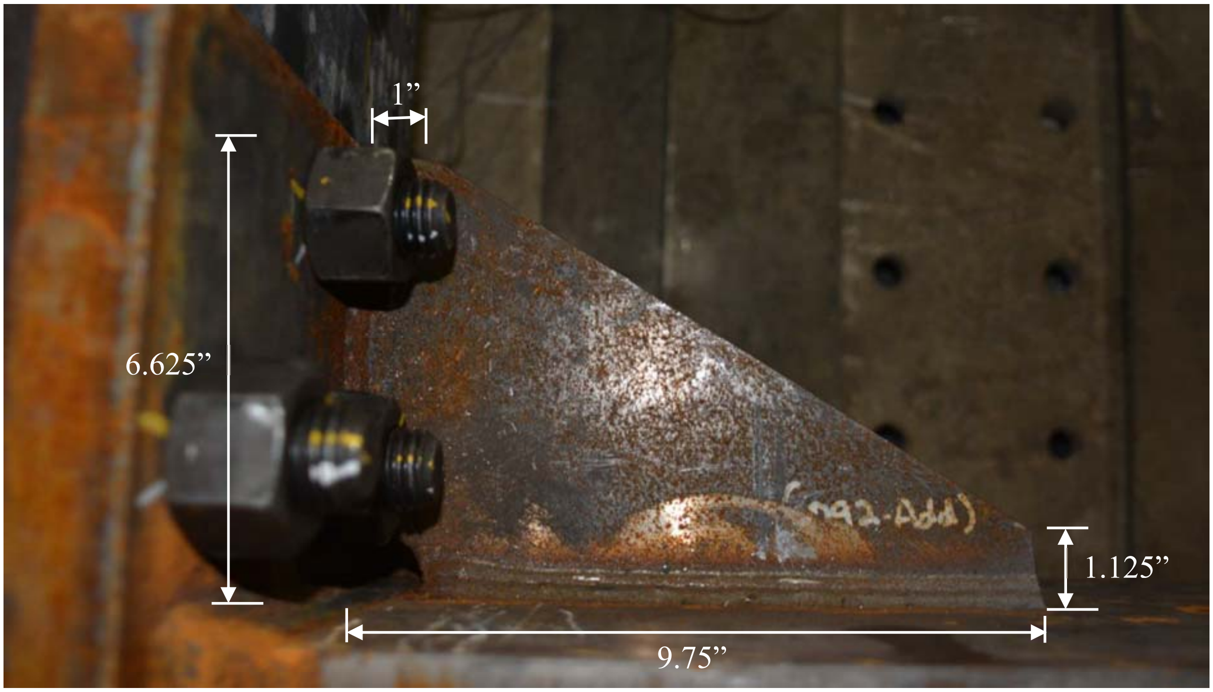


Proposed Stiffener Correction:



Portion to be cut away and ground smooth. Fill flange surface imperfections with weld, and grind surface smooth.

Corrected Stiffener Geometry:



Cut Portion of Stiffener:

Effected surfaces were ground smooth.

APPENDIX S ADDITIONAL PLOTS FOR SPECIMEN 12ES-0.75-1.00-44

Plots from the start until the first failure

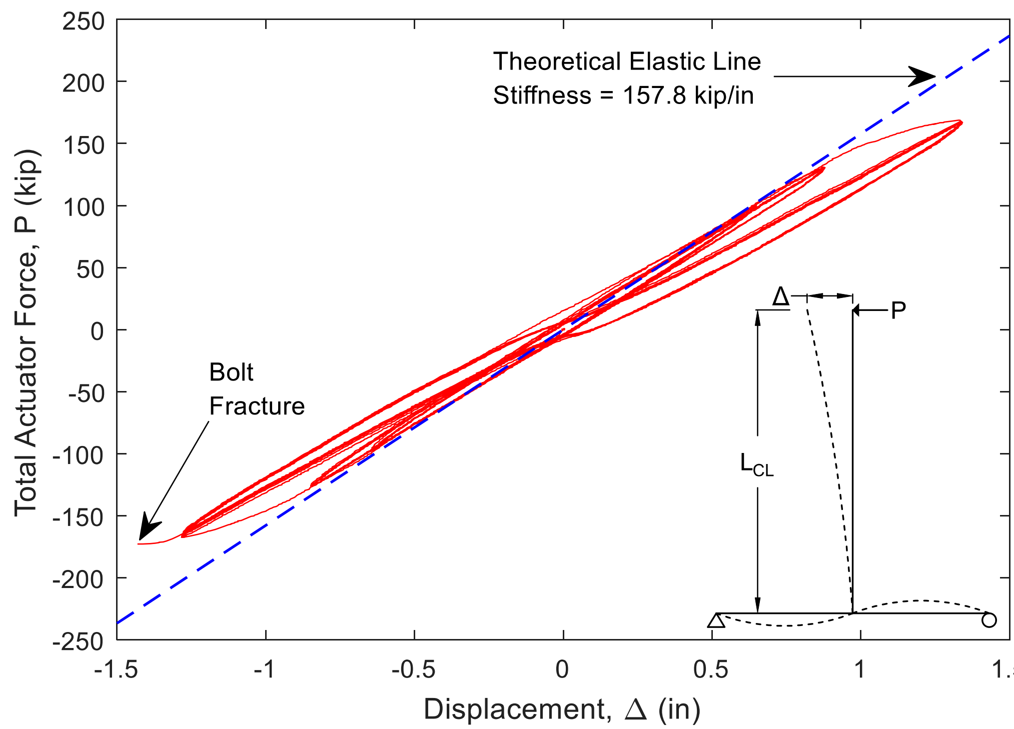


Figure S-1 Total Actuator Force vs. Applied Displacement at End of Beam

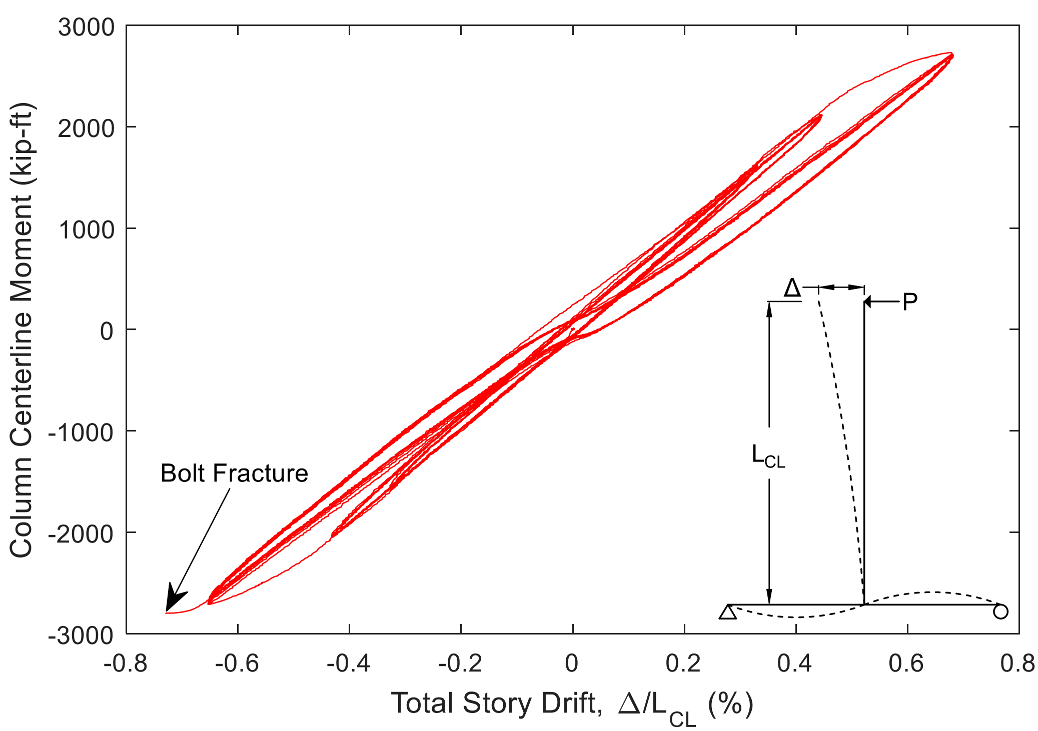


Figure S-2 Column Centerline Moment vs. Total Story Drift

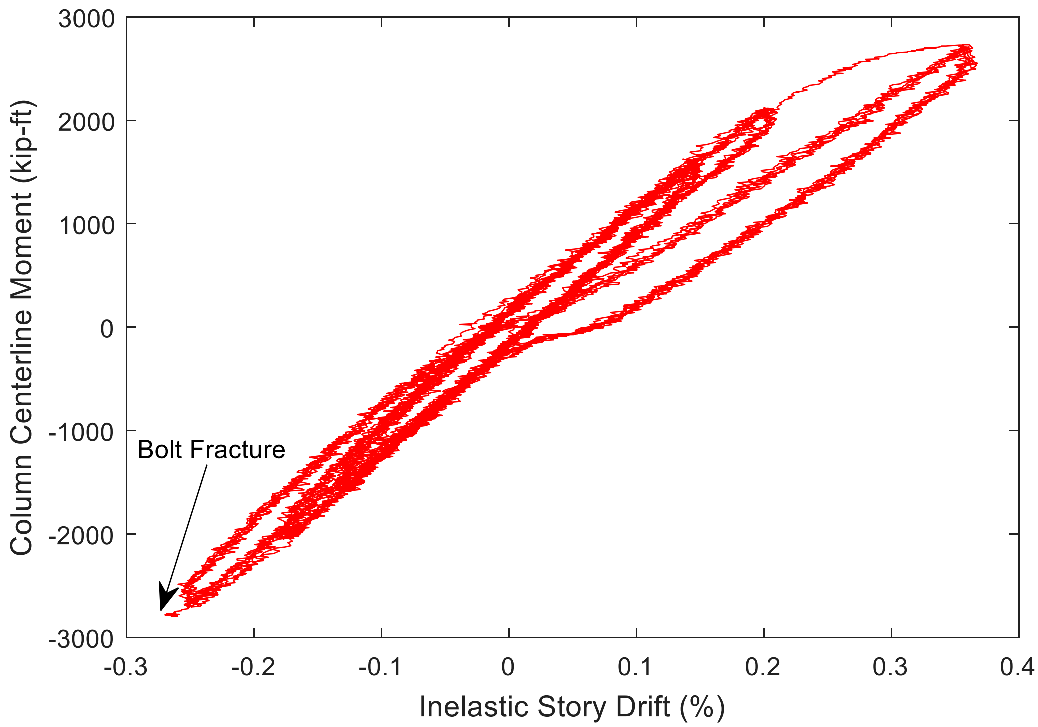


Figure S-3 Column Centerline Moment vs. Inelastic Portion of Story Drift

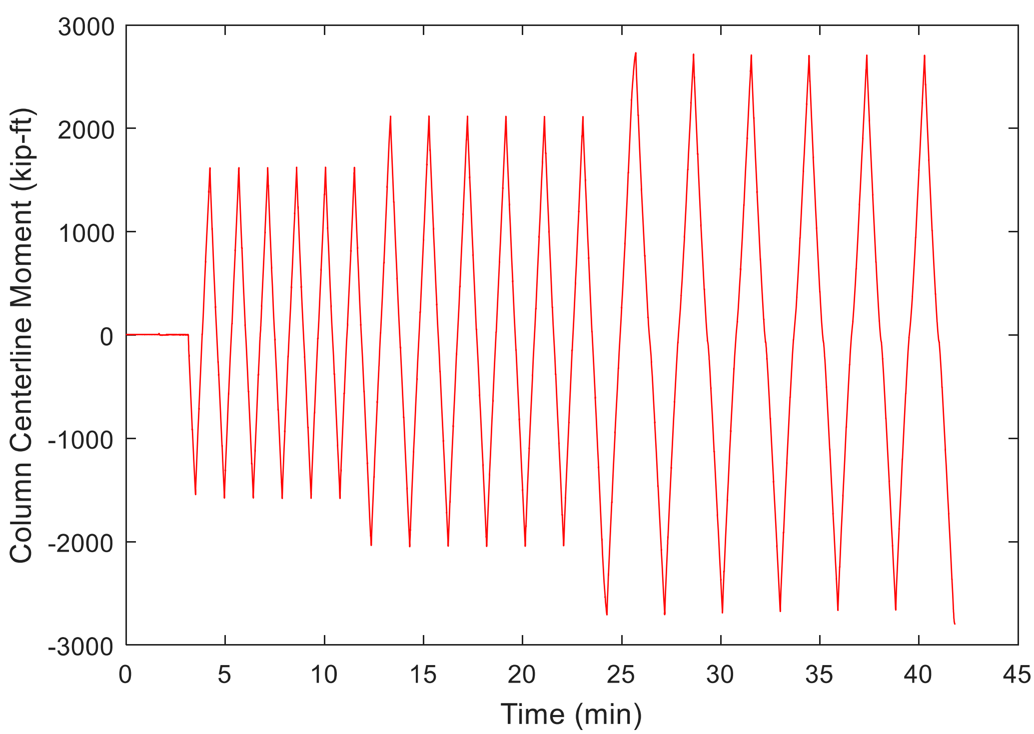


Figure S-4 Column Centerline Moment vs. Time

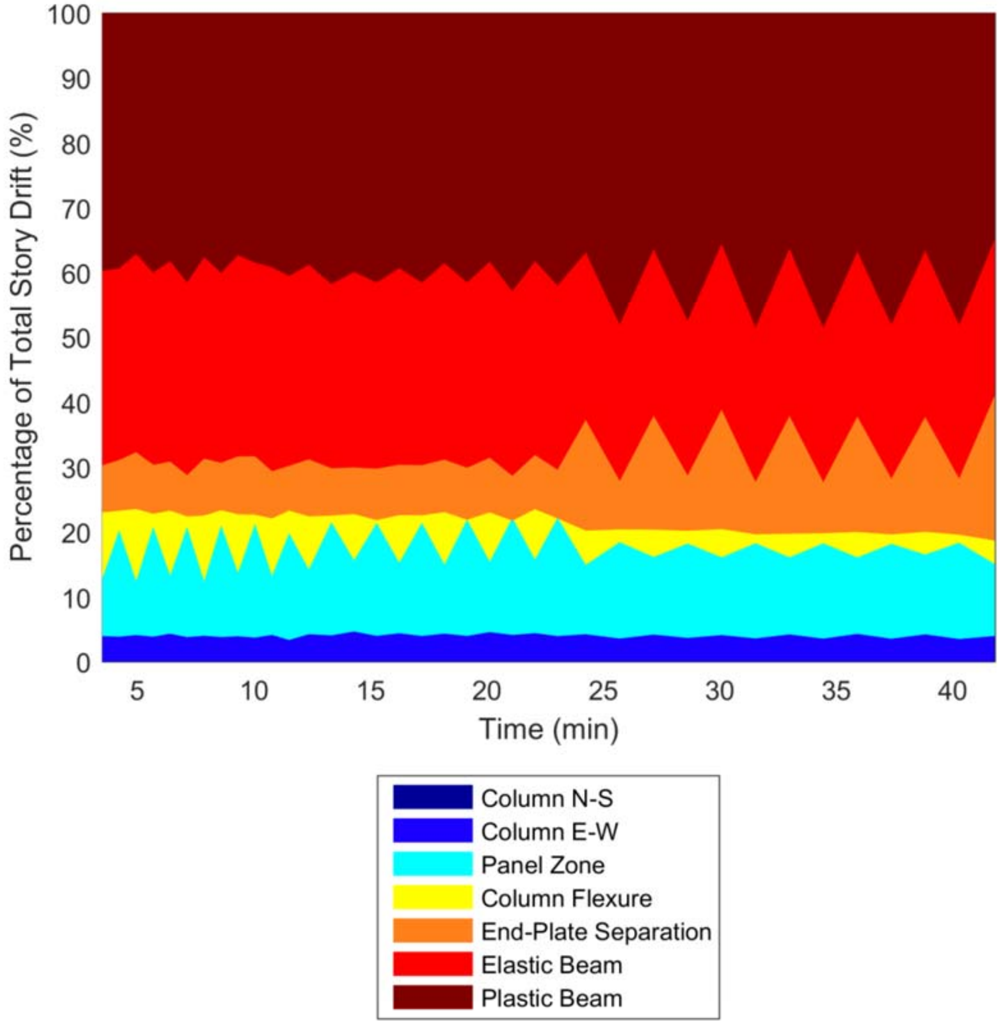


Figure S-5 Story Drift Components Percentage of Total Story Drift vs. Time

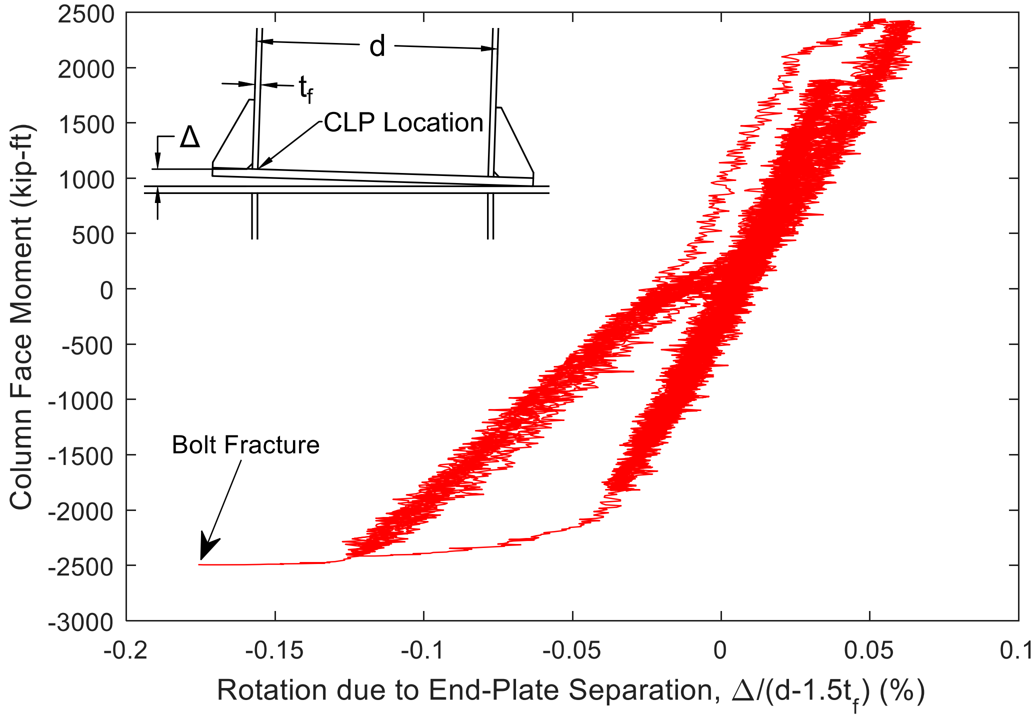


Figure S-6 Column Face Moment vs. Rotation due to End-Plate Separation

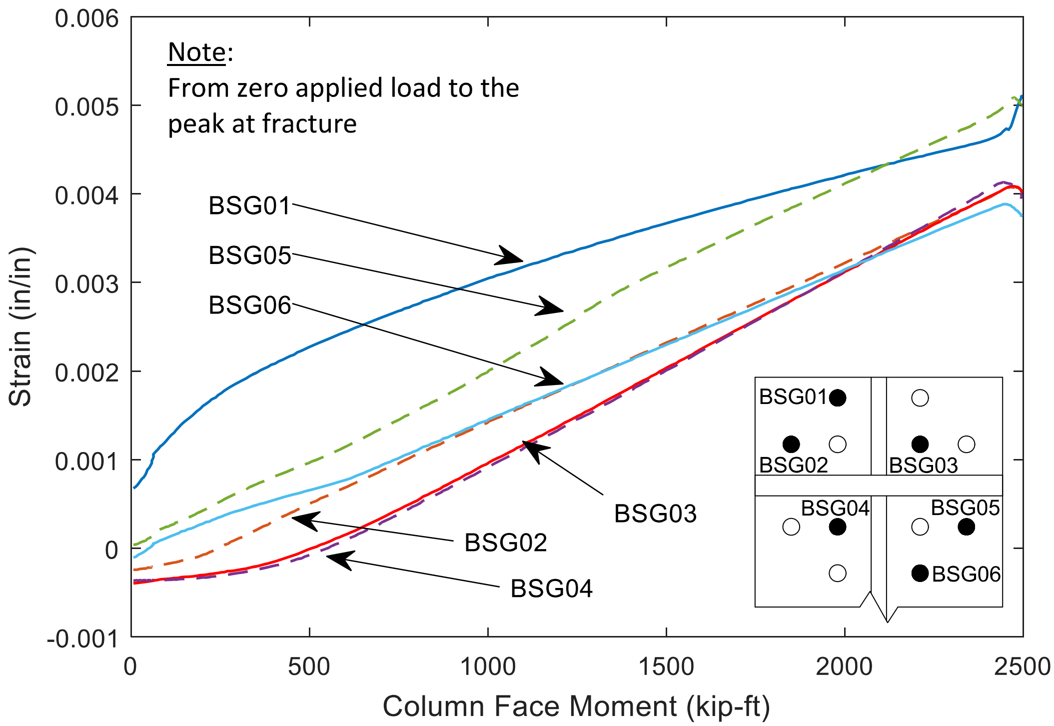


Figure S-7 Strain in Bolts vs. Column Face Moment

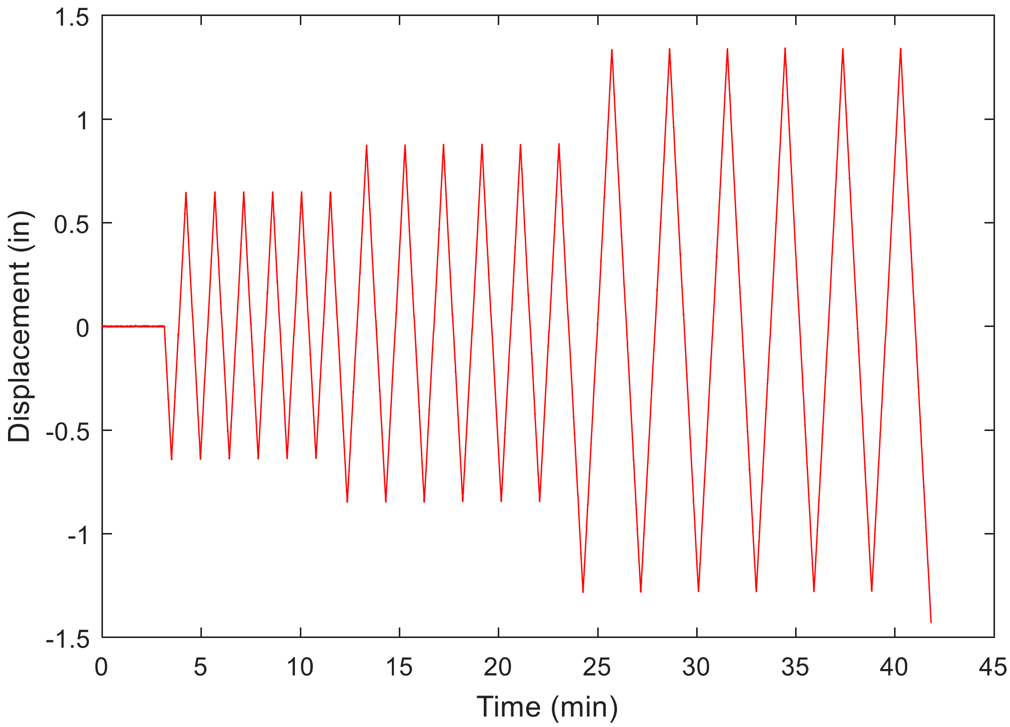


Figure S-8 Displacement at End of Beam vs. Time

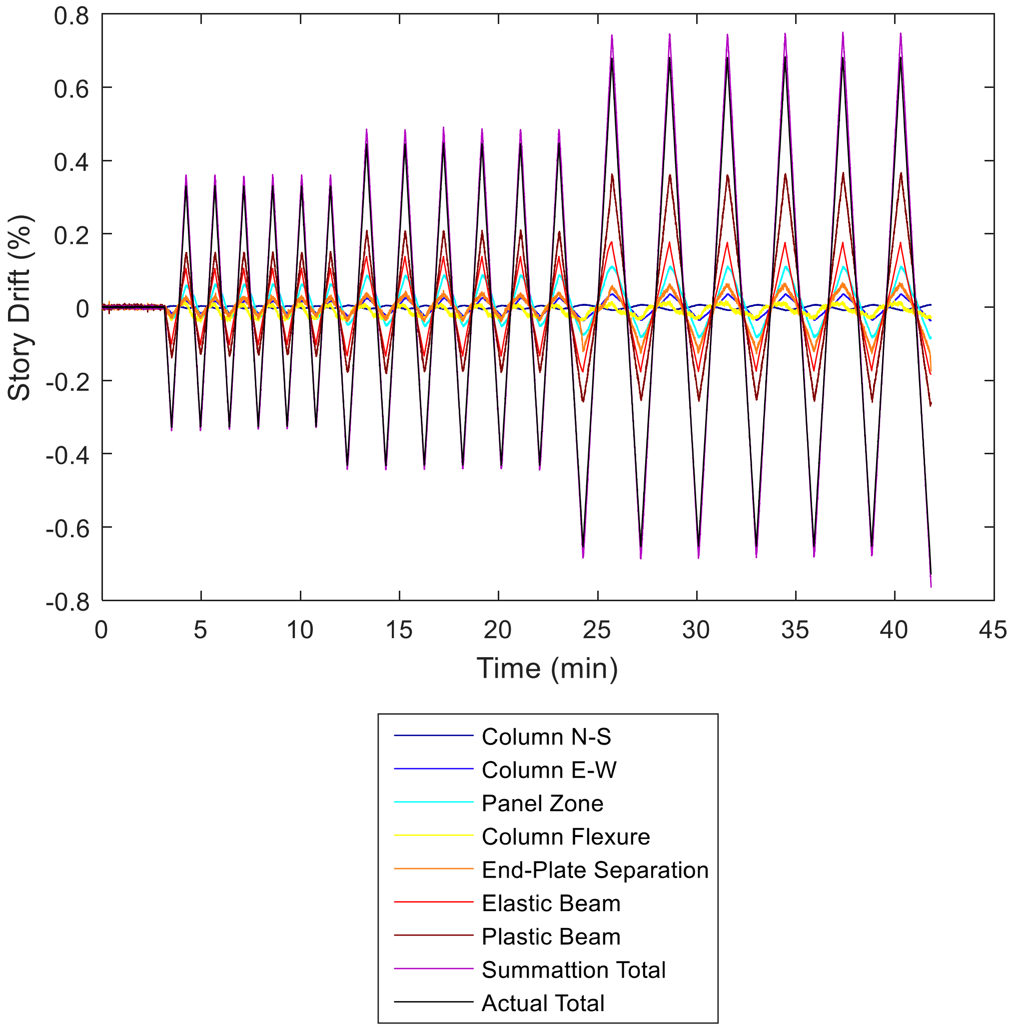


Figure S-9 Story Drift Components vs. Time

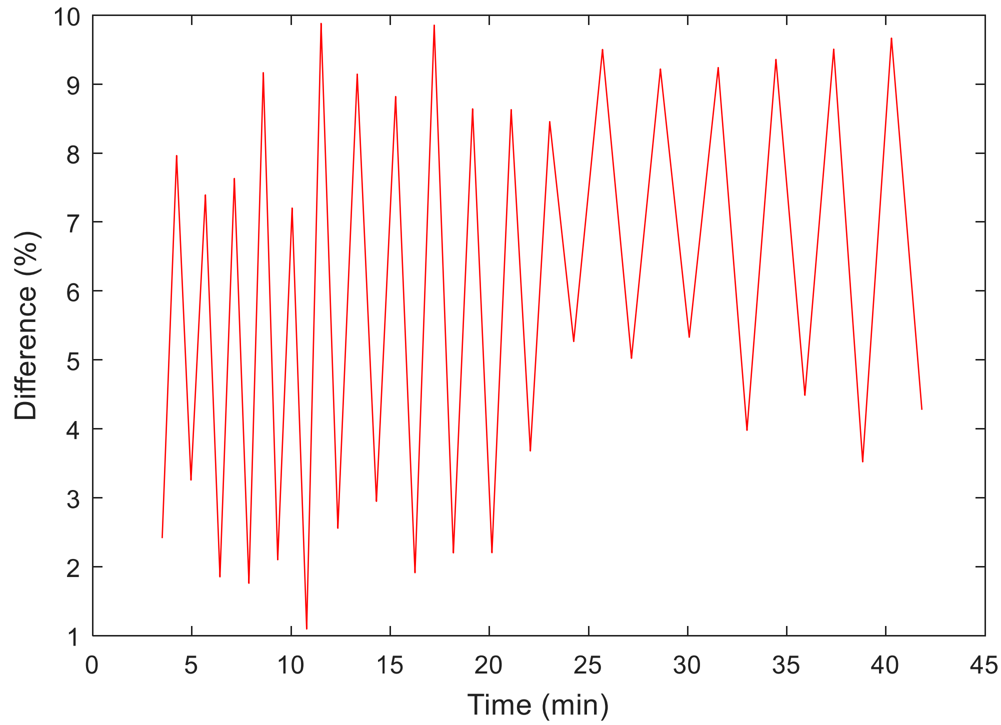


Figure S-10 Percent Difference Between Total Story Drift & Actual Story Drift at Peaks

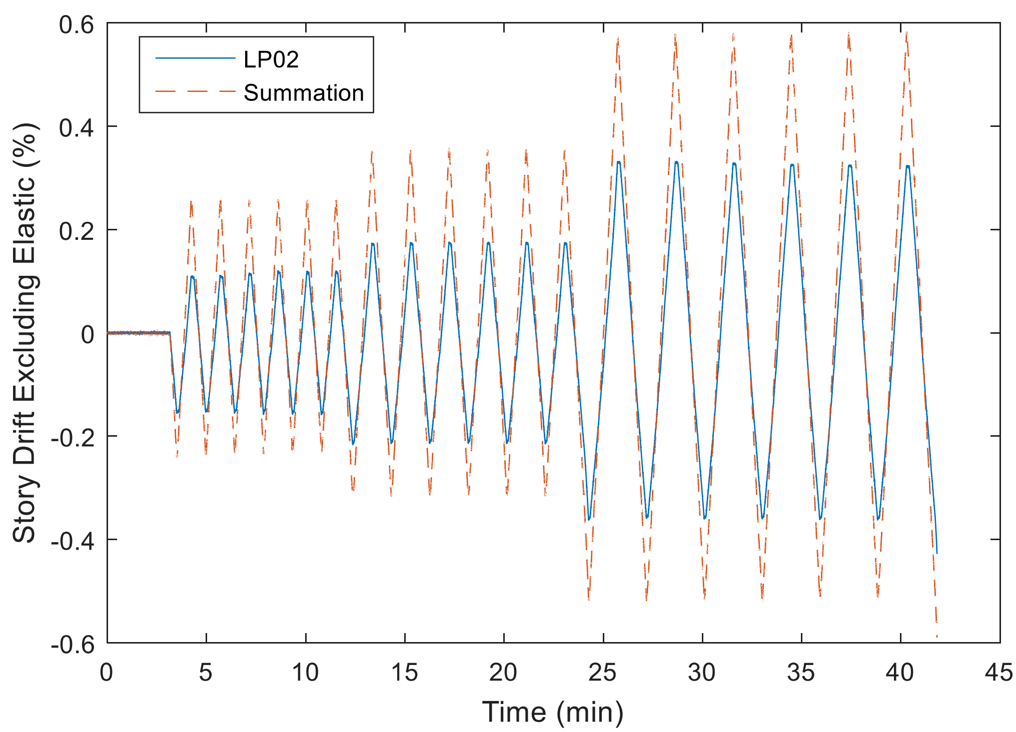


Figure S-11 Large Deformations Check

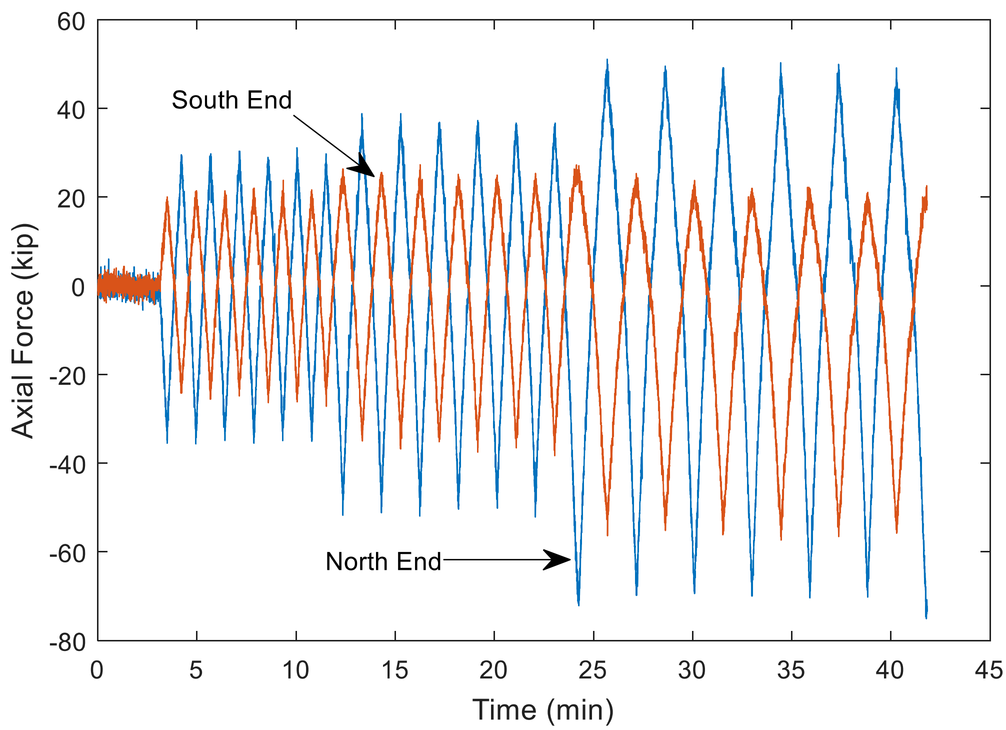


Figure S-12 Column Axial Force Calculated From Strain Gauges on Stub Sections

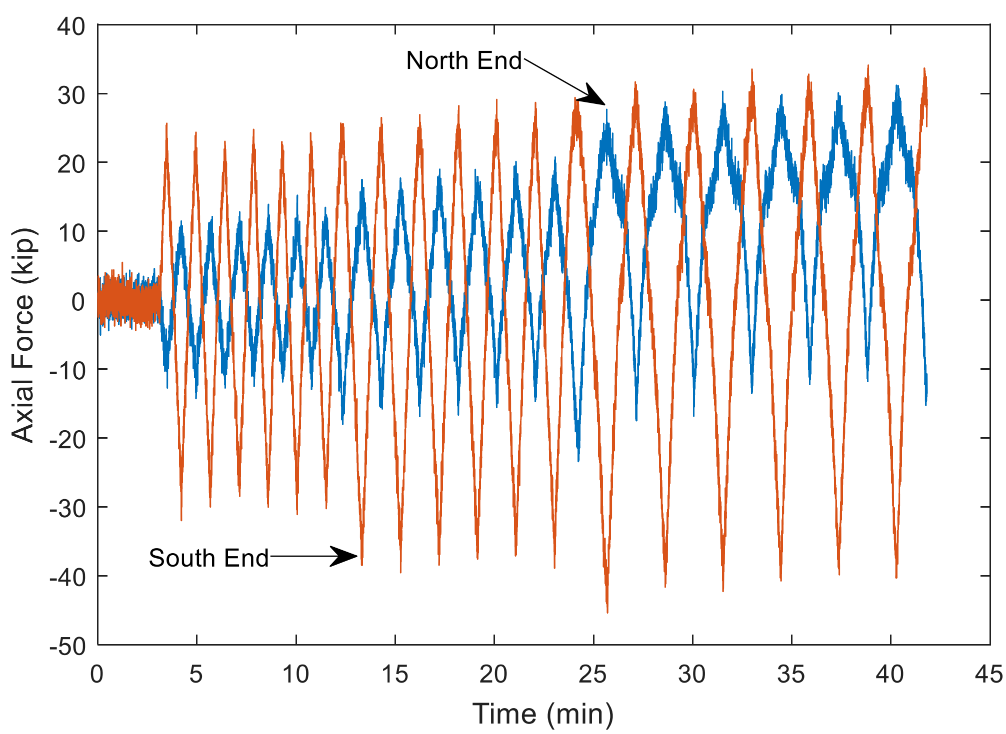


Figure S-13 Column Axial Force Calculated From Strain Gauges on Column

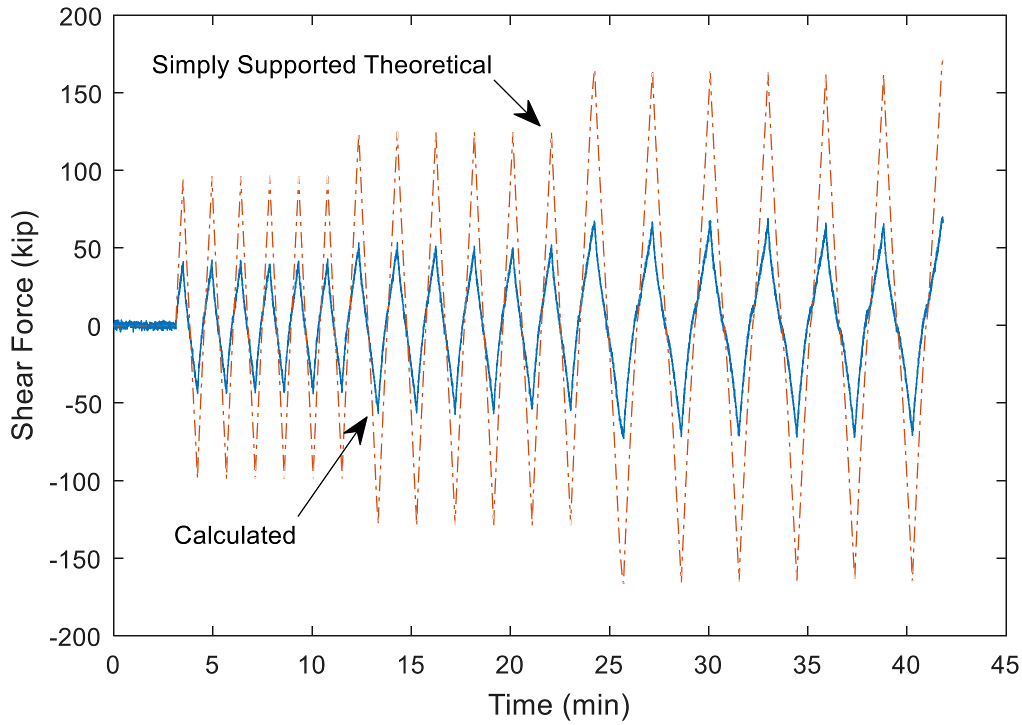


Figure S-14 Column Shear Force – North End

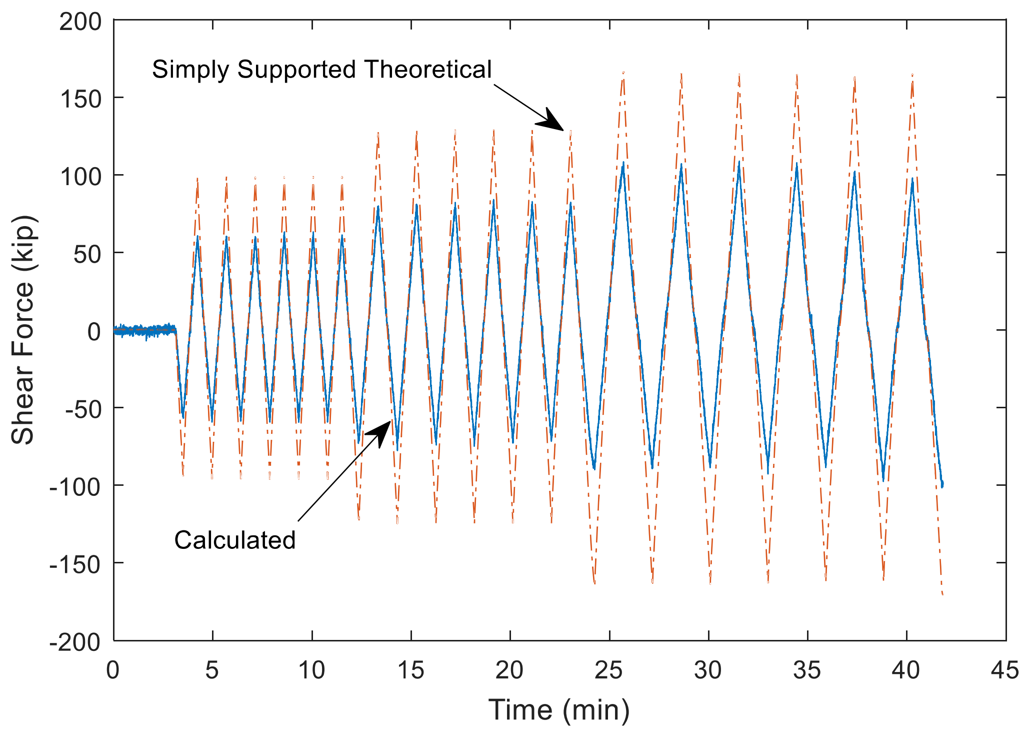


Figure S-15 Column Shear Force – South End

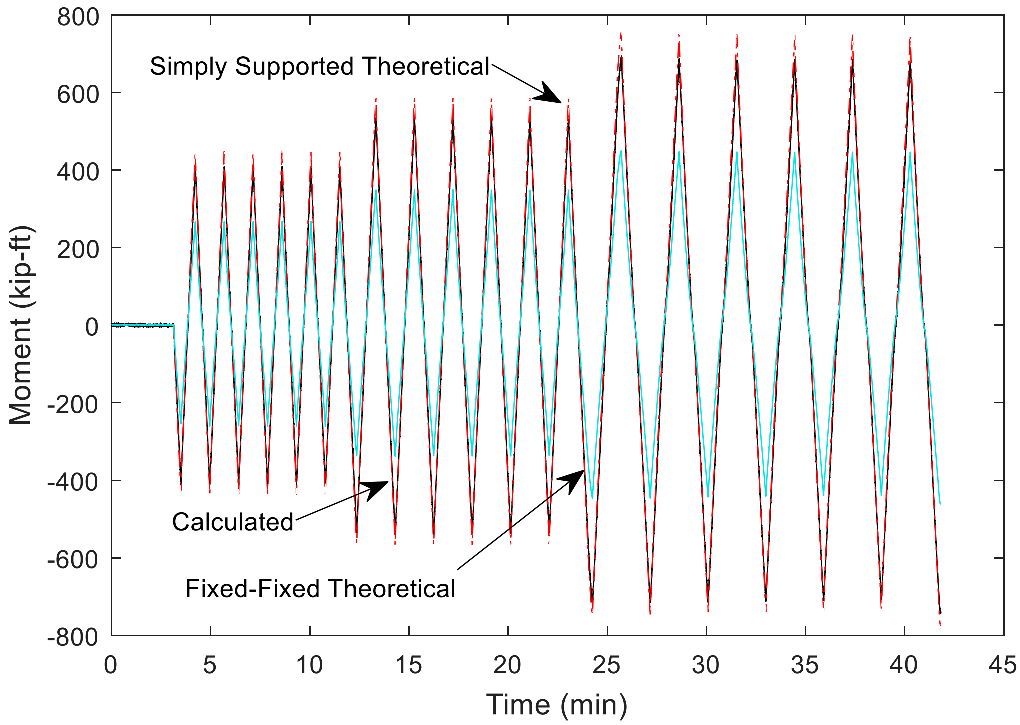


Figure S-16 Moment in Column at Location of Strain Gauges – North End

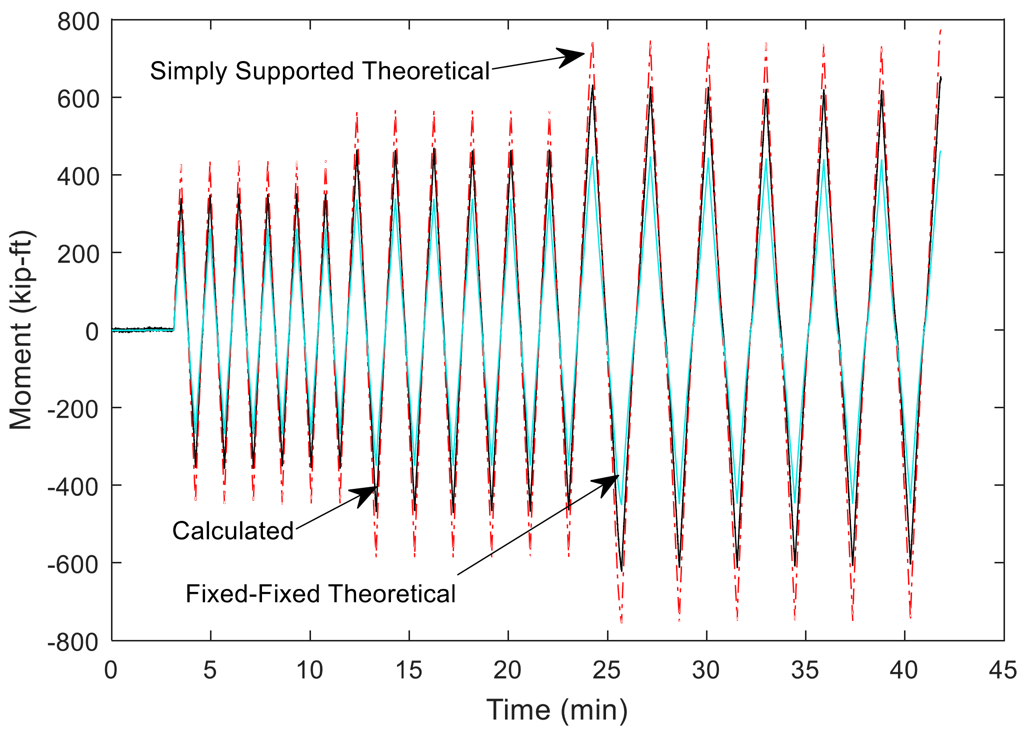


Figure S-17 Moment in Column at Location of Strain Gauges – South End

Instrumentation Data vs. Time

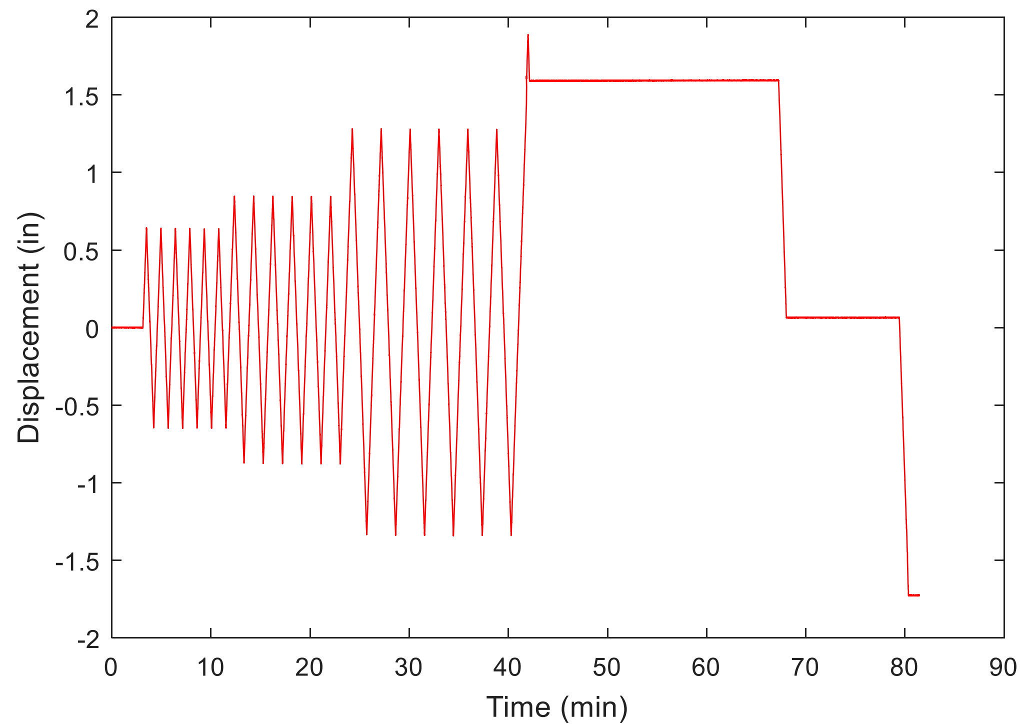


Figure S-18 SP01

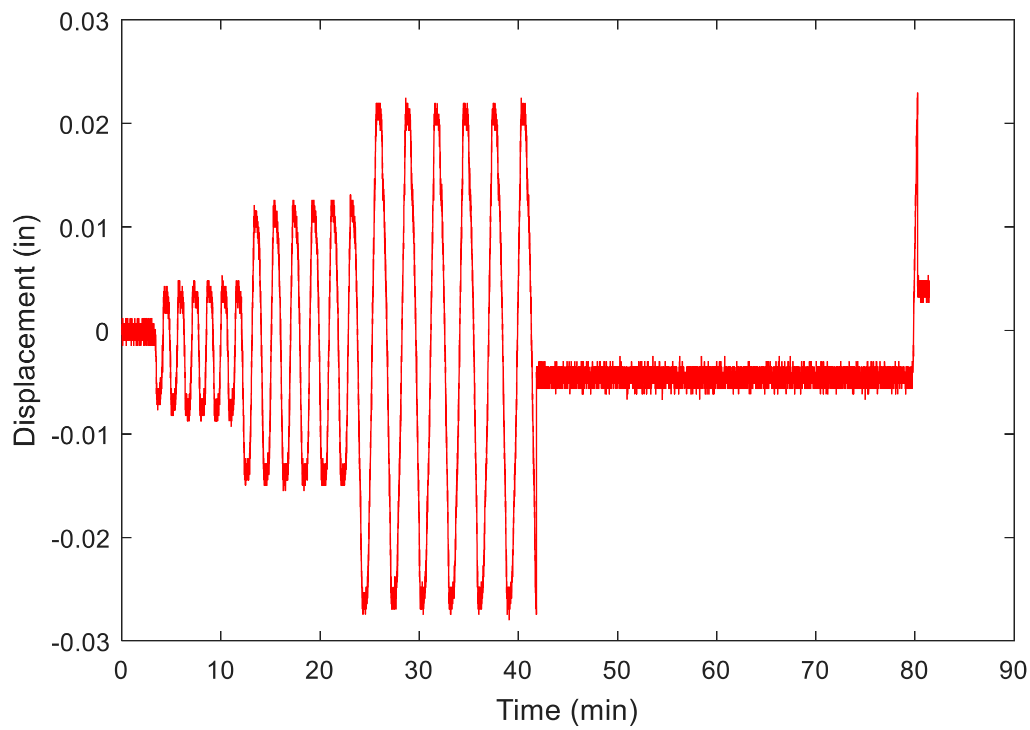


Figure S-19 SP02

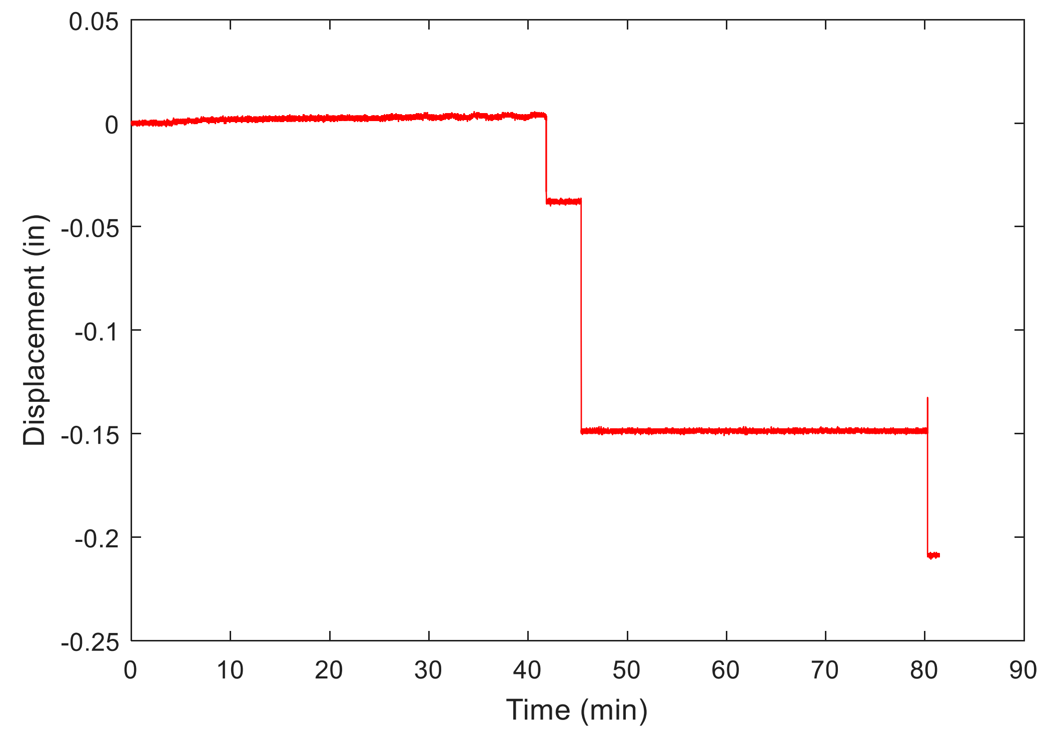


Figure S-20 SP03

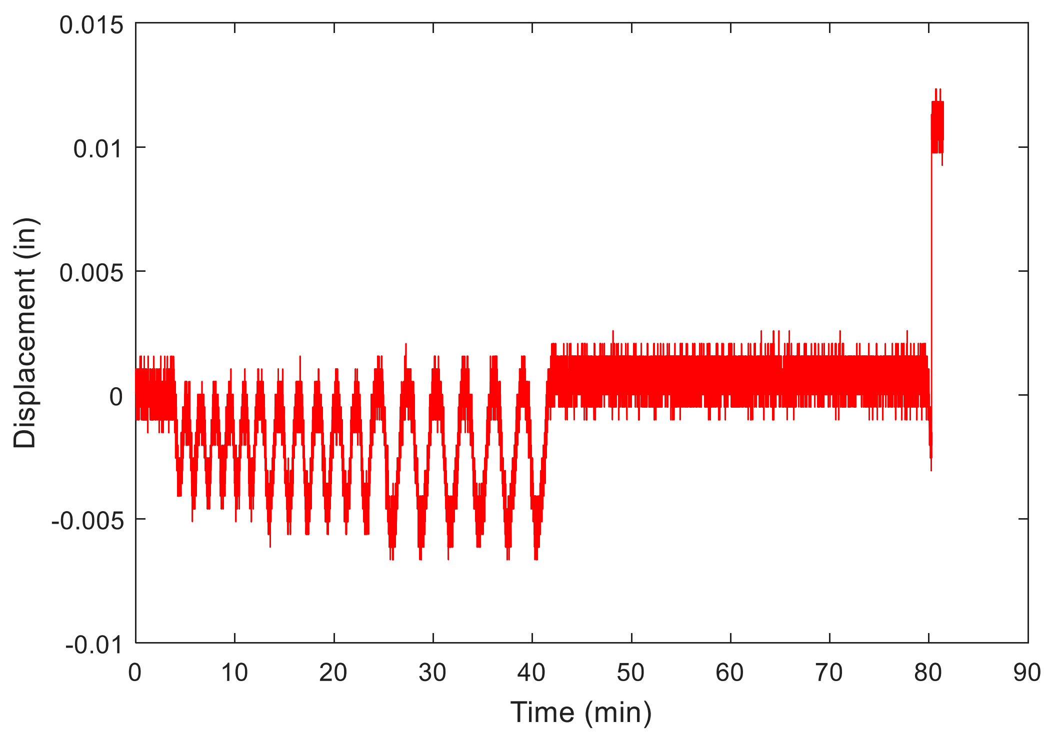


Figure S-21 SP04

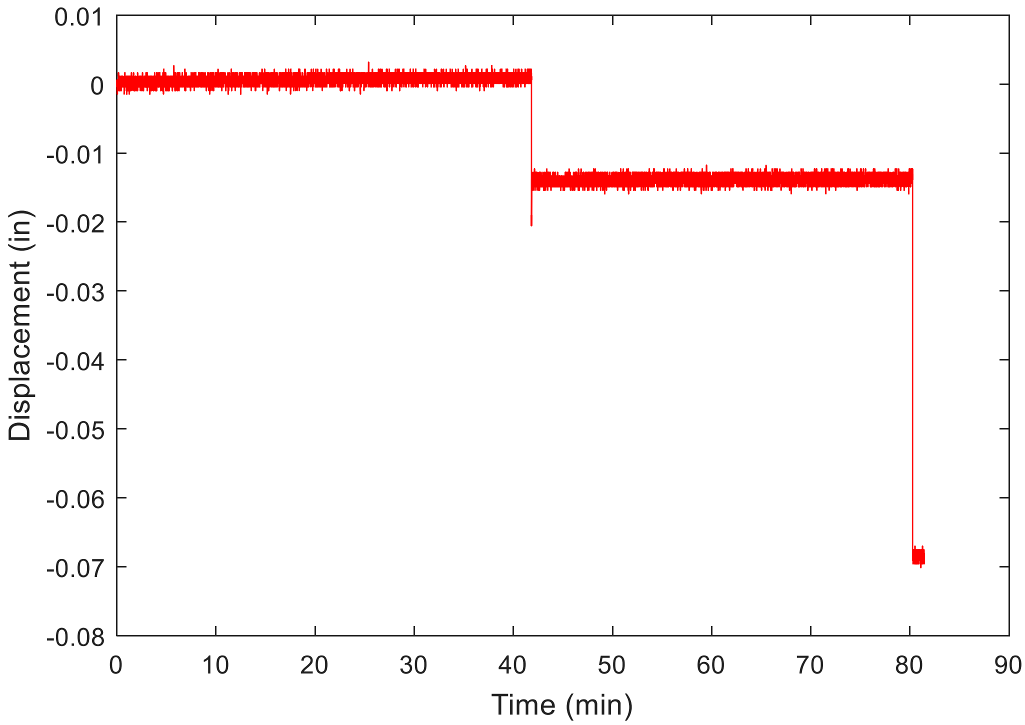


Figure S-22 SP05

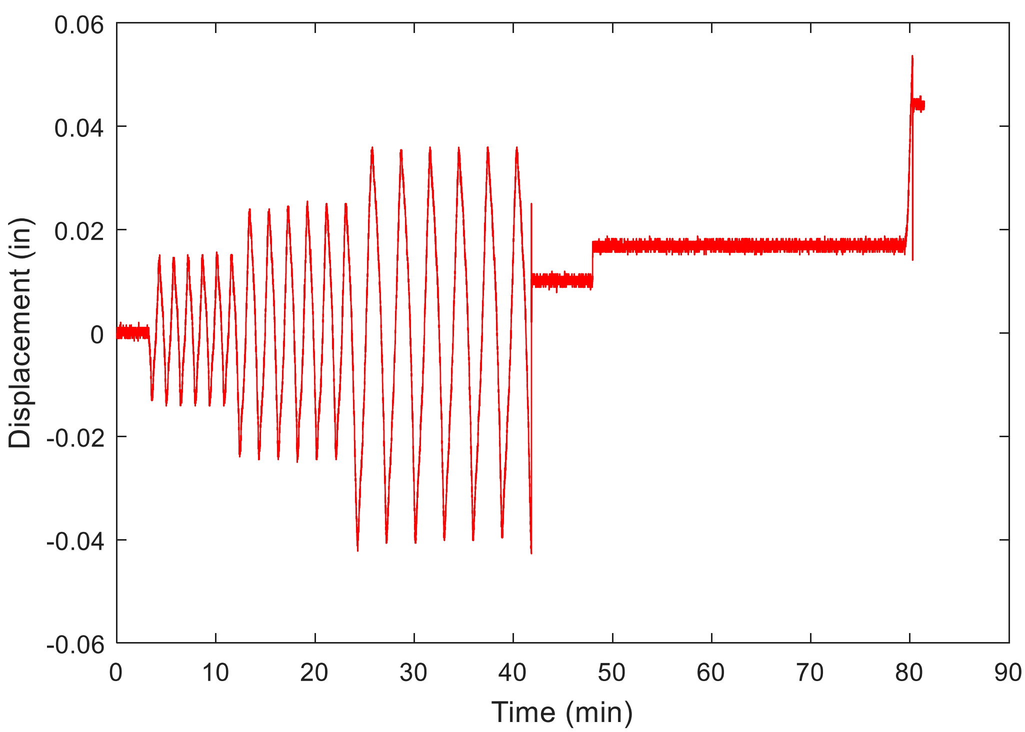


Figure S-23 SP06

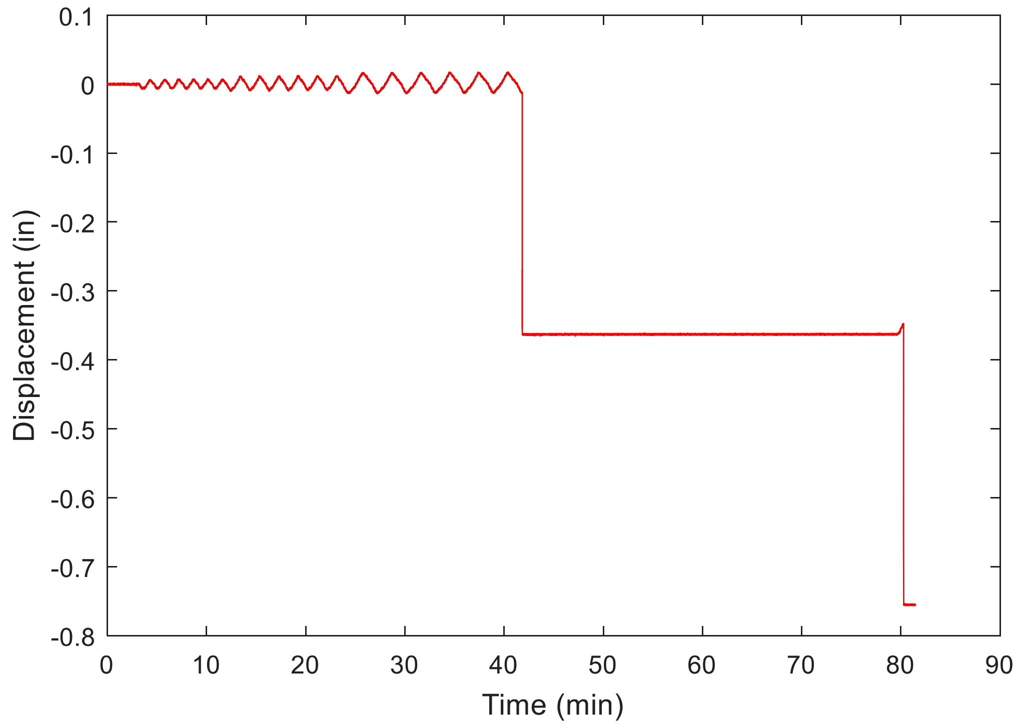


Figure S-24 SP07

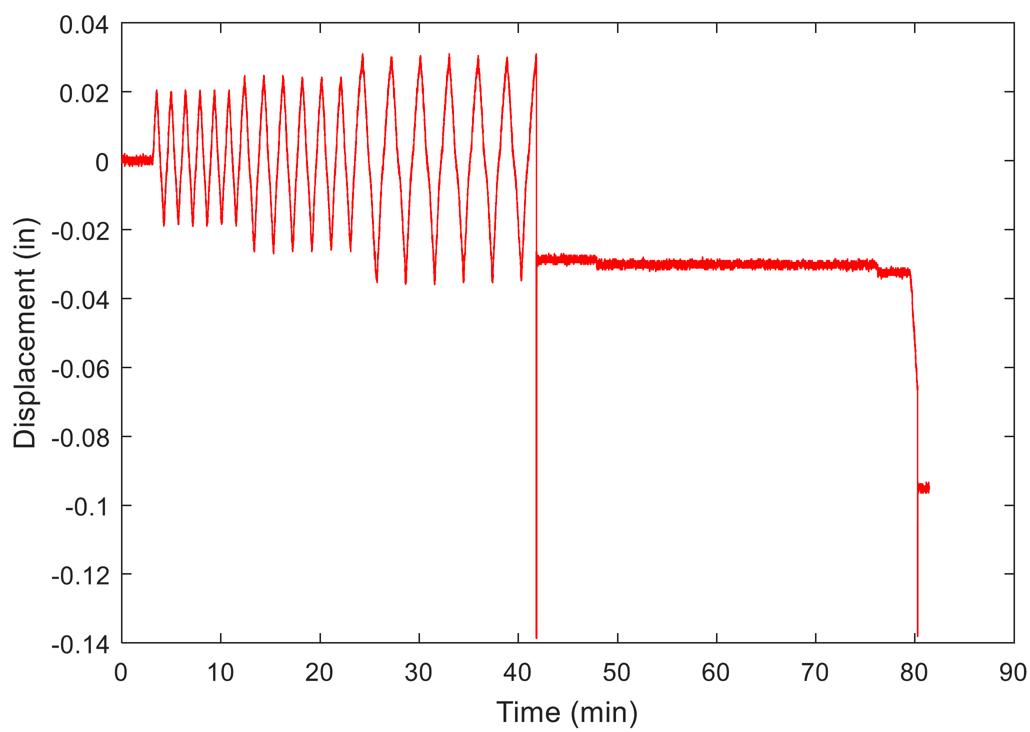


Figure S-25 SP08

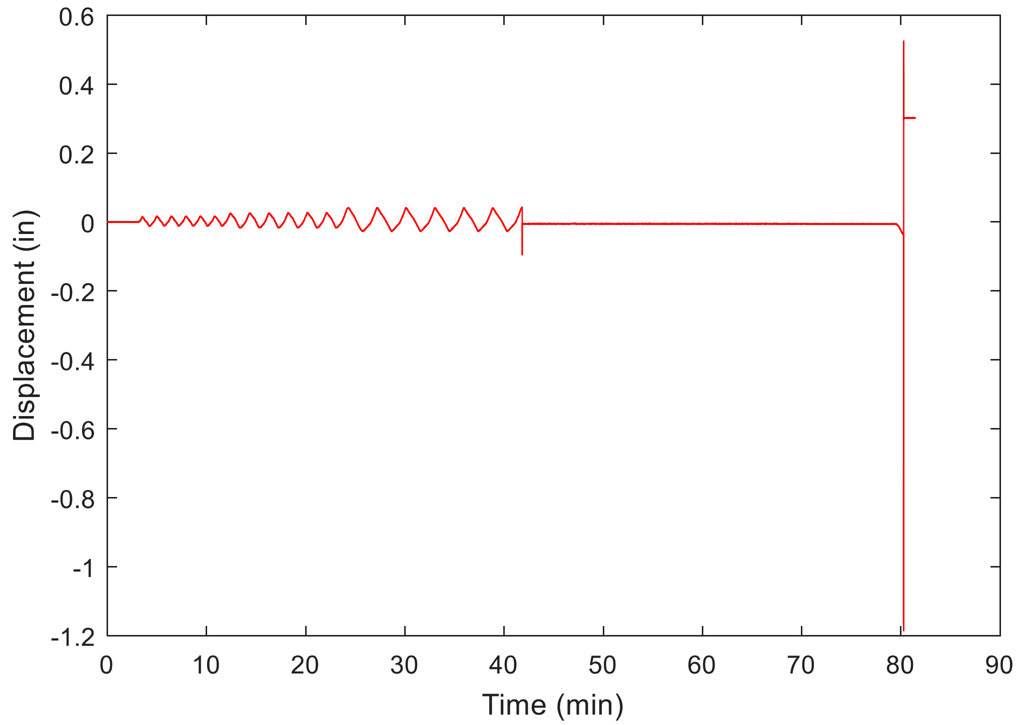


Figure S-26 SP09

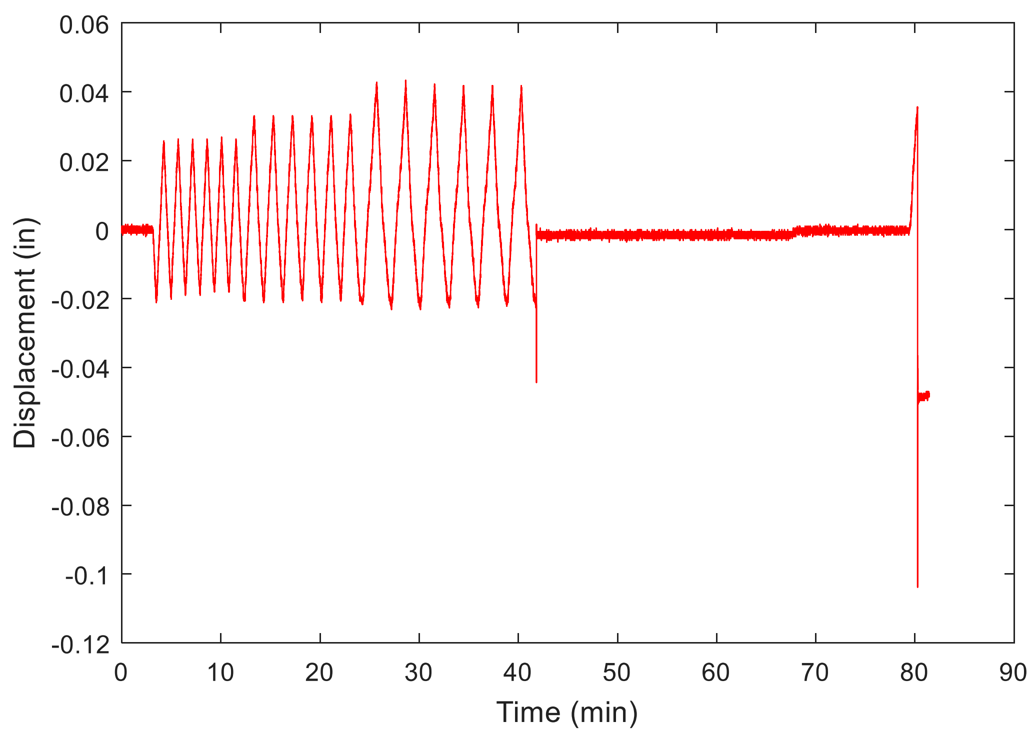


Figure S-27 SP10

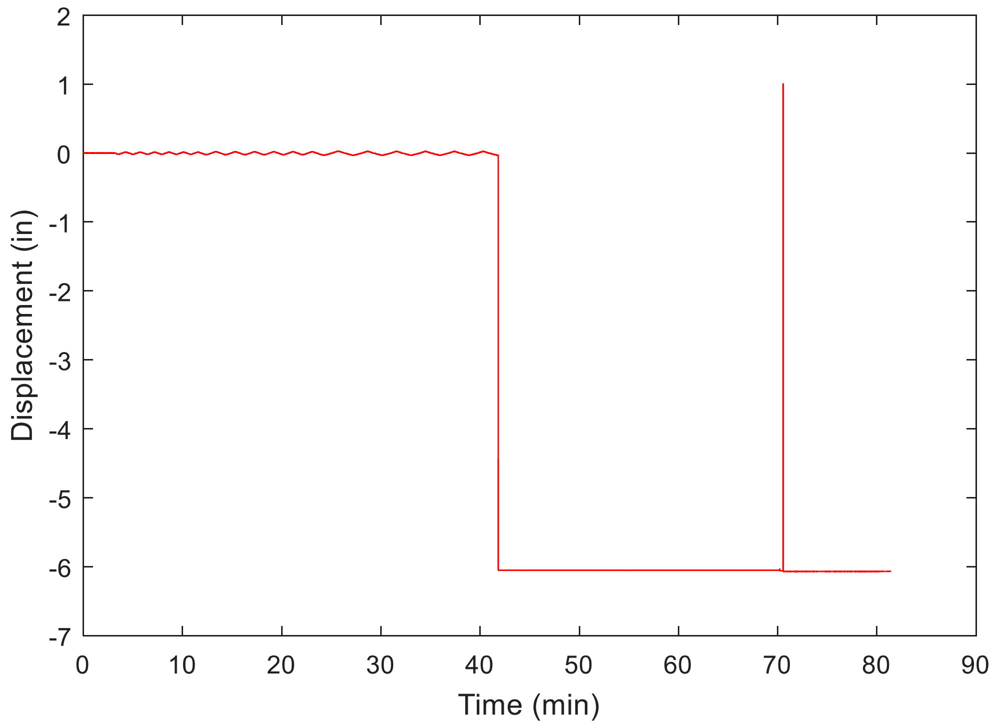


Figure S-28 SP11

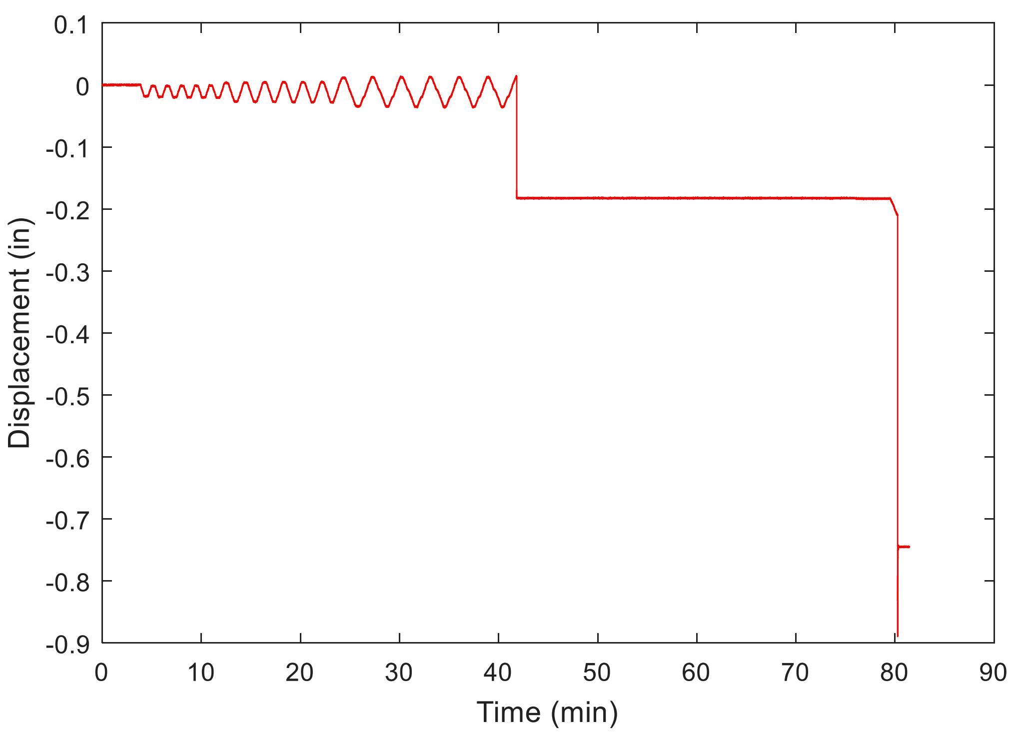


Figure S-29 SP12

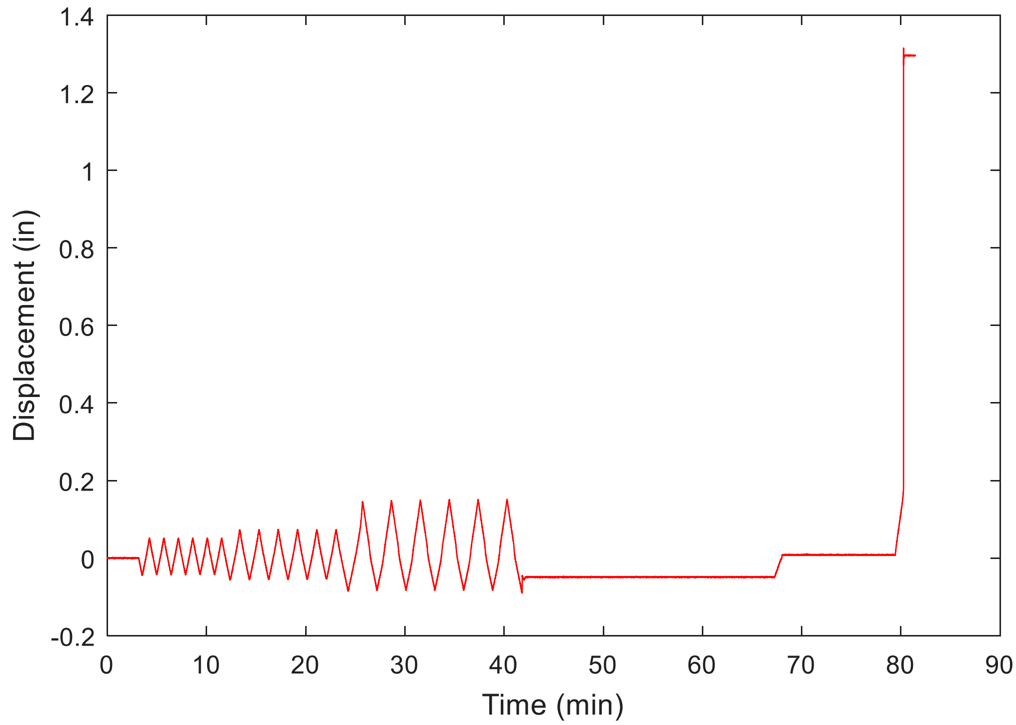


Figure S-30 SP13

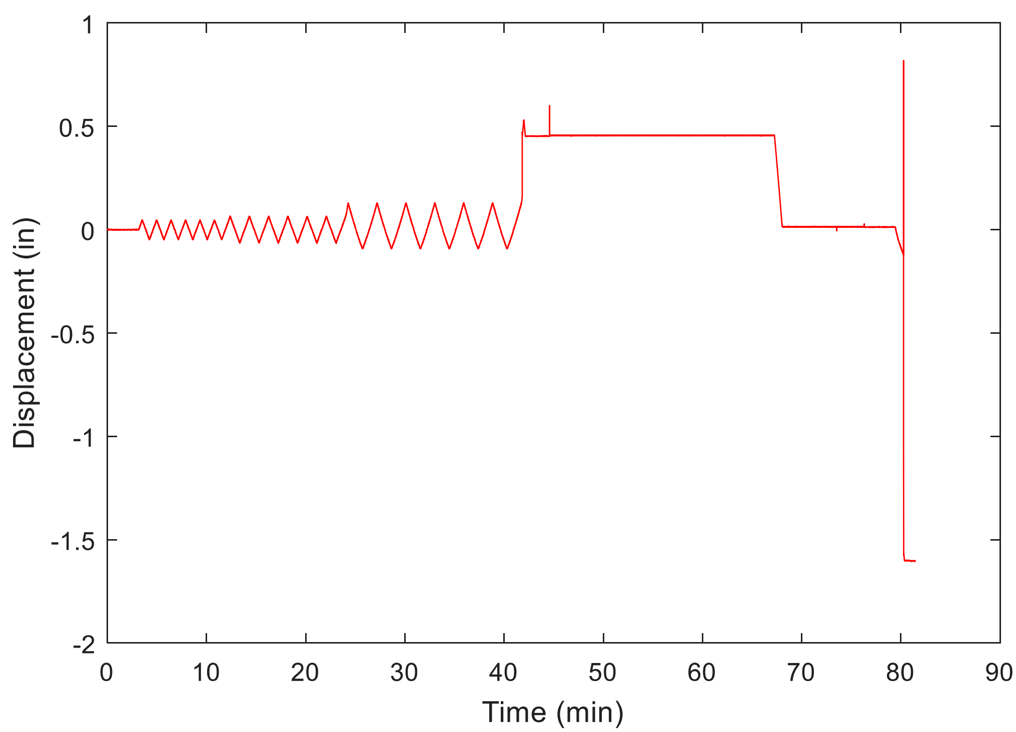


Figure S-31 SP14

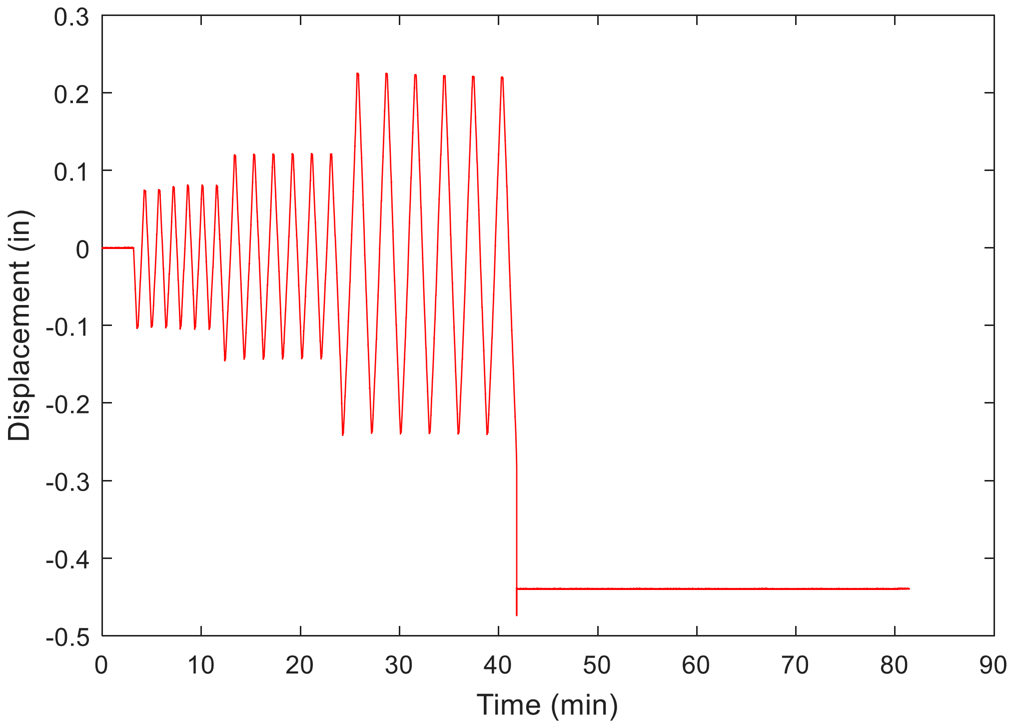


Figure S-32 LP02

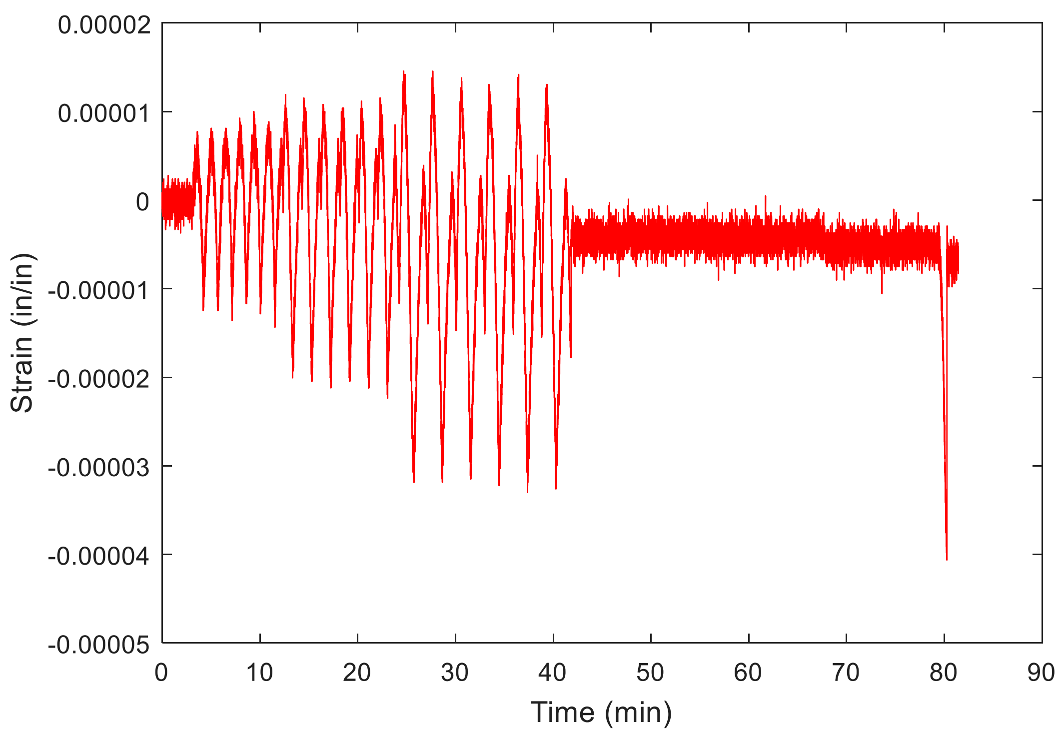


Figure S-33 SG01

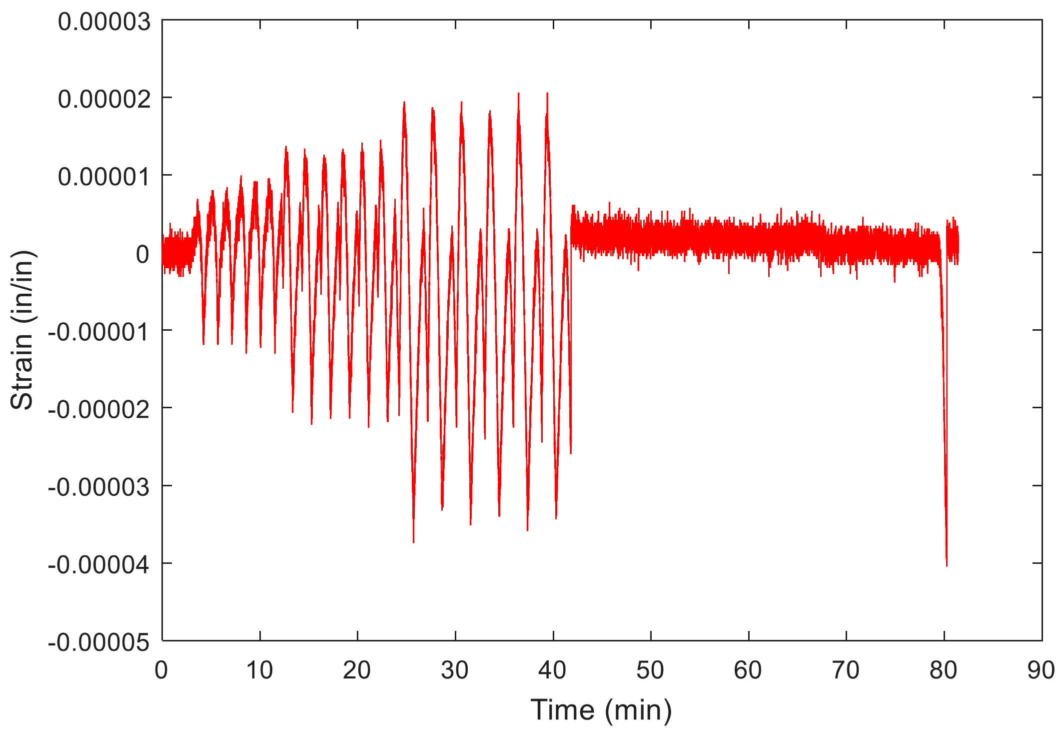


Figure S-34 SG02

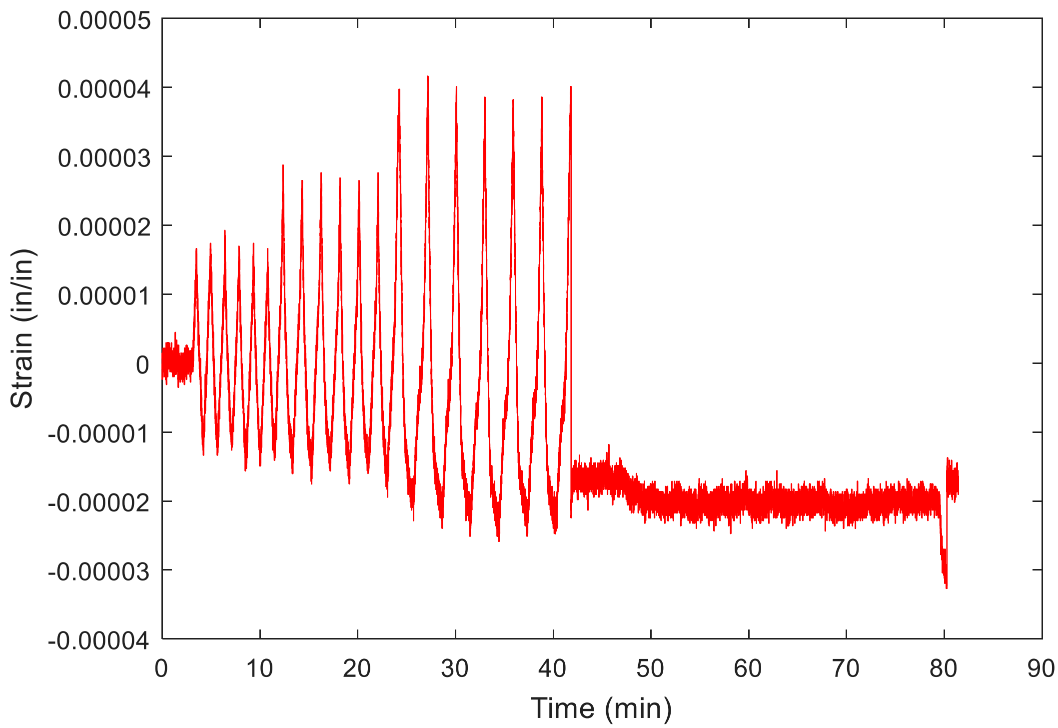


Figure S-35 SG03

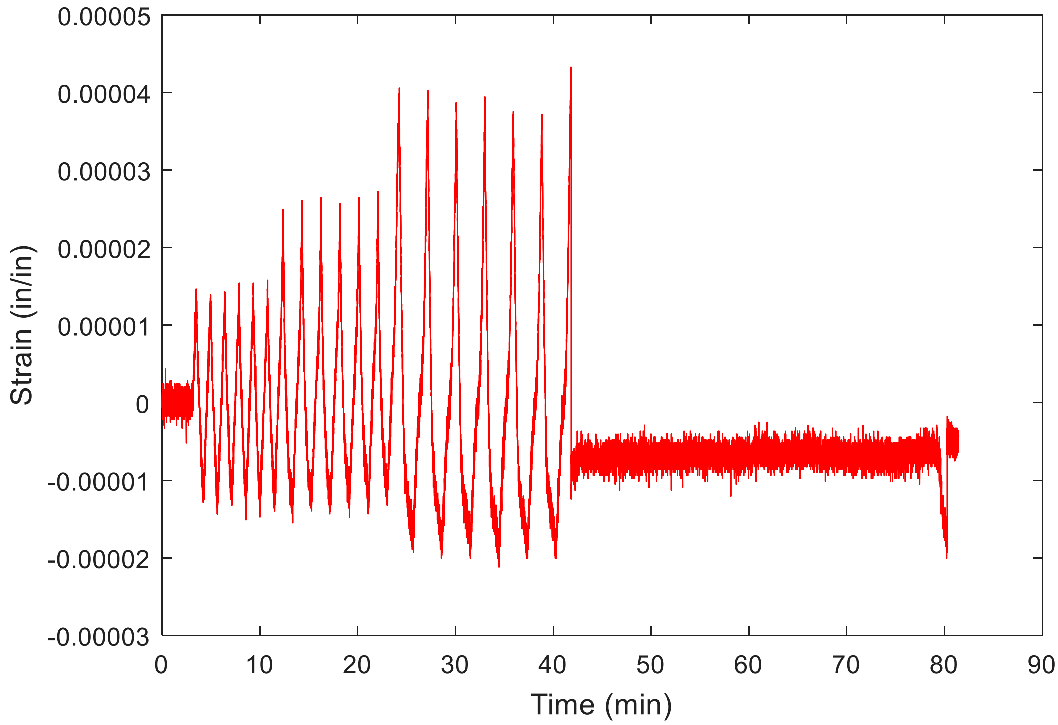


Figure S-36 SG04

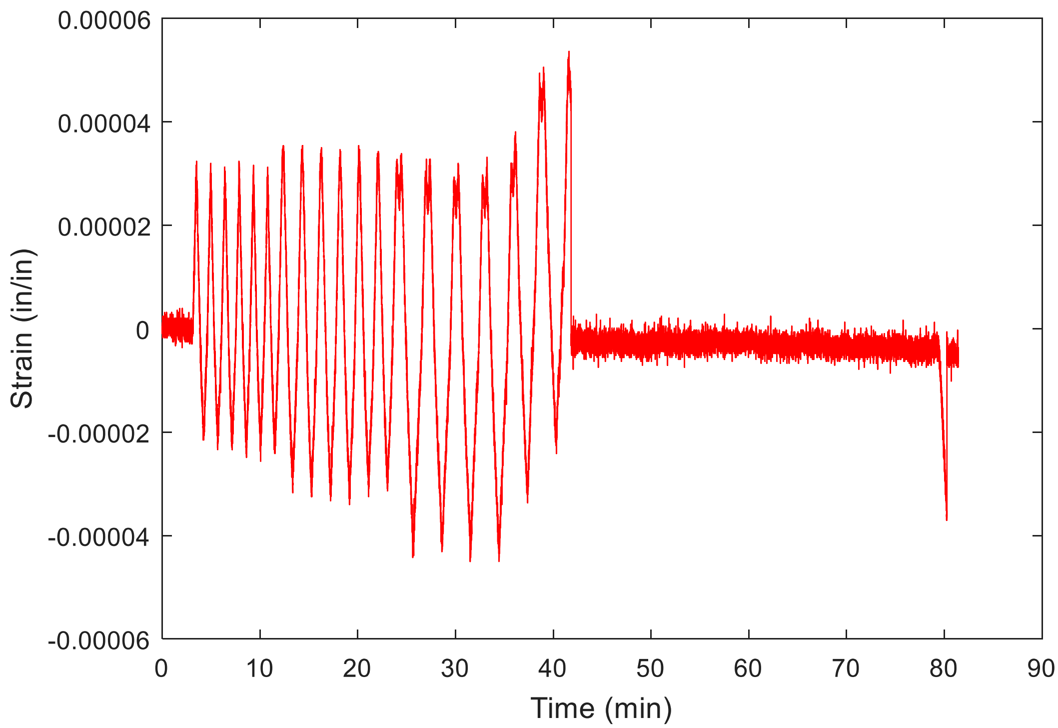


Figure S-37 SG05

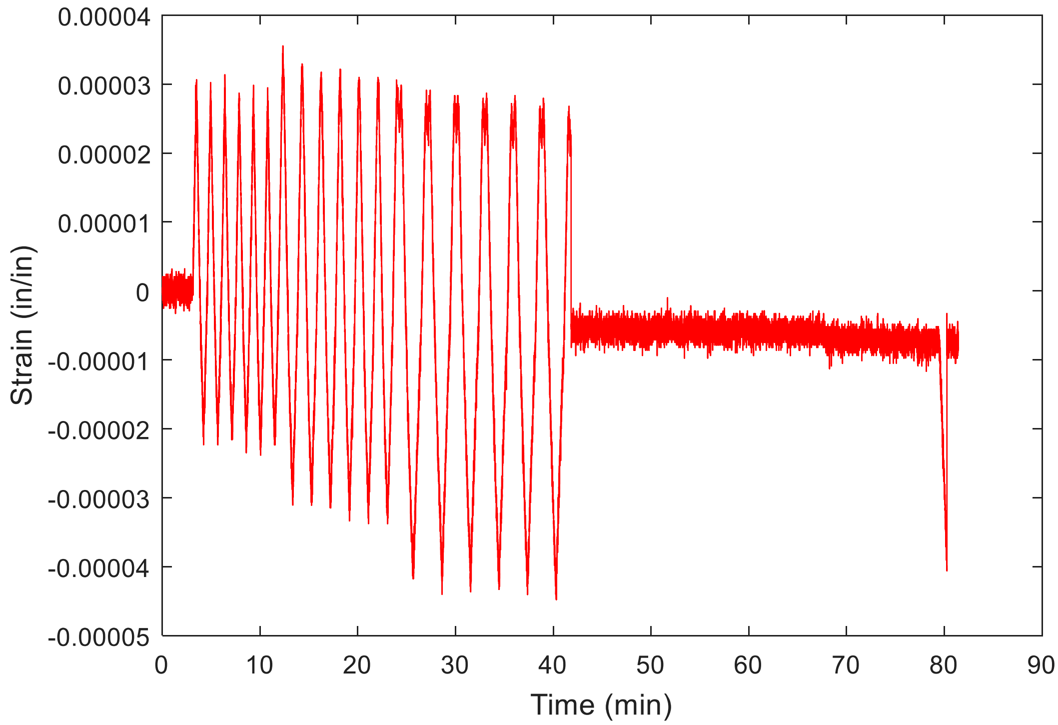


Figure S-38 SG06

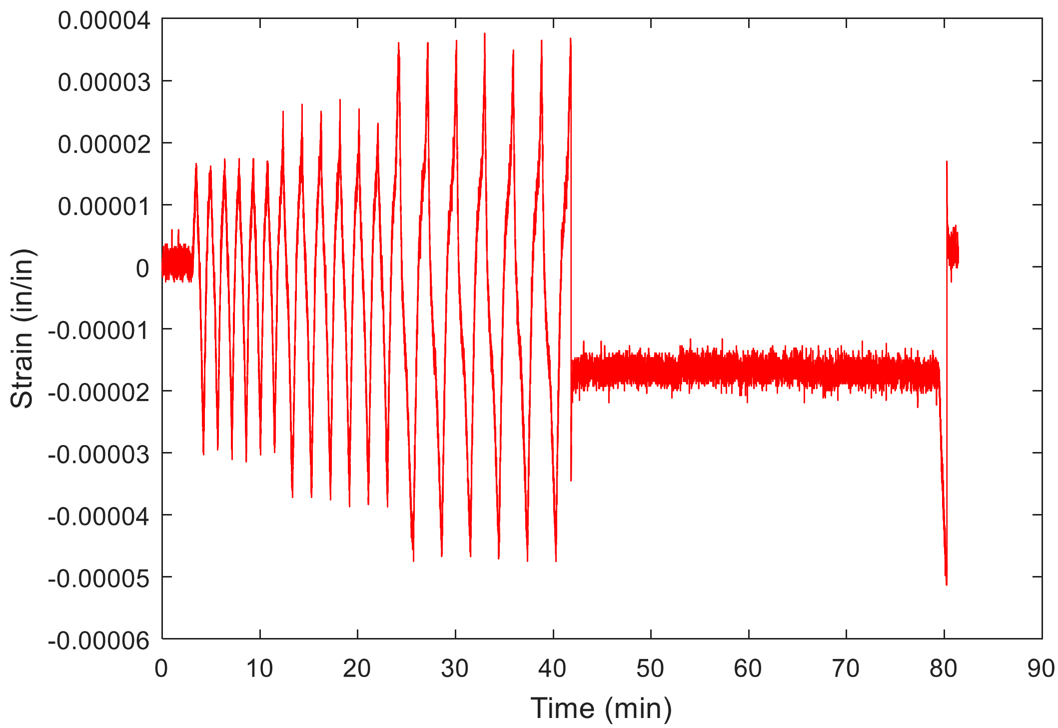


Figure S-39 SG07

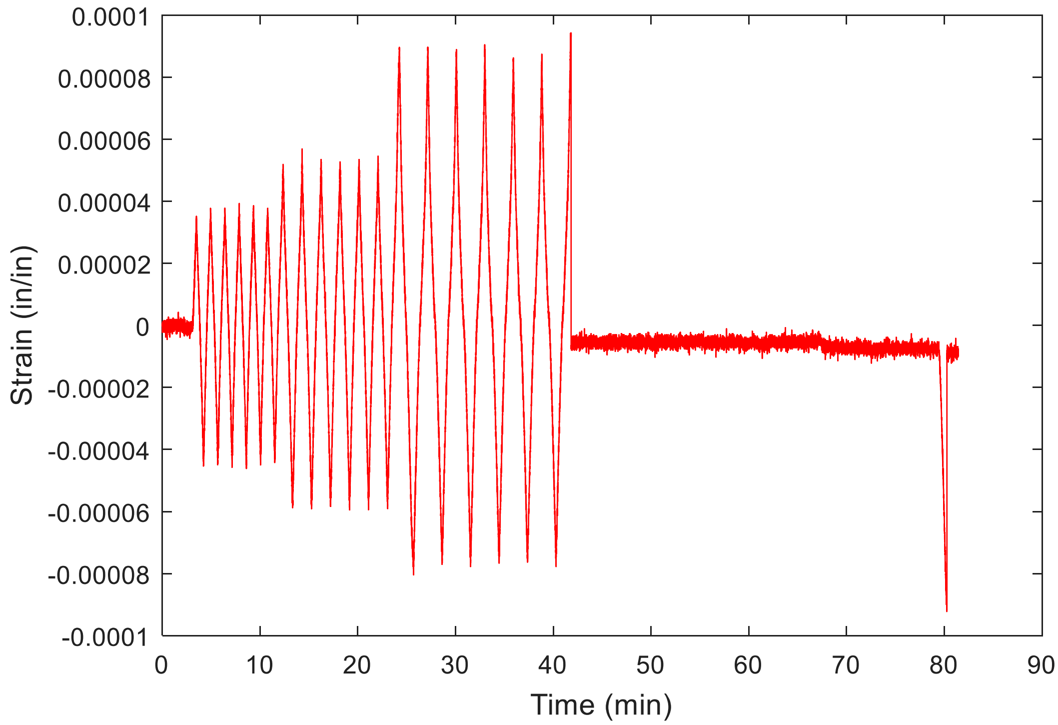


Figure S-40 SG08

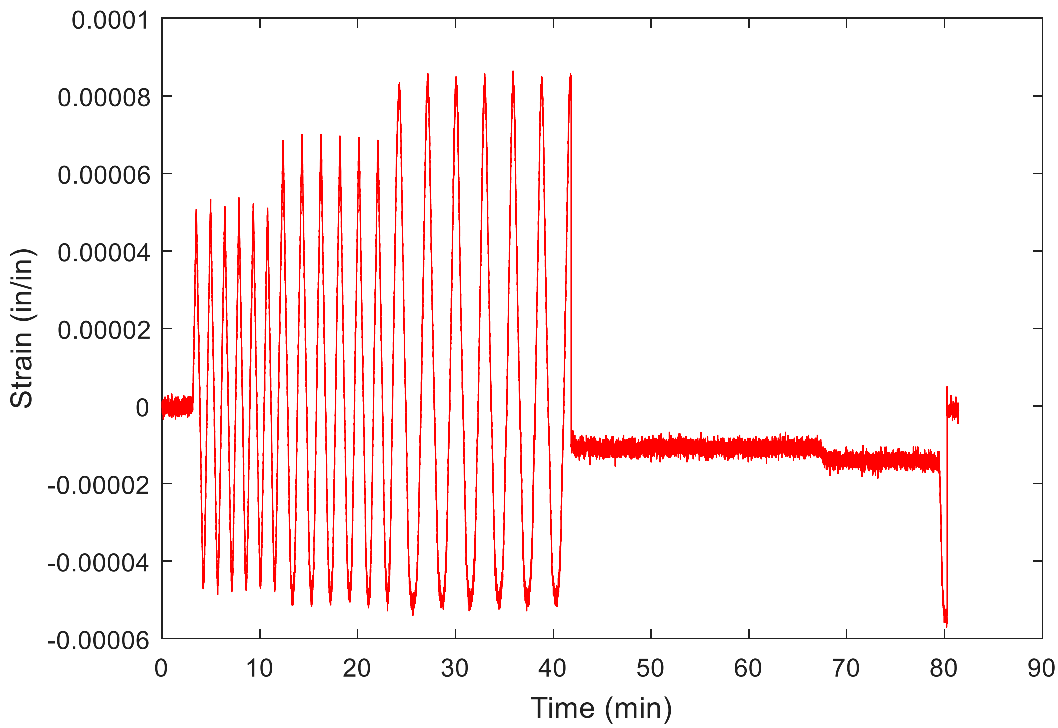


Figure S-41 SG09

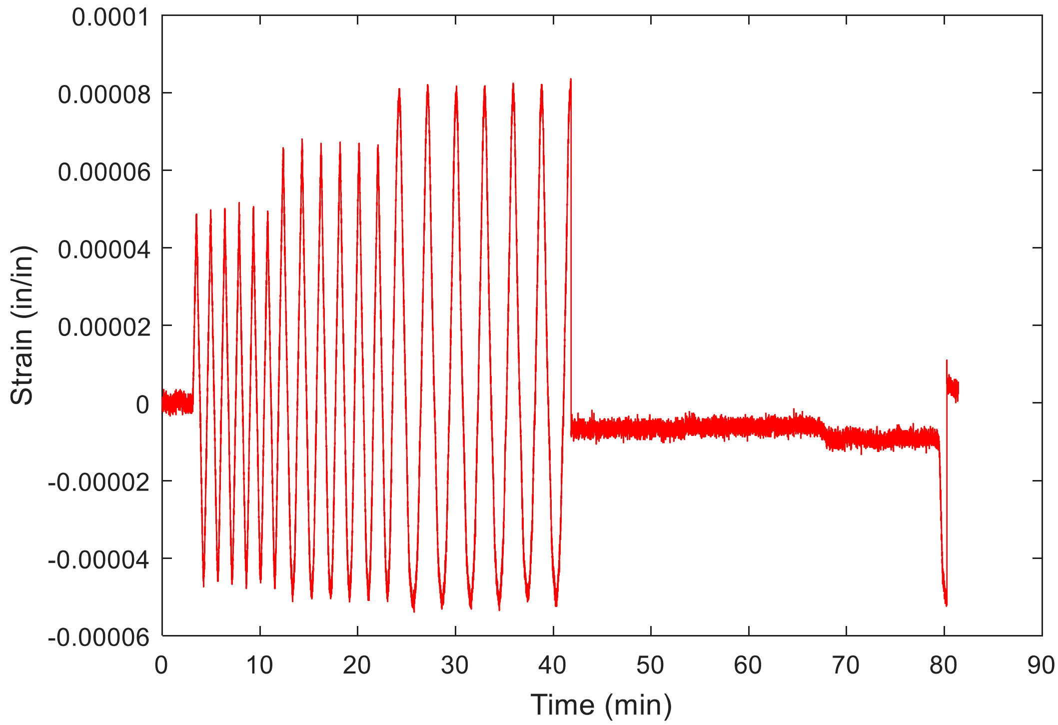


Figure S-42 SG10

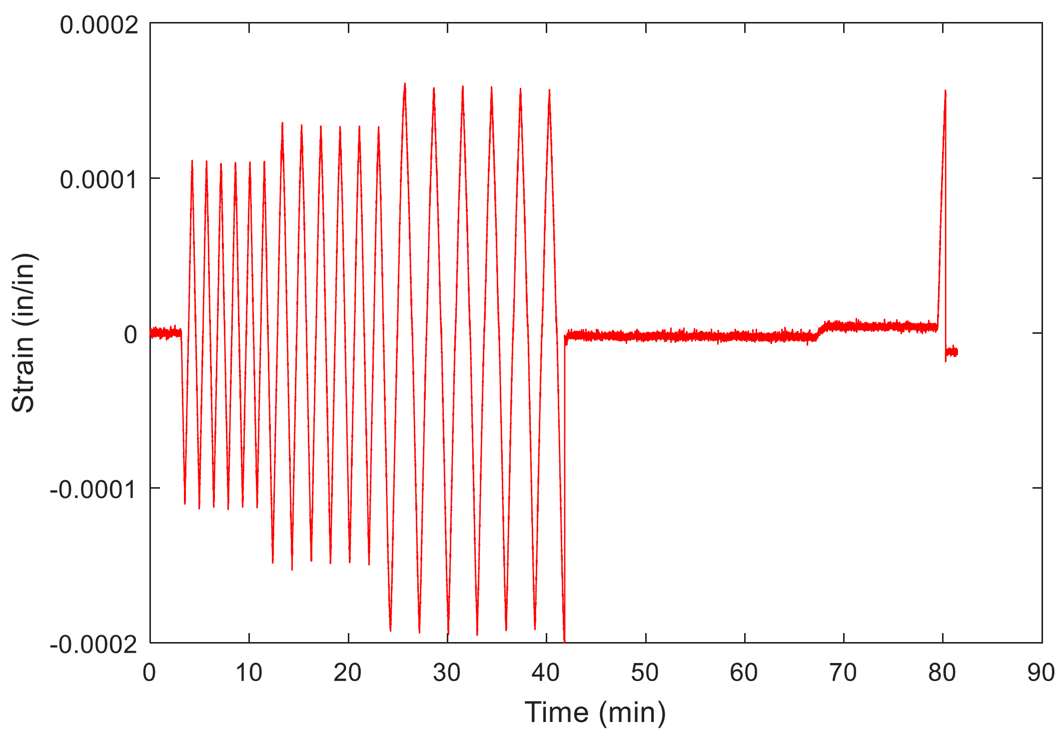


Figure S-43 SG11

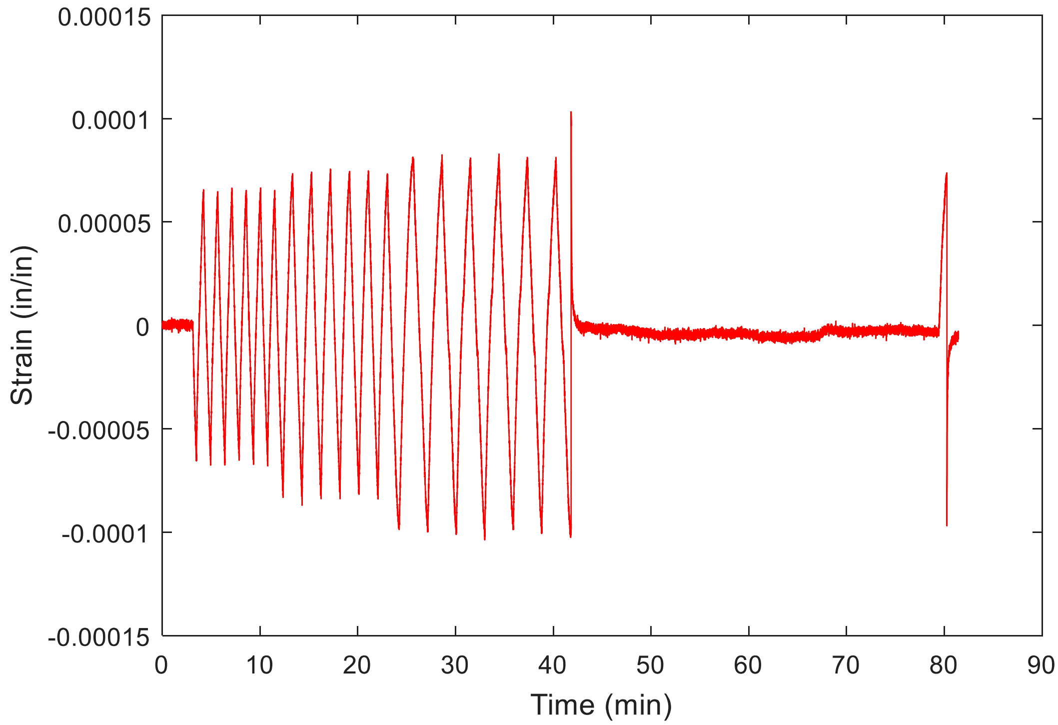


Figure S-44 SG12

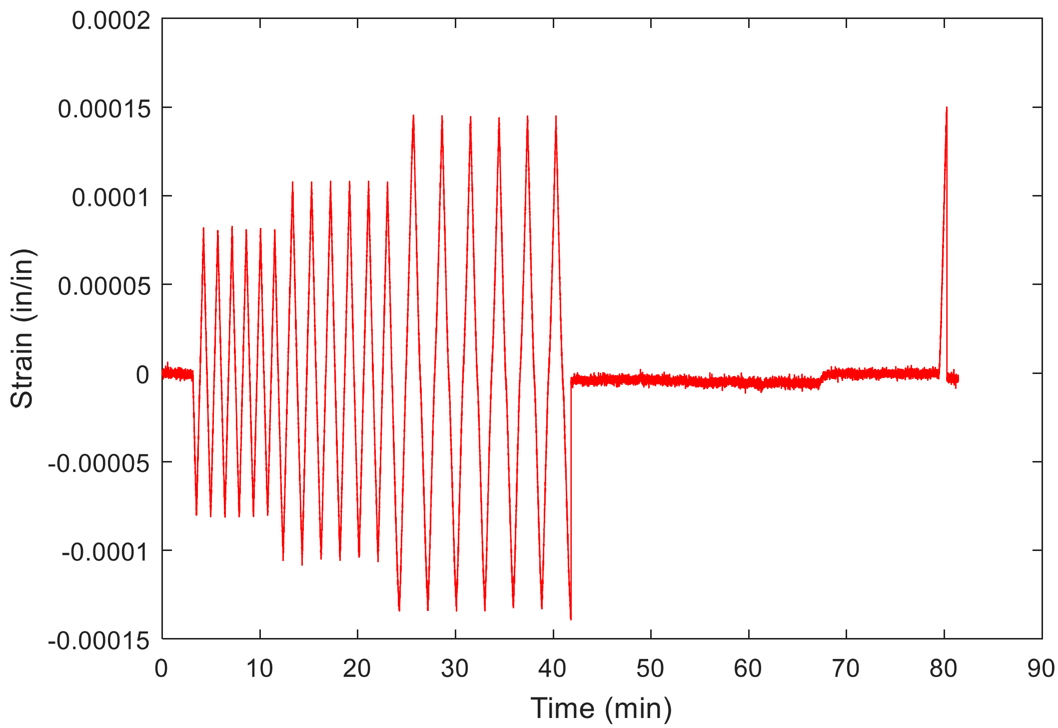


Figure S-45 SG13

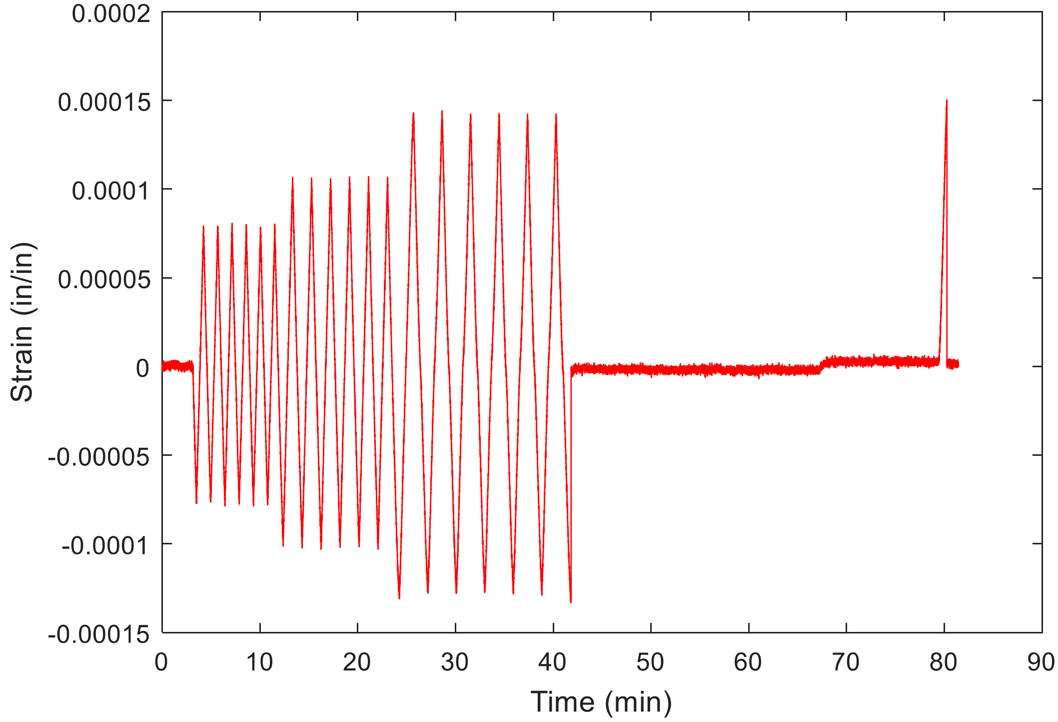


Figure S-46 SG14

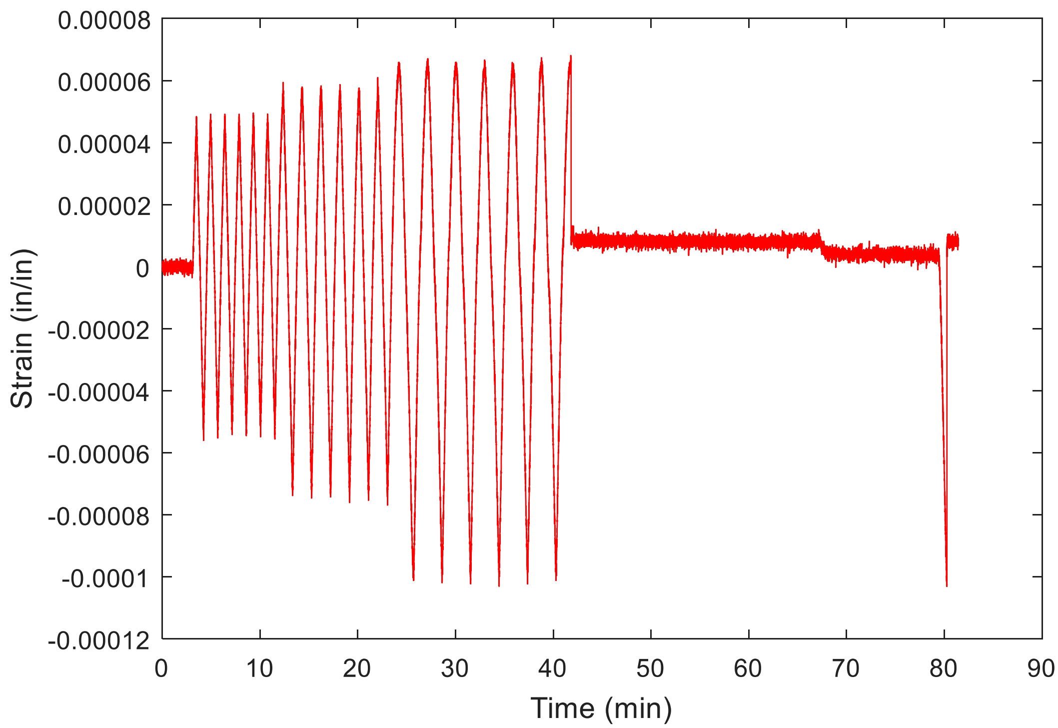


Figure S-47 SG15

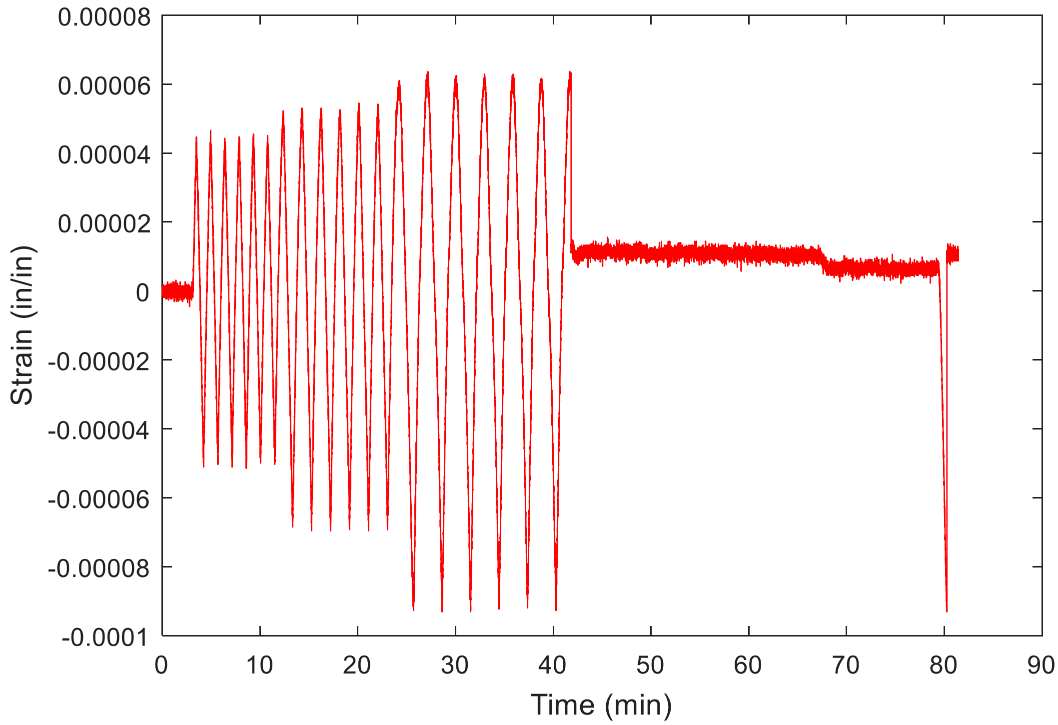


Figure S-48 SG16

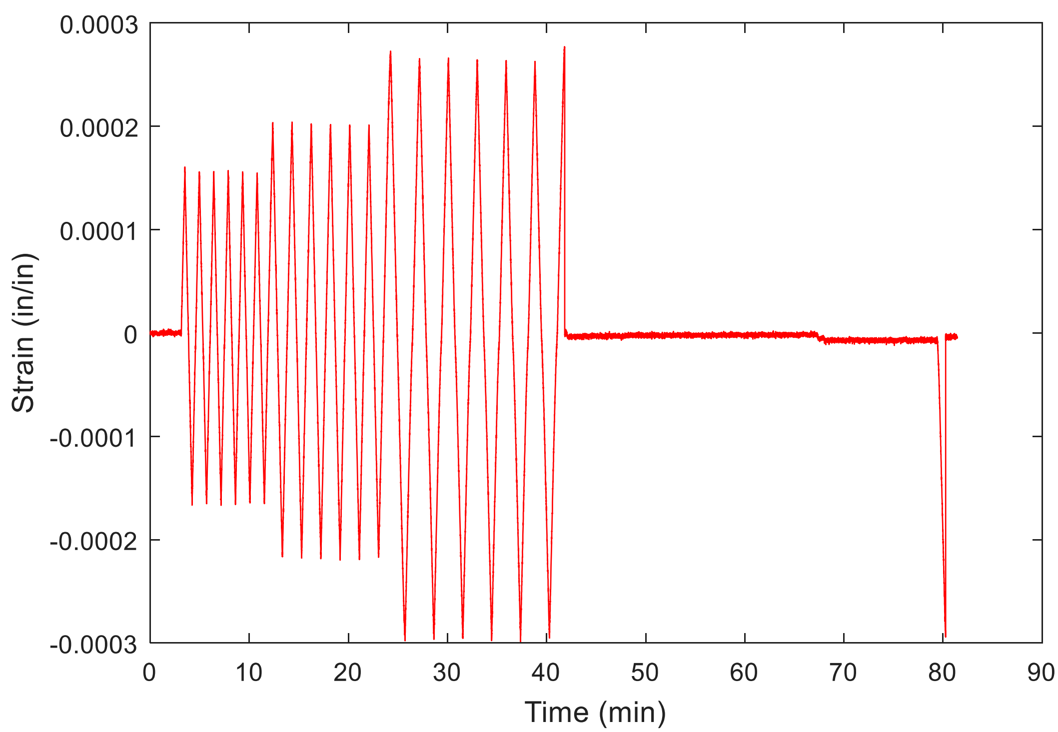


Figure S-49 SG17

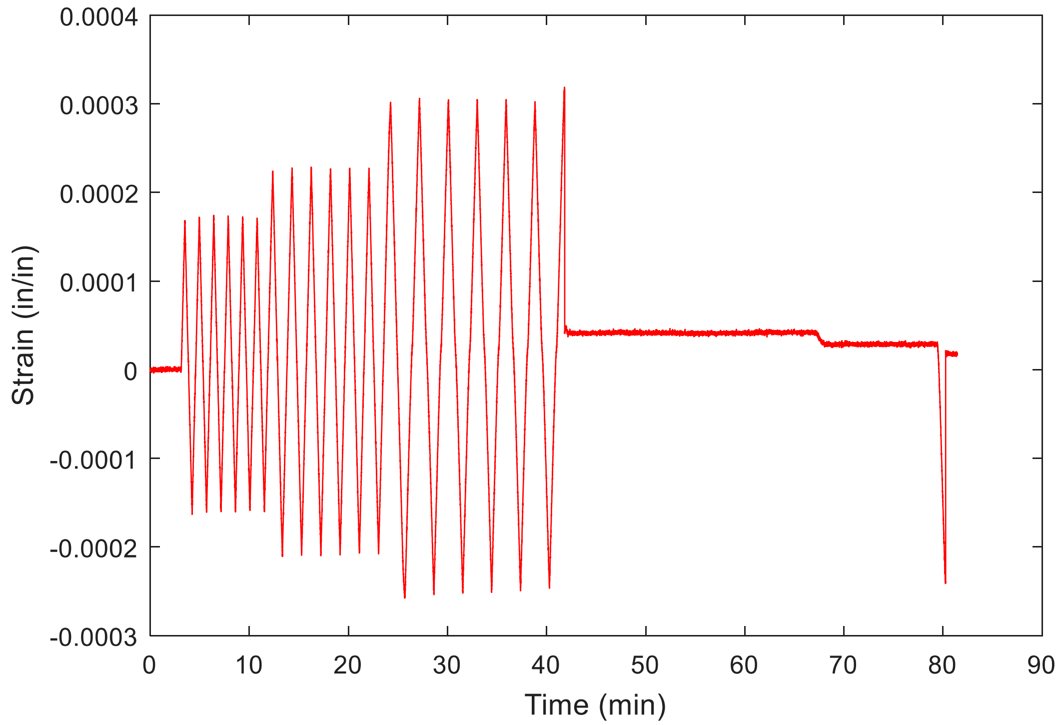


Figure S-50 SG18

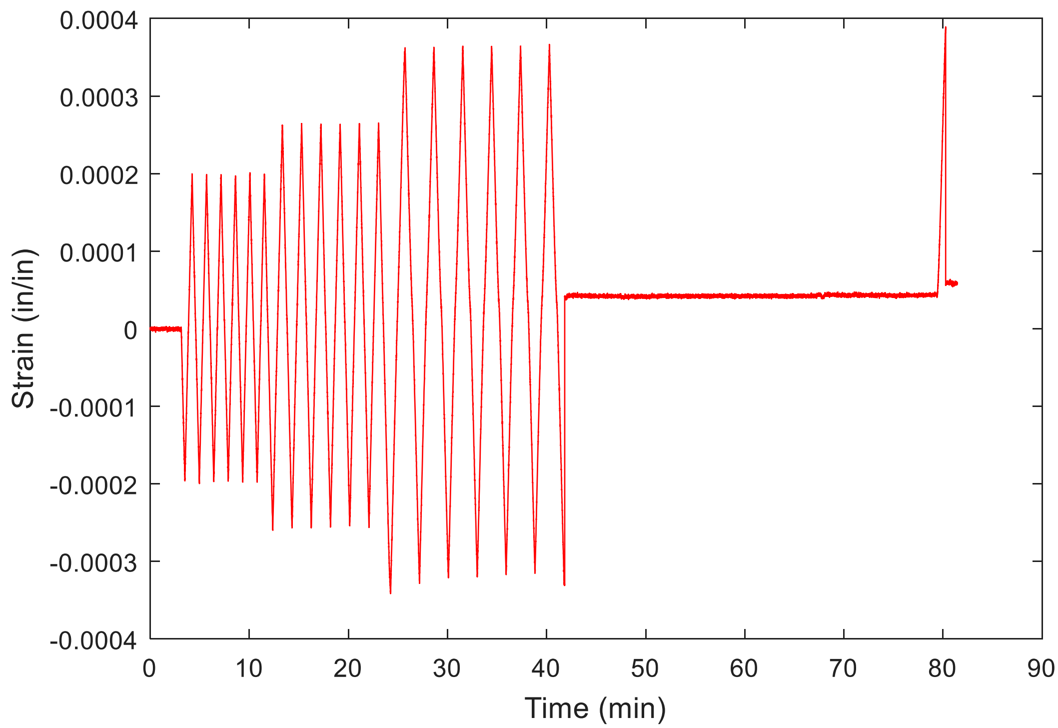


Figure S-51 SG19

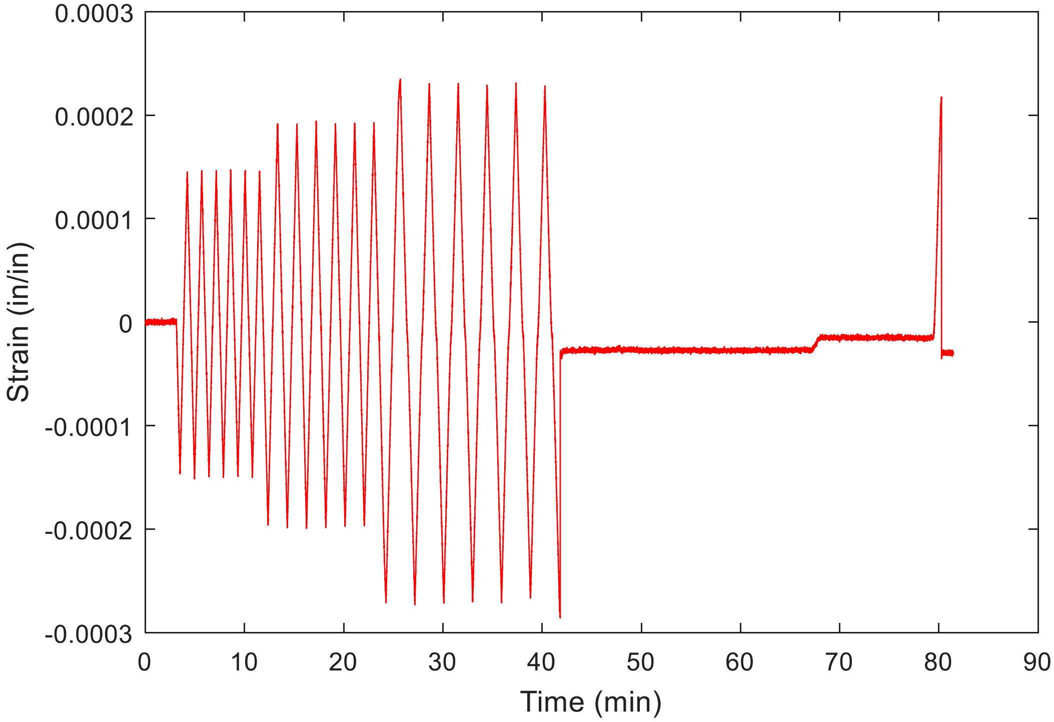


Figure S-52 SG20

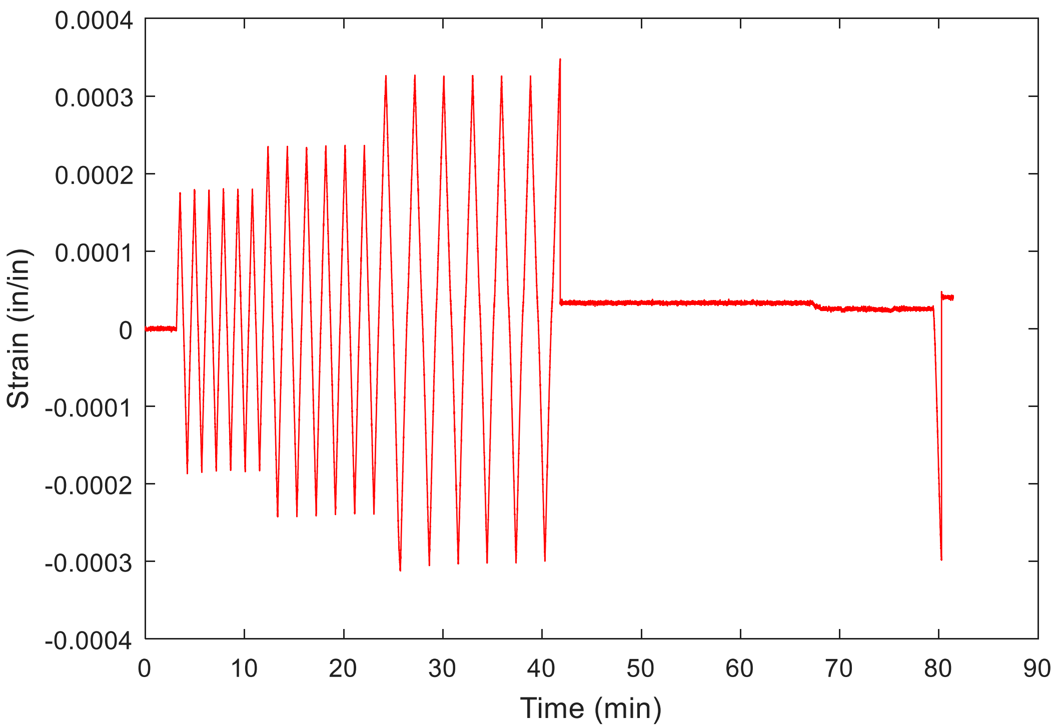


Figure S-53 SG21

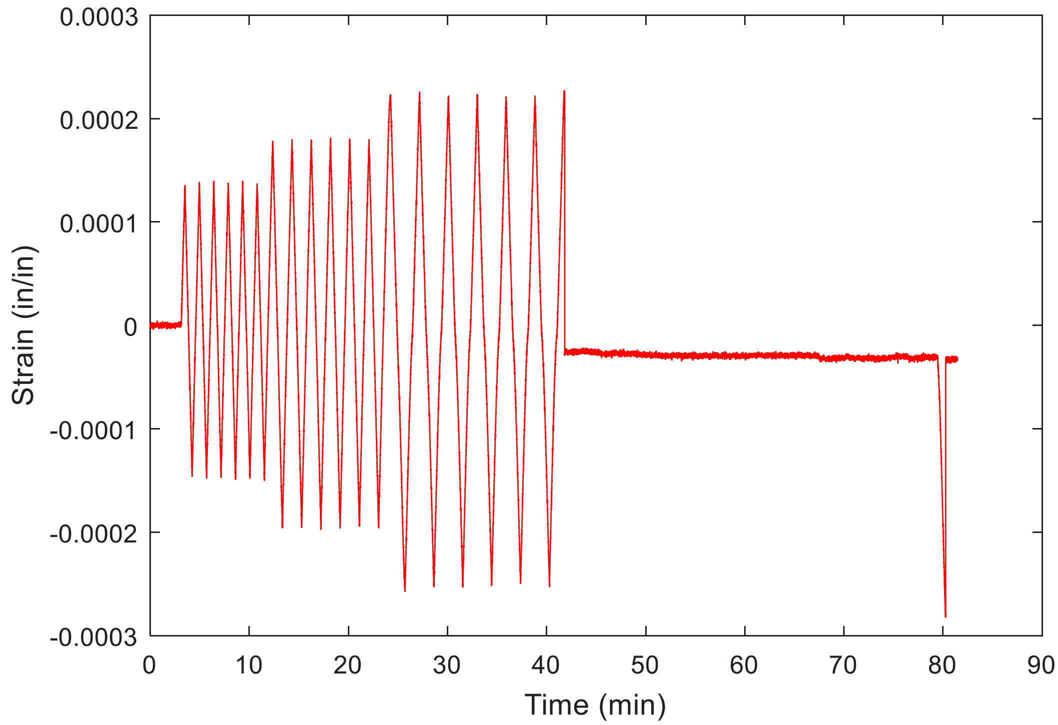


Figure S-54 SG22

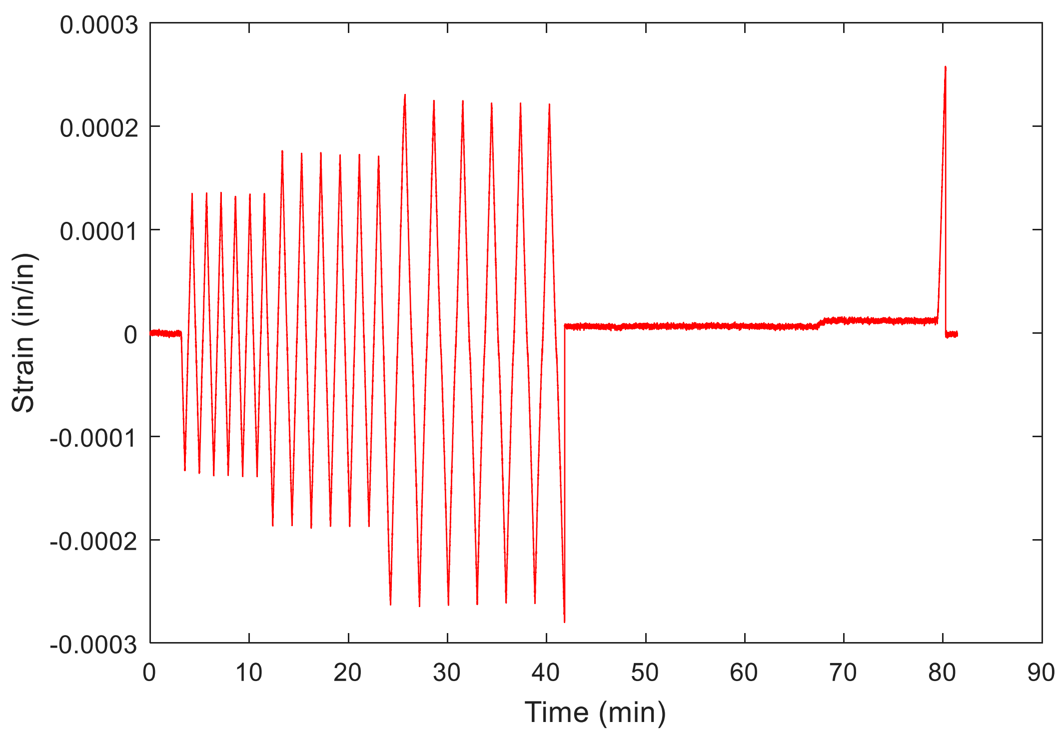


Figure S-55 SG23

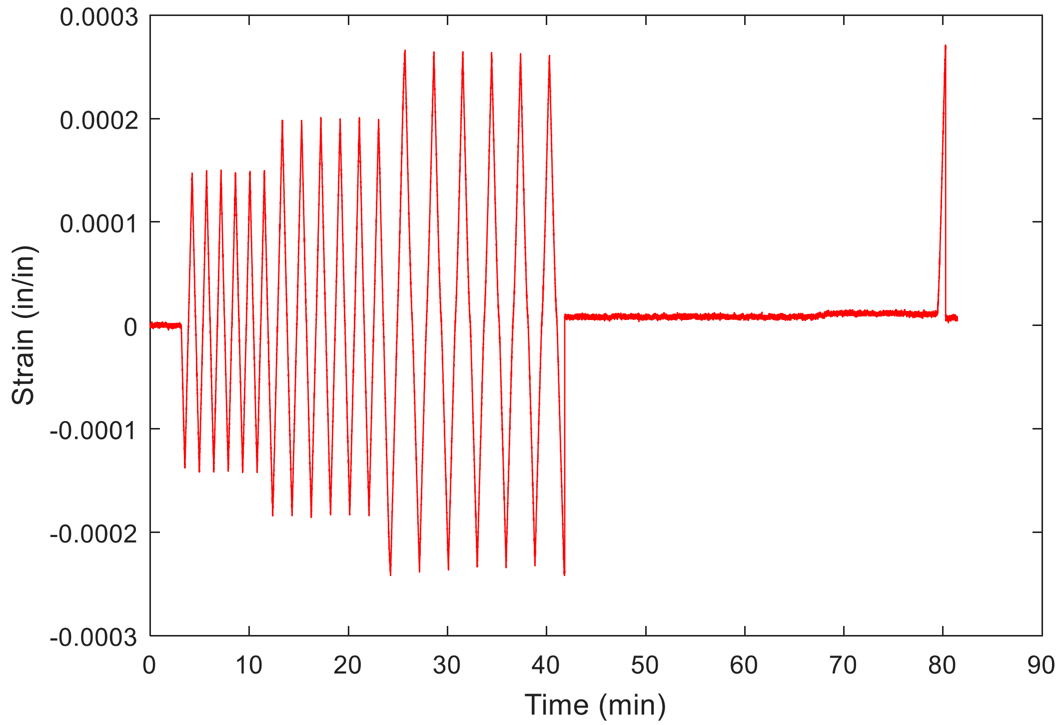


Figure S-56 SG24

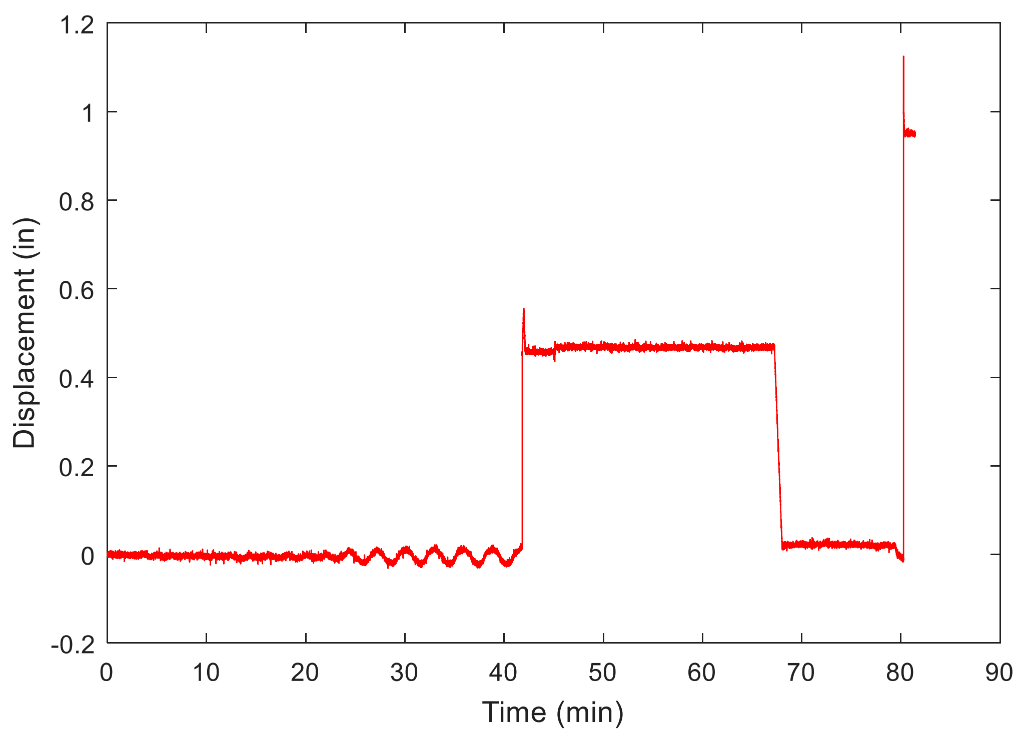


Figure S-57 CLP01

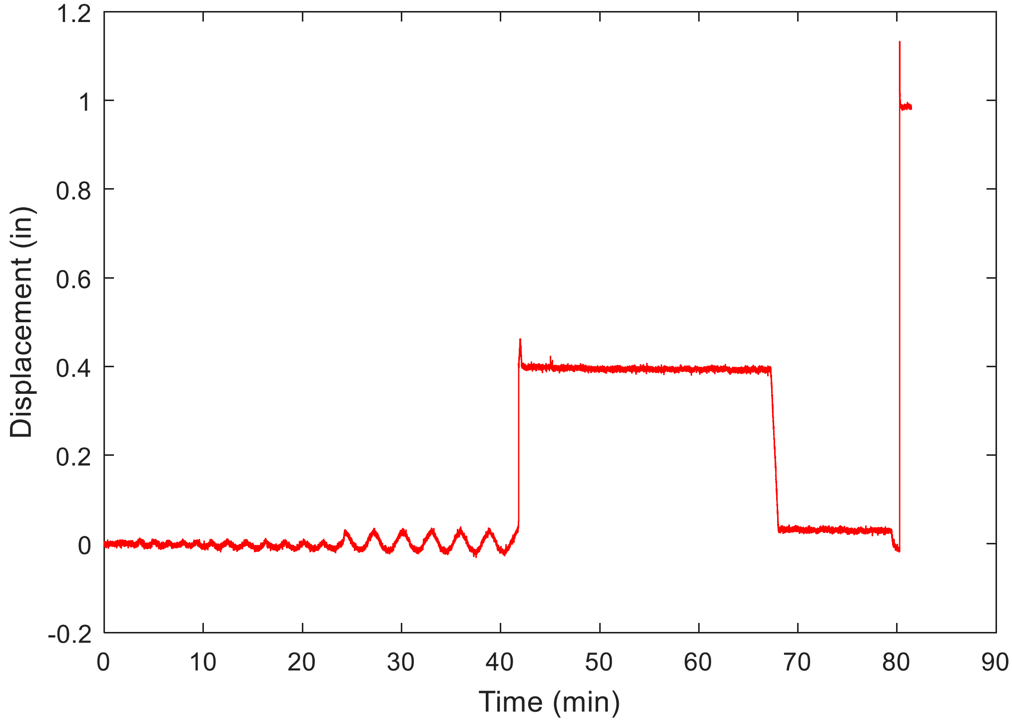


Figure S-58 CLP02

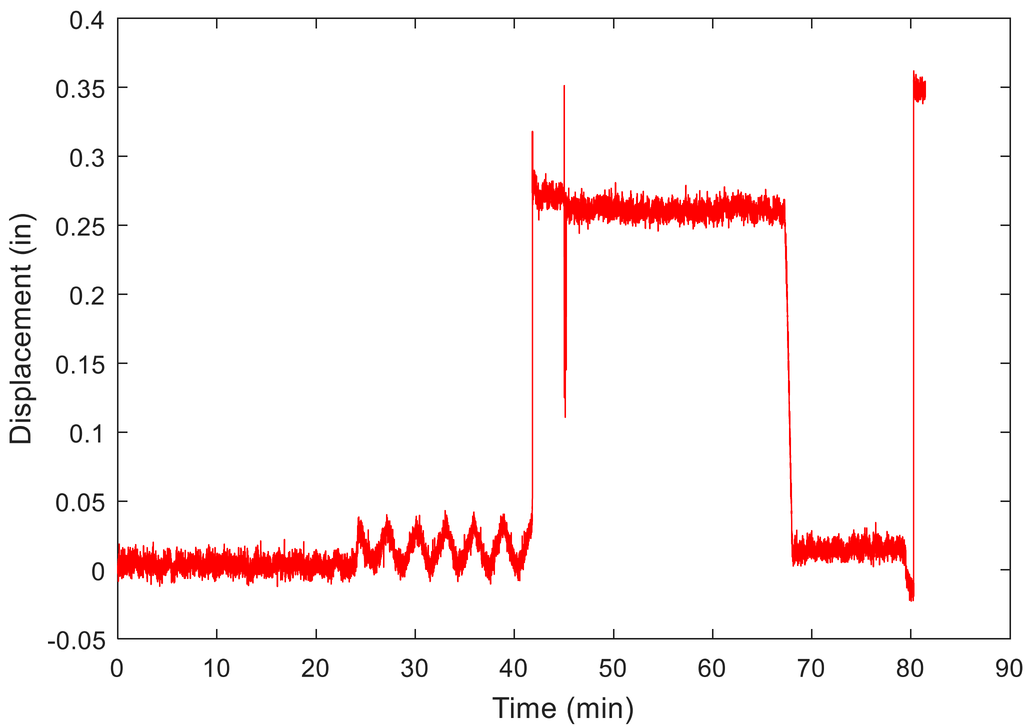


Figure S-59 CLP03

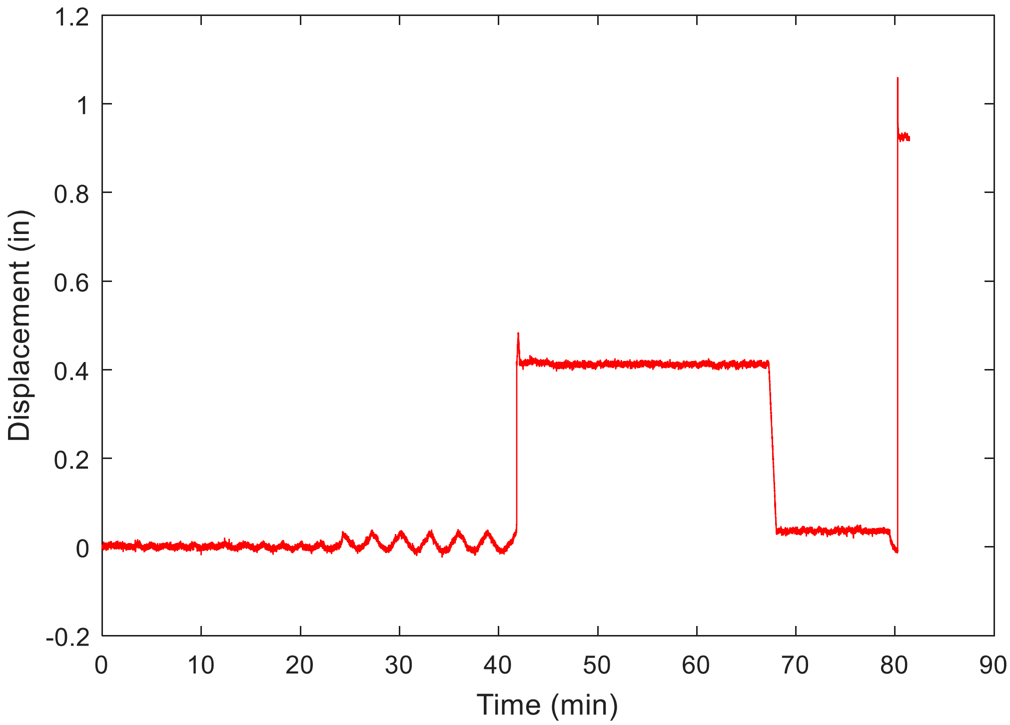


Figure S-60 CLP04

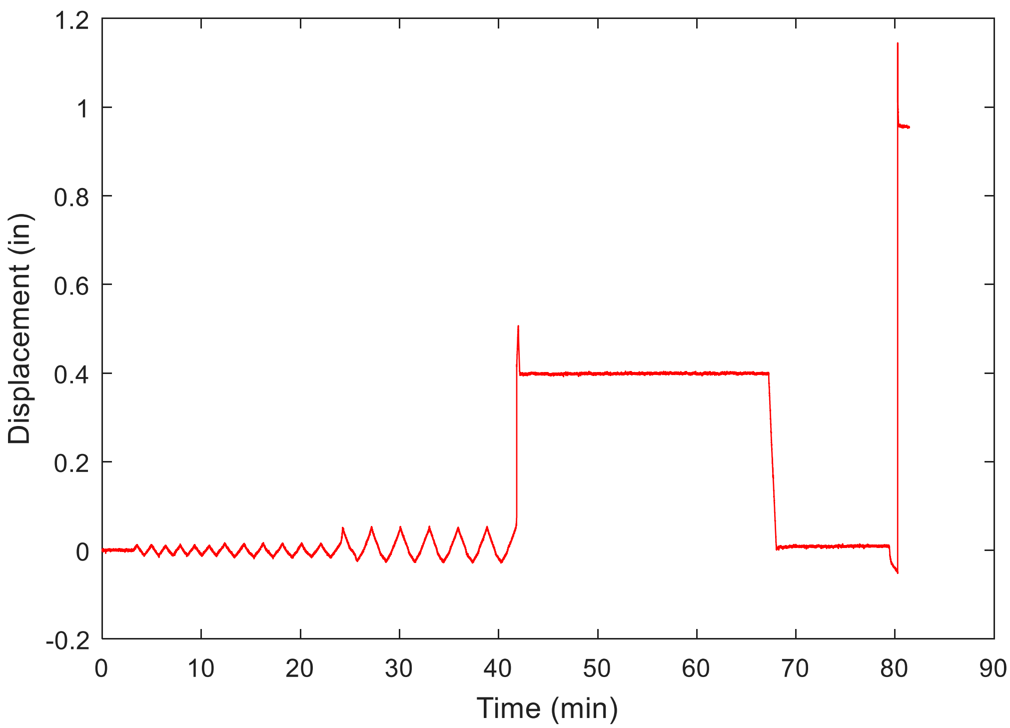


Figure S-61 CLP05

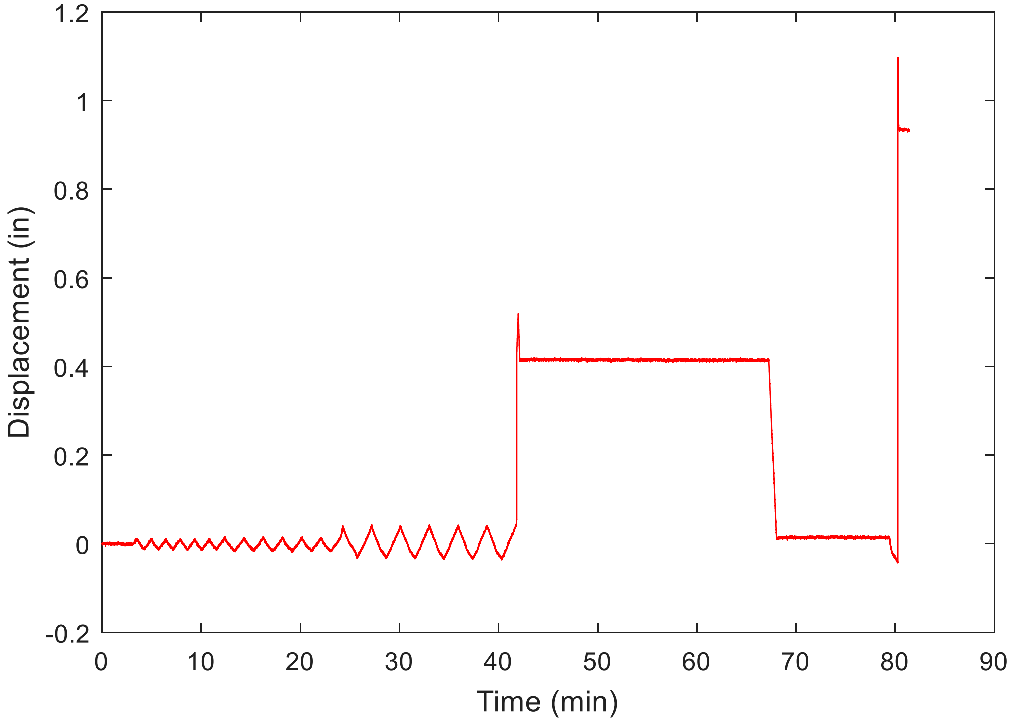


Figure S-62 CLP06

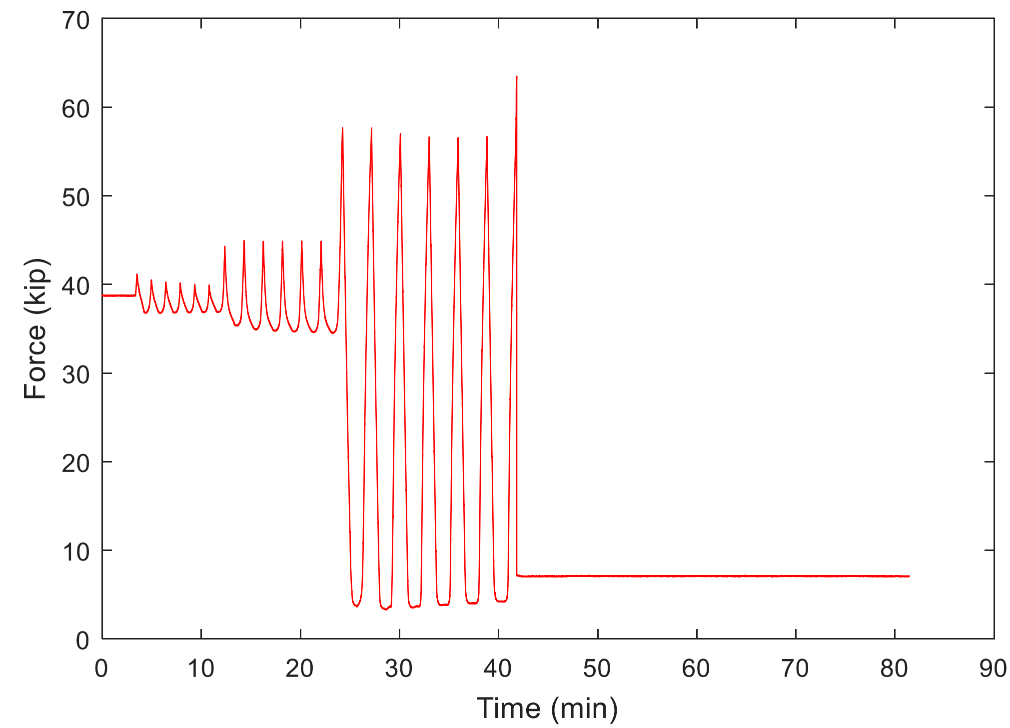


Figure S-63 Force in BSG01

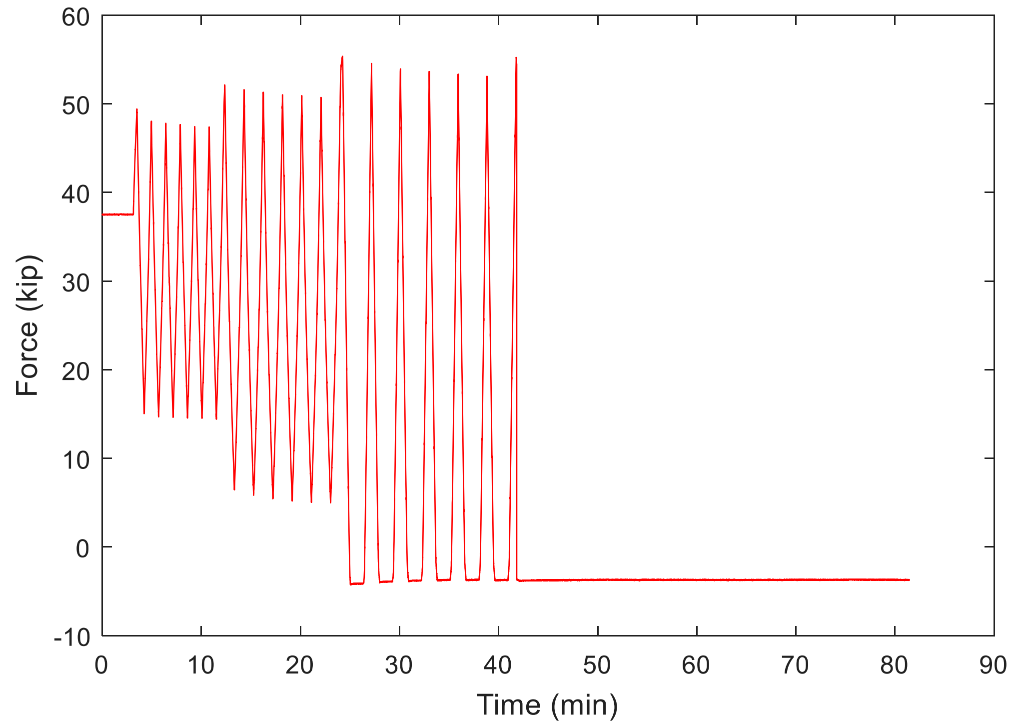


Figure S-64 Force in BSG02

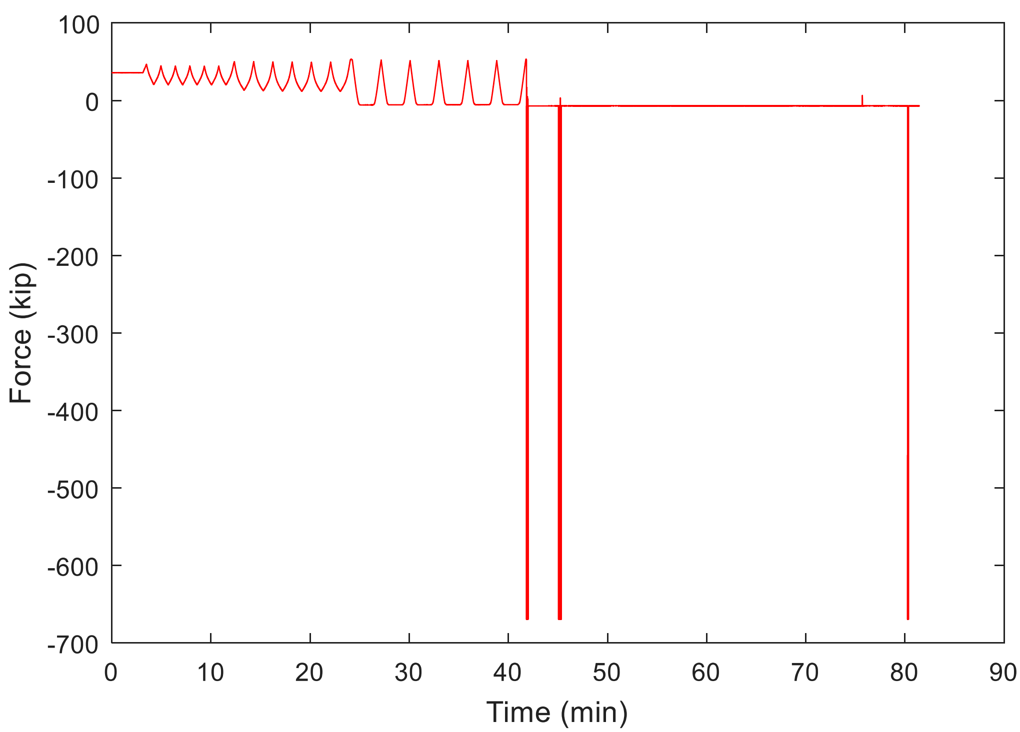


Figure S-65 Force in BSG03

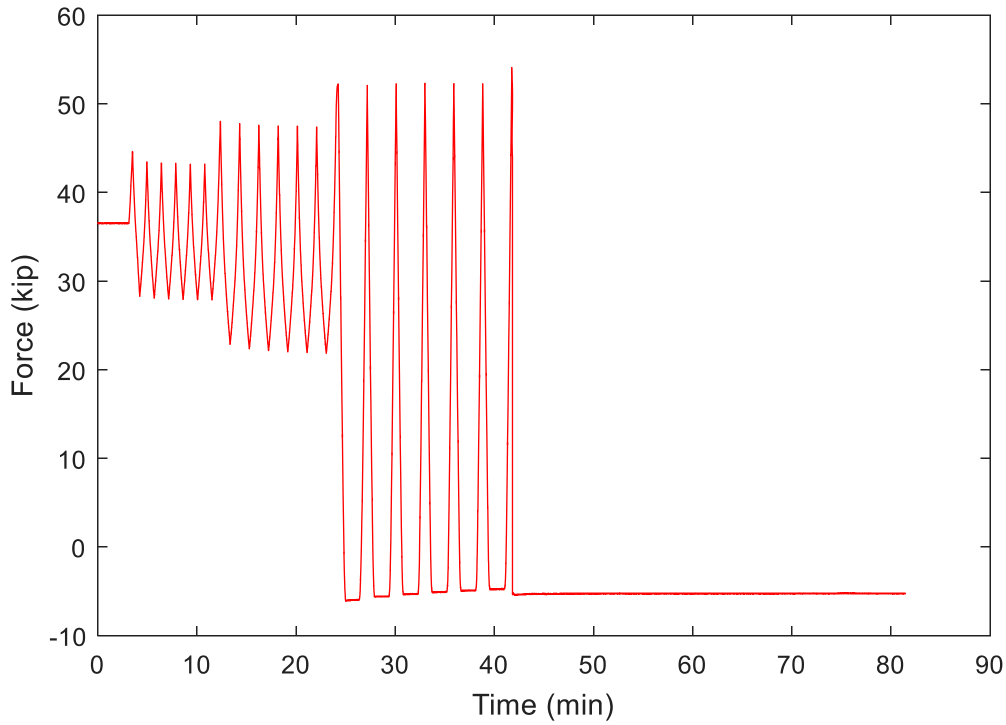


Figure S-66 Force in BSG04

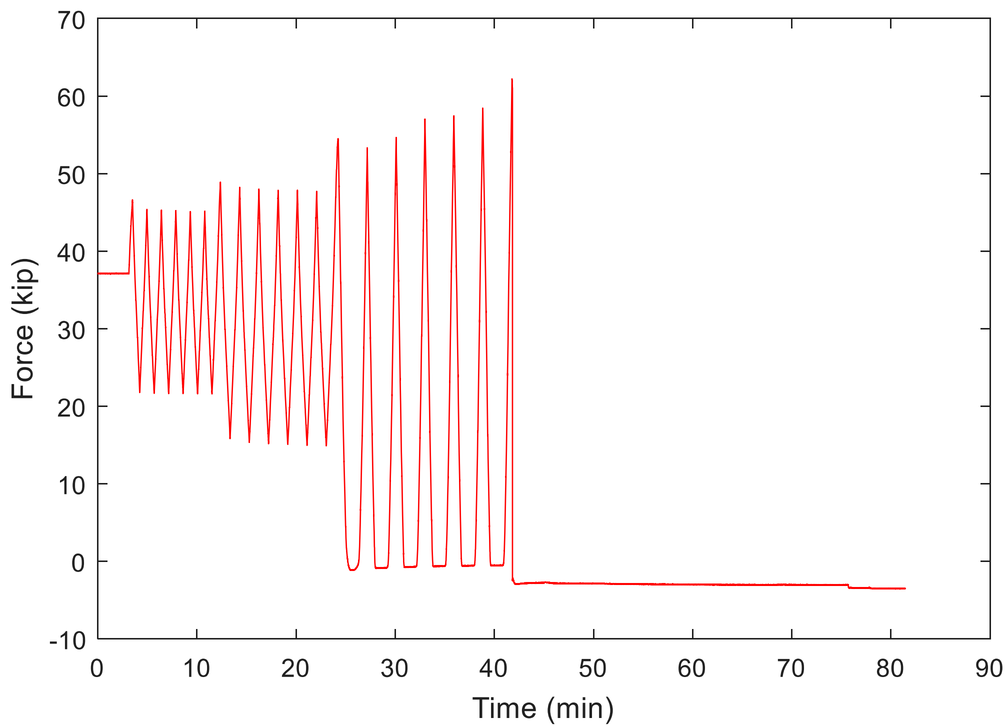


Figure S-67 Force in BSG05

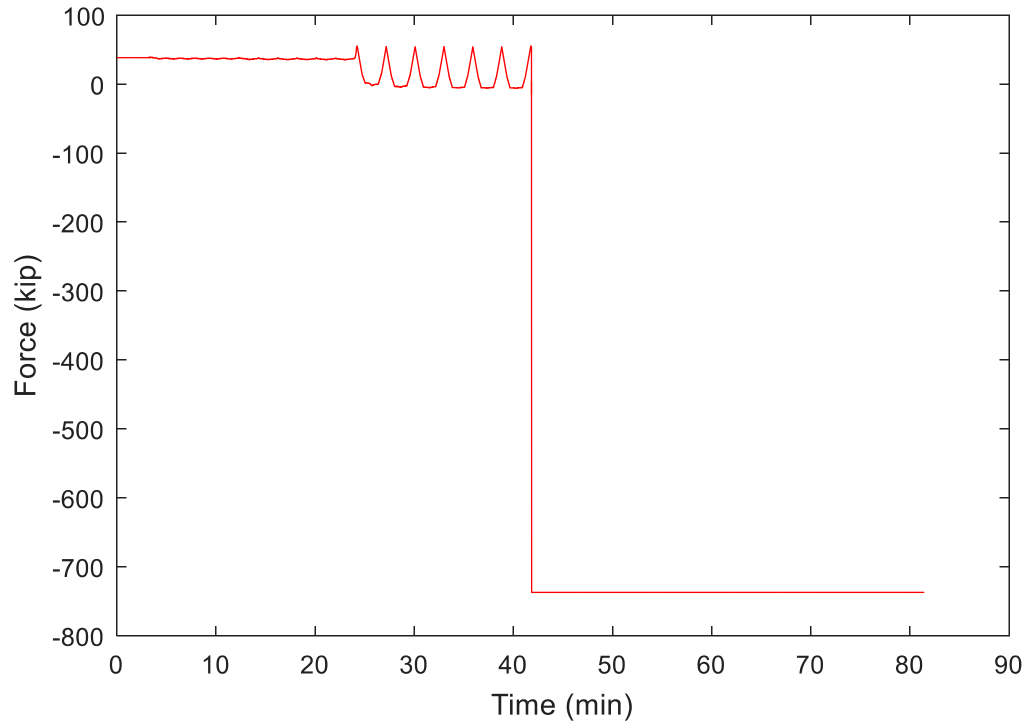


Figure S-68 Force in BSG06

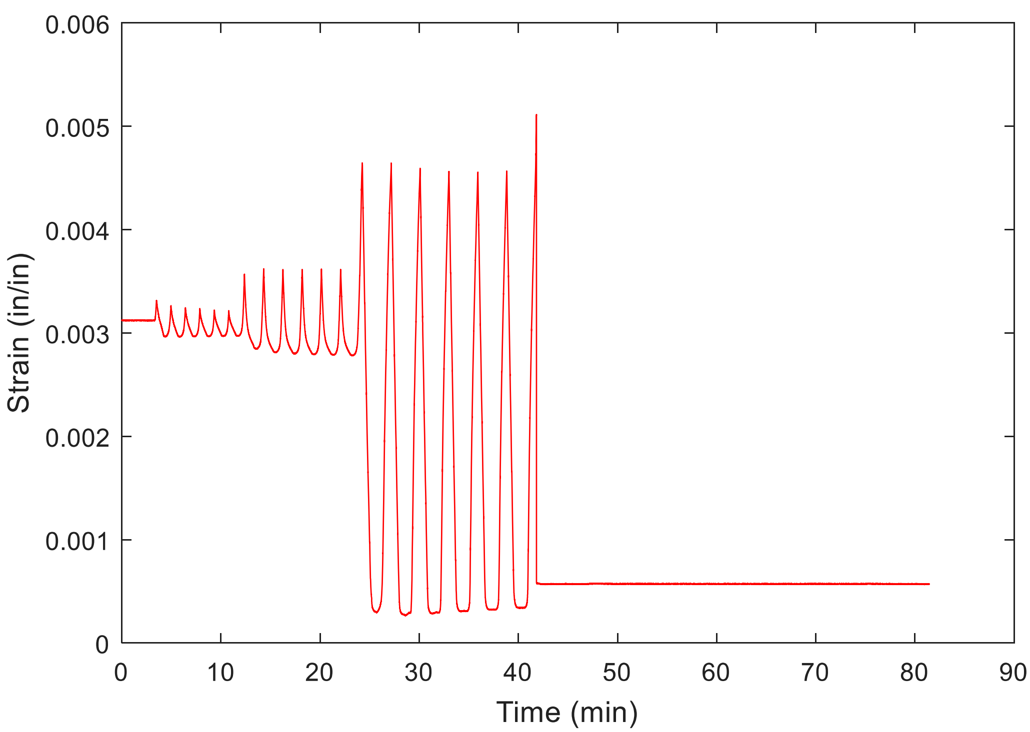


Figure S-69 Strain in BSG01

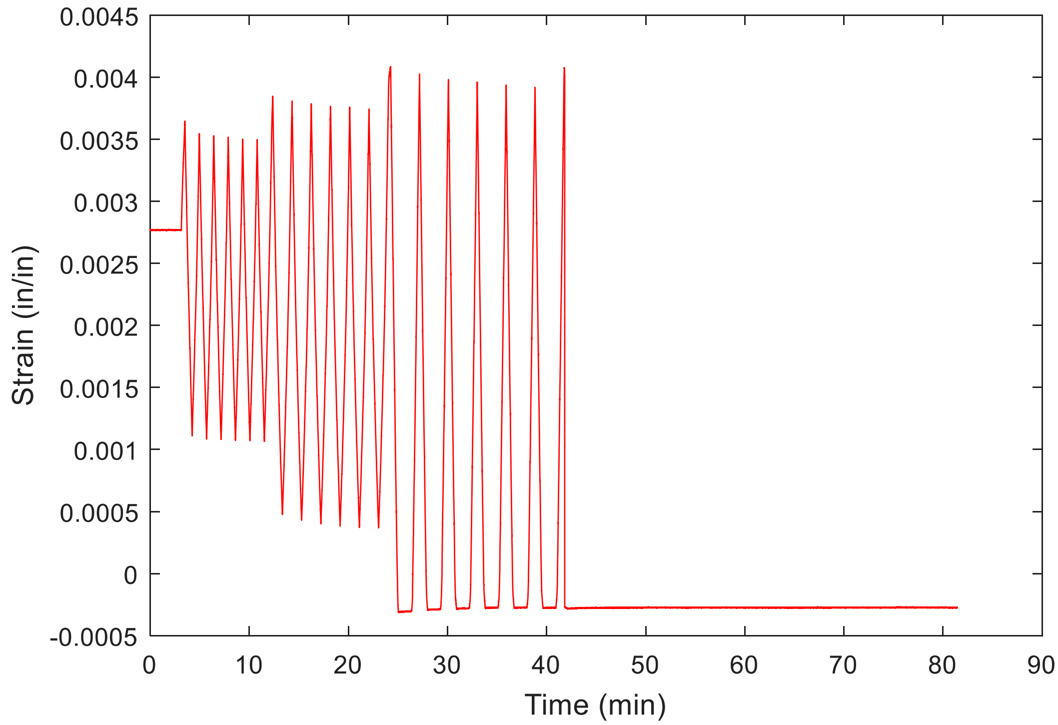


Figure S-70 Strain in BSG02

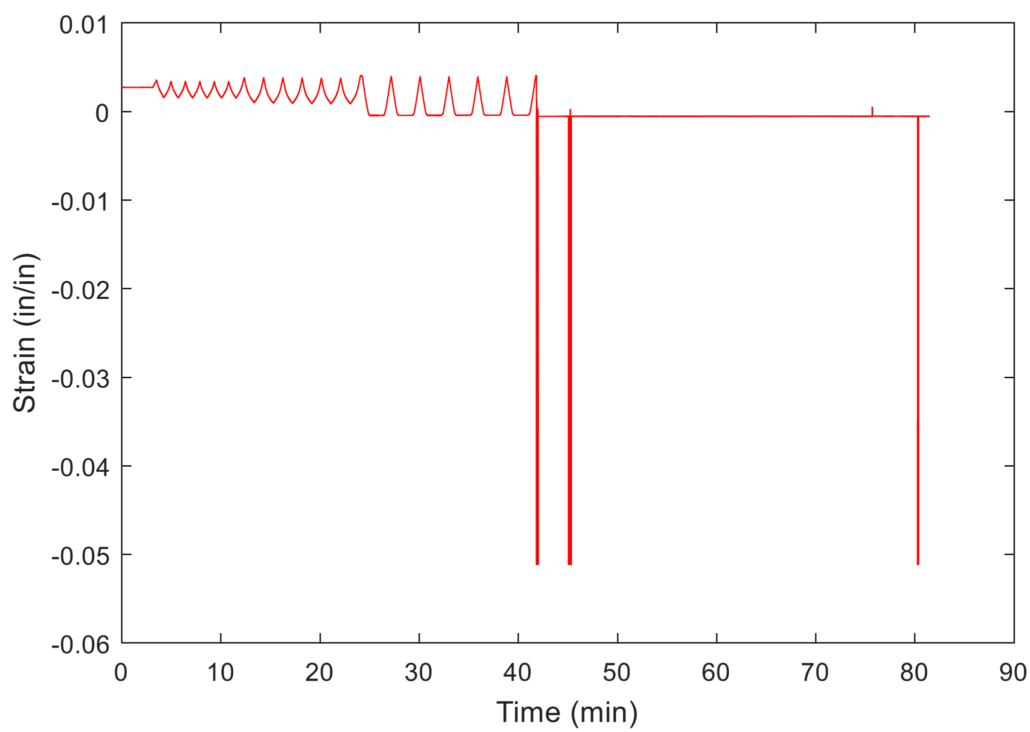


Figure S-71 Strain in BSG03

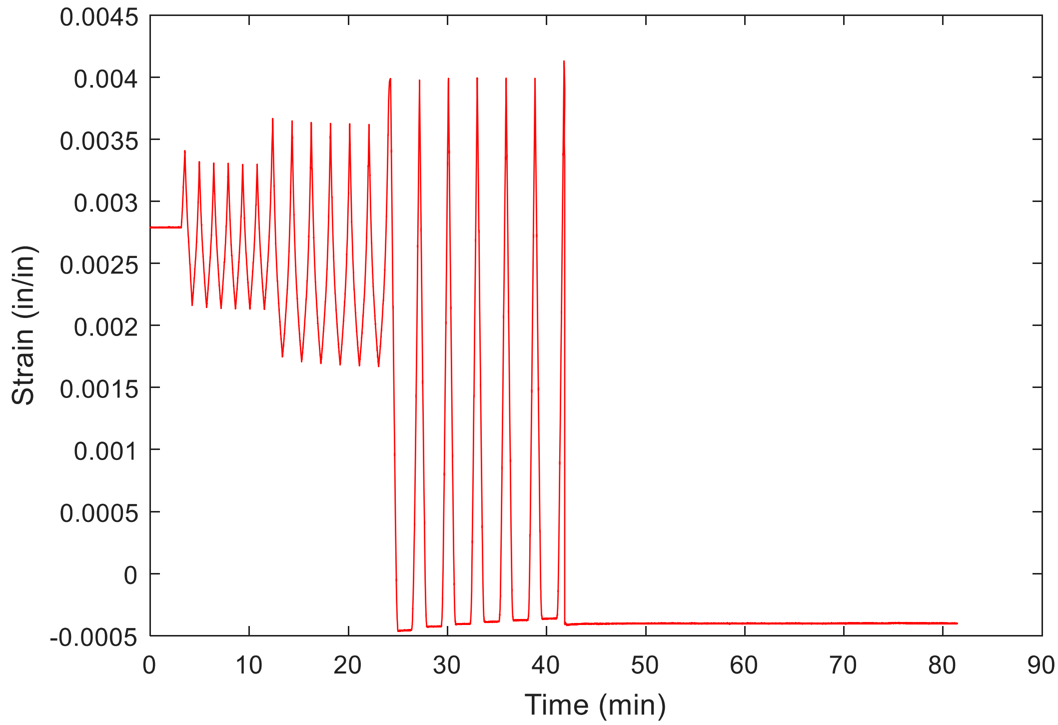


Figure S-72 Strain in BSG04

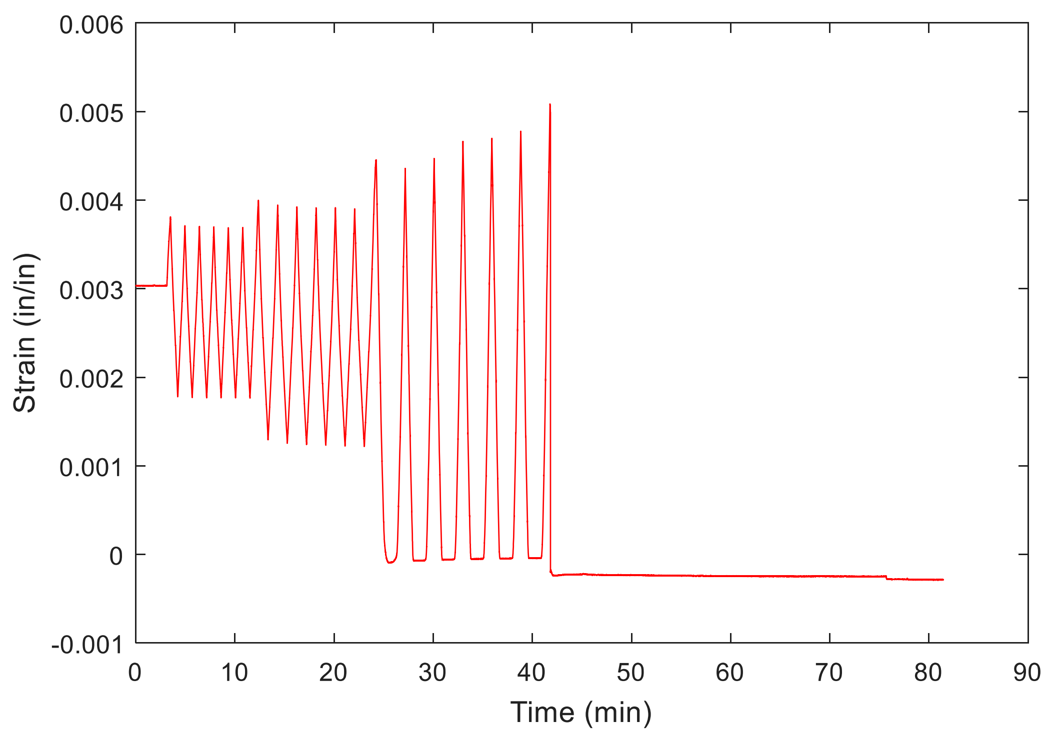


Figure S-73 Strain in BSG05

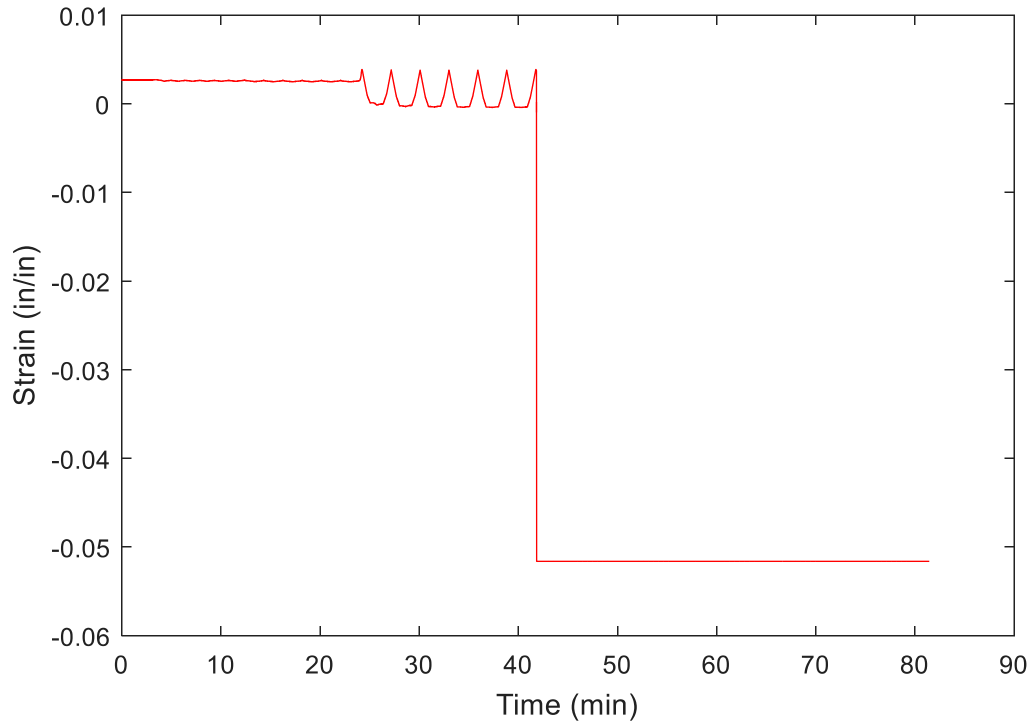


Figure S-74 Strain in BSG06

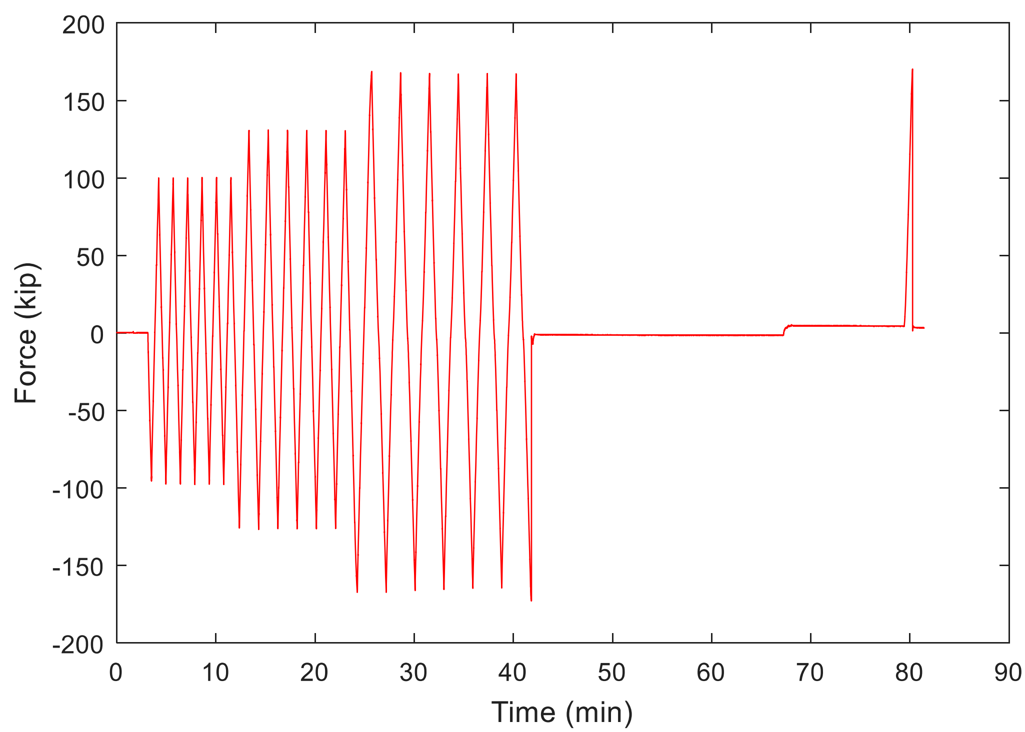


Figure S-75 Actuator Force

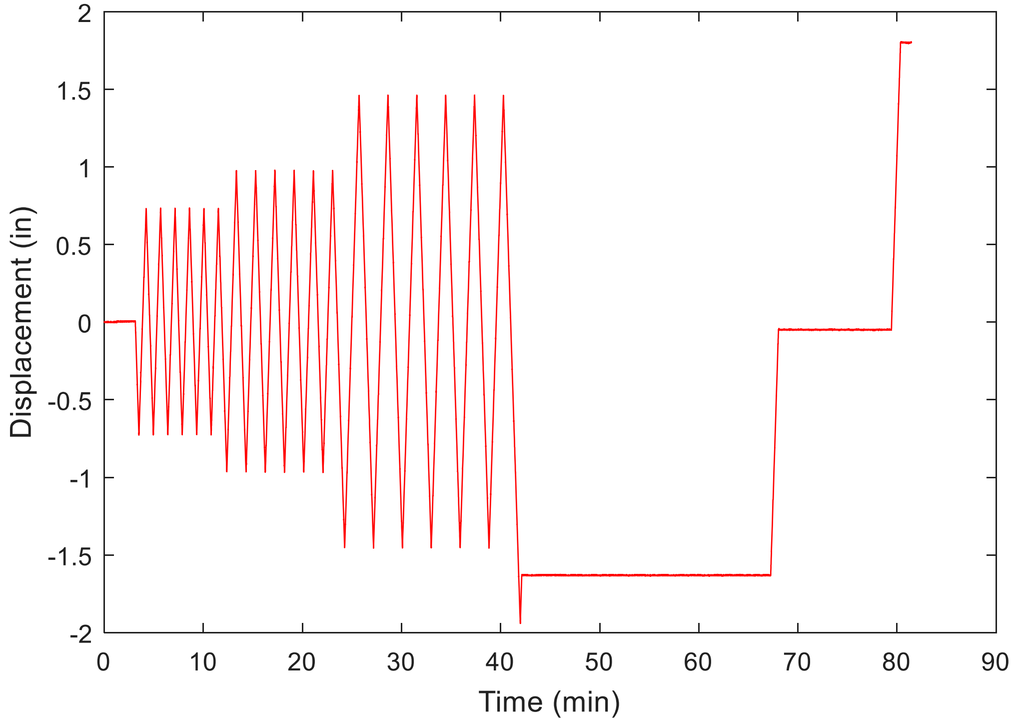


Figure S-76 Actuator Displacement

APPENDIX T ADDITIONAL PLOTS FOR SPECIMEN 12ES-1.25-1.50-44

Plots from the start until the beam fractured

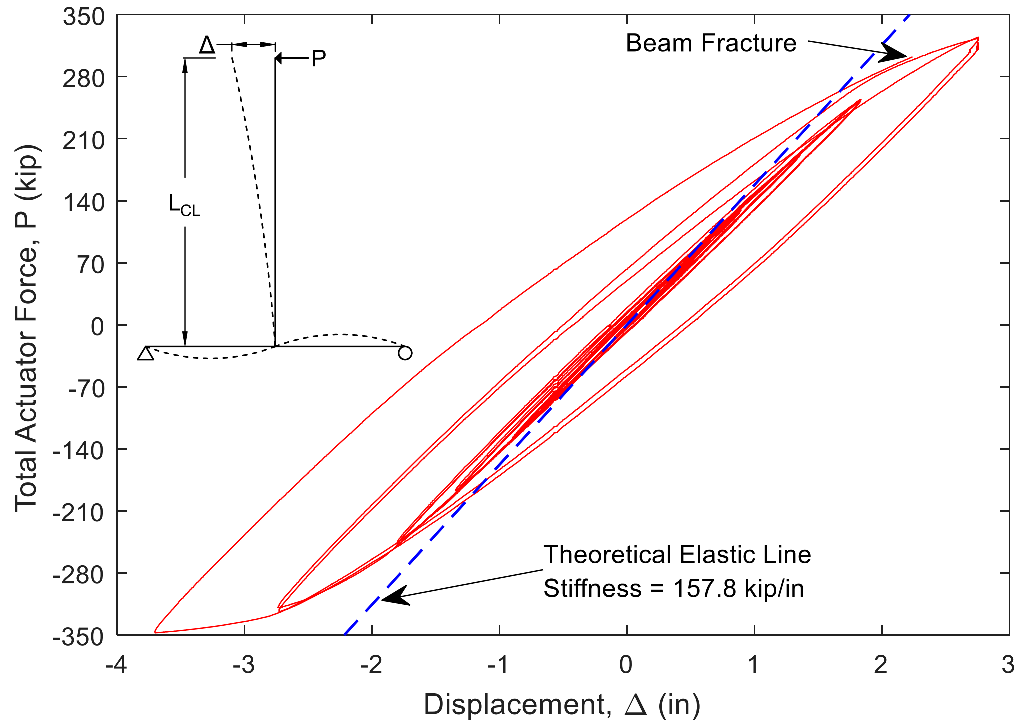


Figure T-1 Total Actuator Force vs. Applied Displacement at End of Beam

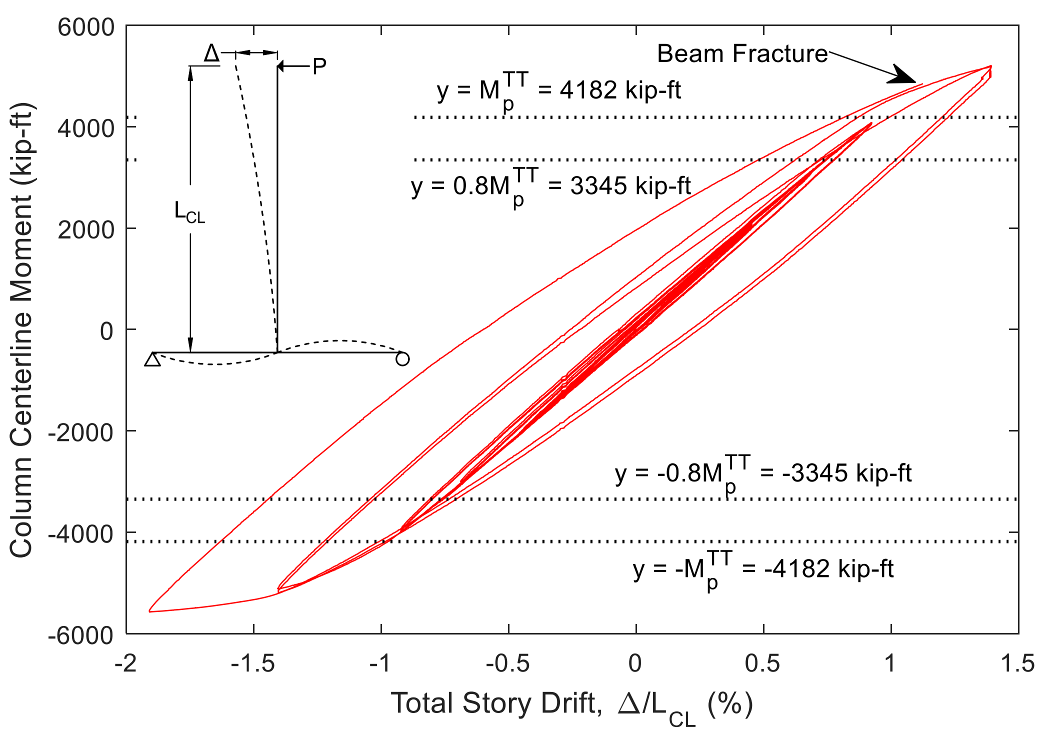


Figure T-2 Column Centerline Moment vs. Total Story Drift

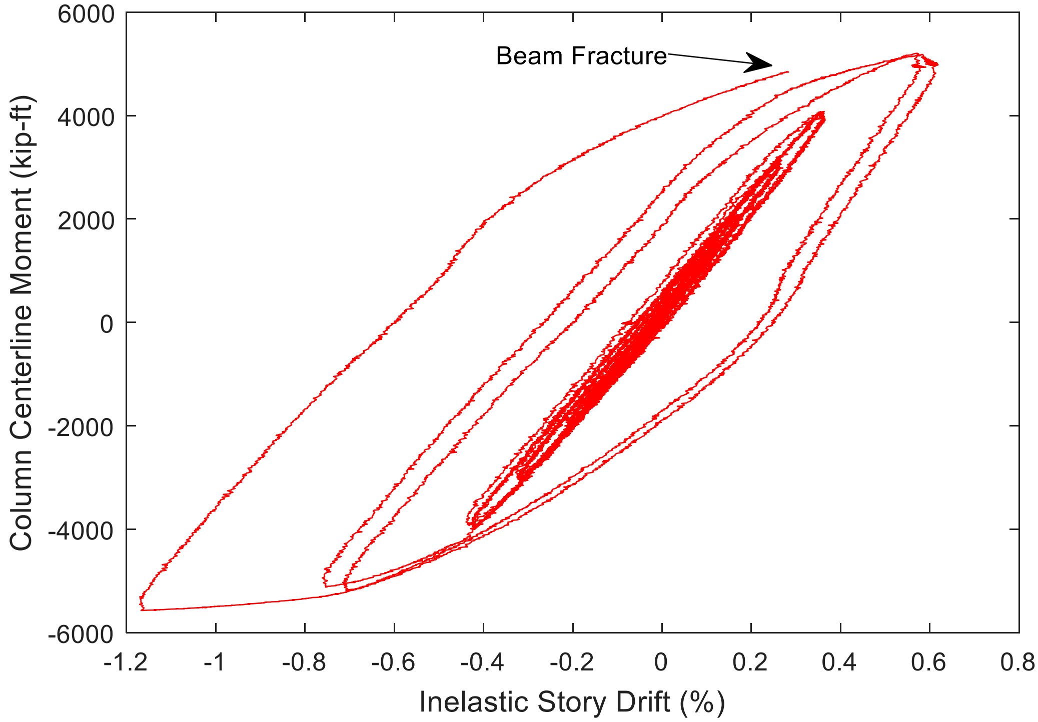


Figure T-3 Column Centerline Moment vs. Inelastic Portion of Story Drift

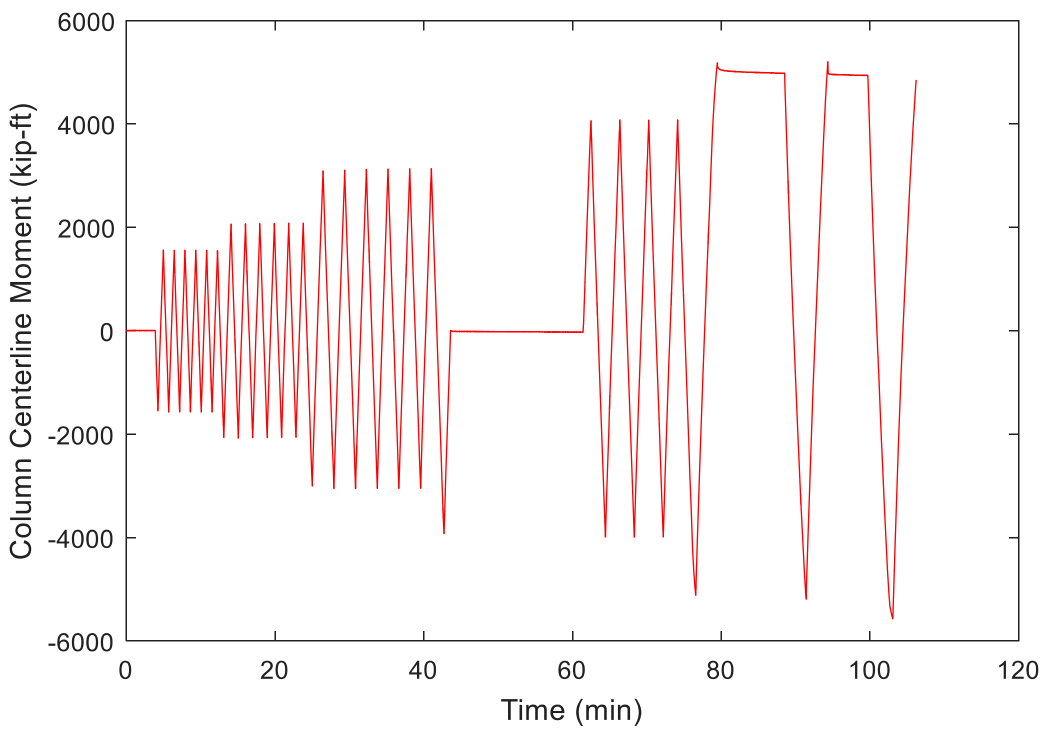


Figure T-4 Column Centerline Moment vs. Time

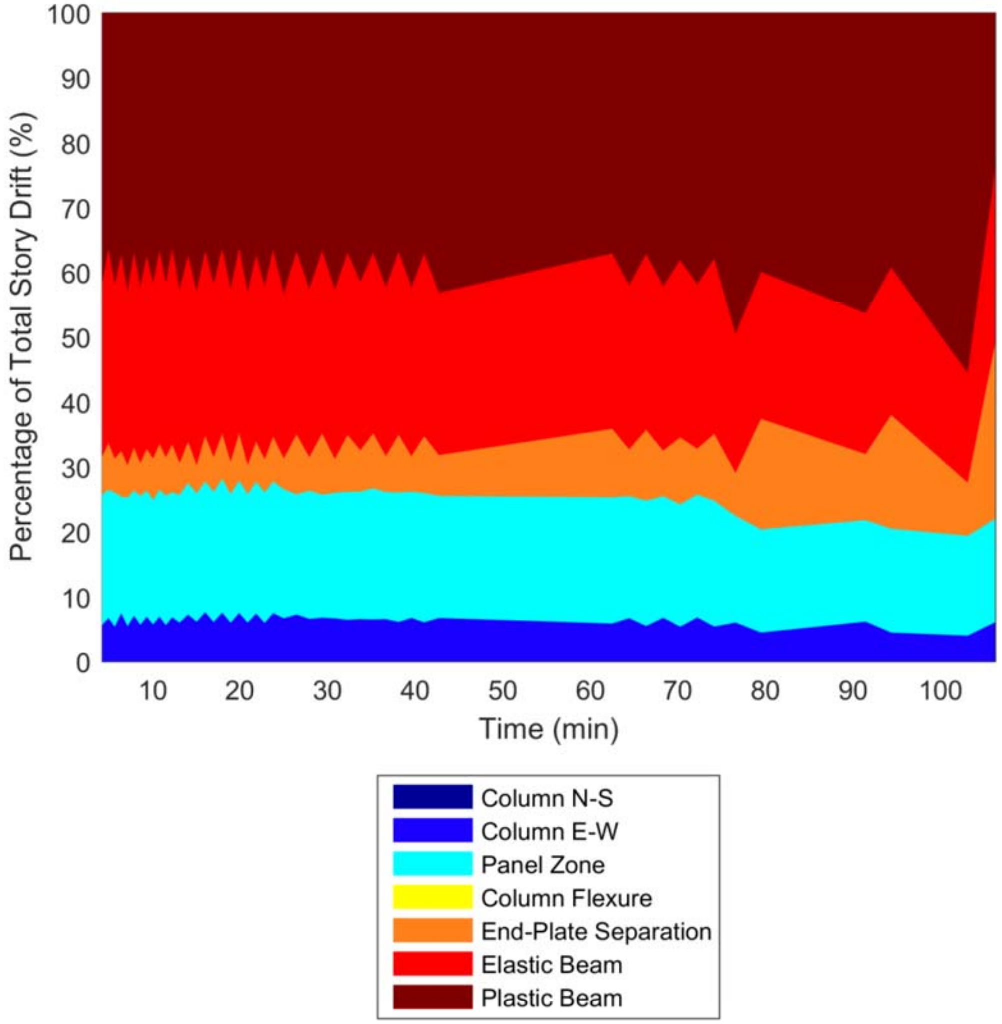


Figure T-5 Story Drift Components Percentage of Total Story Drift vs. Time

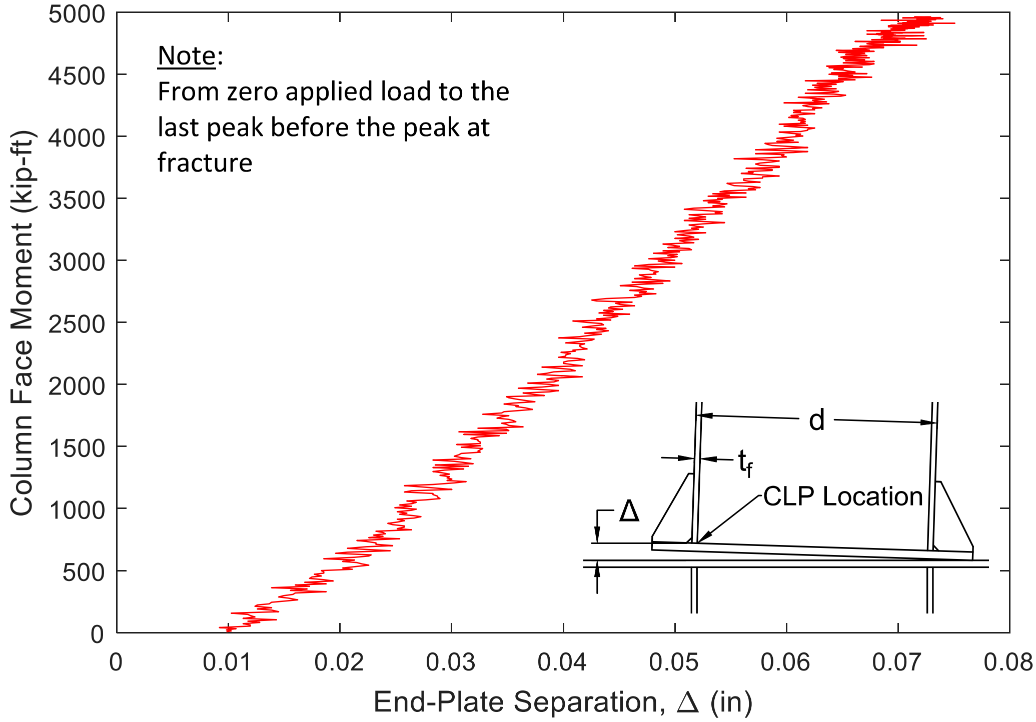


Figure T-6 Column Face Moment vs. End-Plate Separation

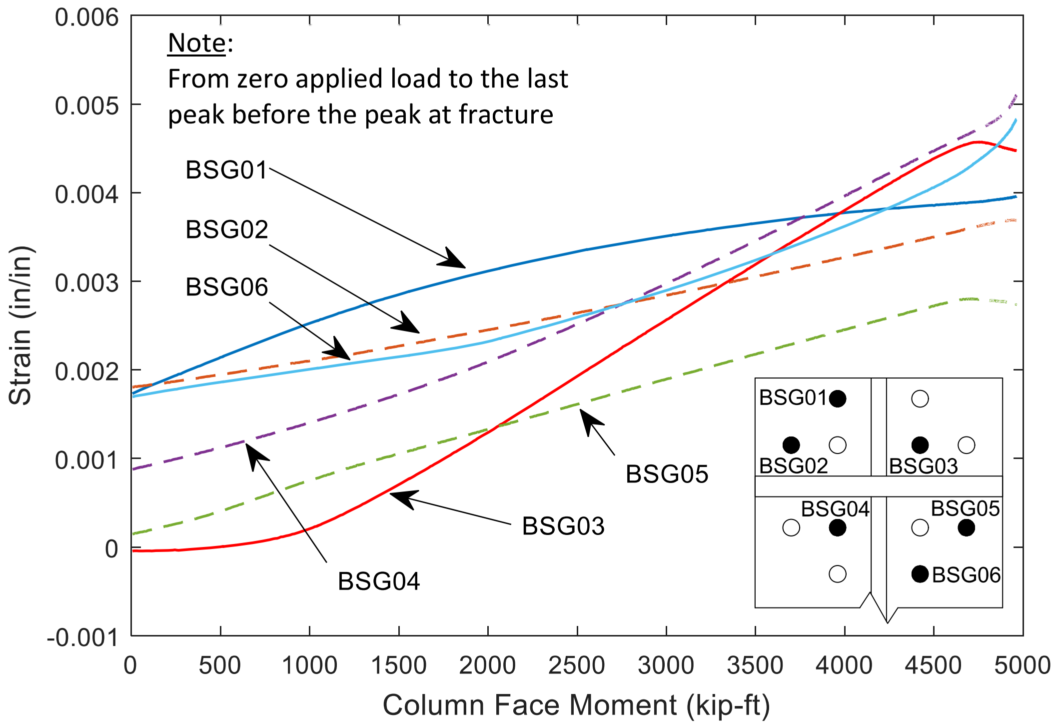


Figure T-7 Strain in Bolts vs. Column Face Moment

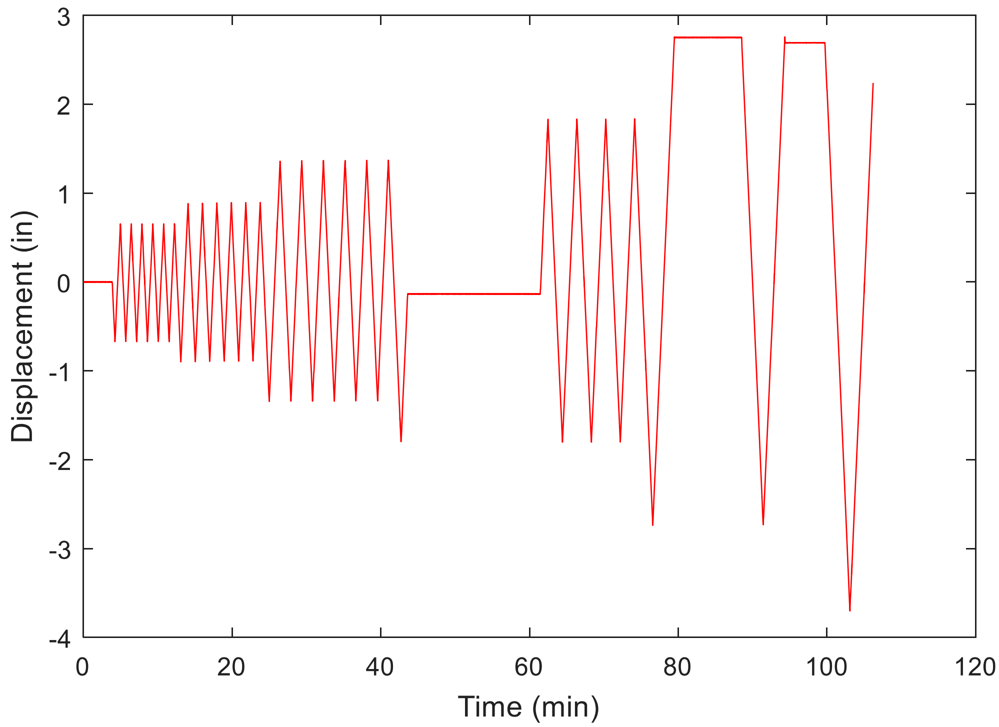


Figure T-8 Displacement at End of Beam vs. Time

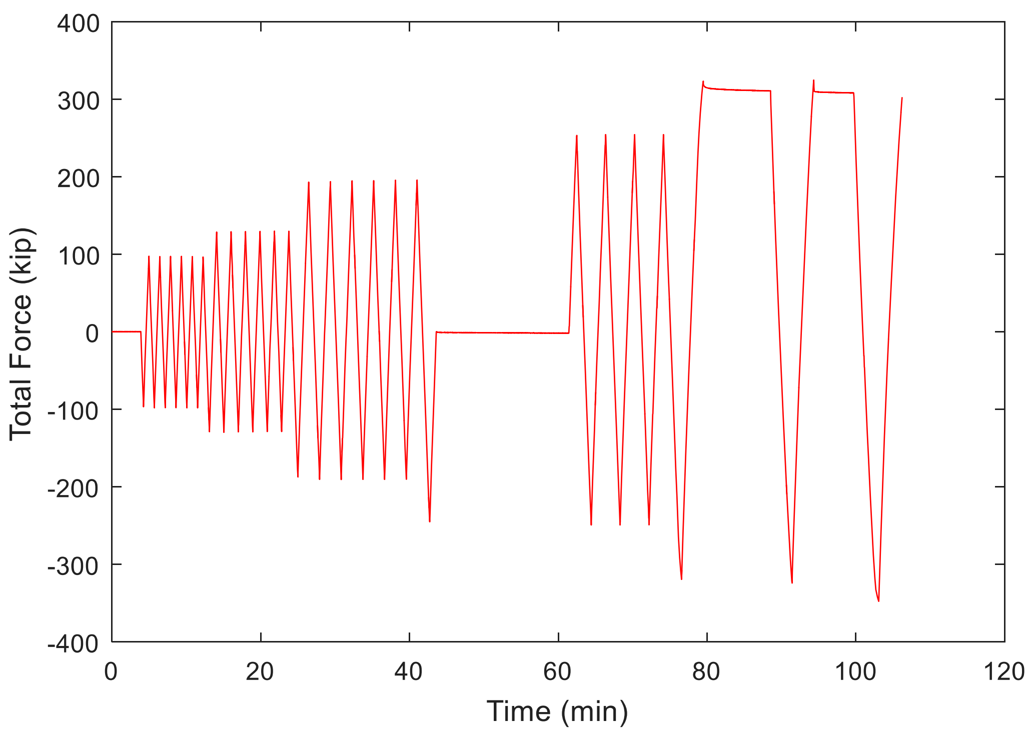


Figure T-9 Total Applied Force vs. Time

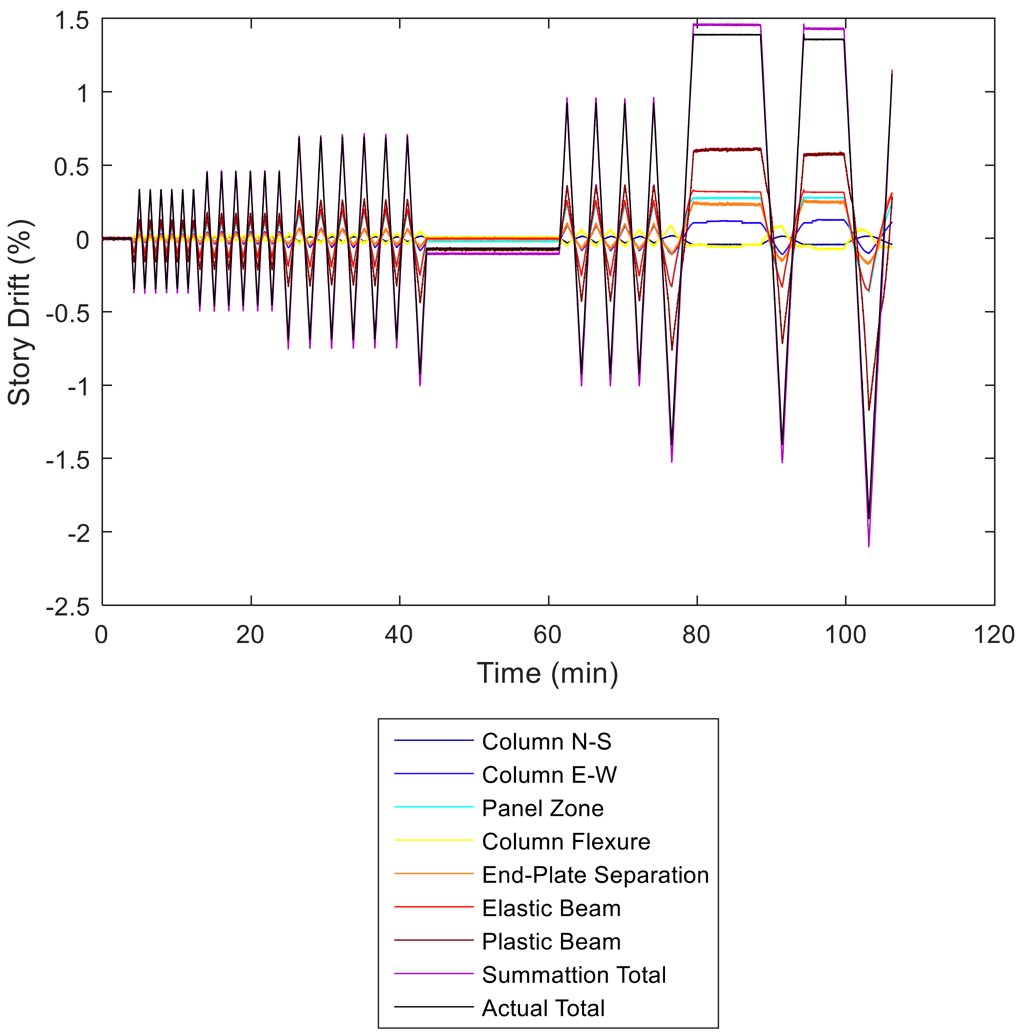


Figure T-10 Story Drift Components vs. Time

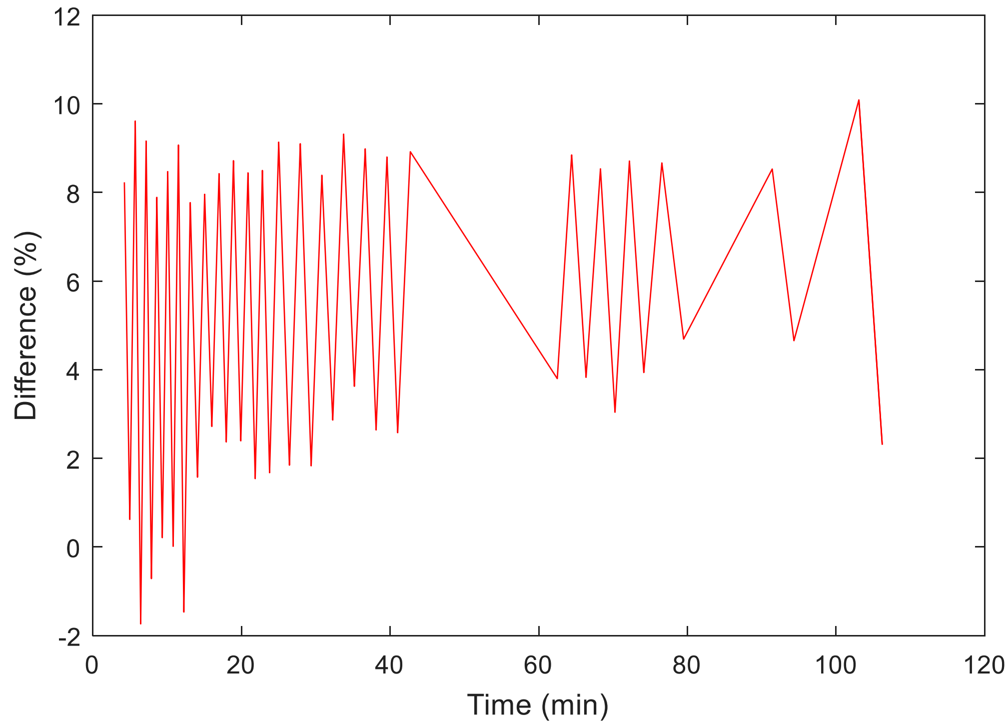


Figure T-11 Percent Difference Between Total Story Drift & Actual Story Drift at Peaks

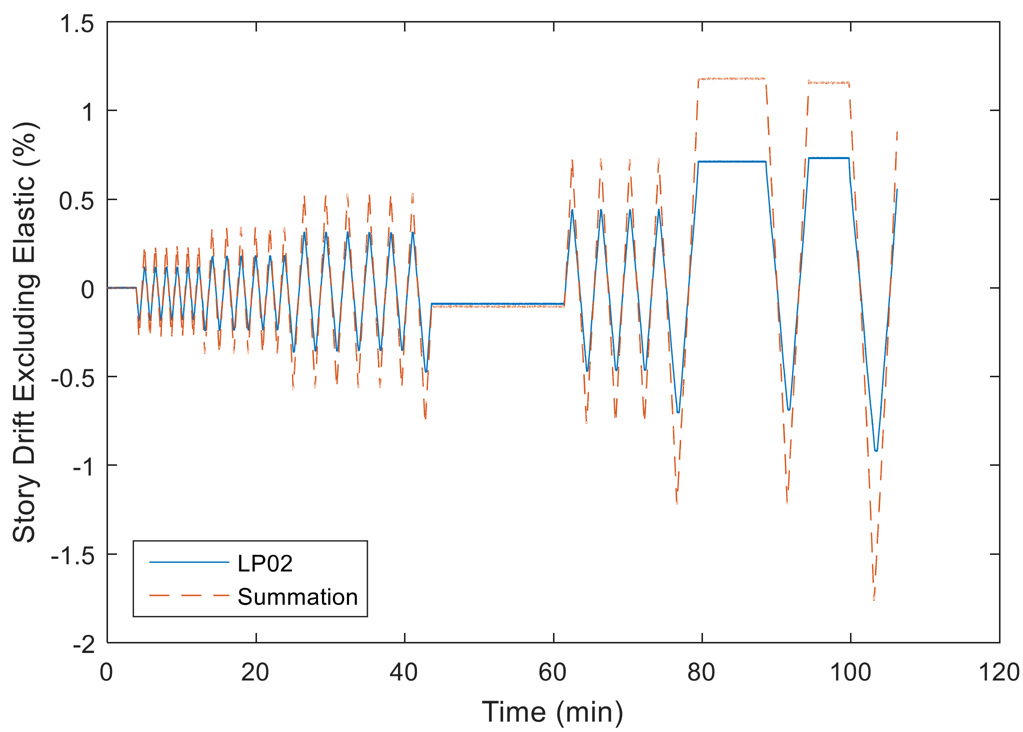


Figure T-12 Large Deformations Check

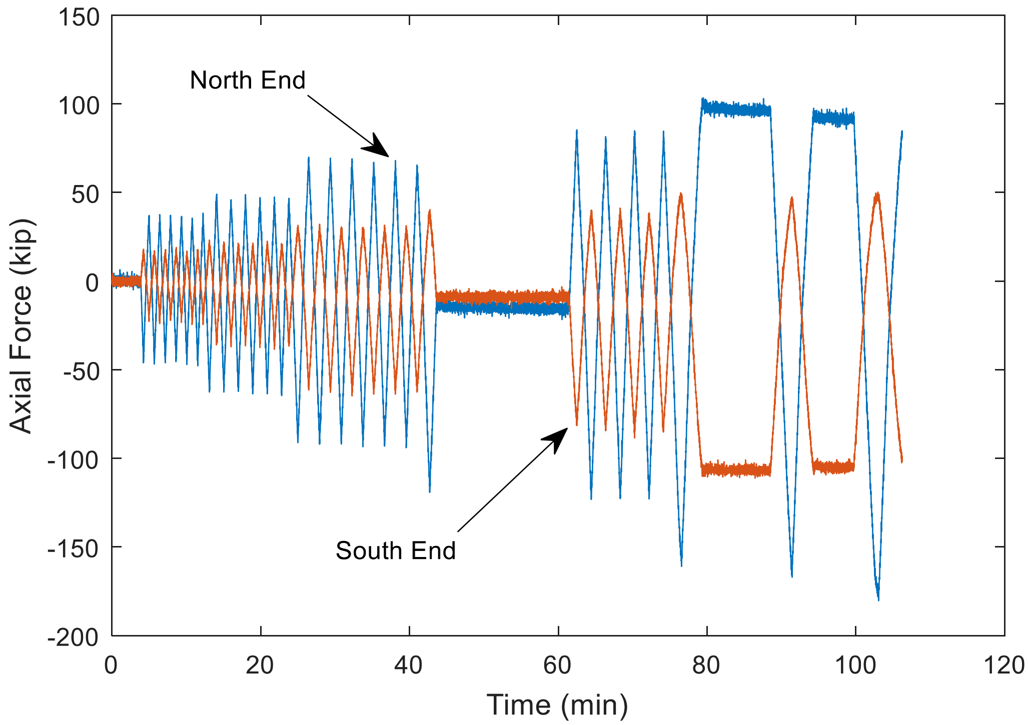


Figure T-13 Column Axial Force Calculated From Strain Gauges on Stub Sections

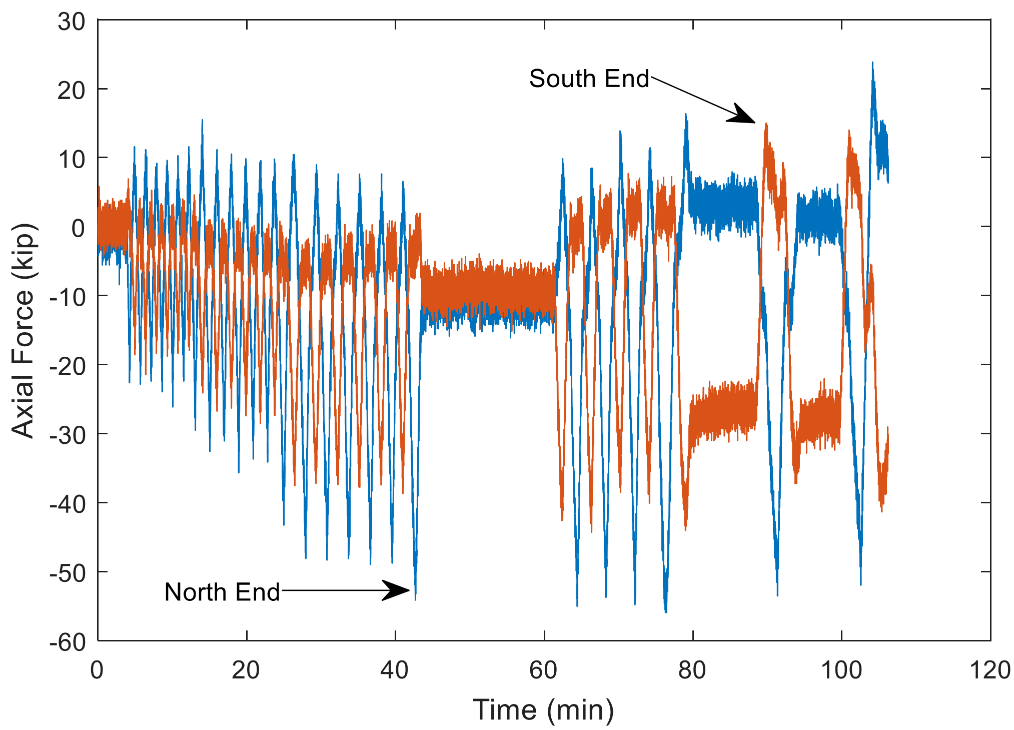


Figure T-14 Column Axial Force Calculated From Strain Gauges on Column

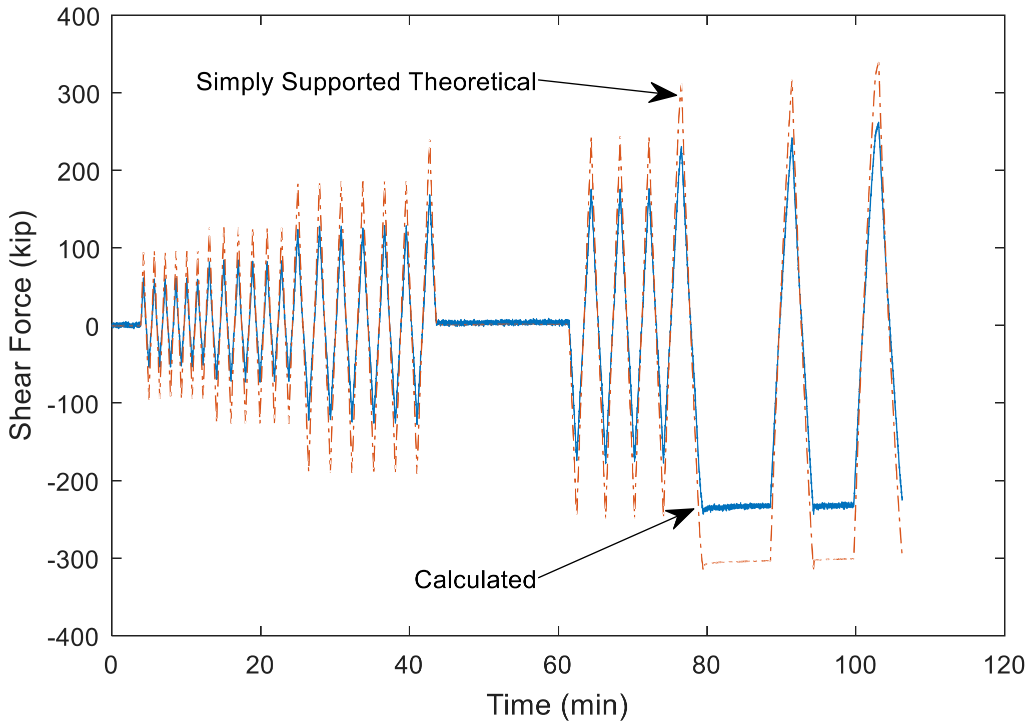


Figure T-15 Column Shear Force – North End

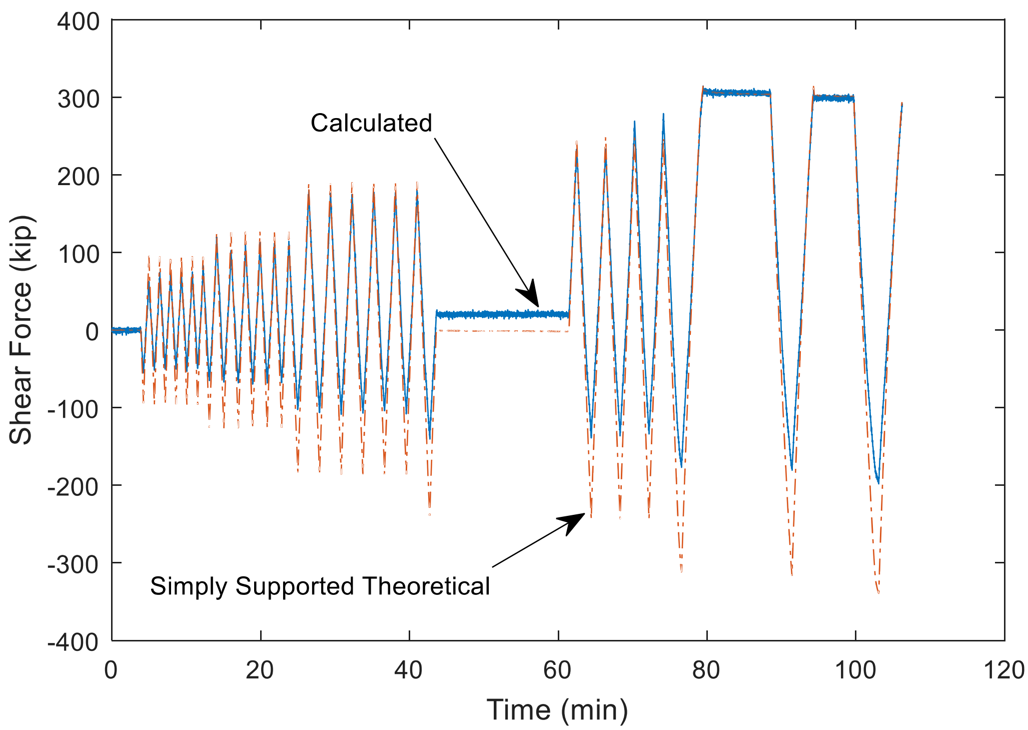


Figure T-16 Column Shear Force – South End

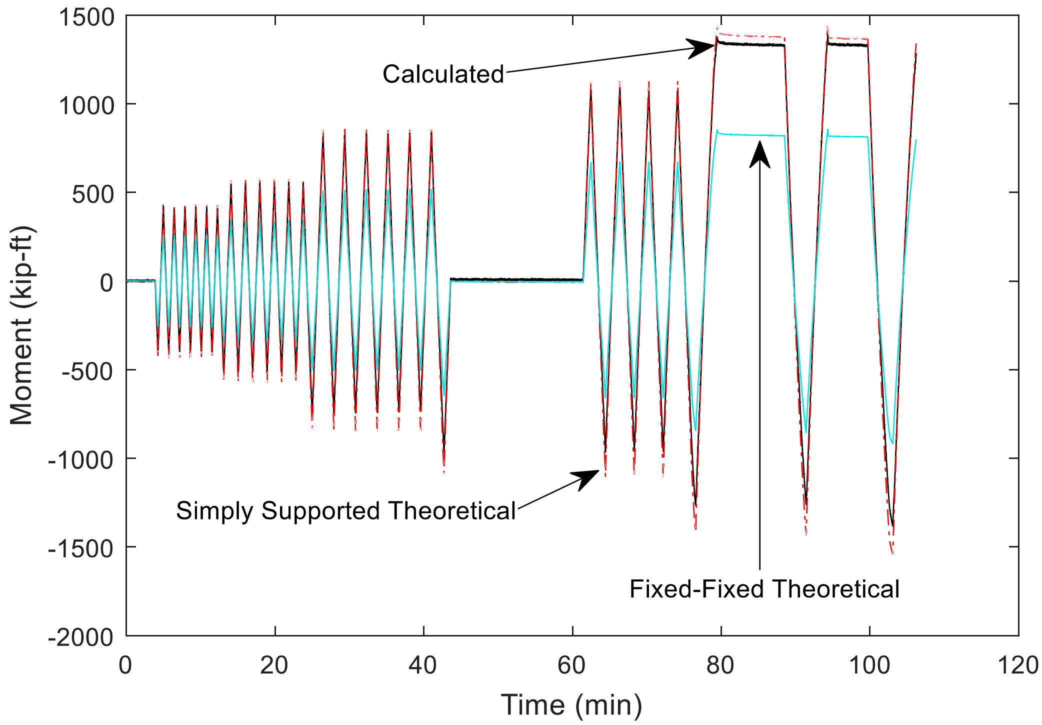


Figure T-17 Moment in Column at Location of Strain Gauges – North End

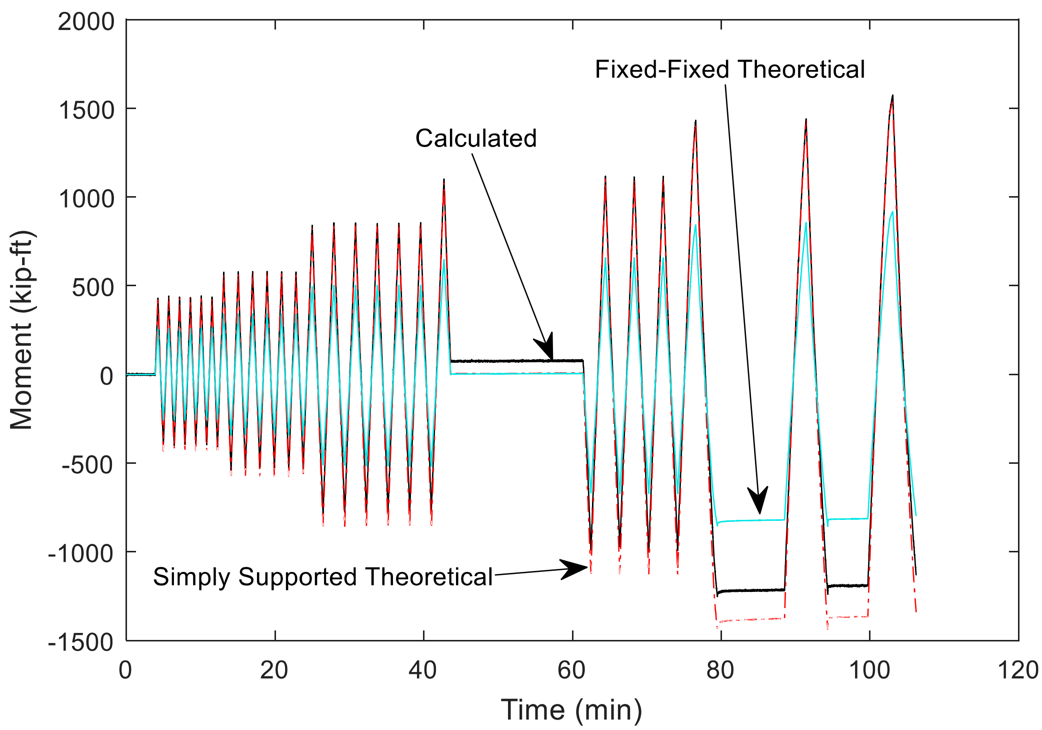


Figure T-18 Moment in Column at Location of Strain Gauges – South End

Instrumentation Data vs. Time

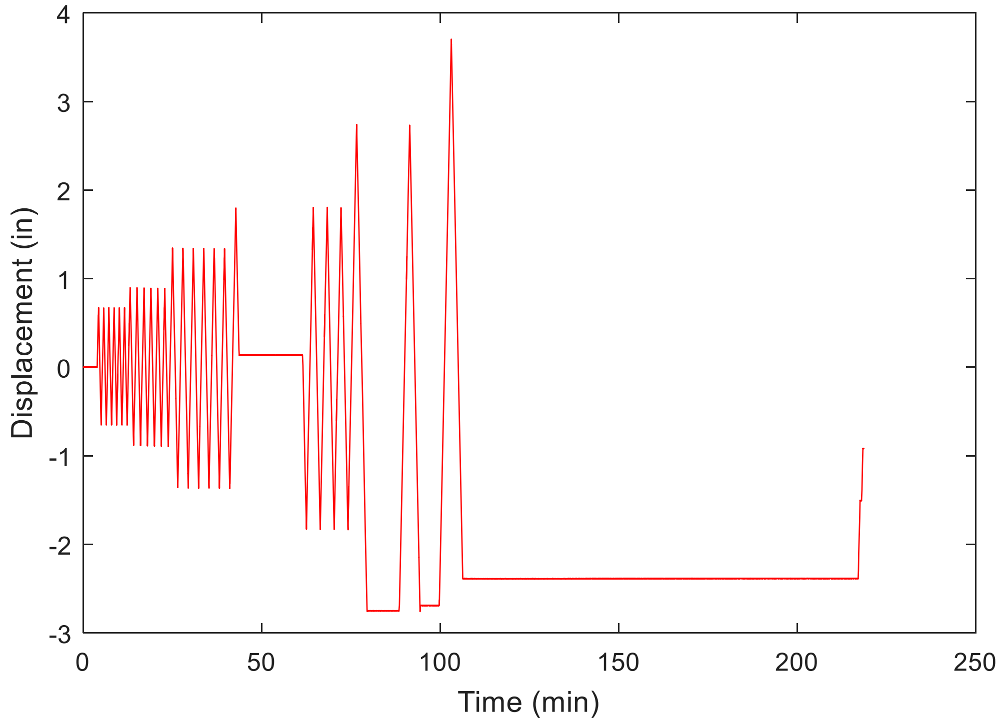


Figure T-19 SP01

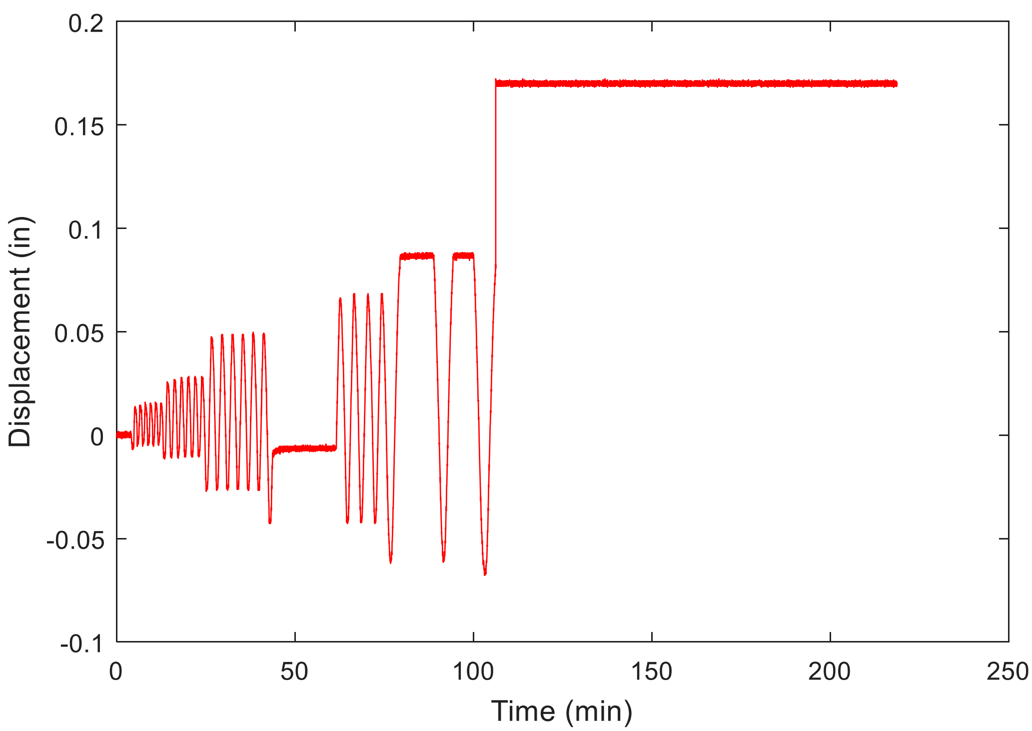


Figure T-20 SP02

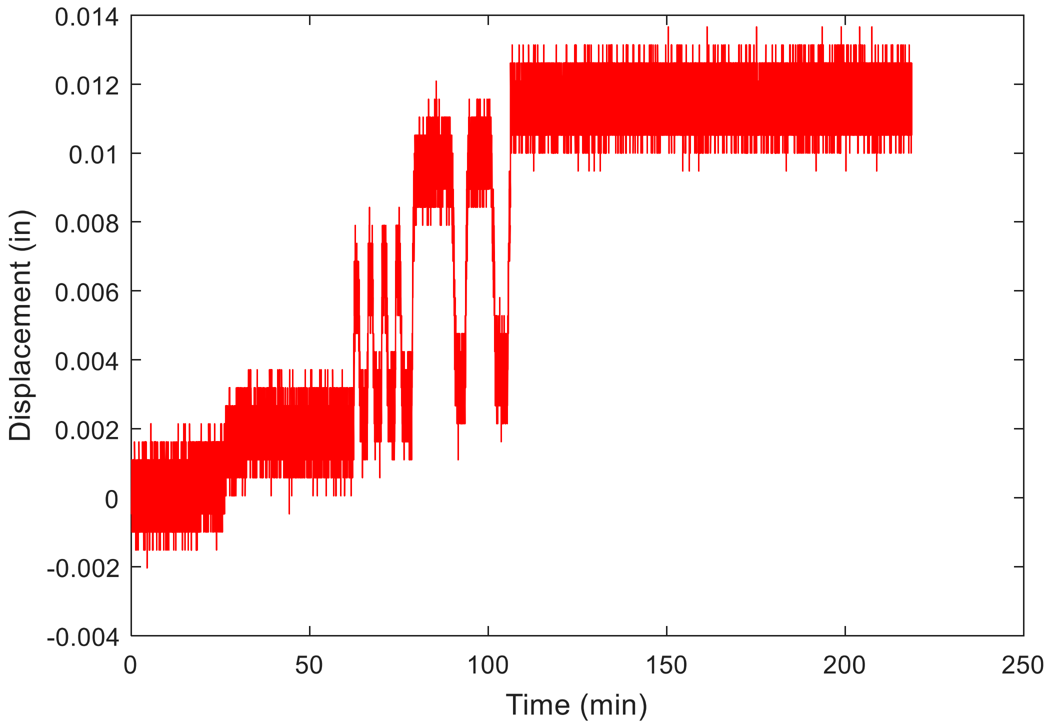


Figure T-21 SP03

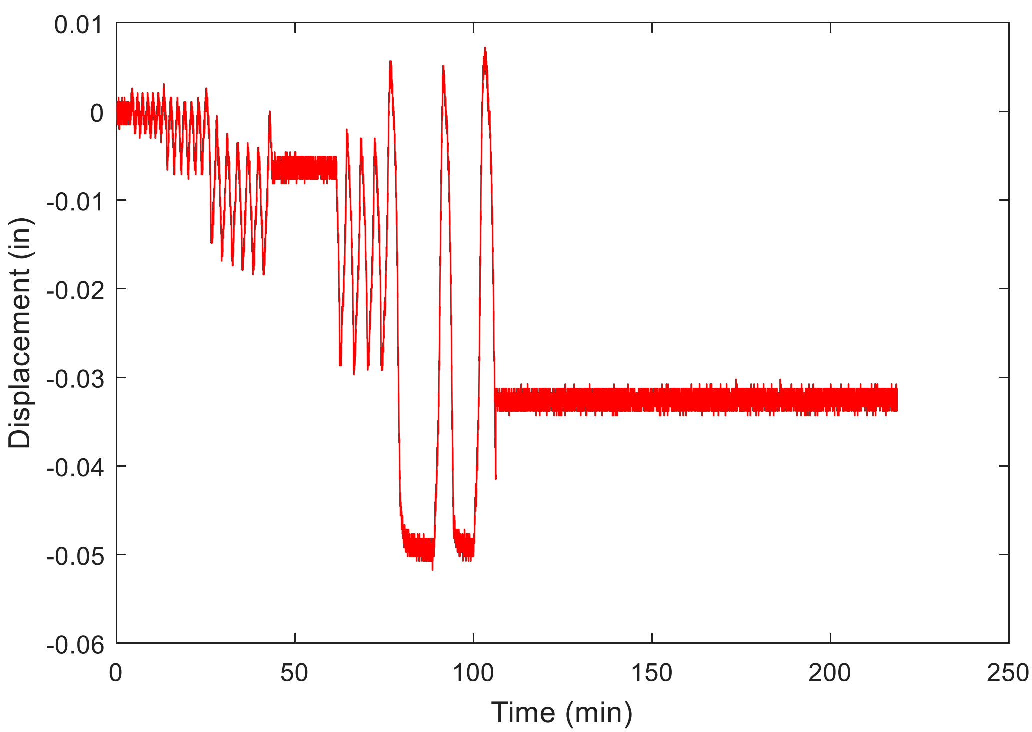


Figure T-22 SP04

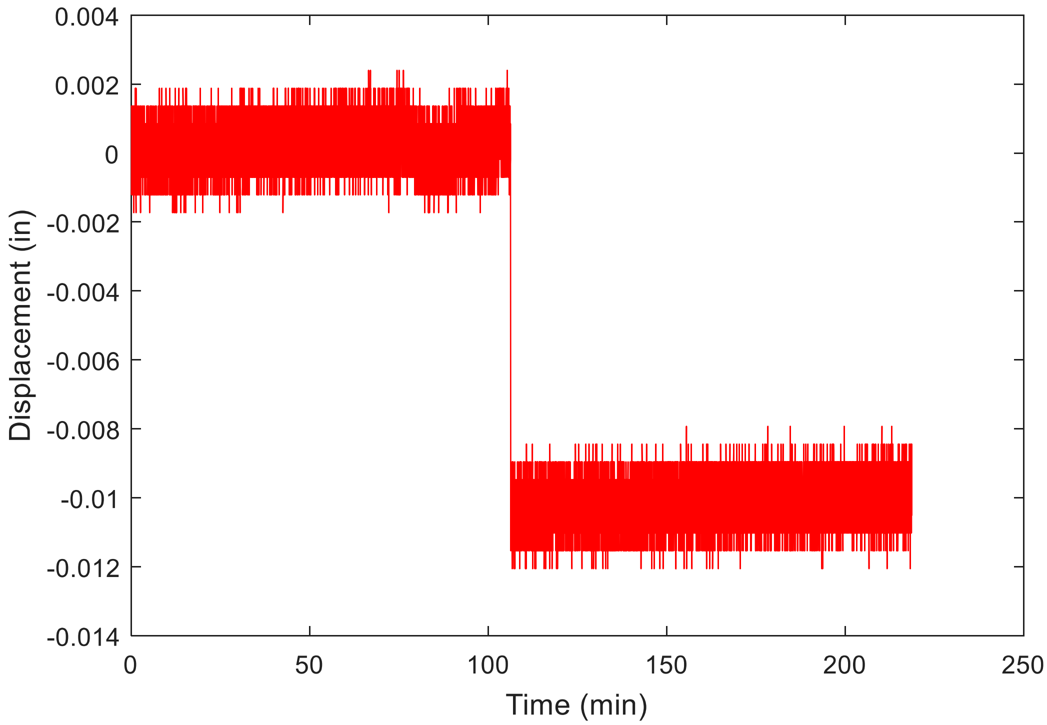


Figure T-23 SP05

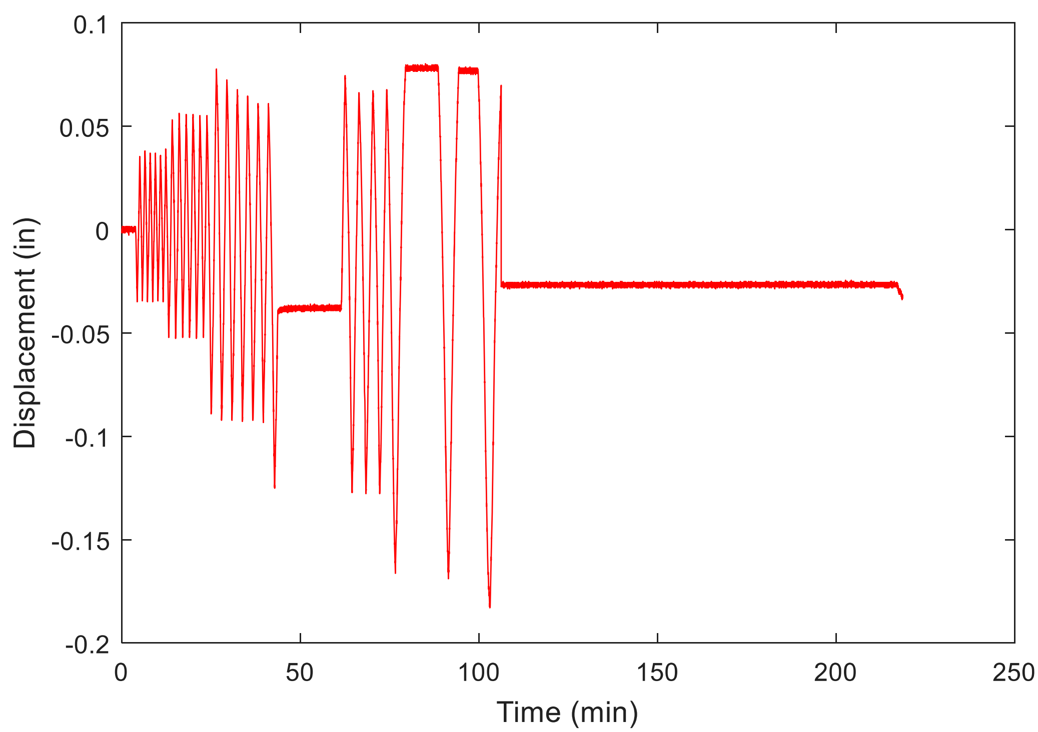


Figure T-24 SP06

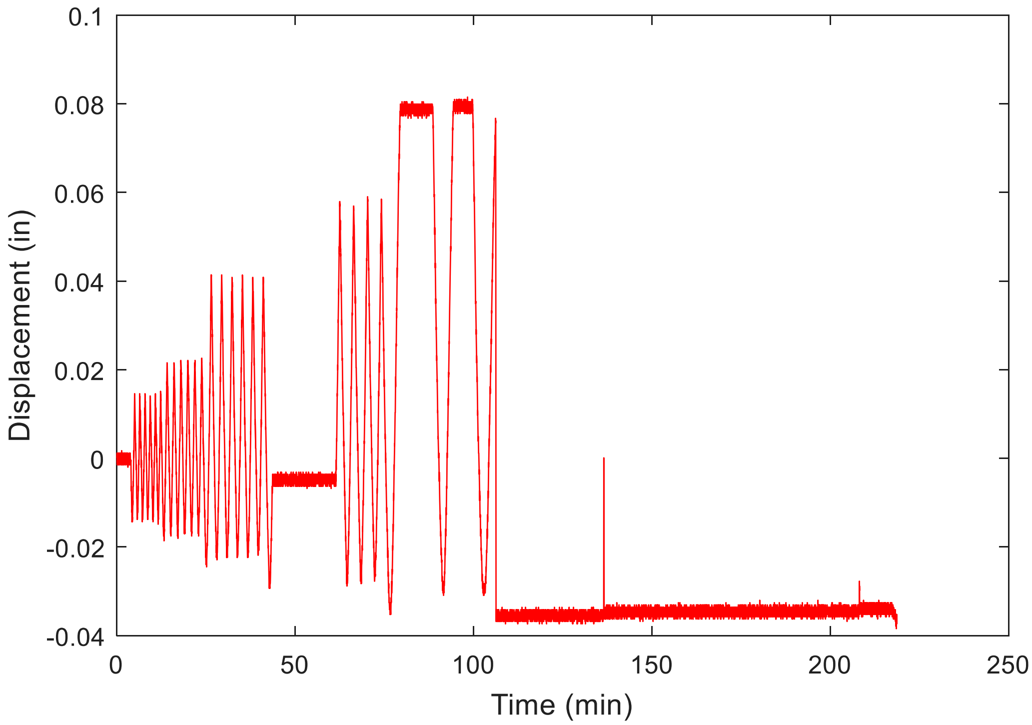


Figure T-25 SP07

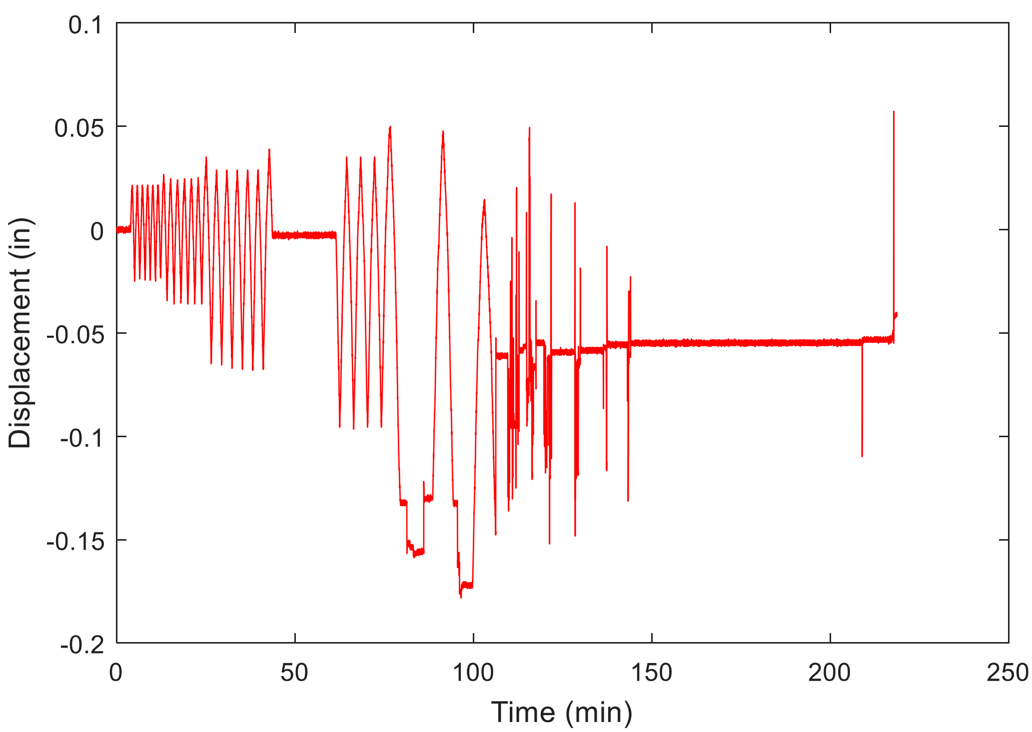


Figure T-26 SP08

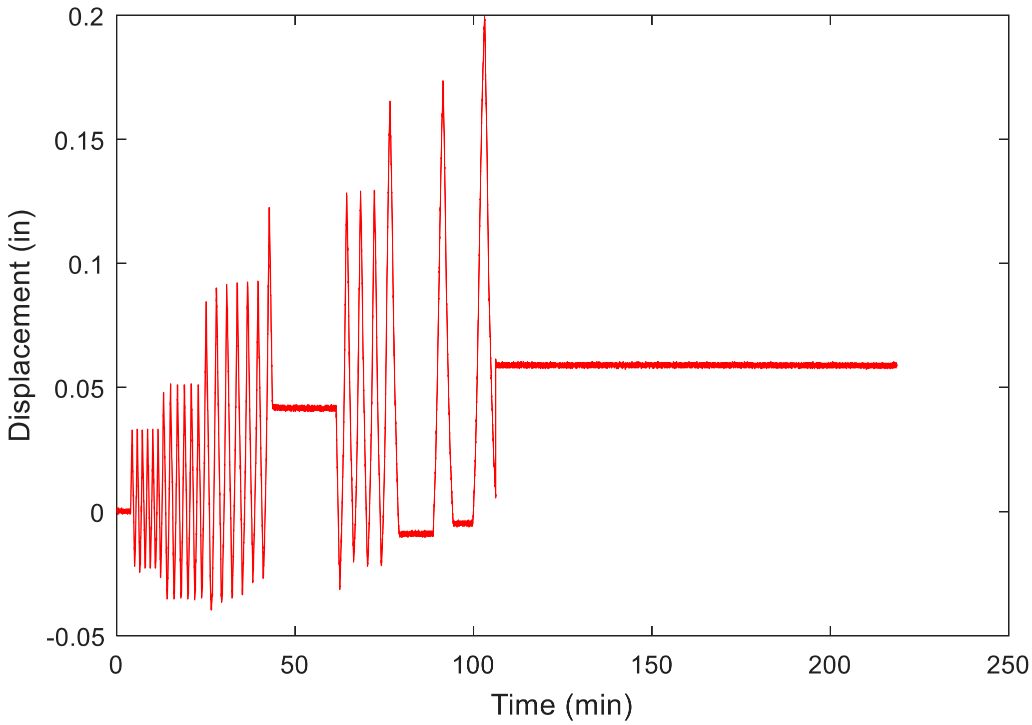


Figure T-27 SP09

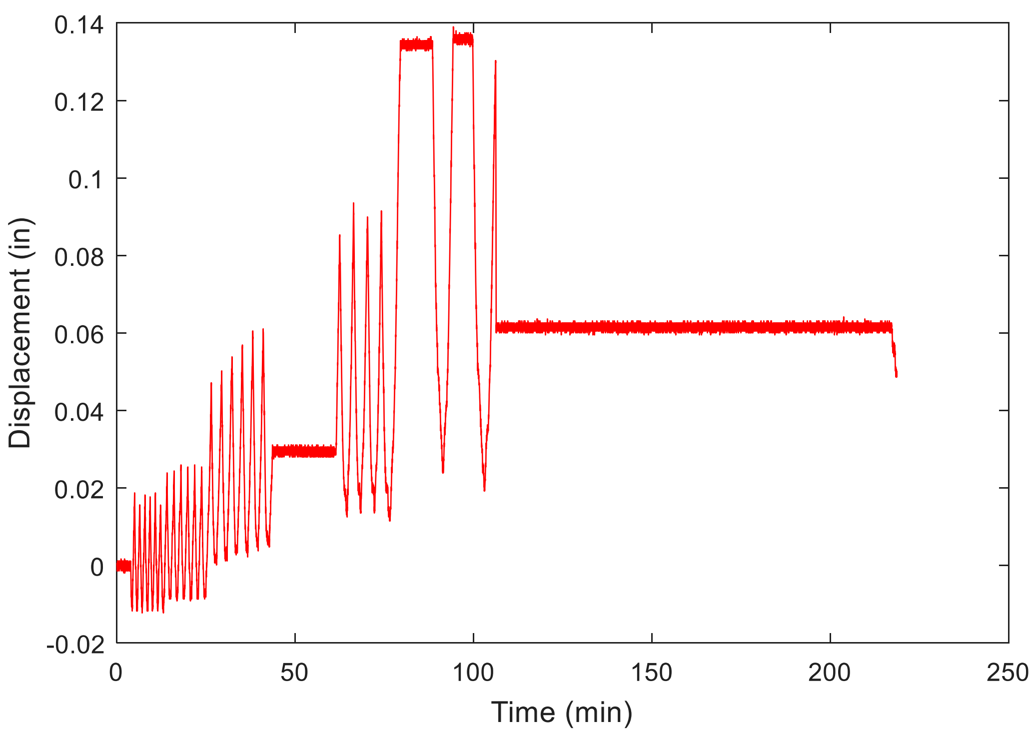


Figure T-28 SP10

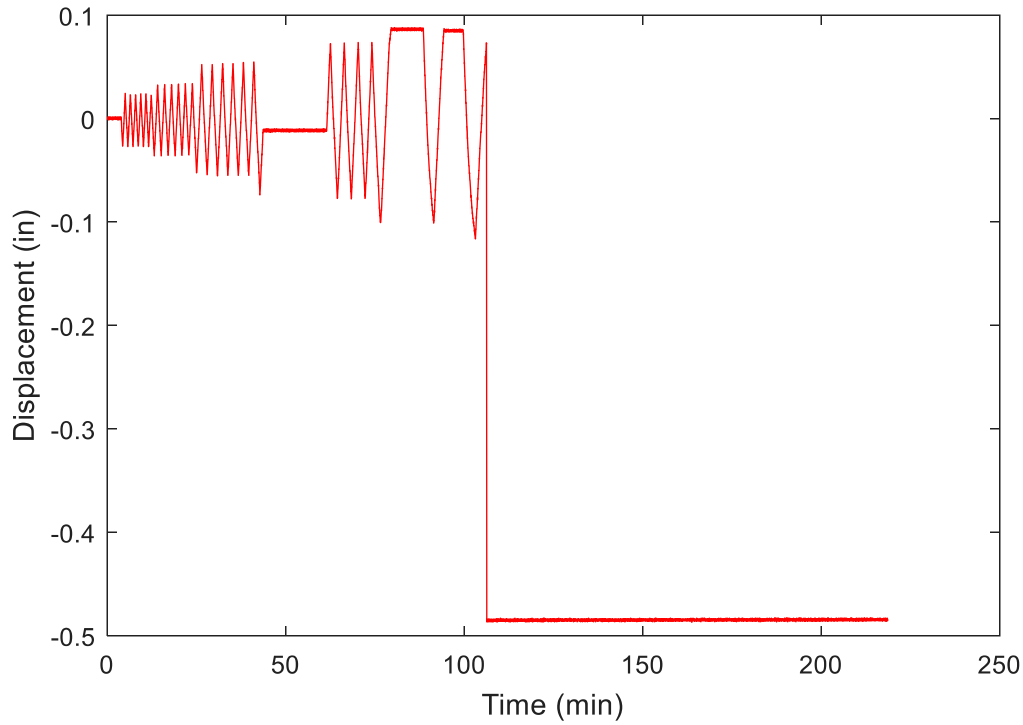


Figure T-29 SP11

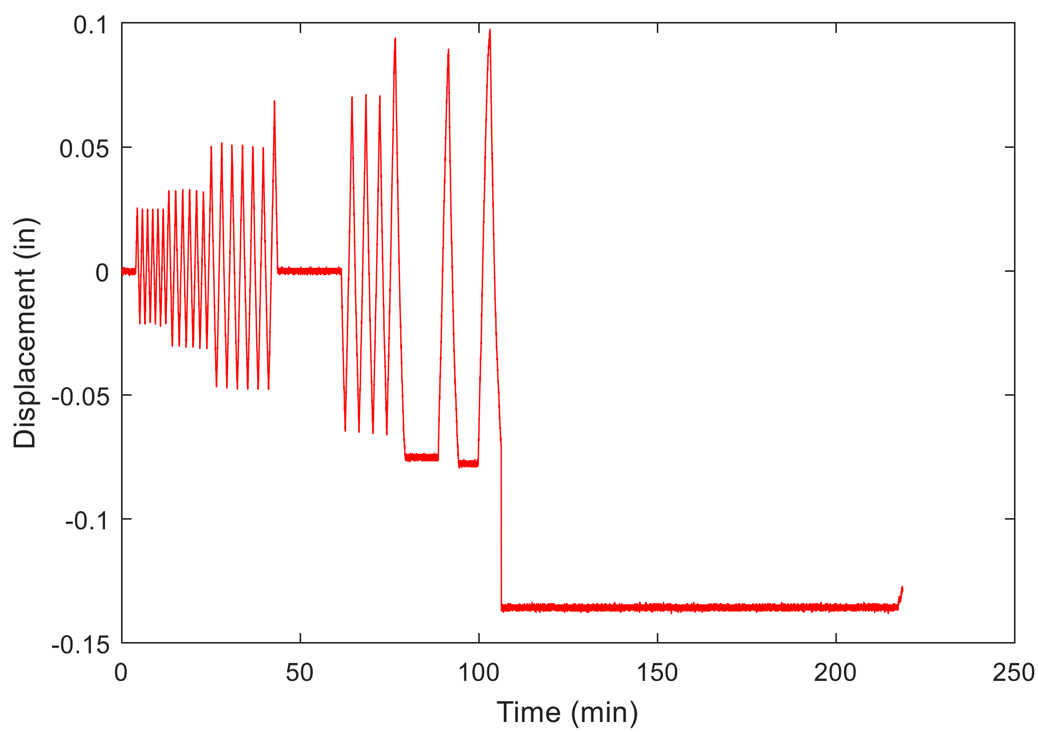


Figure T-30 SP12

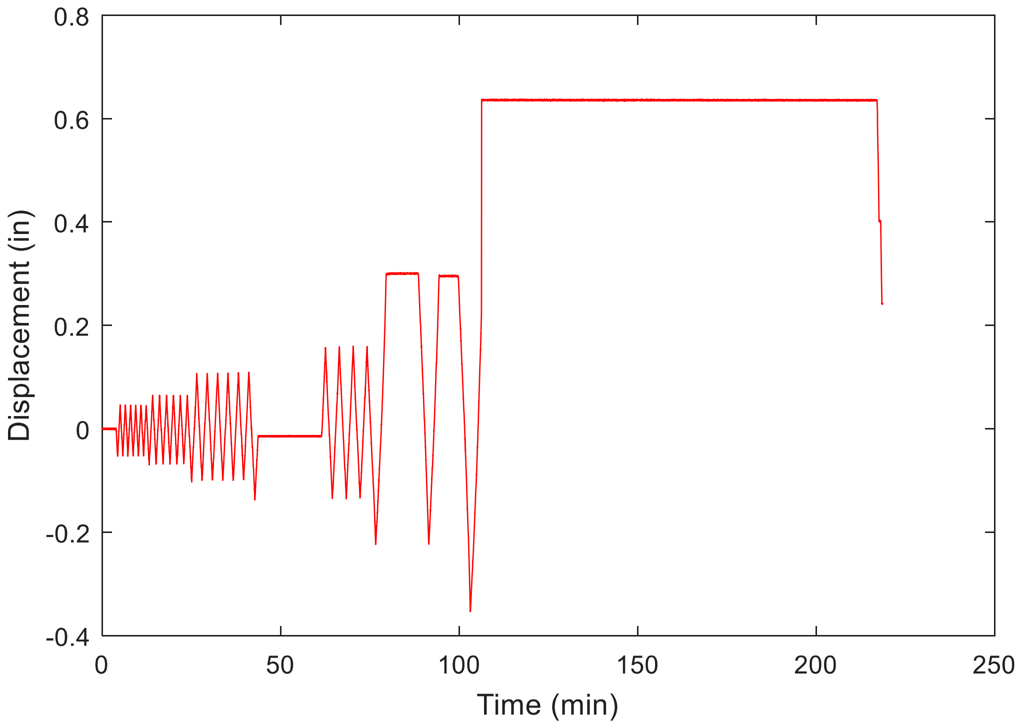


Figure T-31 SP13

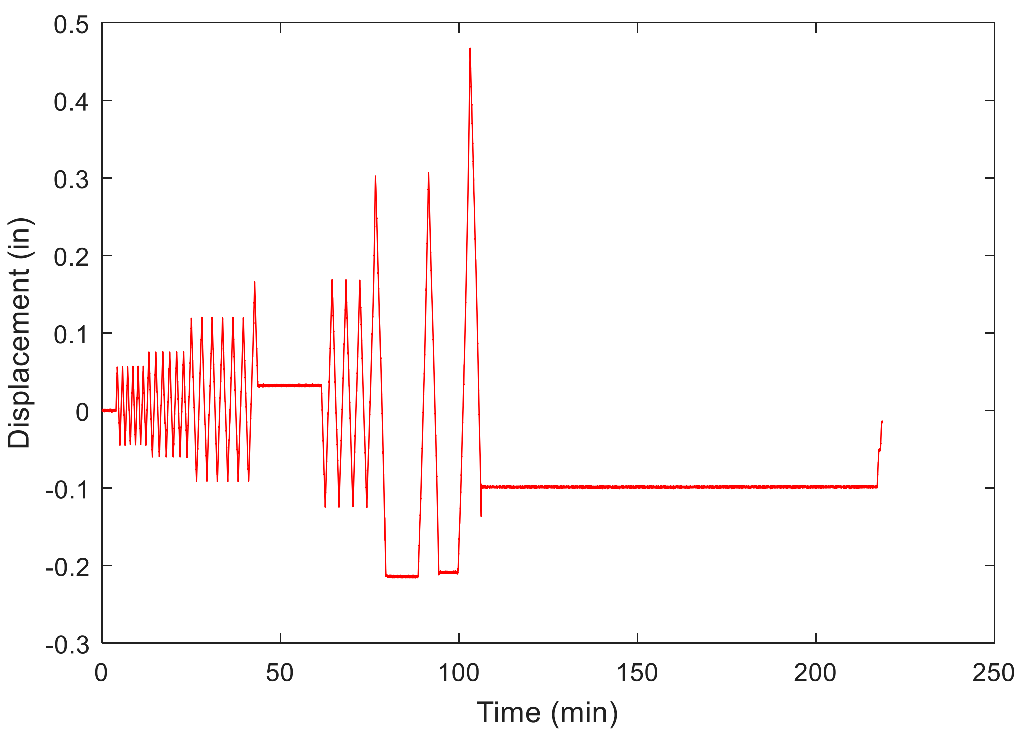


Figure T-32 SP14

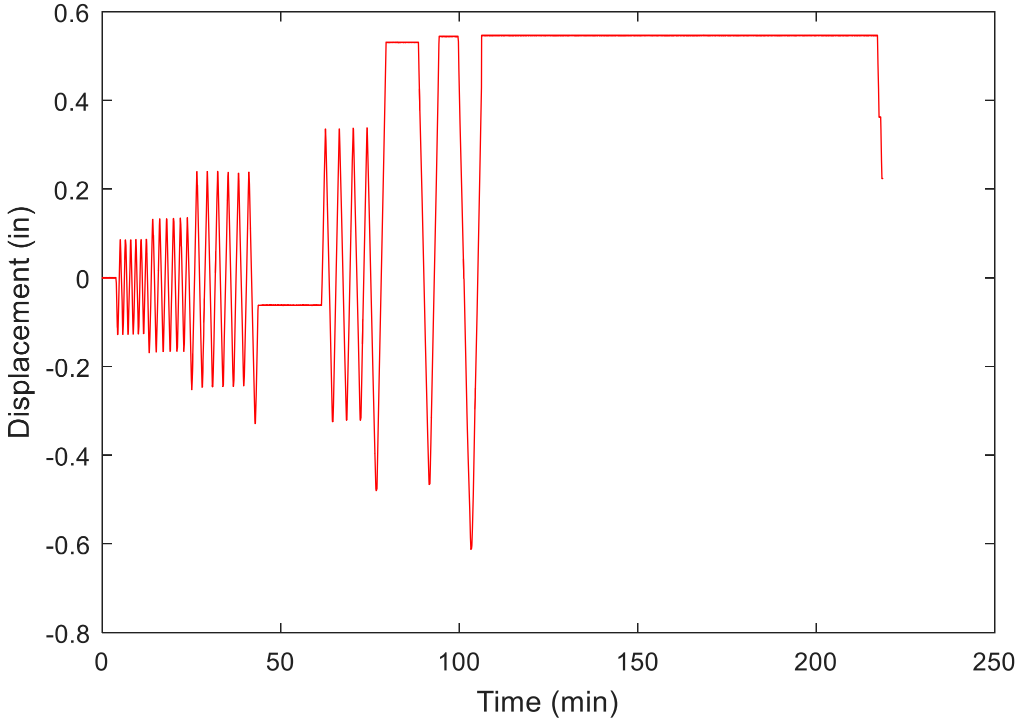


Figure T-33 LP02

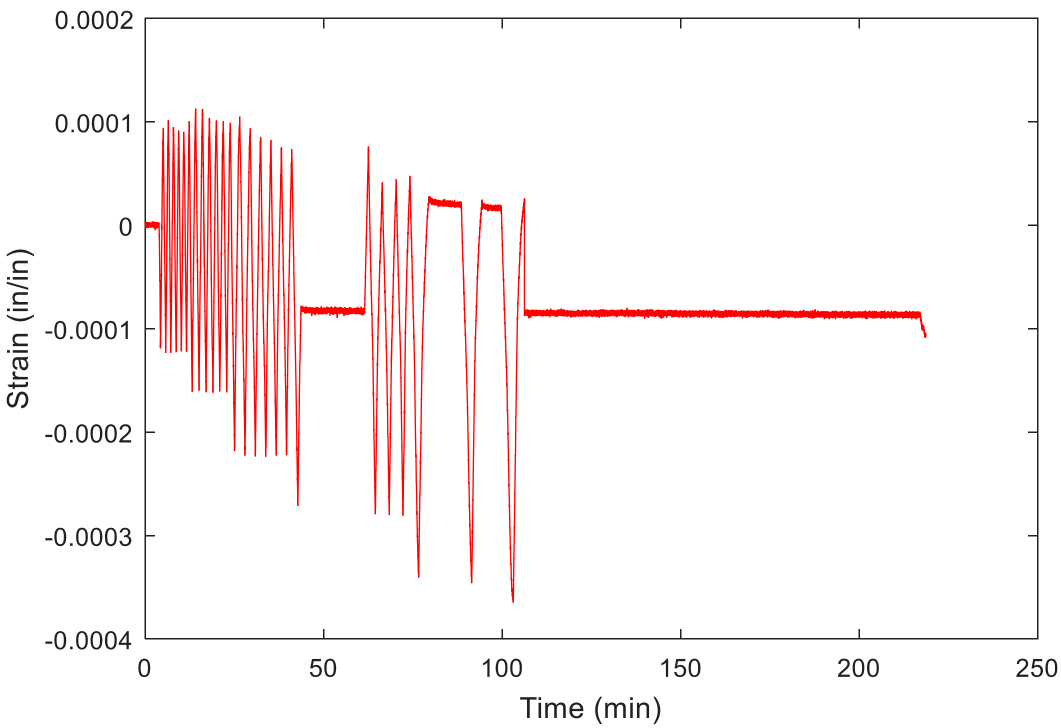


Figure T-34 SG01

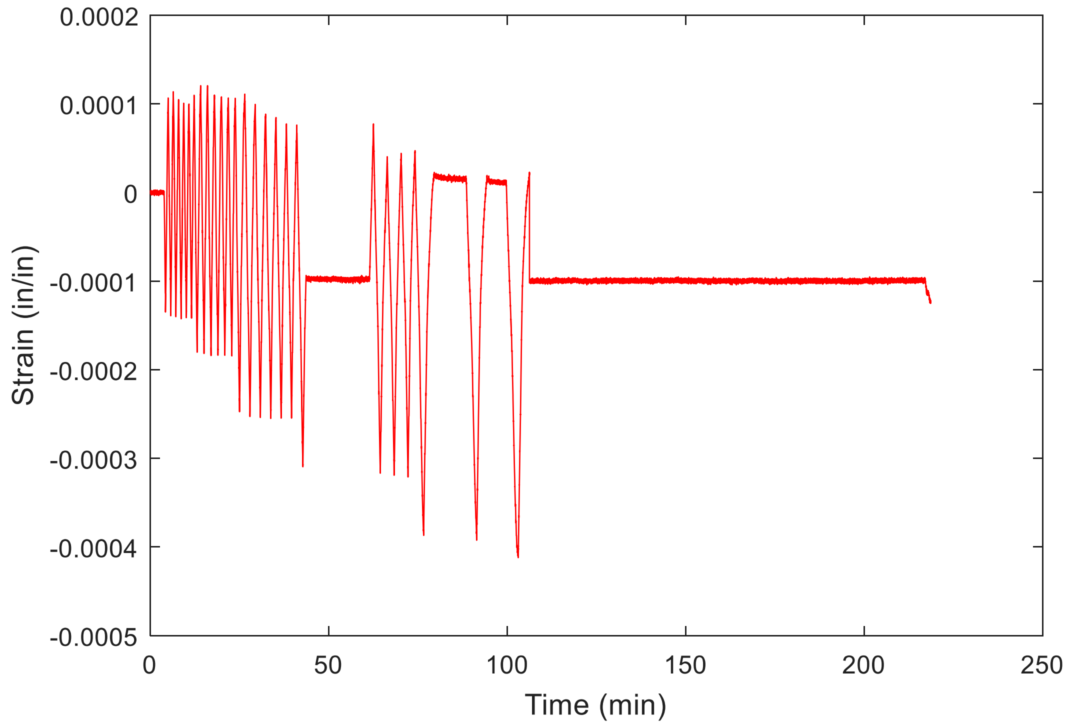


Figure T-35 SG02

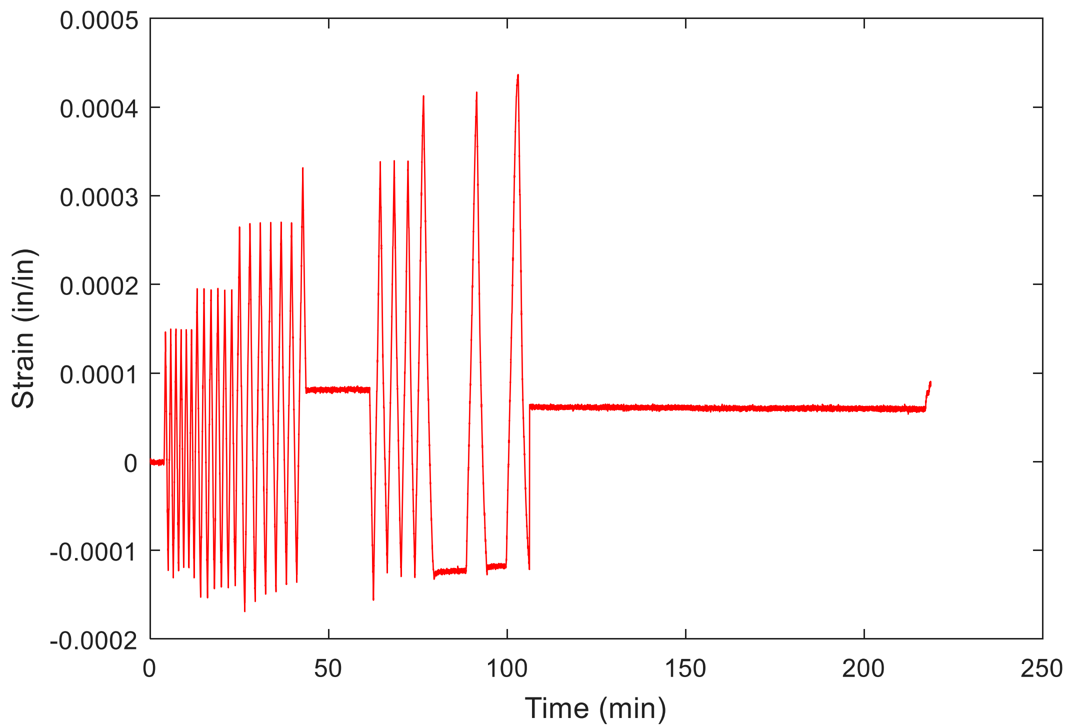


Figure T-36 SG03

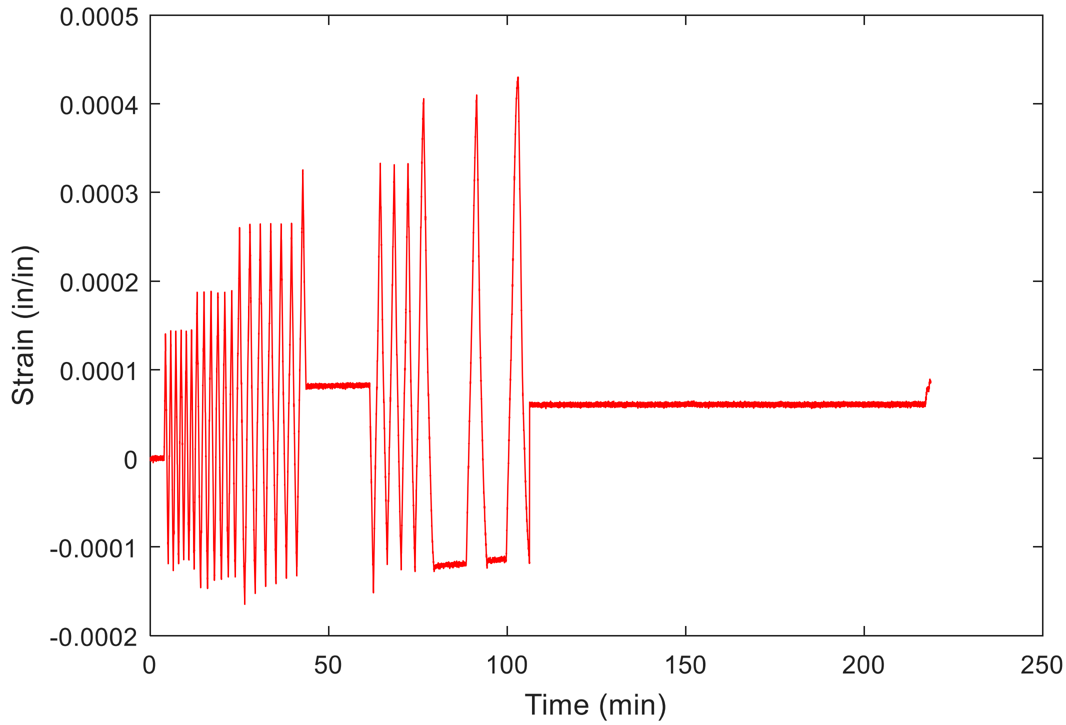


Figure T-37 SG04

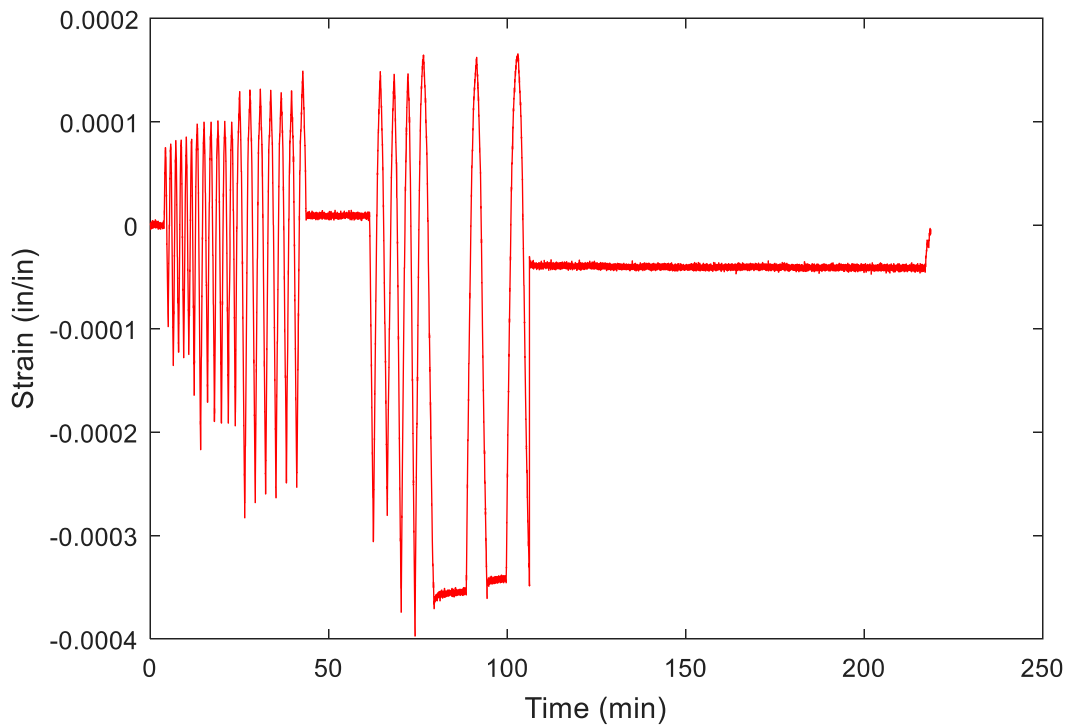


Figure T-38 SG05

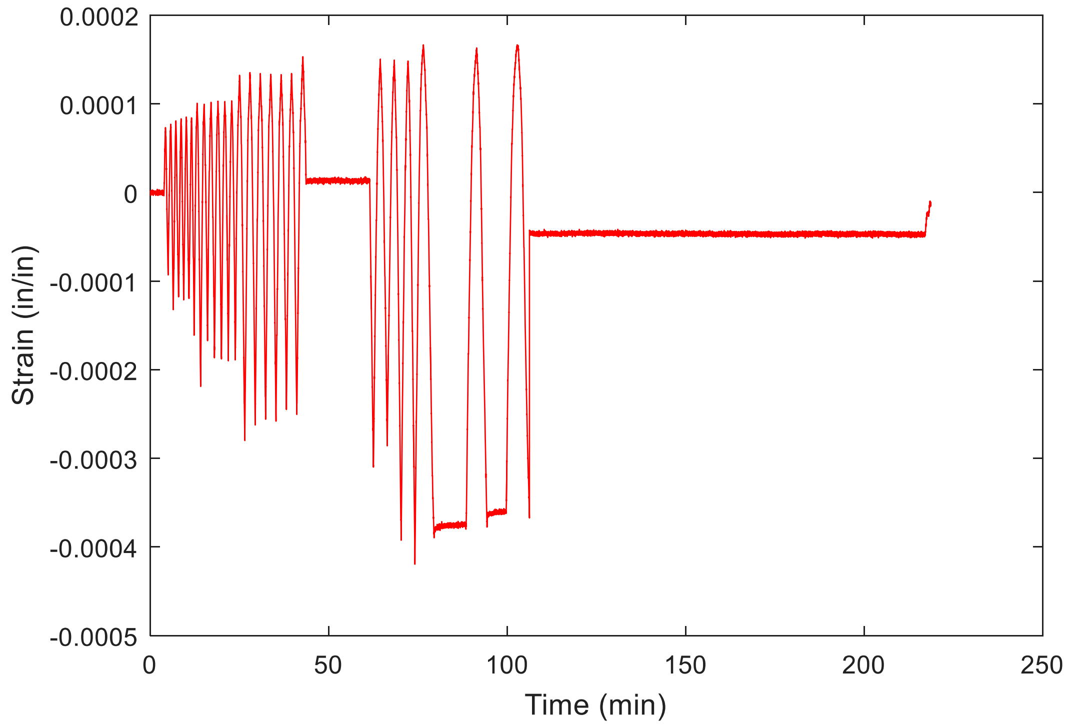


Figure T-39 SG06

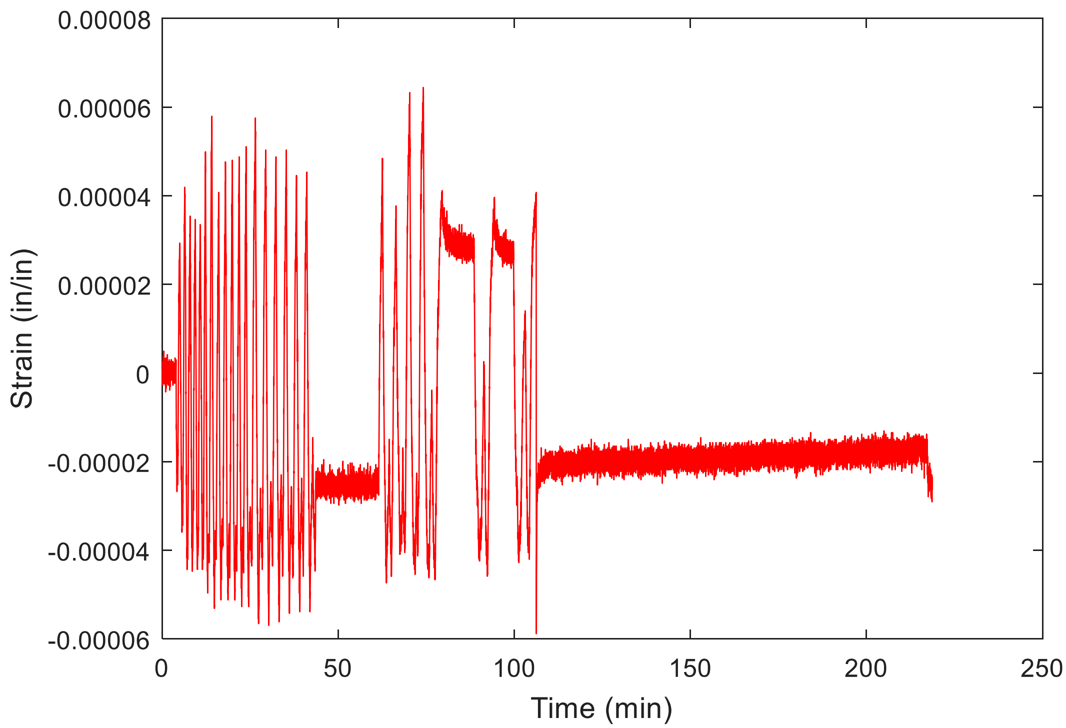


Figure T-40 SG07

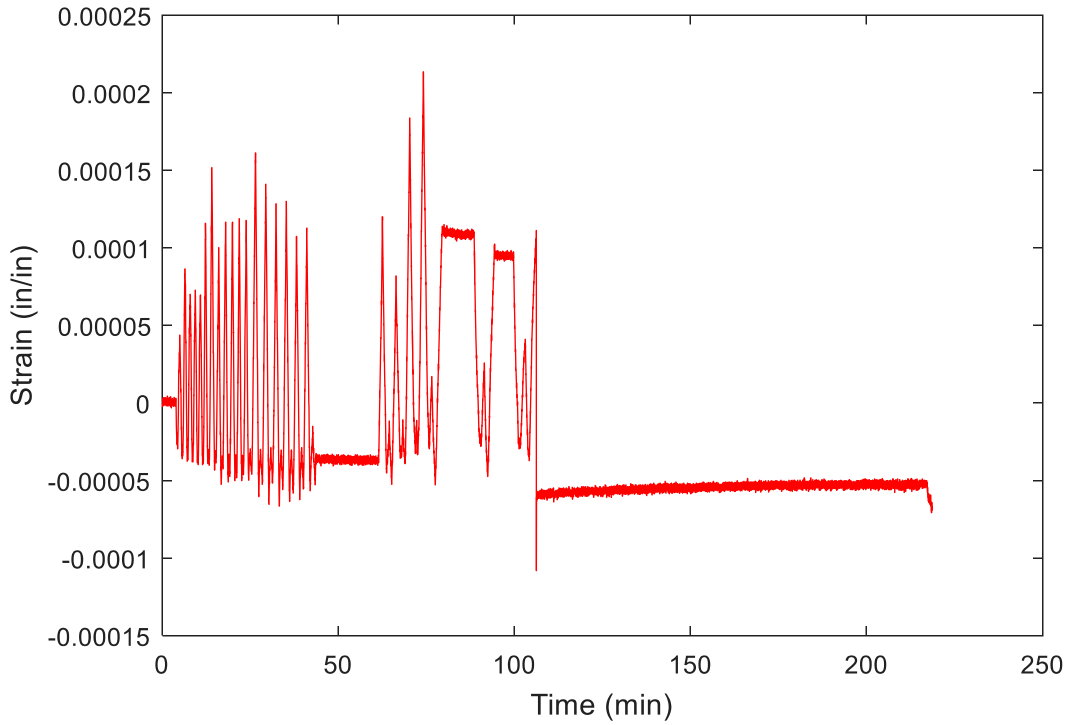


Figure T-41 SG08

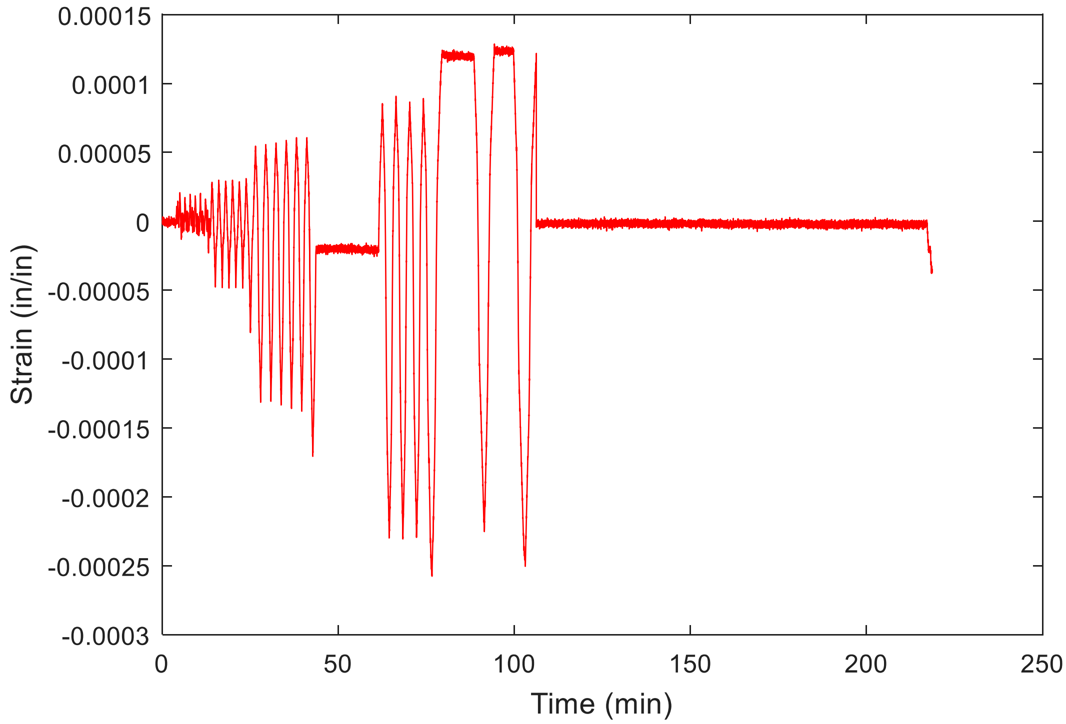


Figure T-42 SG09

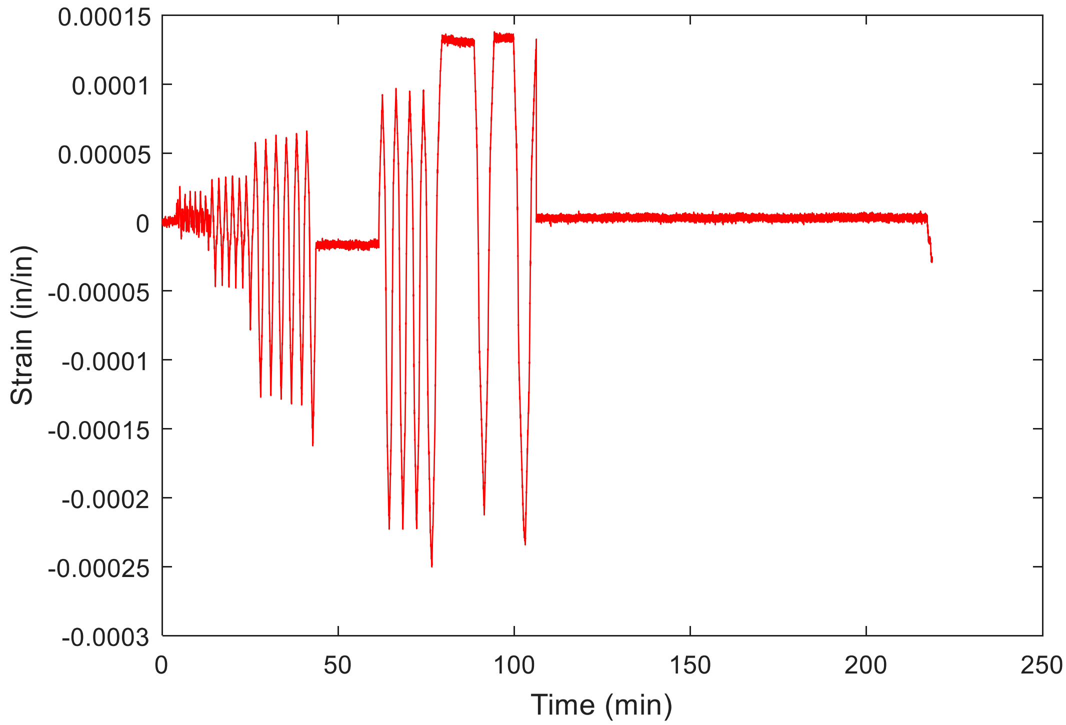


Figure T-43 SG10

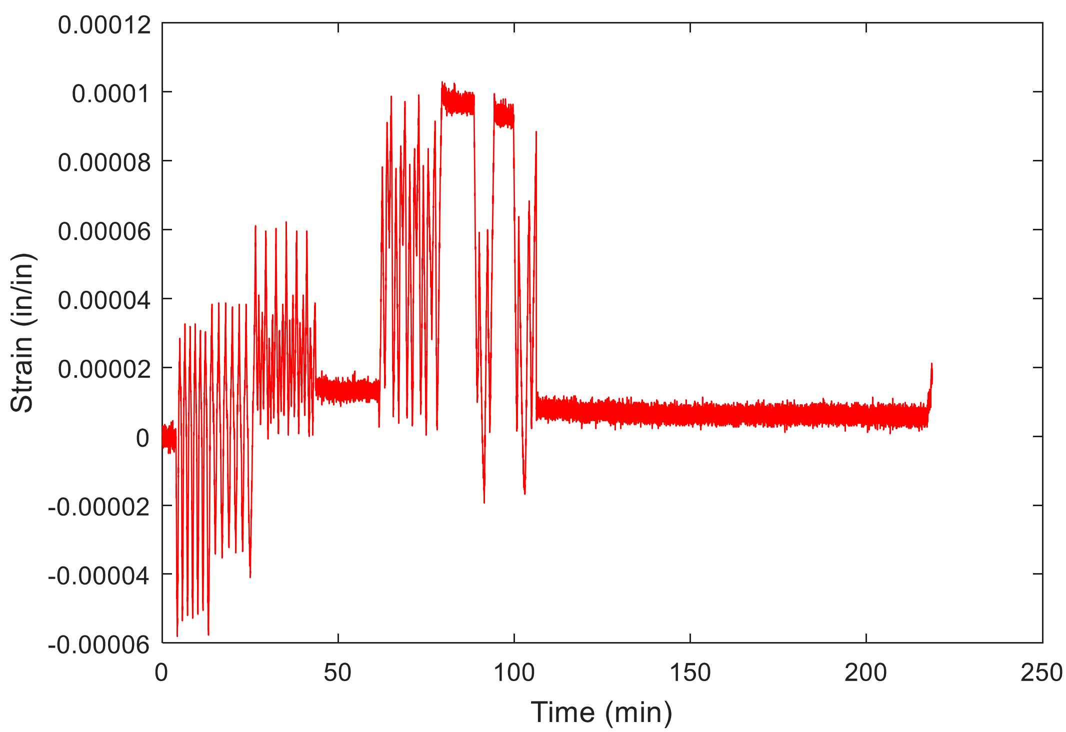


Figure T-44 SG11

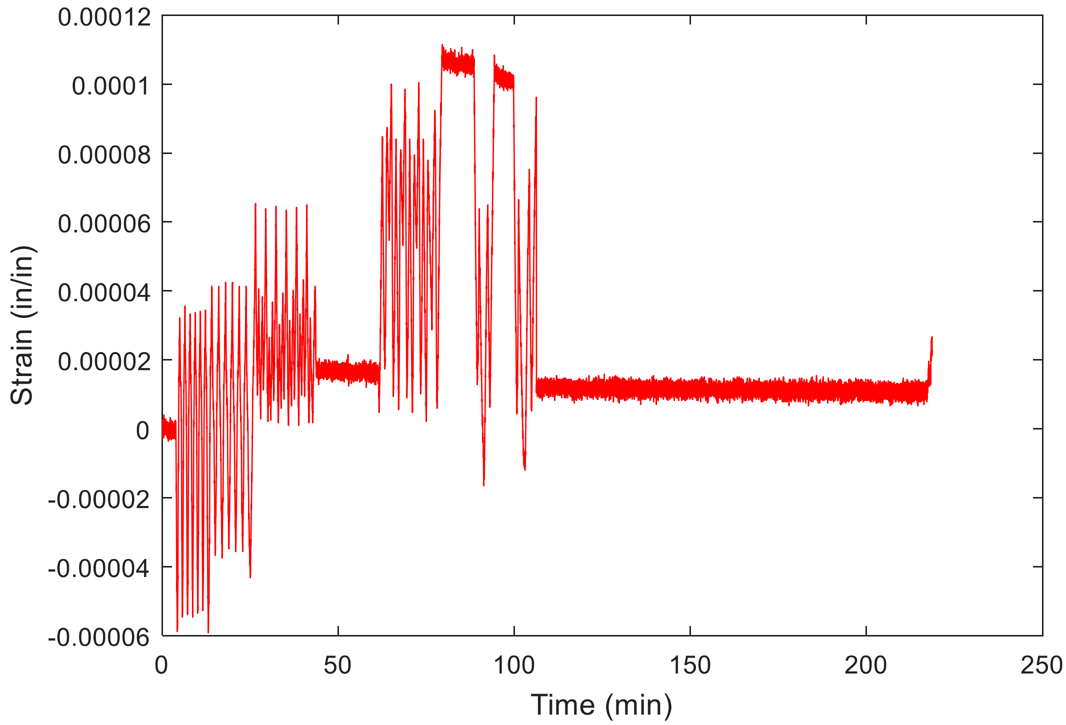


Figure T-45 SG12

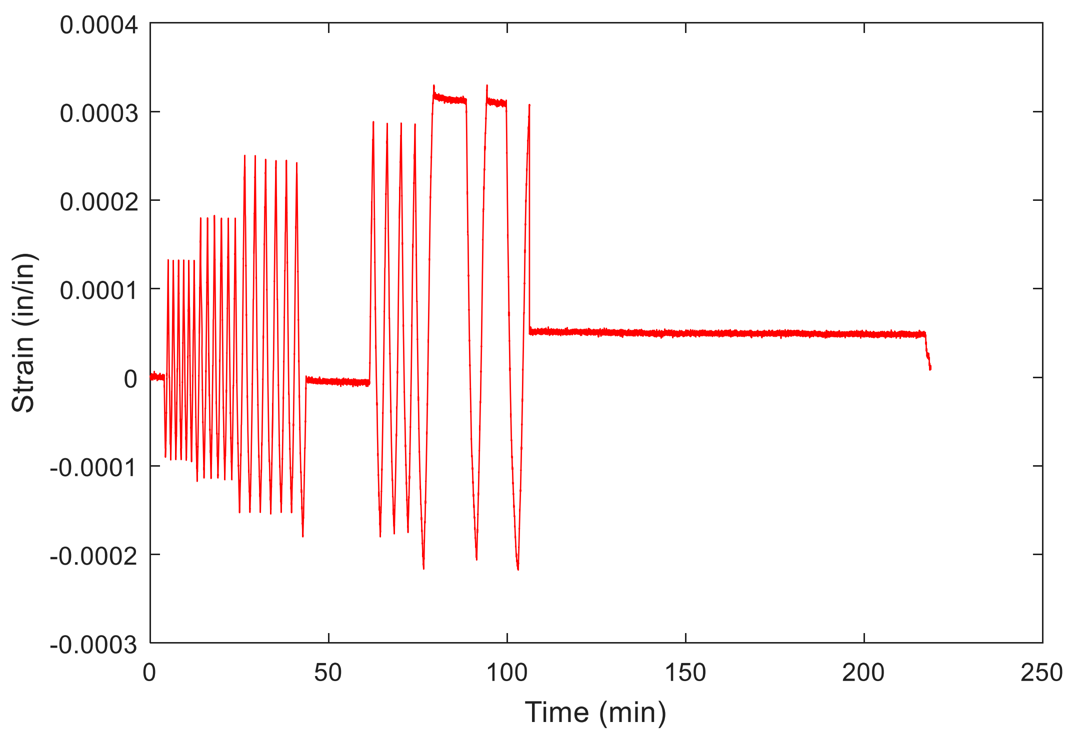


Figure T-46 SG13

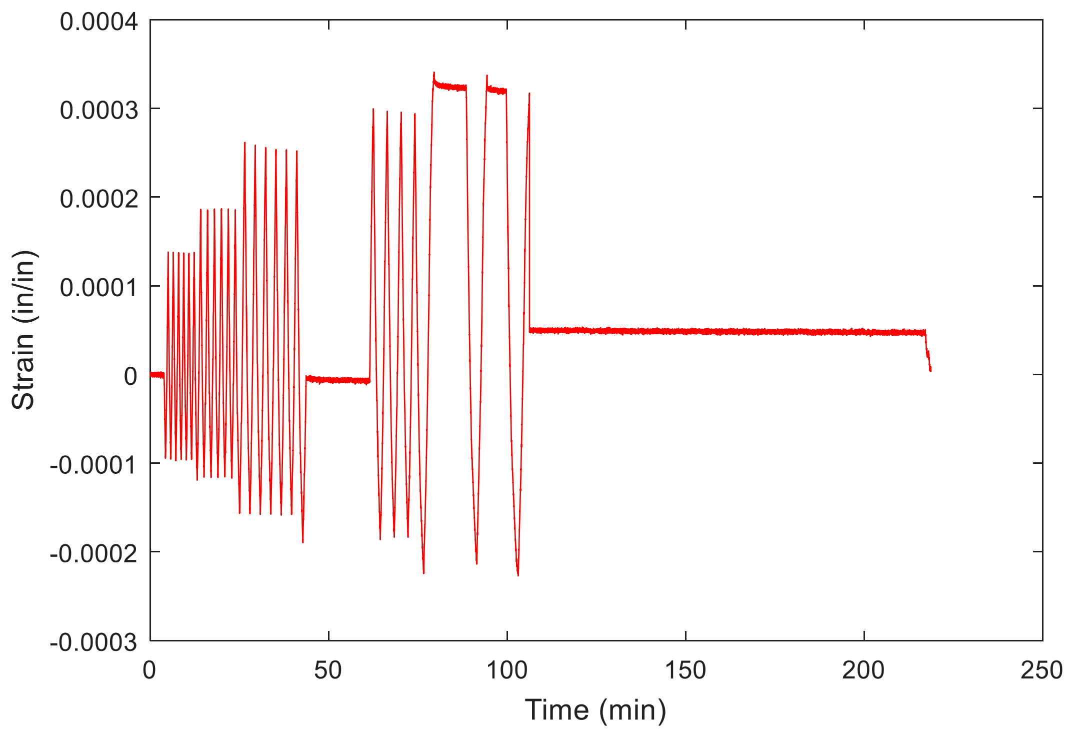


Figure T-47 SG14

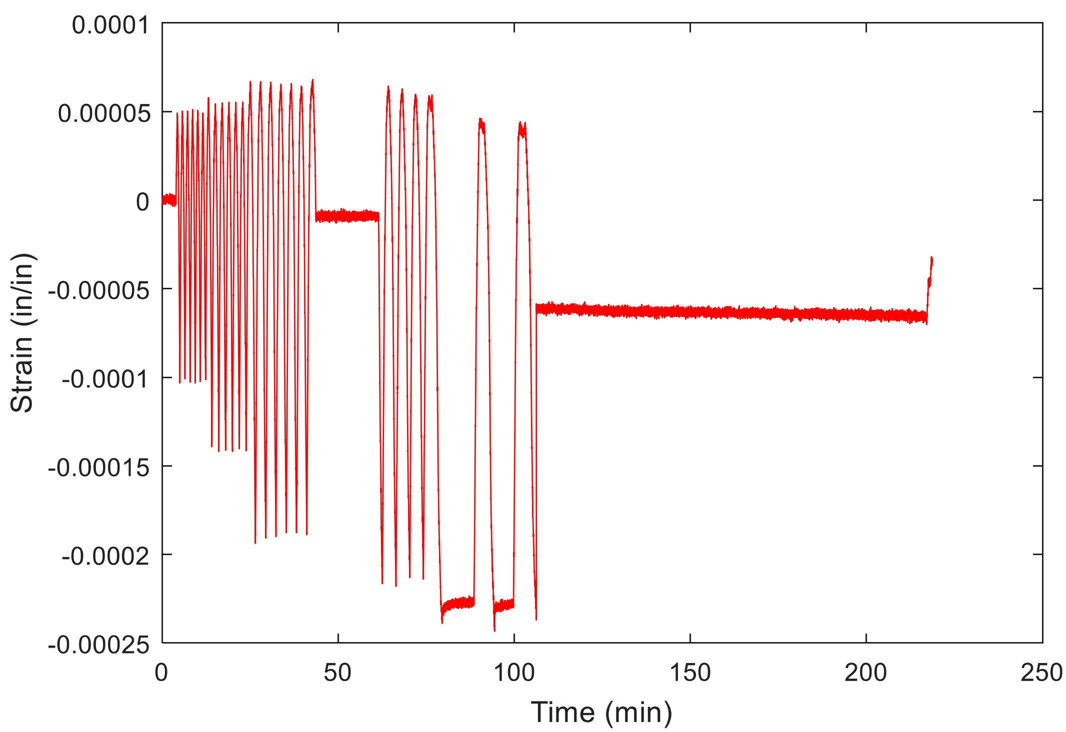


Figure T-48 SG15

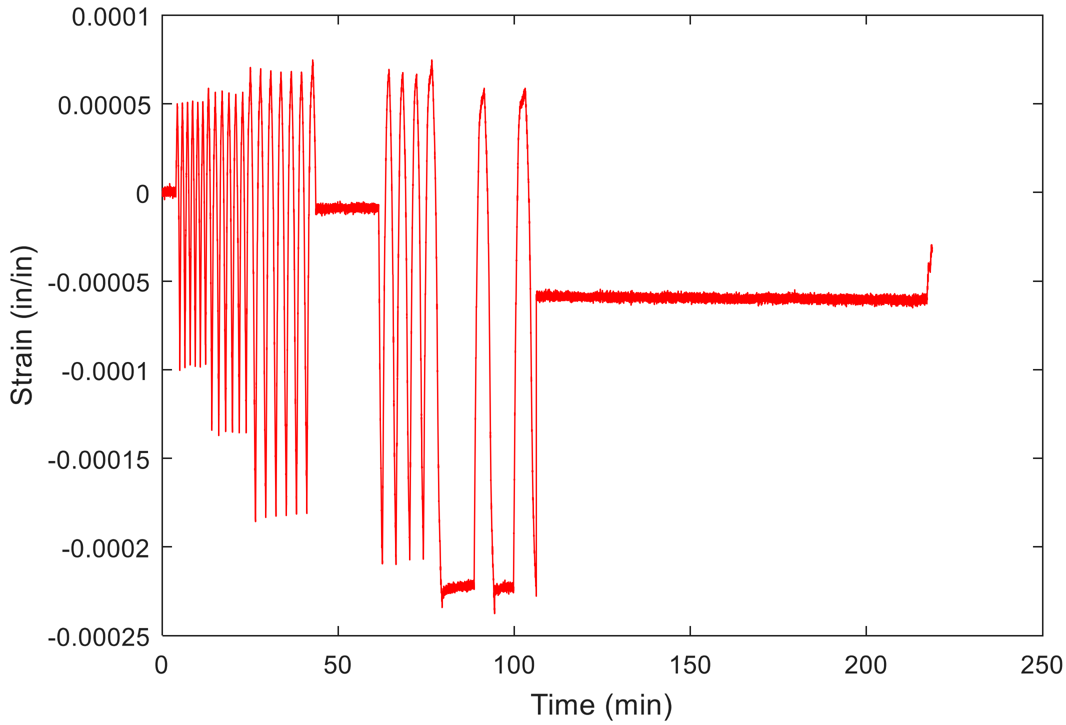


Figure T-49 SG16

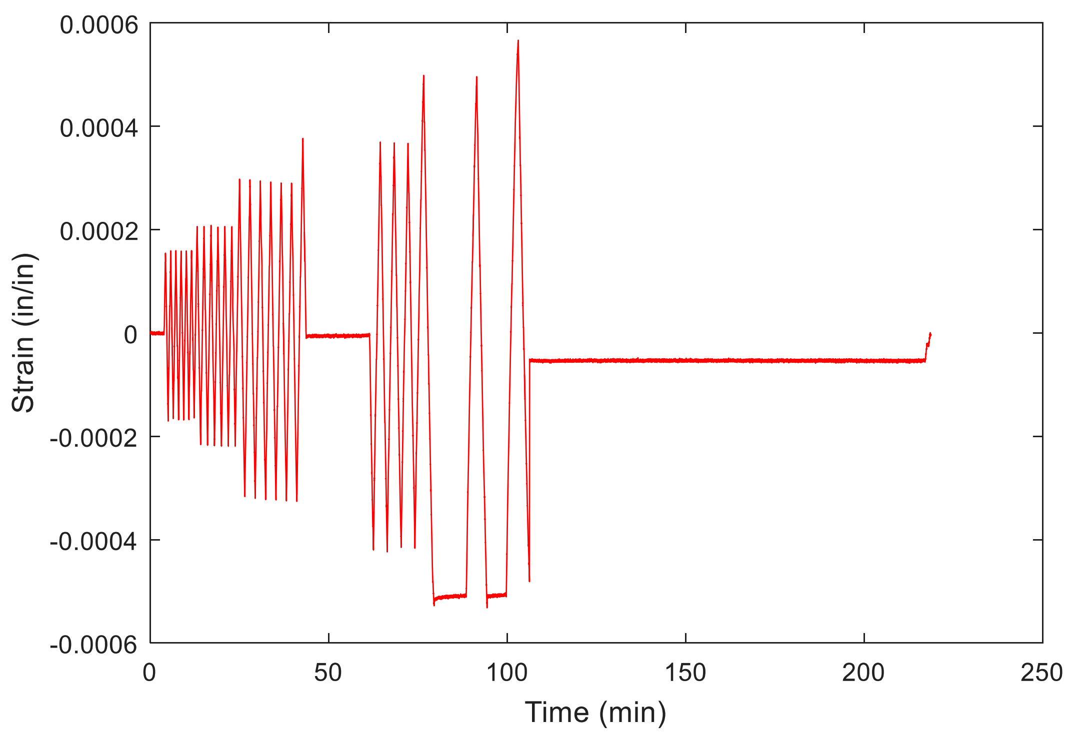


Figure T-50 SG17

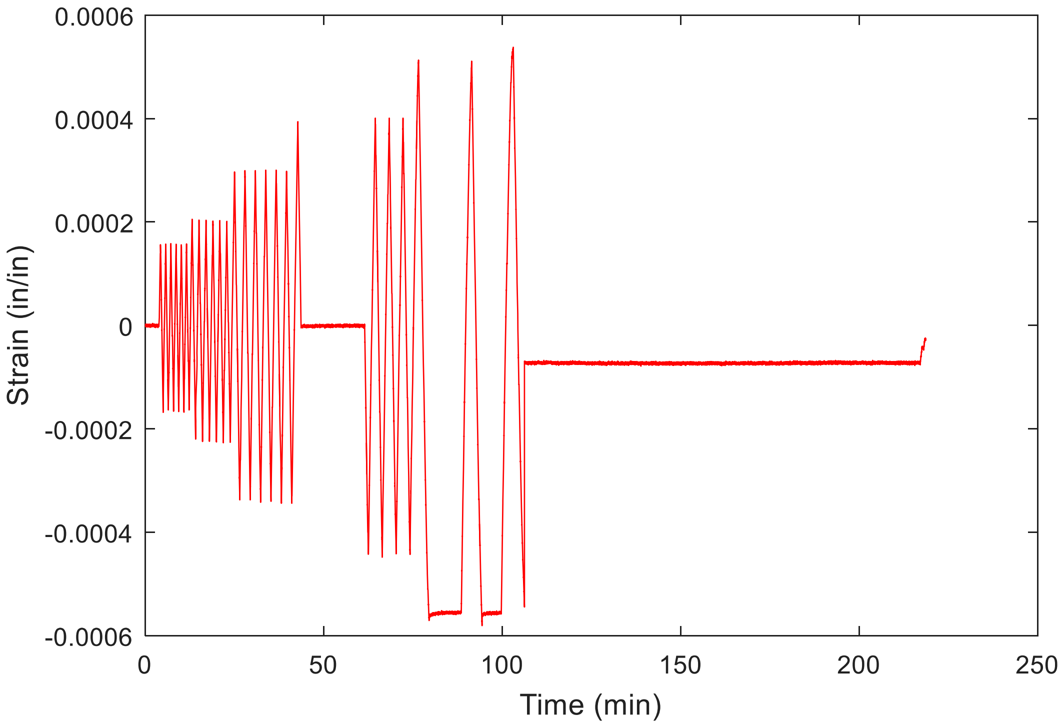


Figure T-51 SG18

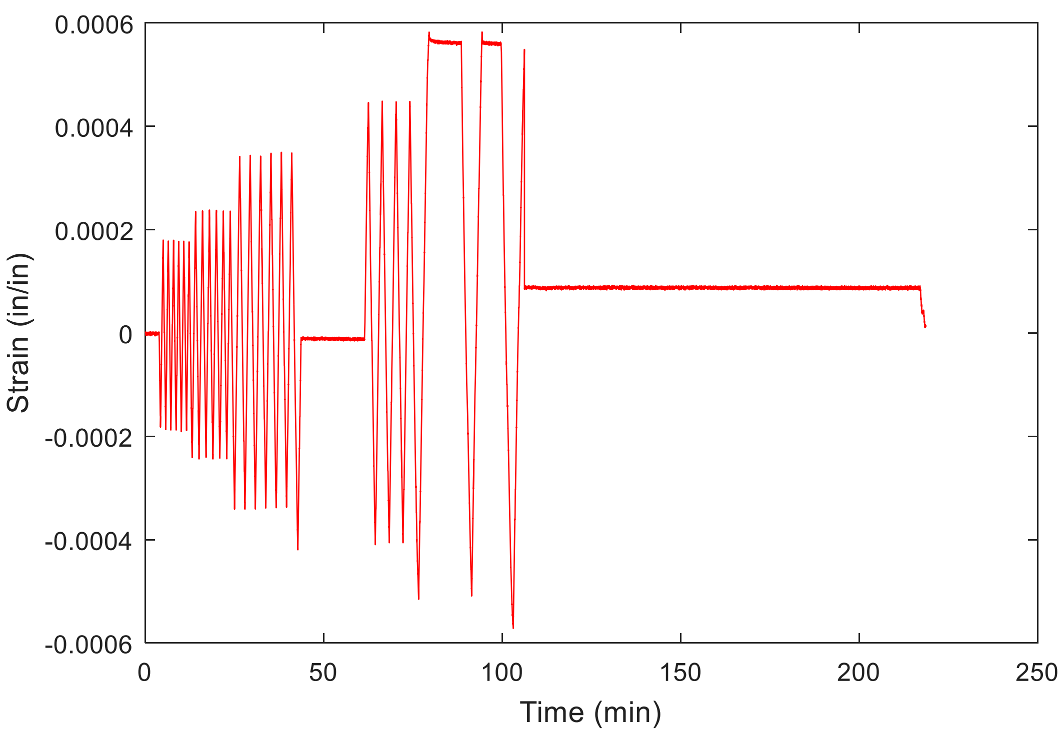


Figure T-52 SG19

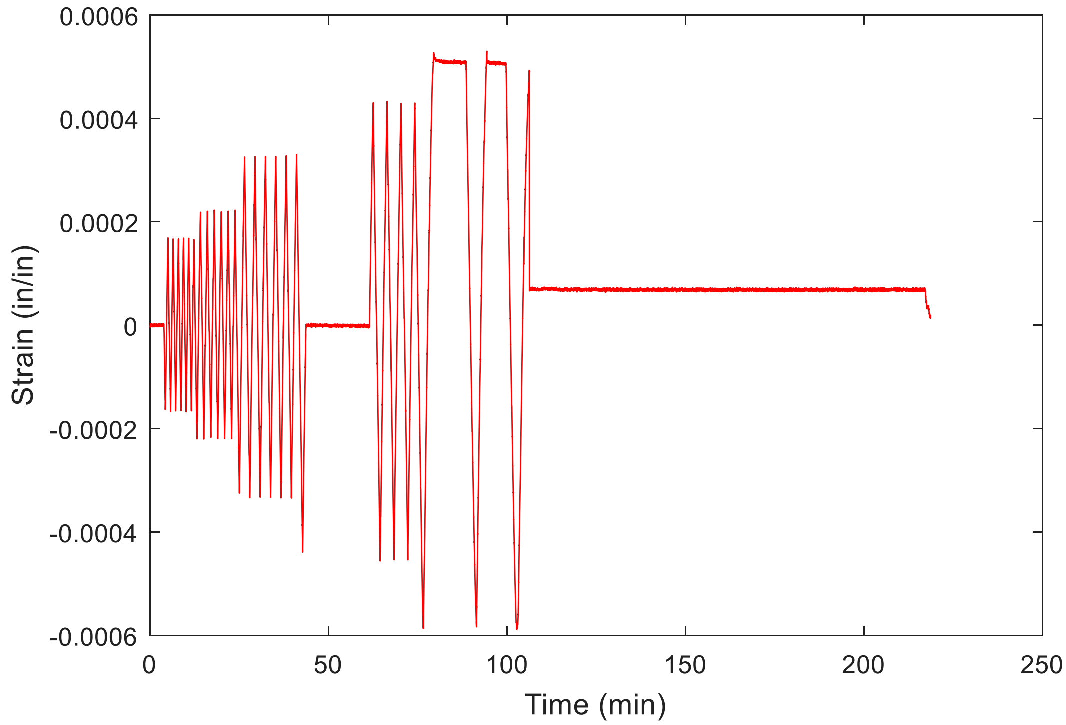


Figure T-53 SG20

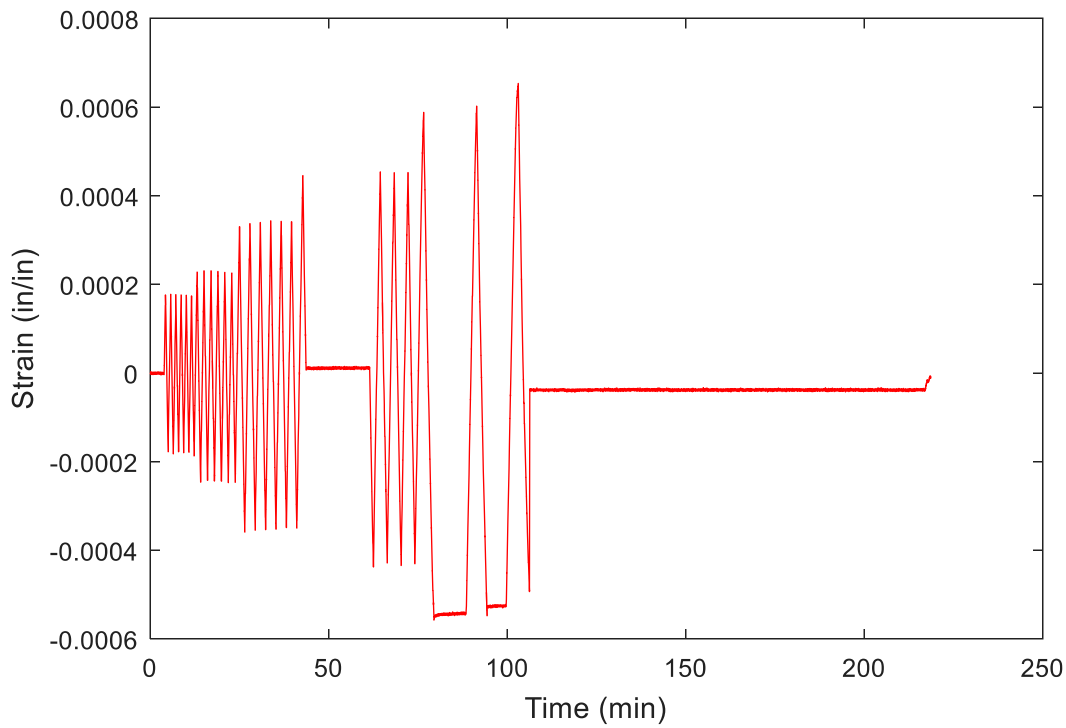


Figure T-54 SG21

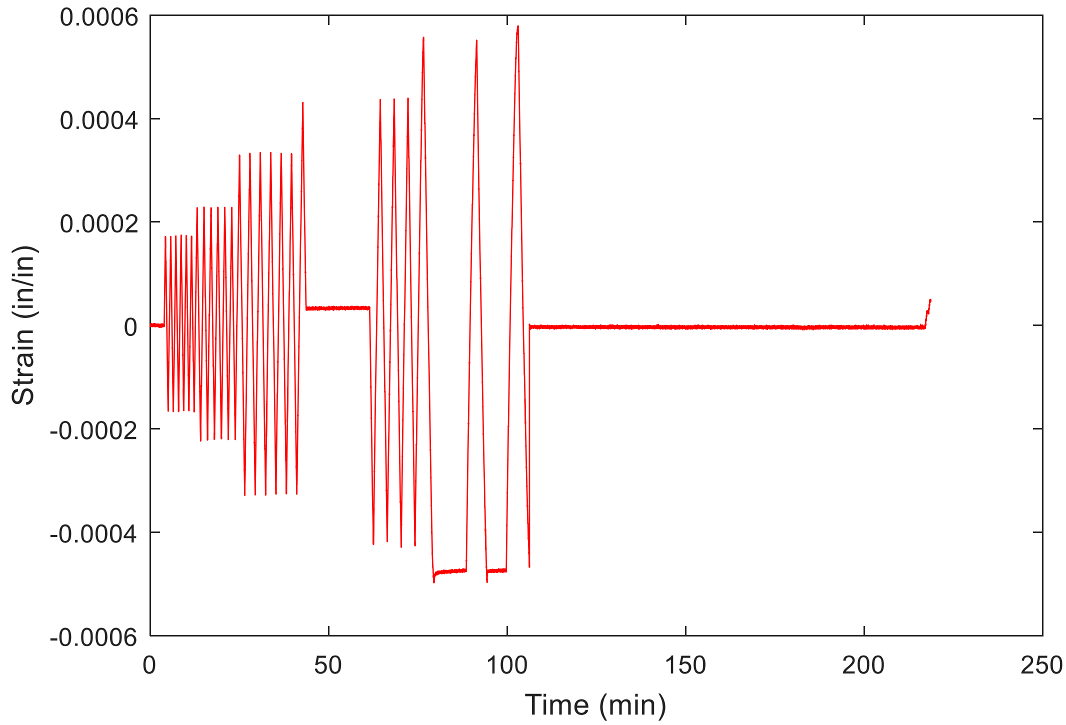


Figure T-55 SG22

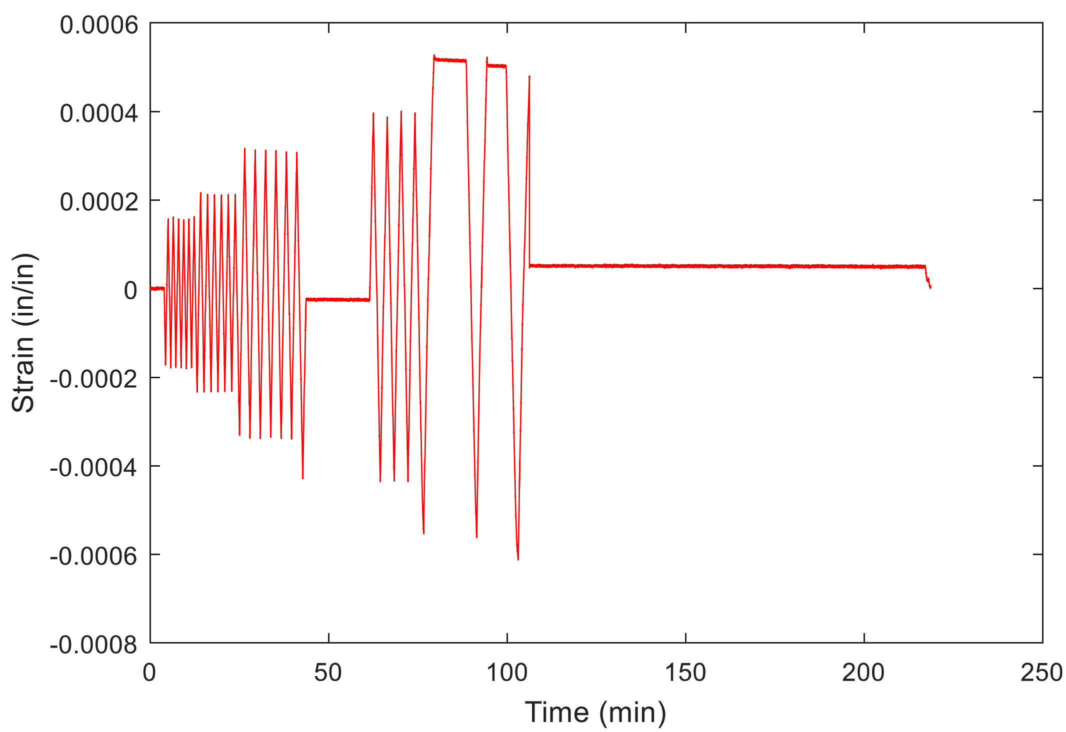


Figure T-56 SG23

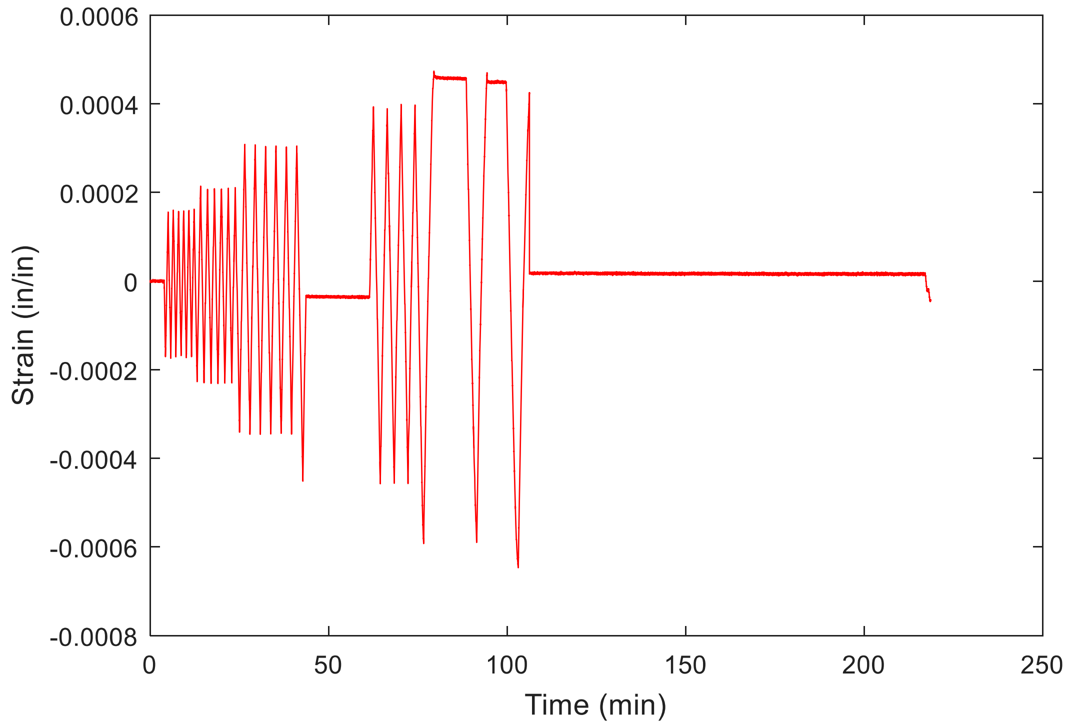


Figure T-57 SG24

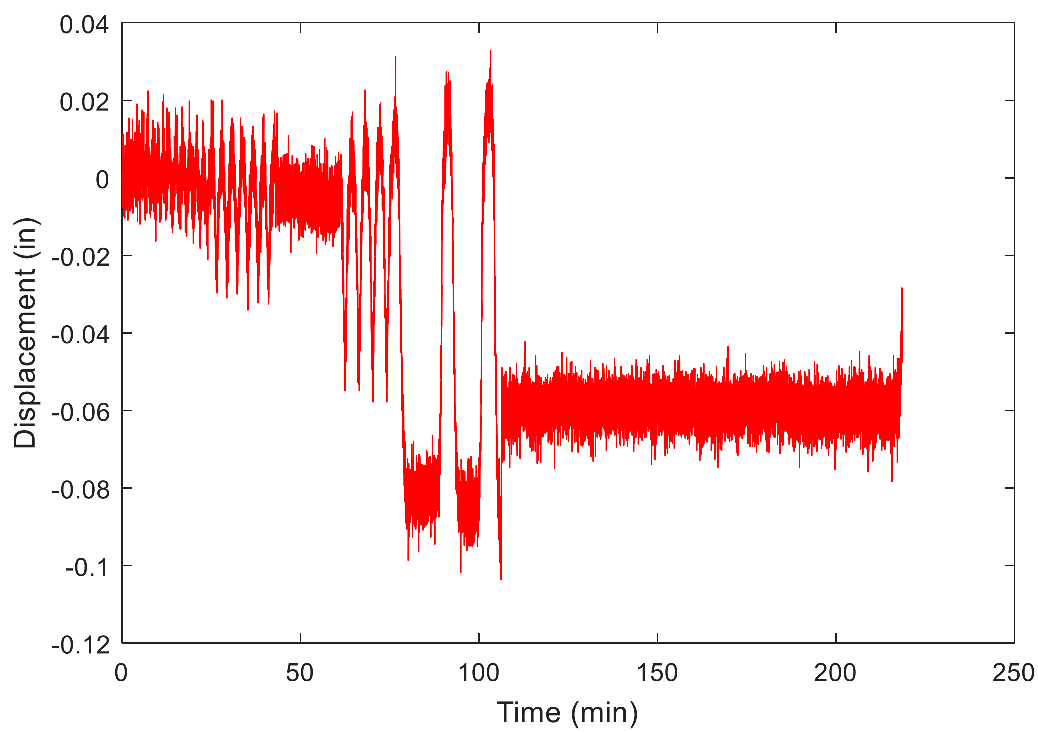


Figure T-58 CLP01

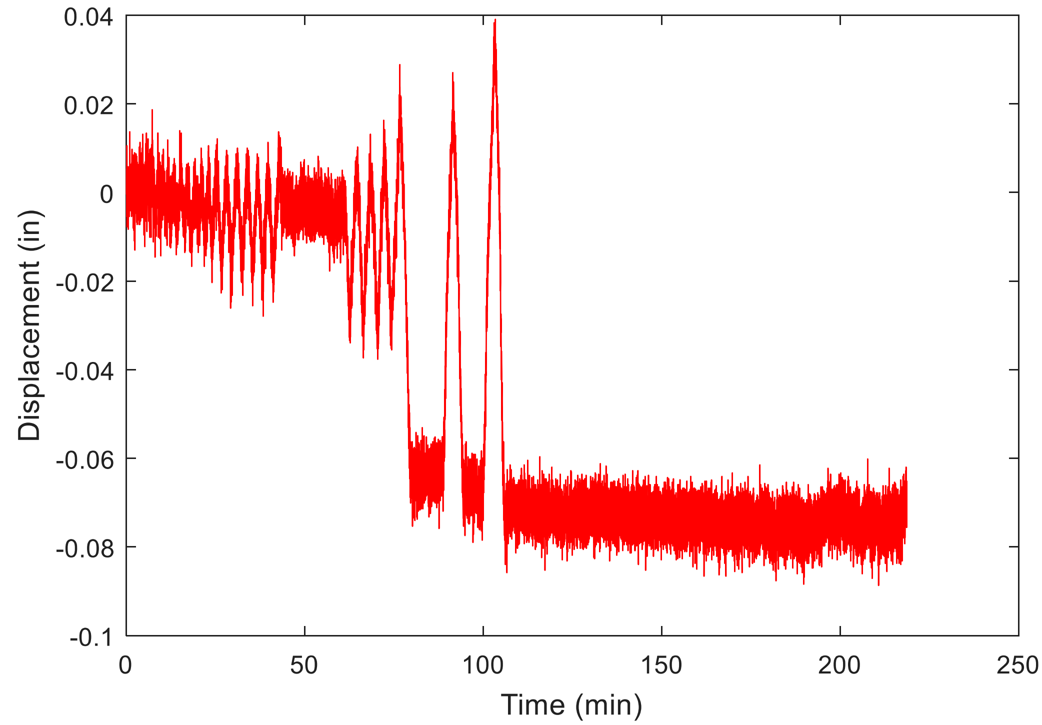


Figure T-59 CLP02

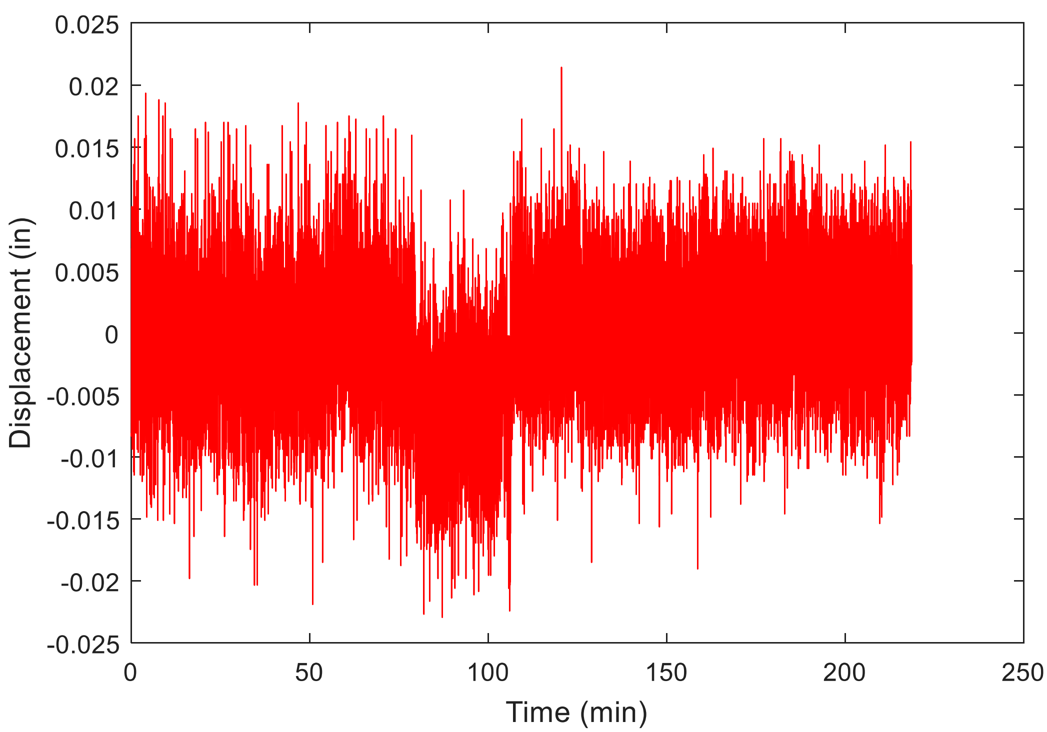


Figure T-60 CLP03

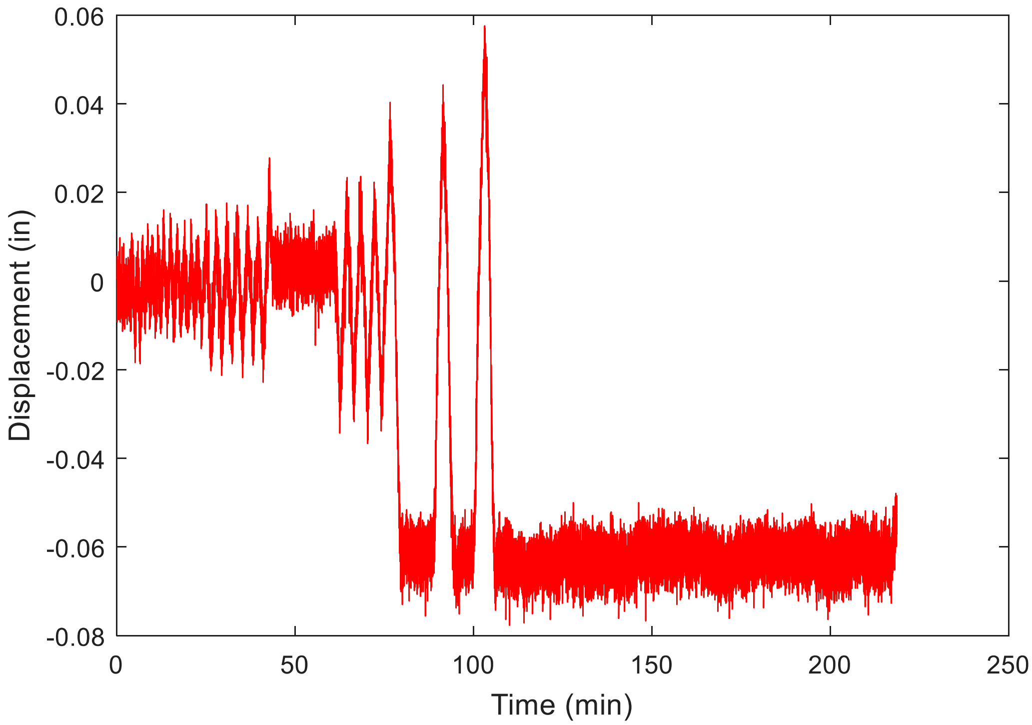


Figure T-61 CLP04

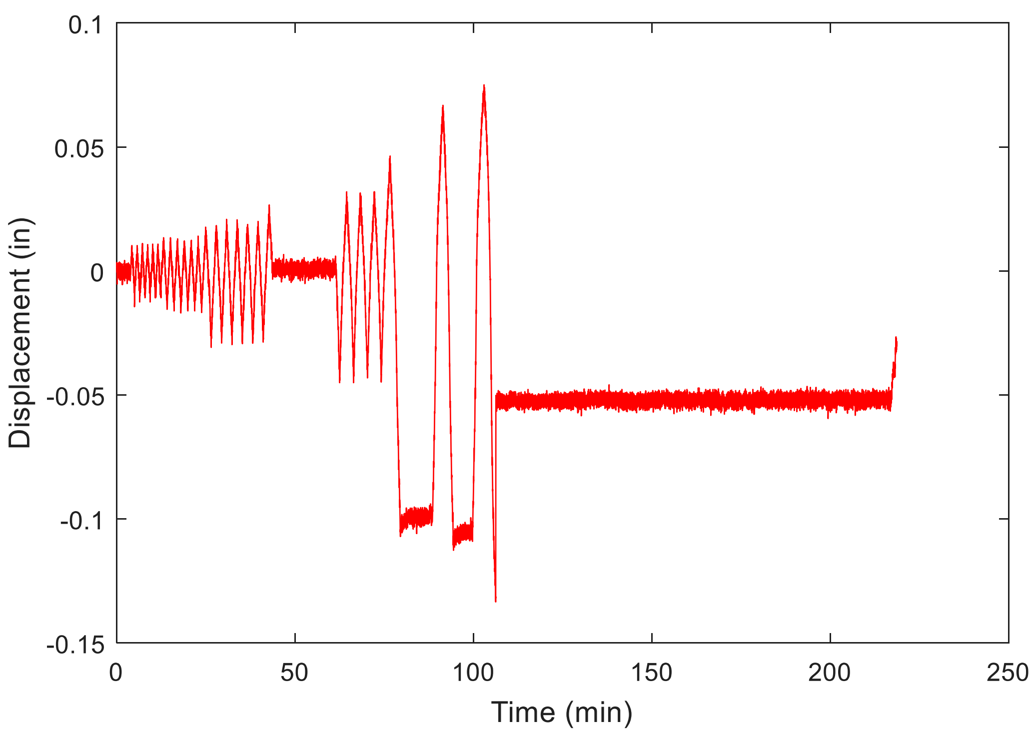


Figure T-62 CLP05

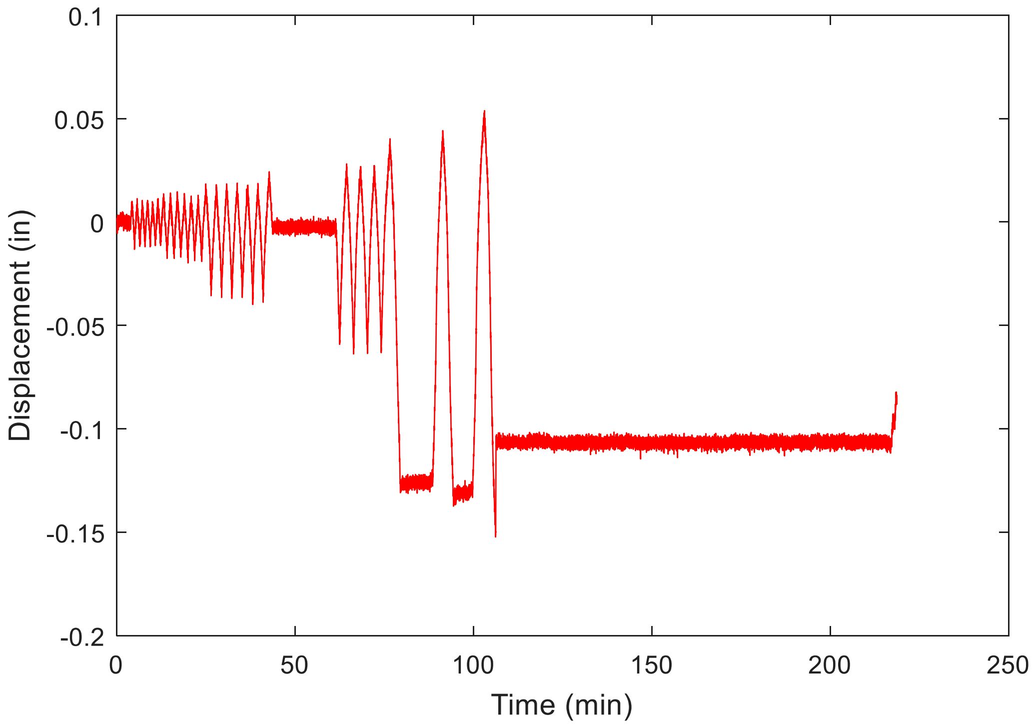


Figure T-63 CLP06

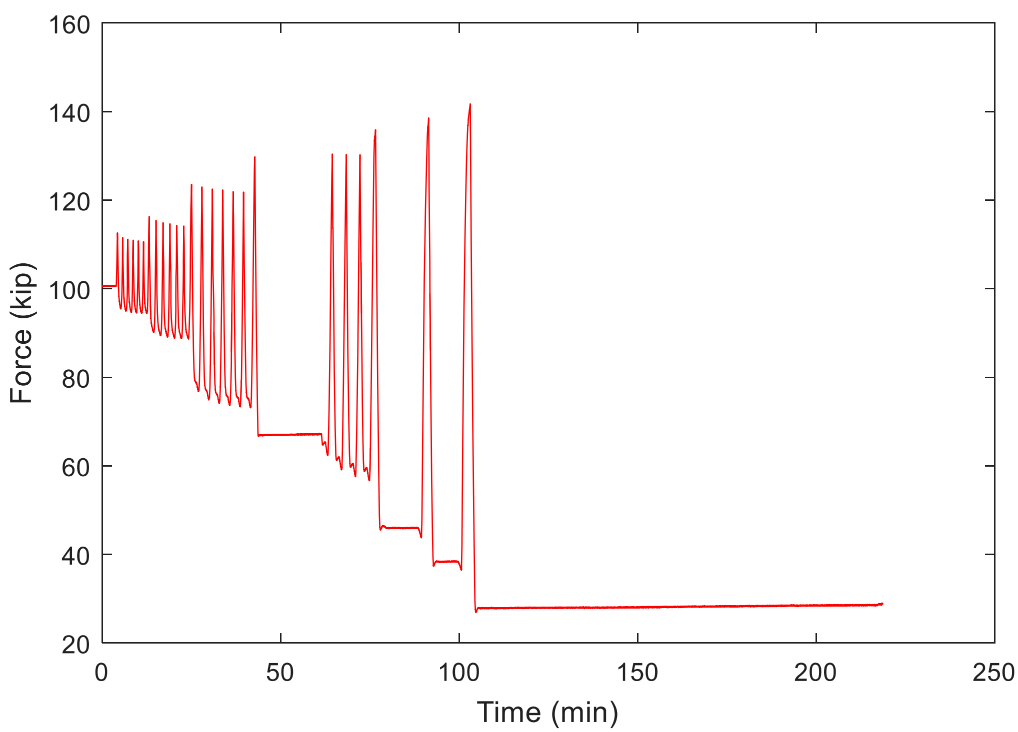


Figure T-64 Force in BSG01

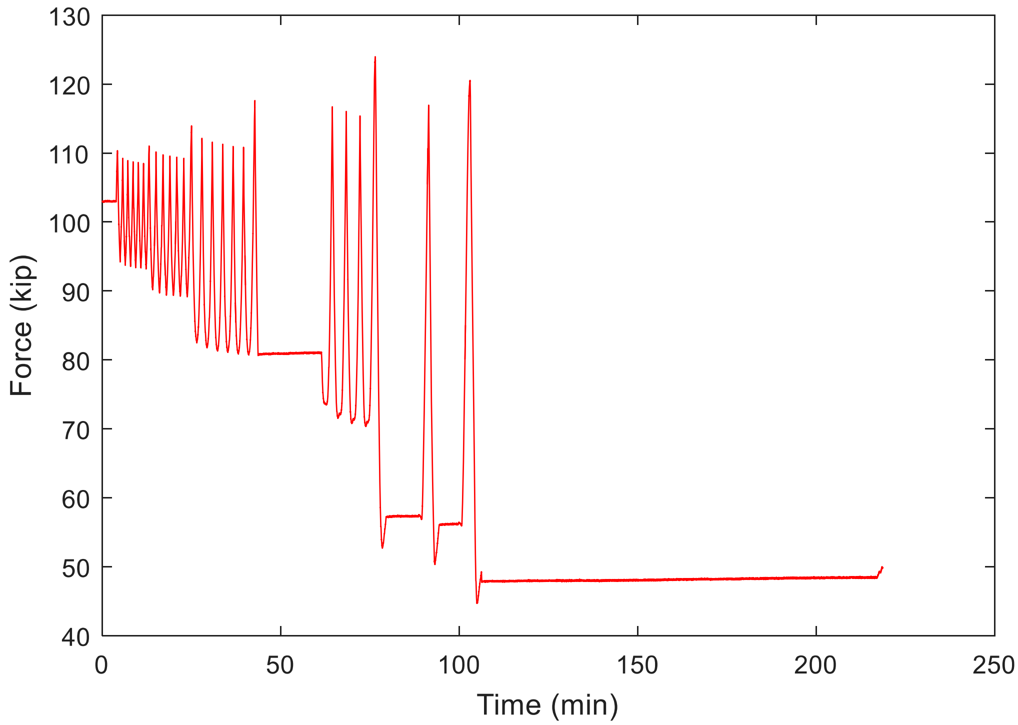


Figure T-65 Force in BSG02

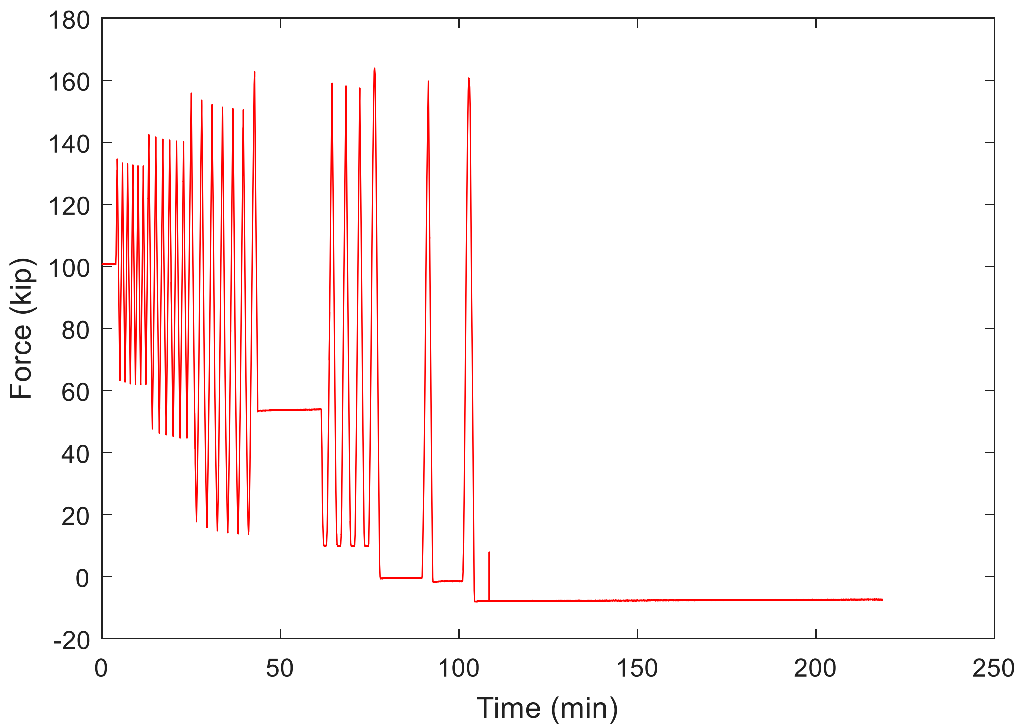


Figure T-66 Force in BSG03

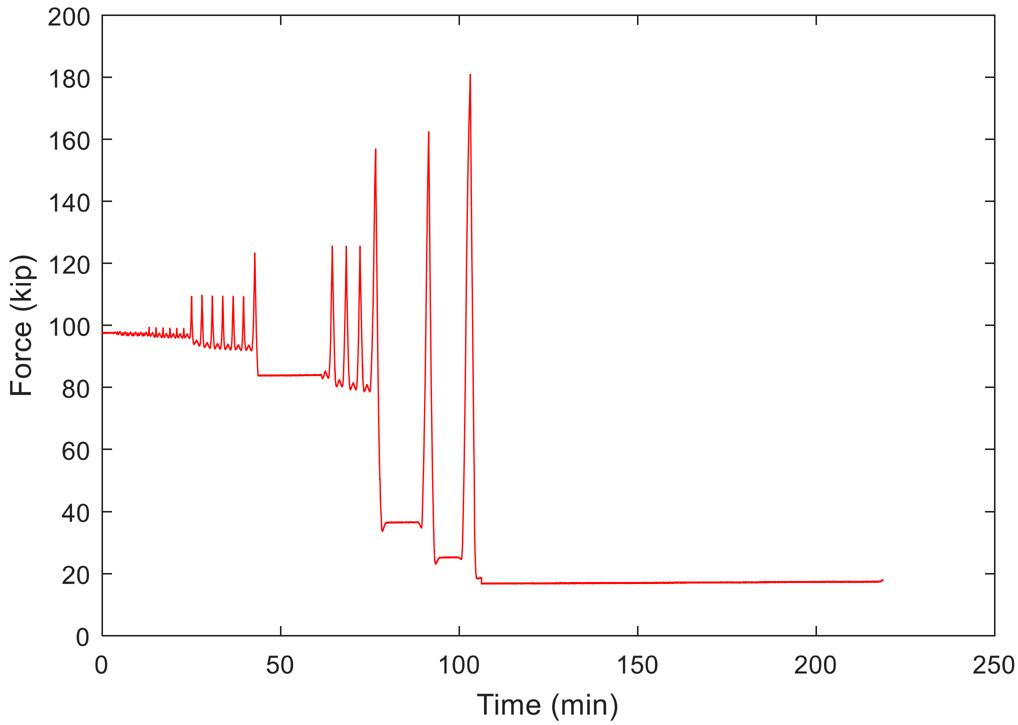


Figure T-67 Force in BSG04

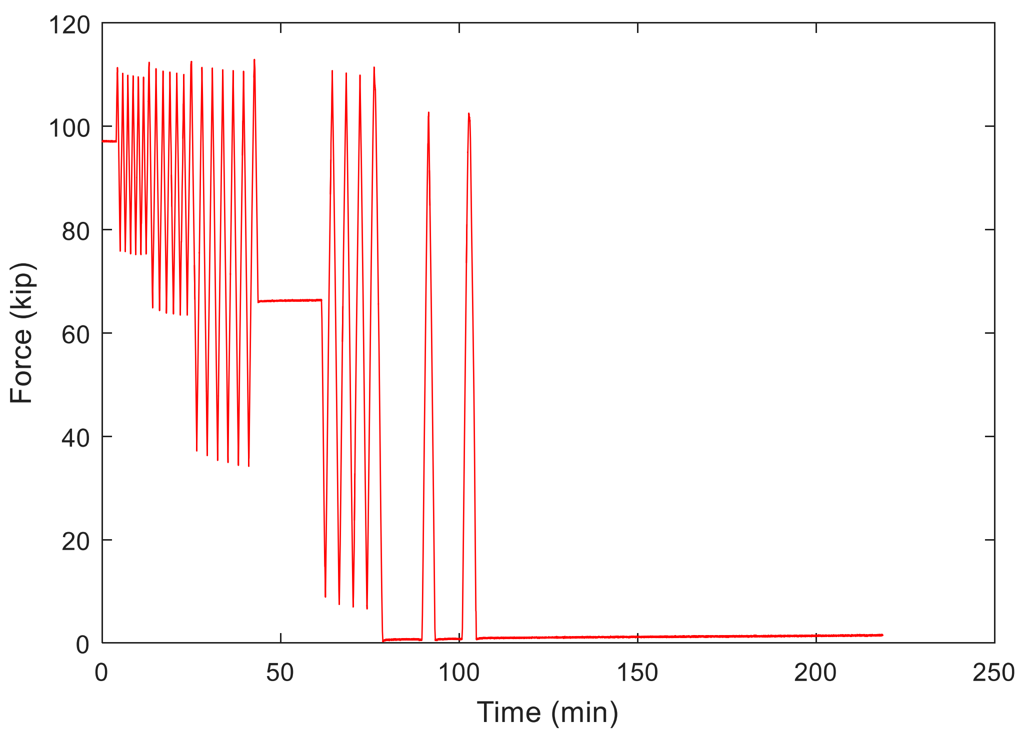


Figure T-68 Force in BSG05

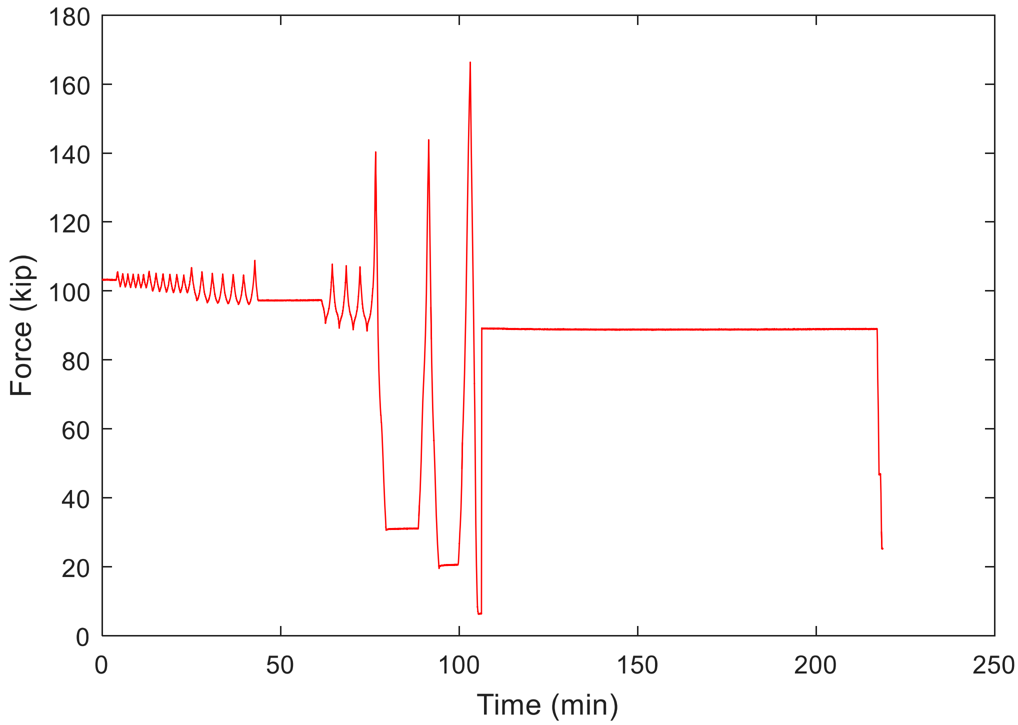


Figure T-69 Force in BSG06

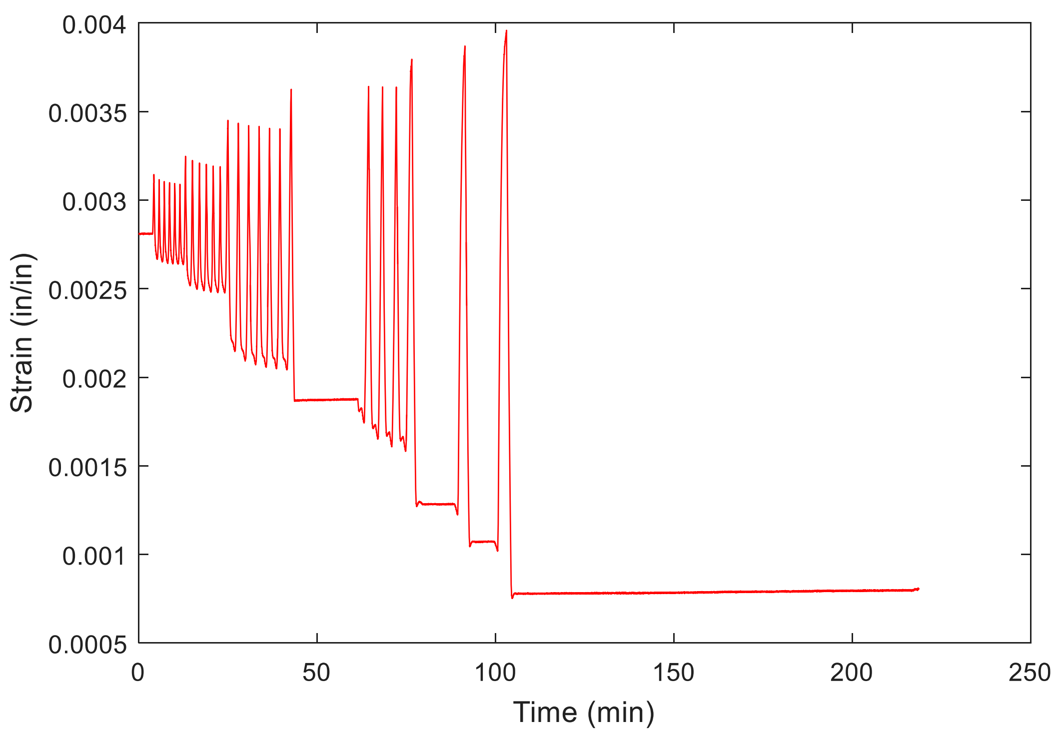


Figure T-70 Strain in BSG01

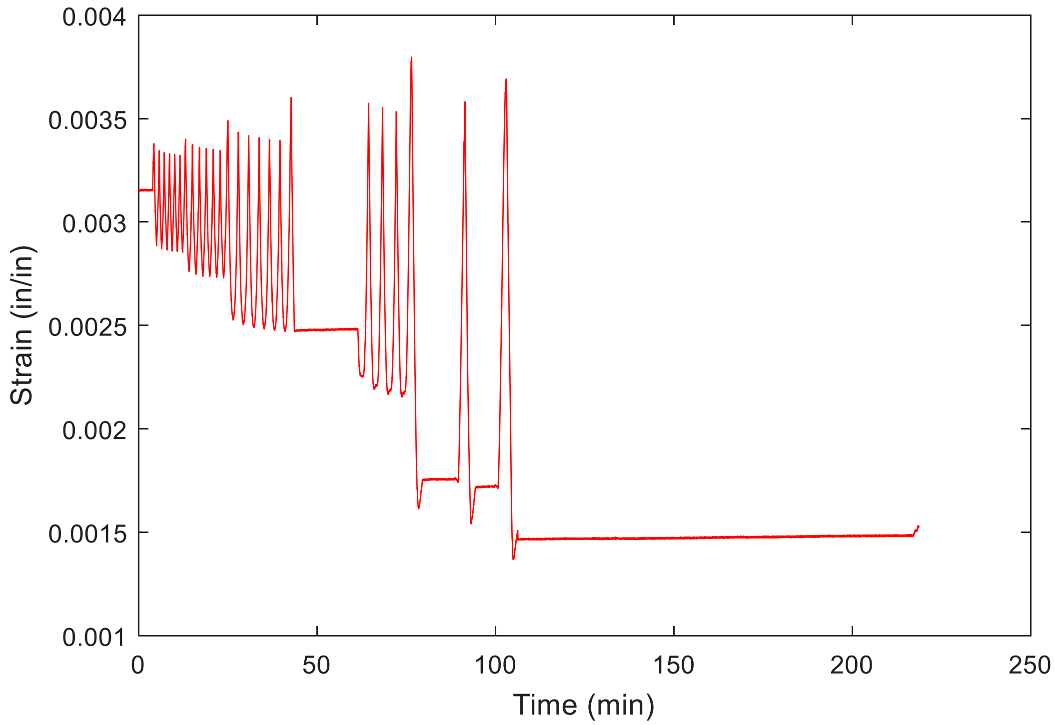


Figure T-71 Strain in BSG02

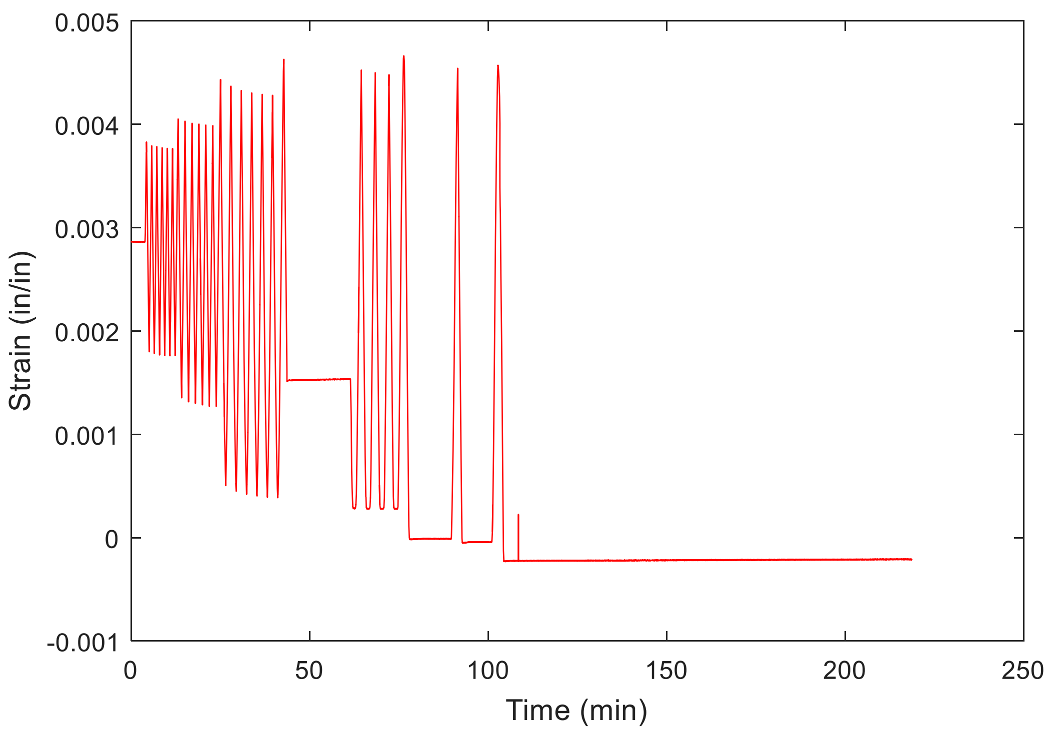


Figure T-72 Strain in BSG03

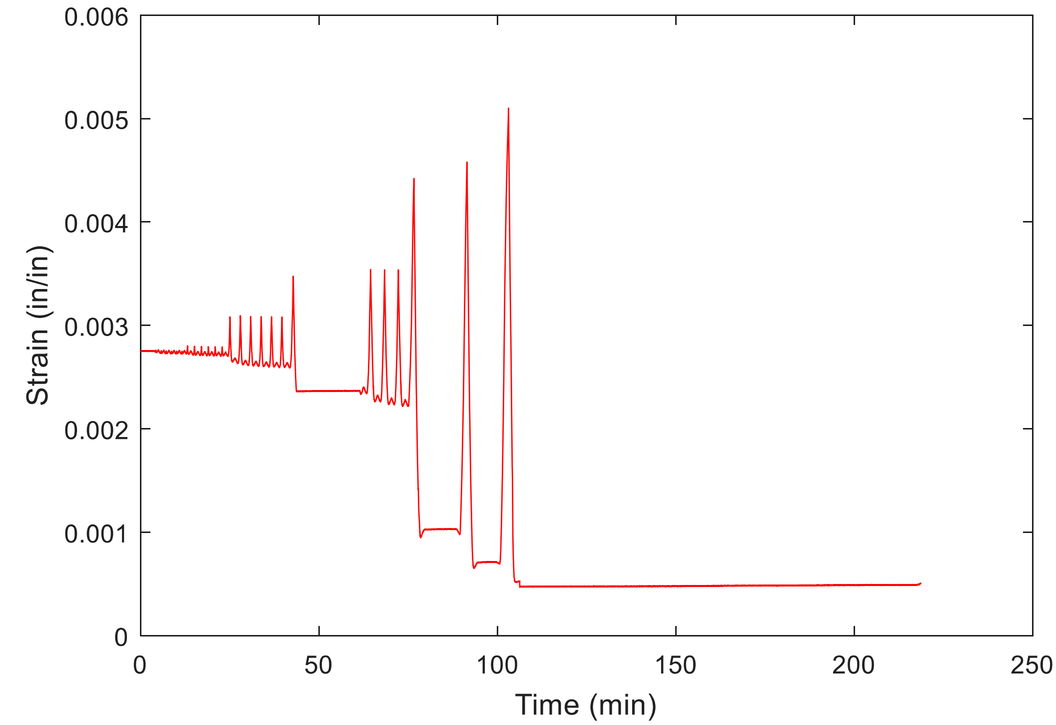


Figure T-73 Strain in BSG04

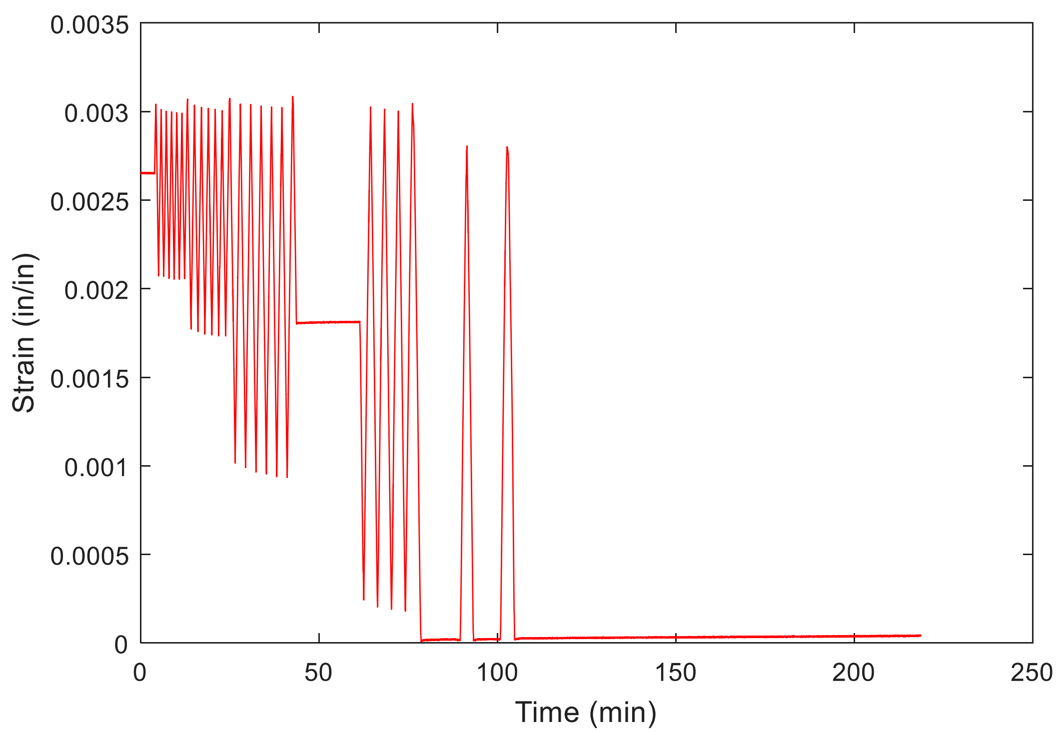


Figure T-74 Strain in BSG05

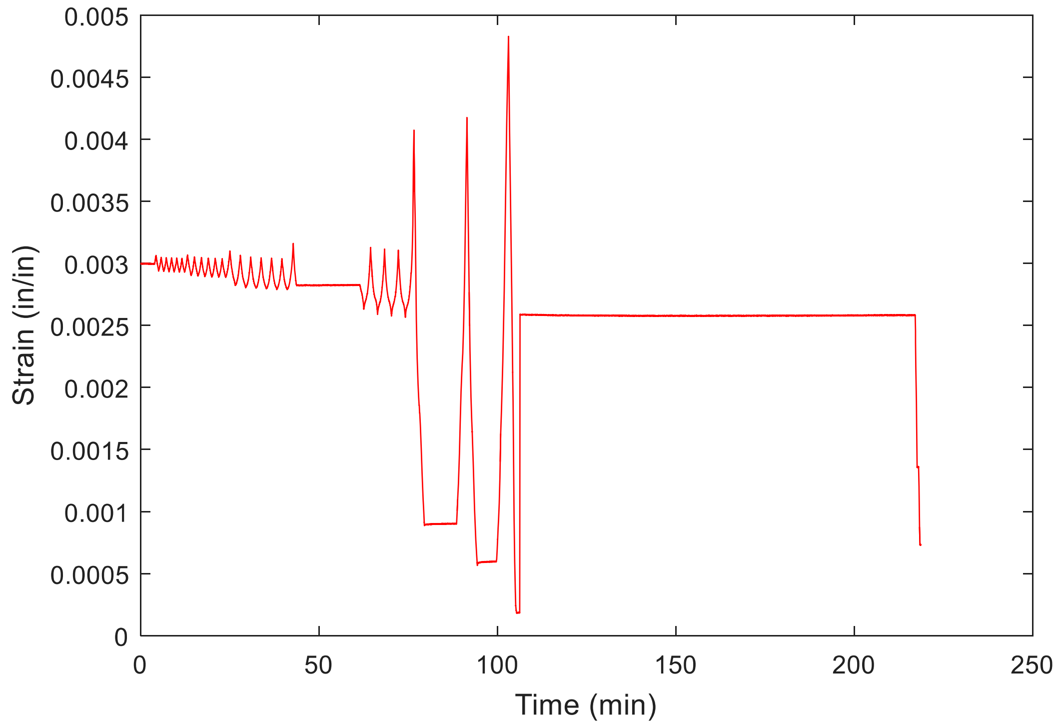


Figure T-75 Strain in BSG06

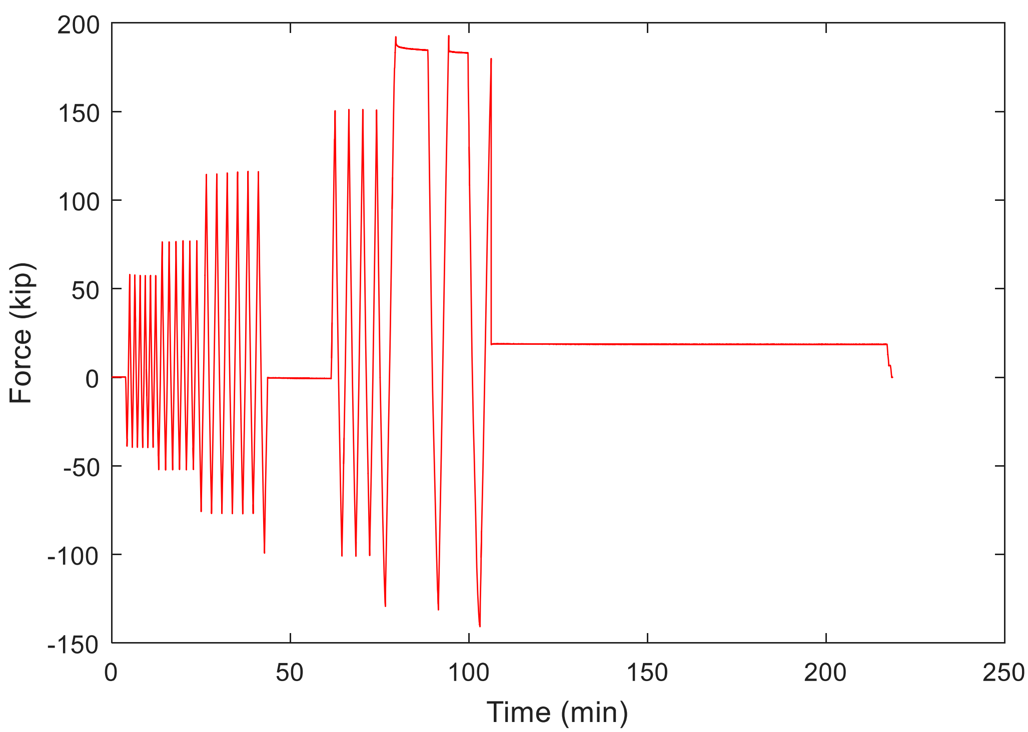


Figure T-76 Force in Actuator A

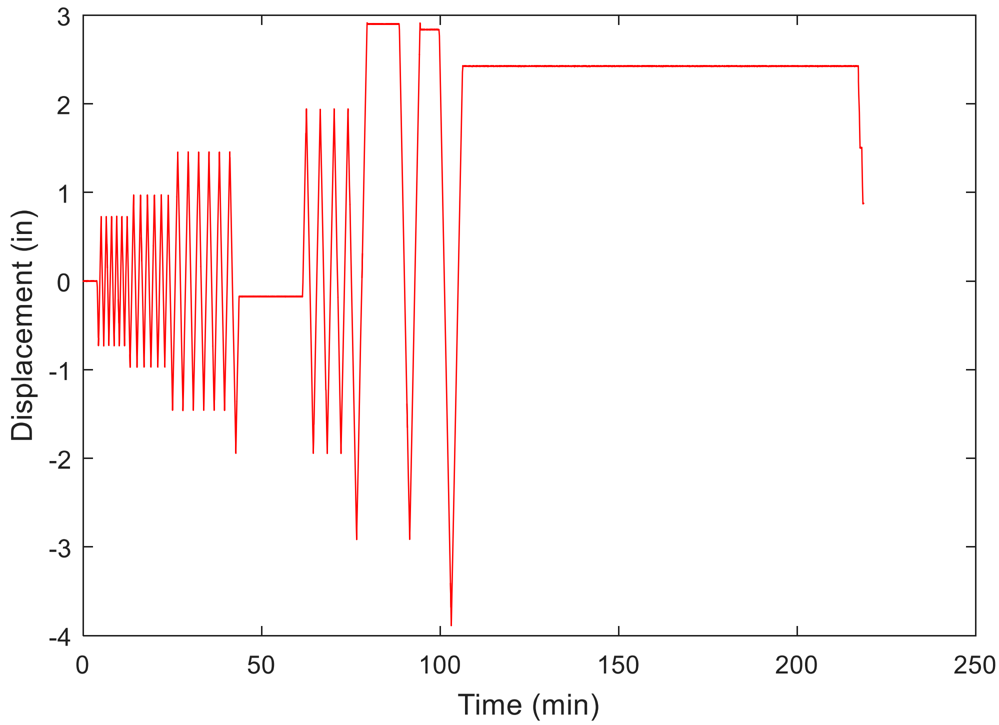


Figure T-77 Actuator A Displacement

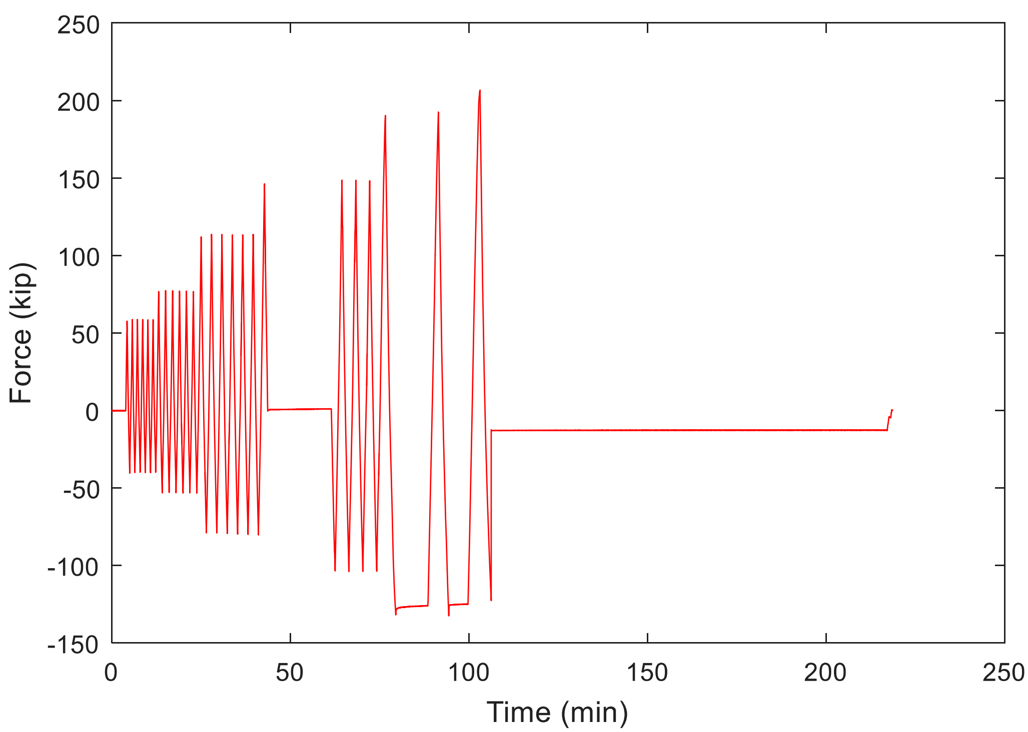


Figure T-78 Force in Actuator B

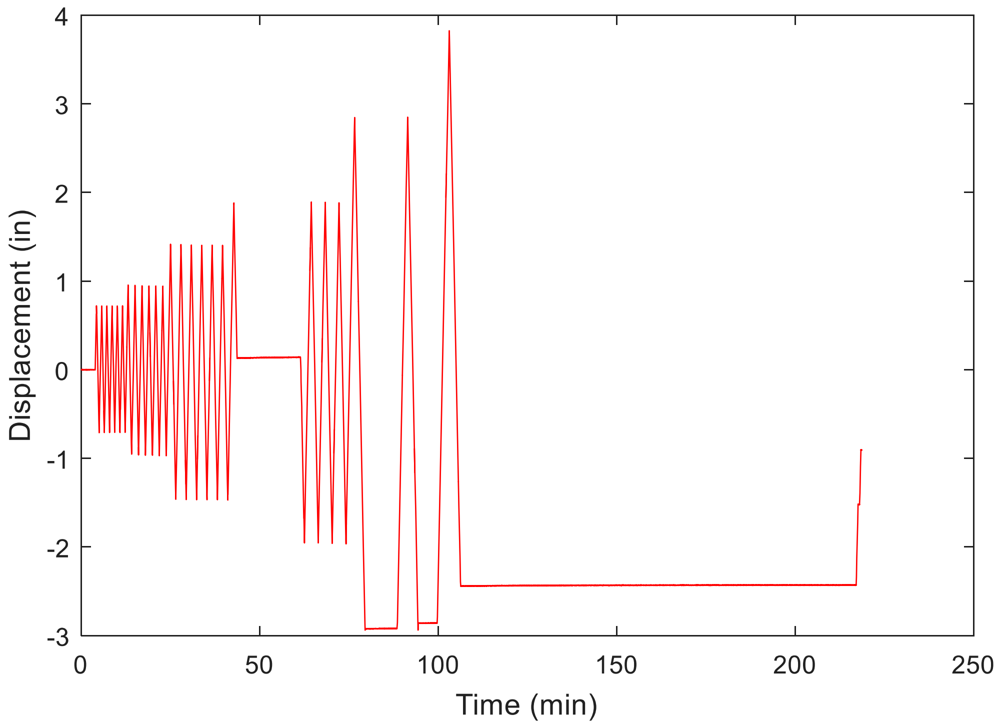


Figure T-79 Actuator B Displacement

APPENDIX U ADDITIONAL PLOTS FOR SPECIMEN 12ES-0.875-0.75-24

Plots from the start until the first failure

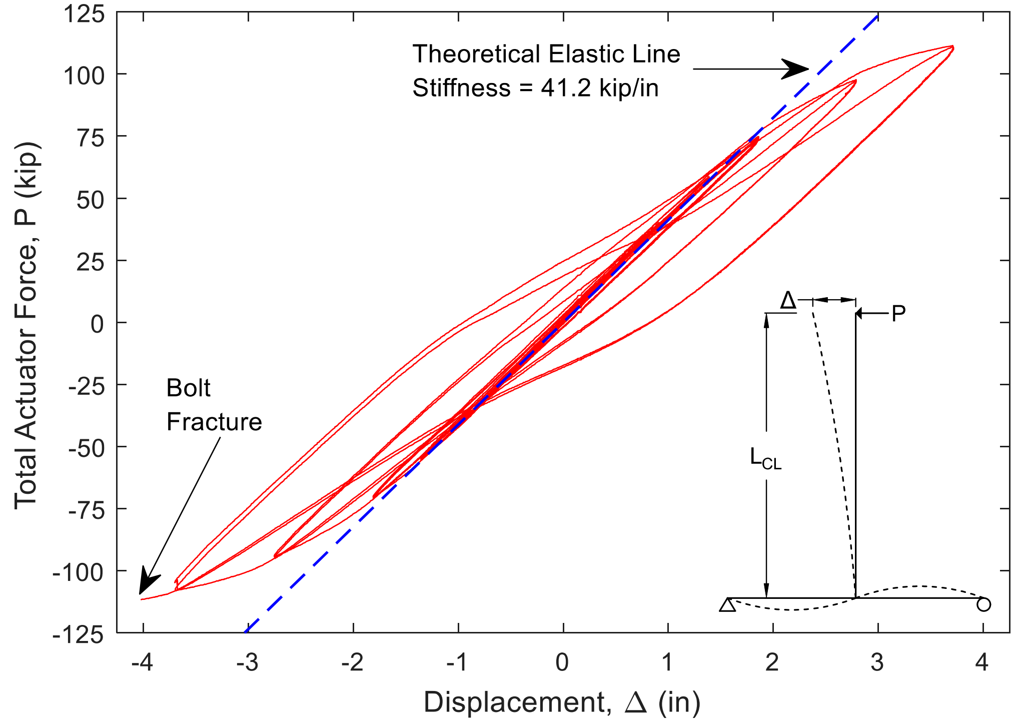


Figure U-1 Total Actuator Force vs. Applied Displacement at End of Beam

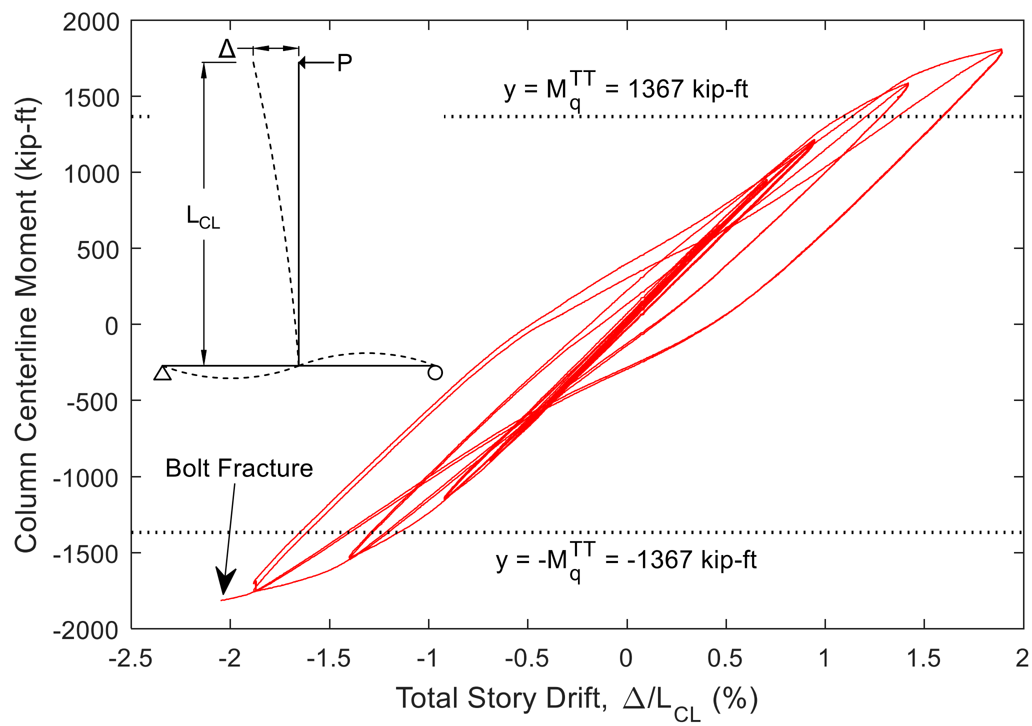


Figure U-2 Column Centerline Moment vs. Total Story Drift

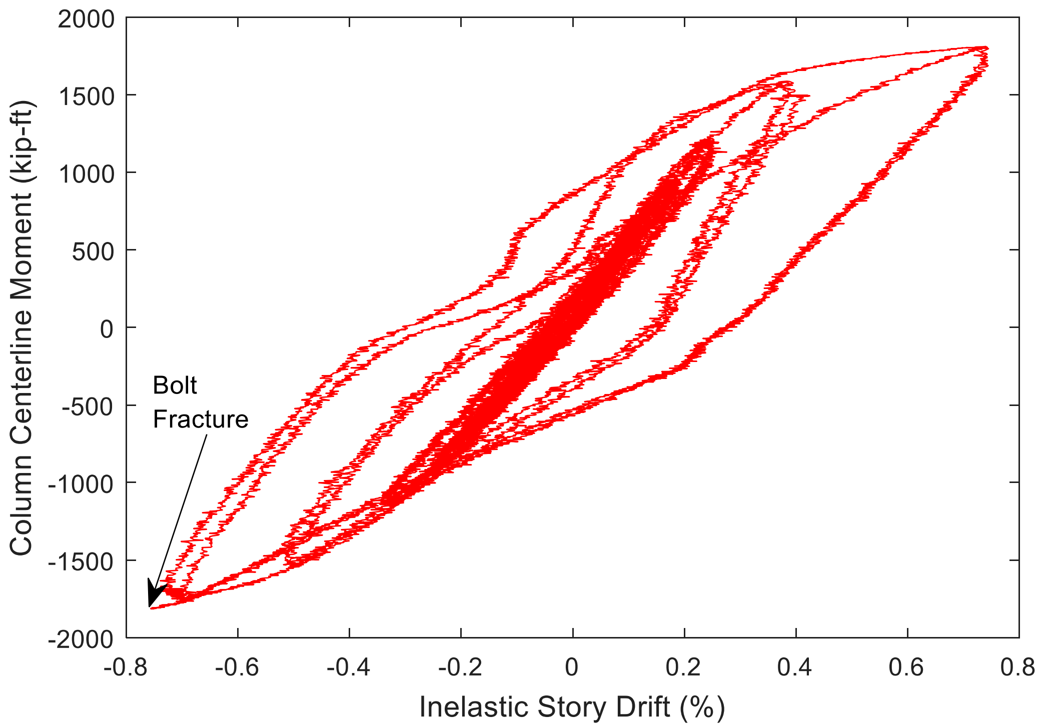


Figure U-3 Column Centerline Moment vs. Inelastic Portion of Story Drift

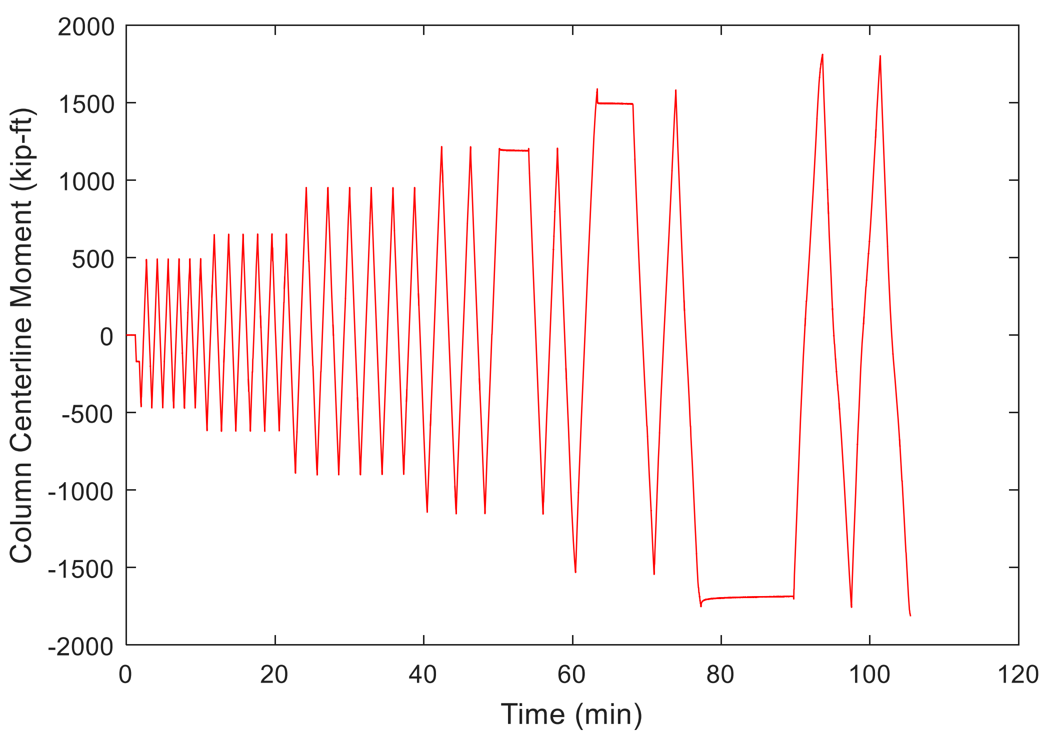


Figure U-4 Column Centerline Moment vs. Time

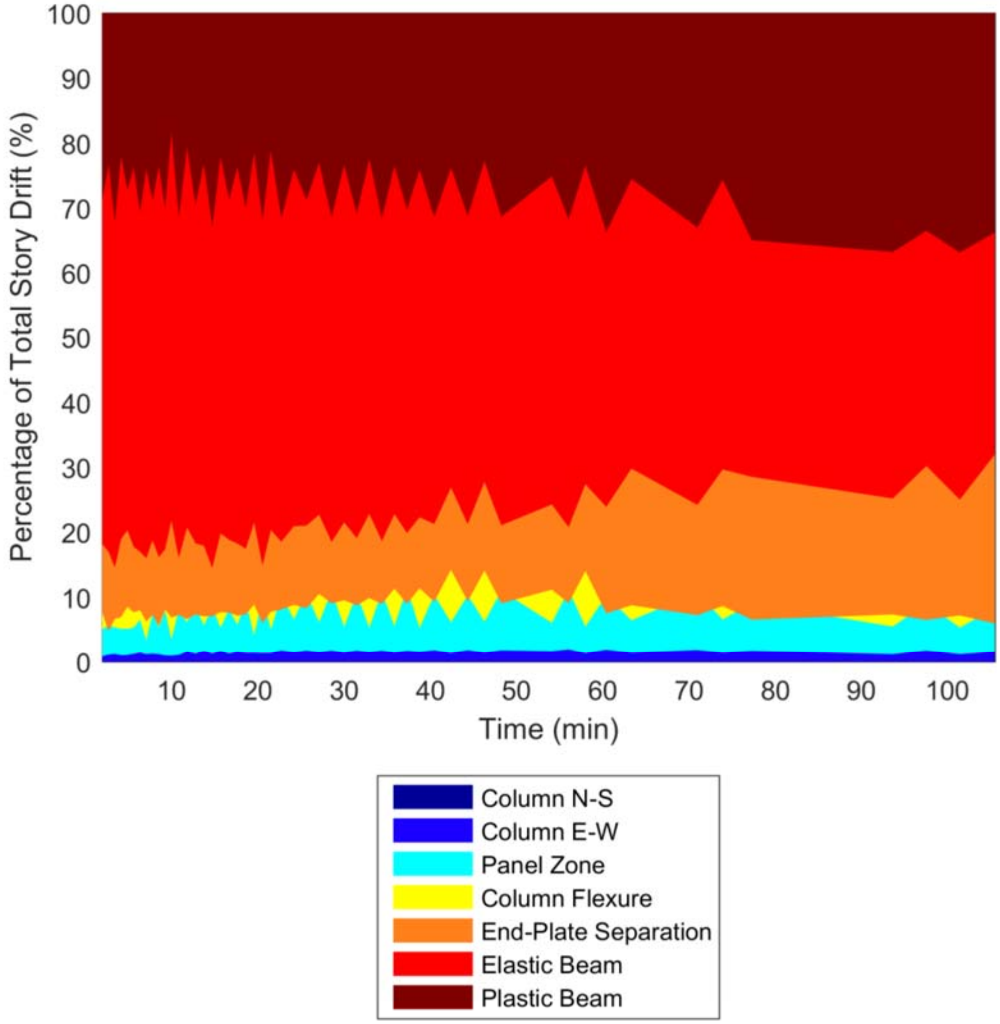


Figure U-5 Story Drift Components Percentage of Total Story Drift vs. Time

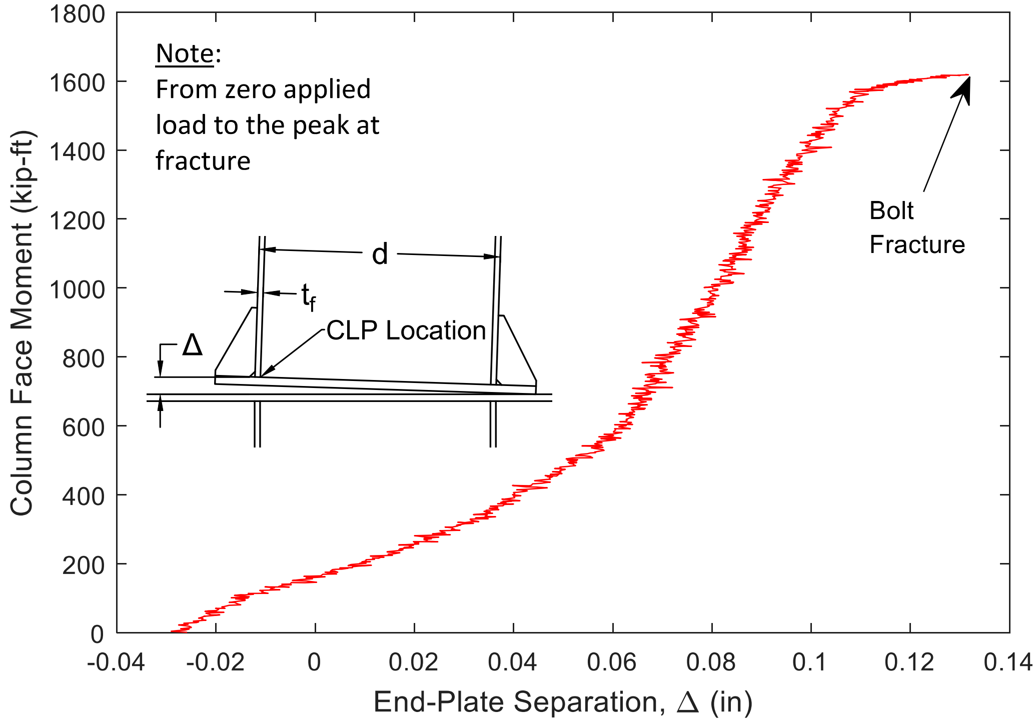


Figure U-6 Column Face Moment vs. End-Plate Separation

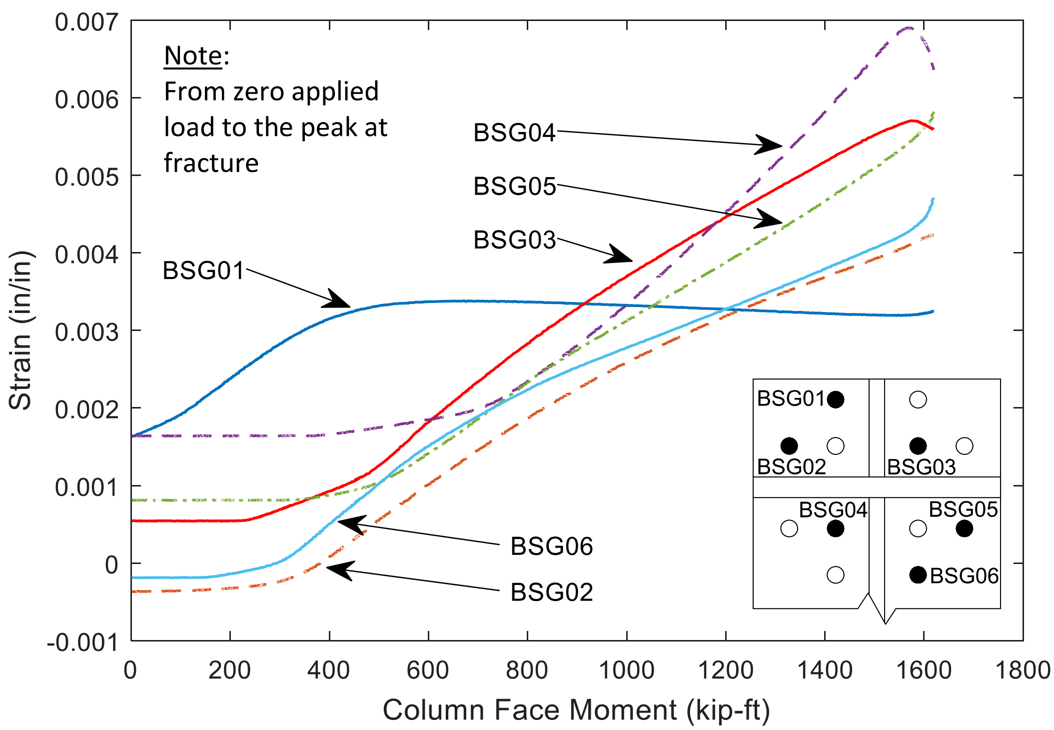


Figure U-7 Strain in Bolts vs. Column Face Moment

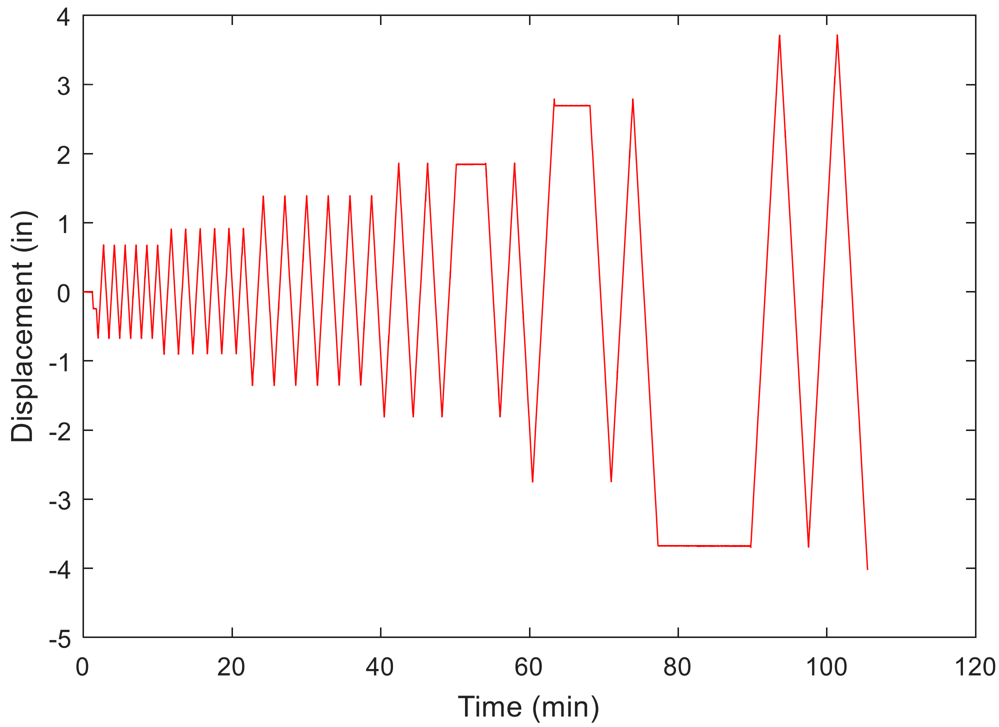


Figure U-8 Displacement at End of Beam vs. Time

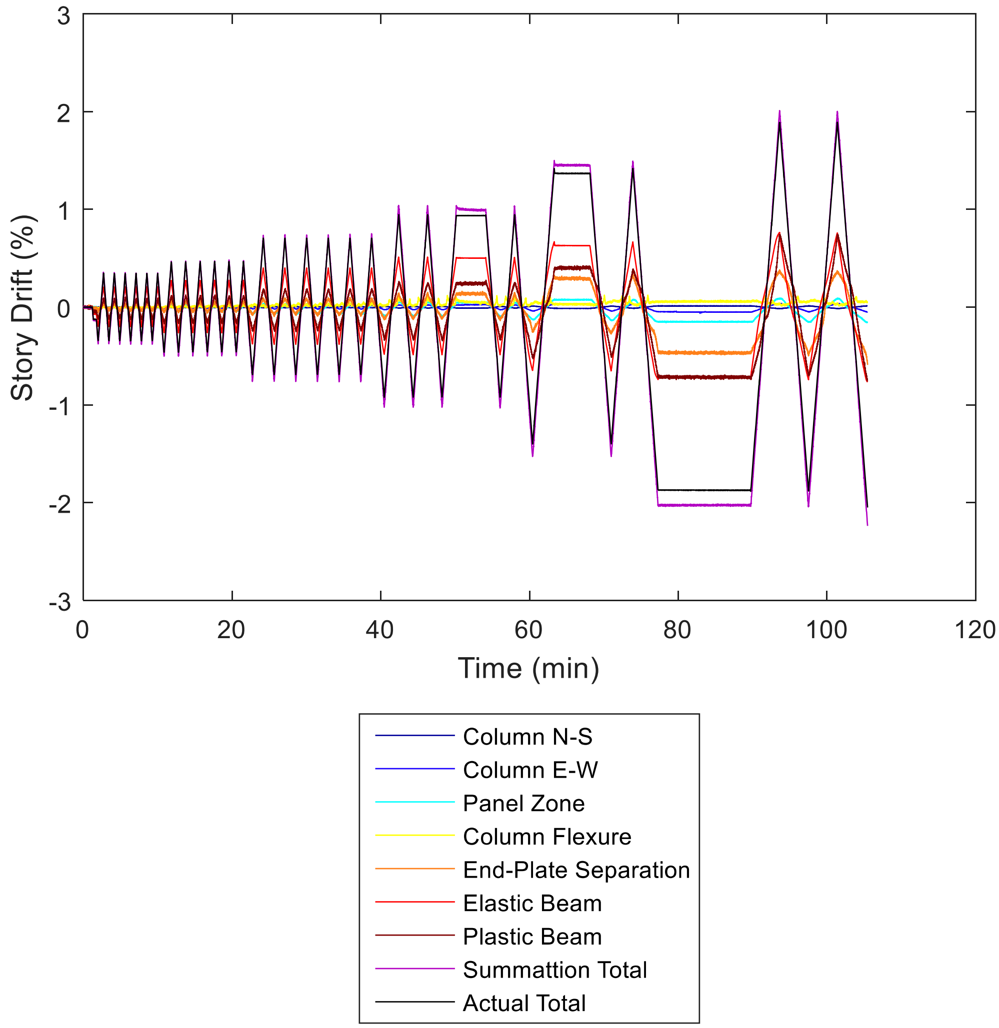


Figure U-9 Story Drift Components vs. Time

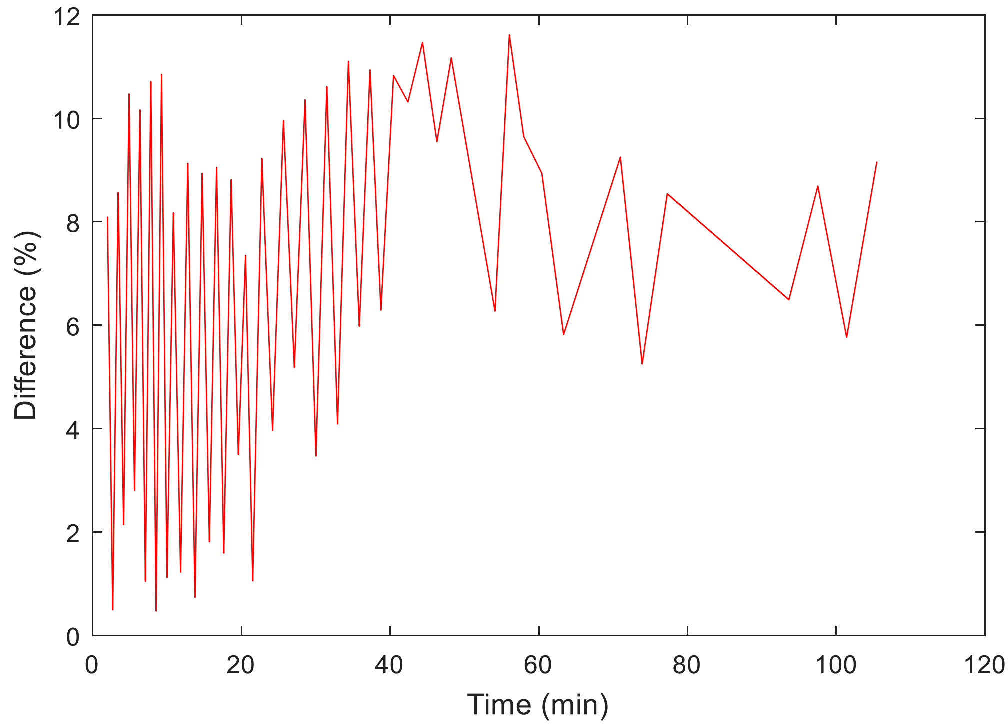


Figure U-10 Percent Difference Between Total Story Drift & Actual Story Drift at Peaks

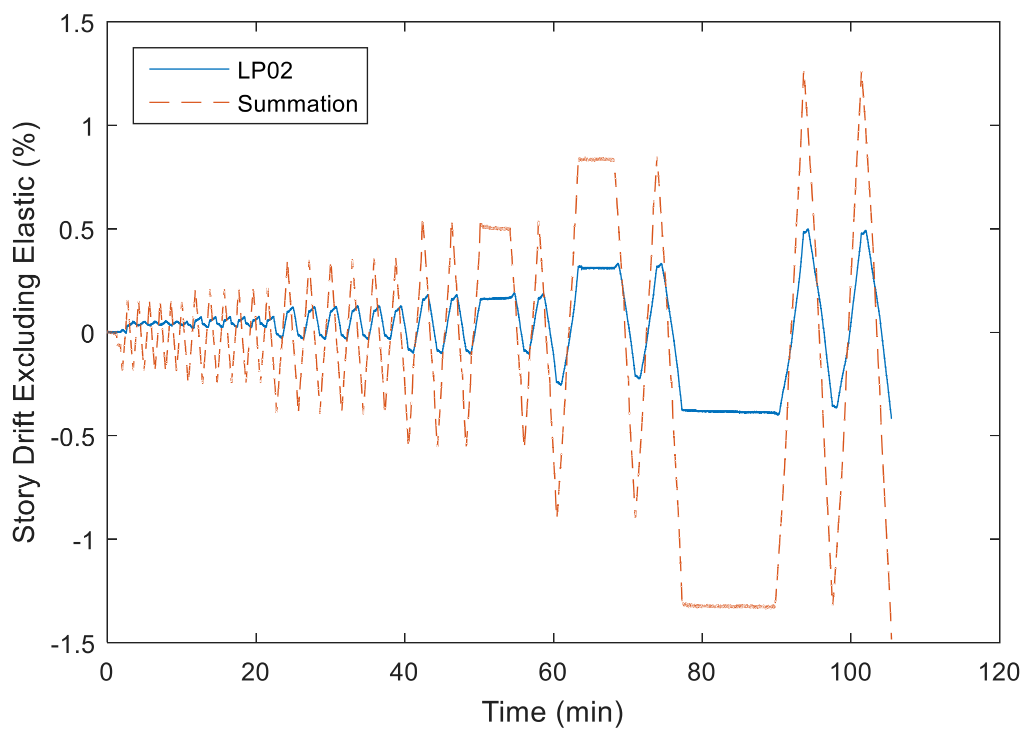


Figure U-11 Large Deformations Check

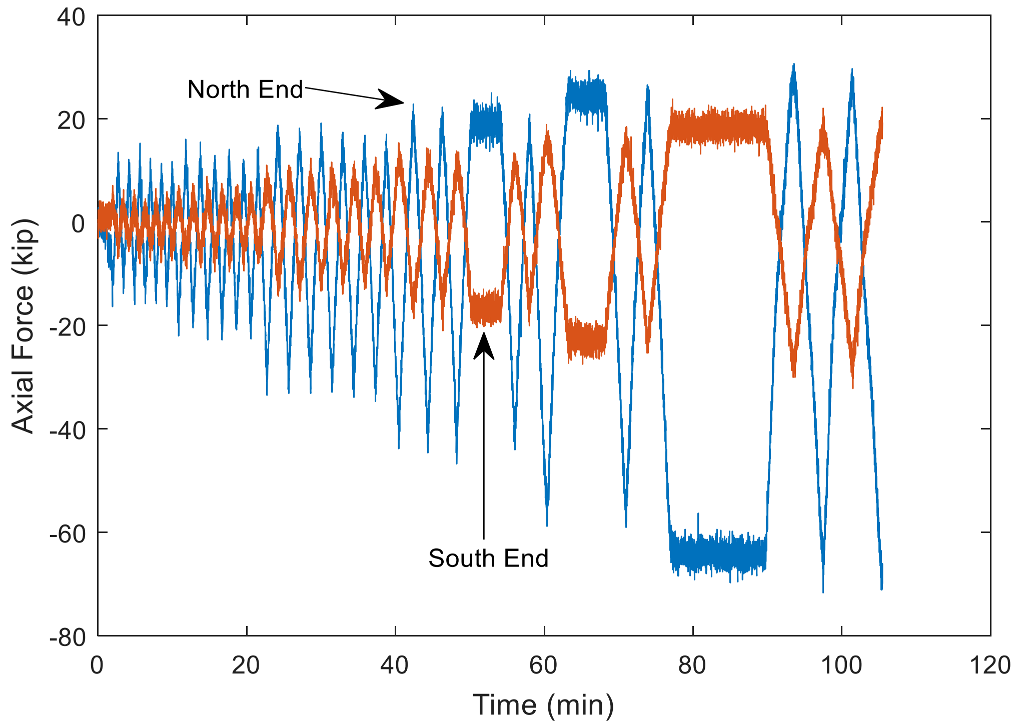


Figure U-12 Column Axial Force Calculated From Strain Gauges on Stub Sections

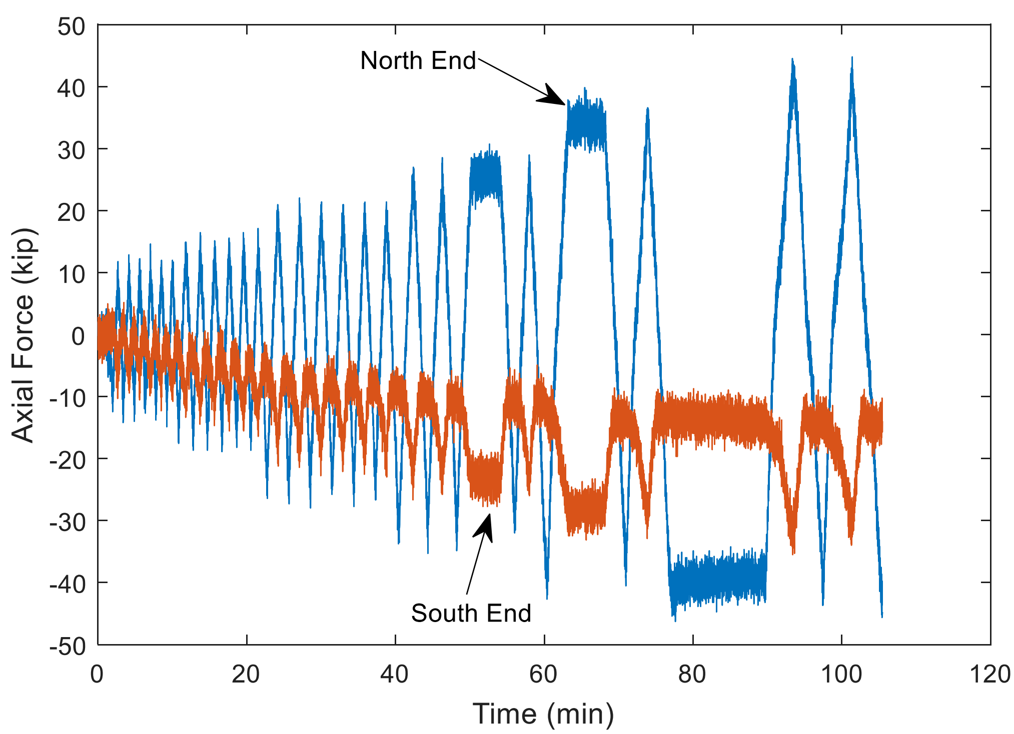


Figure U-13 Column Axial Force Calculated From Strain Gauges on Column

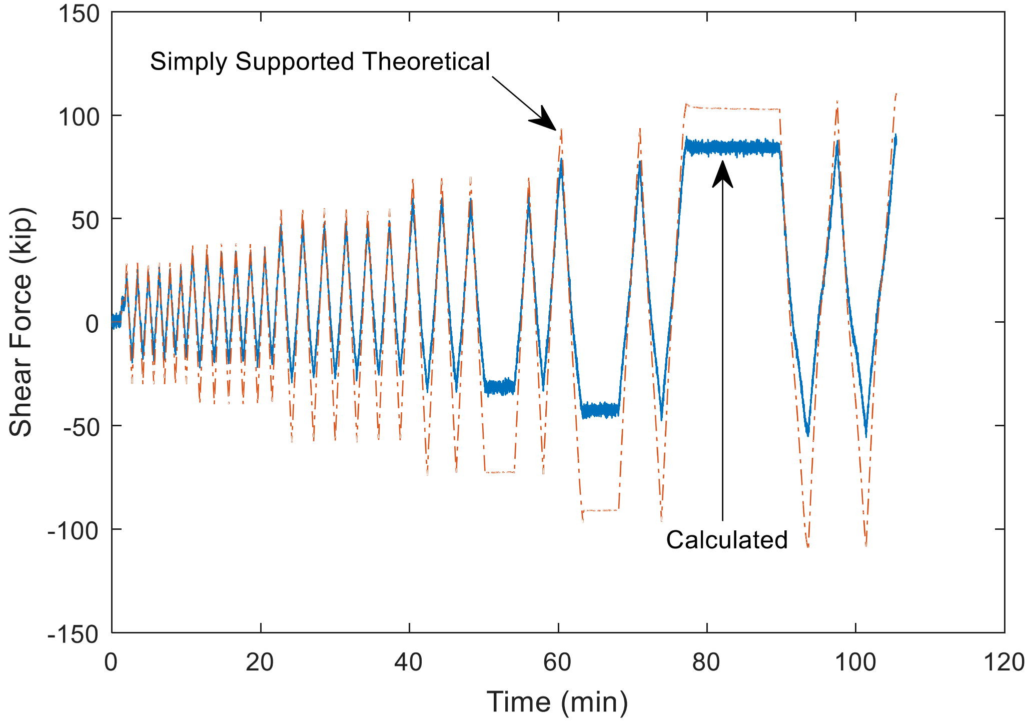


Figure U-14 Column Shear Force – North End

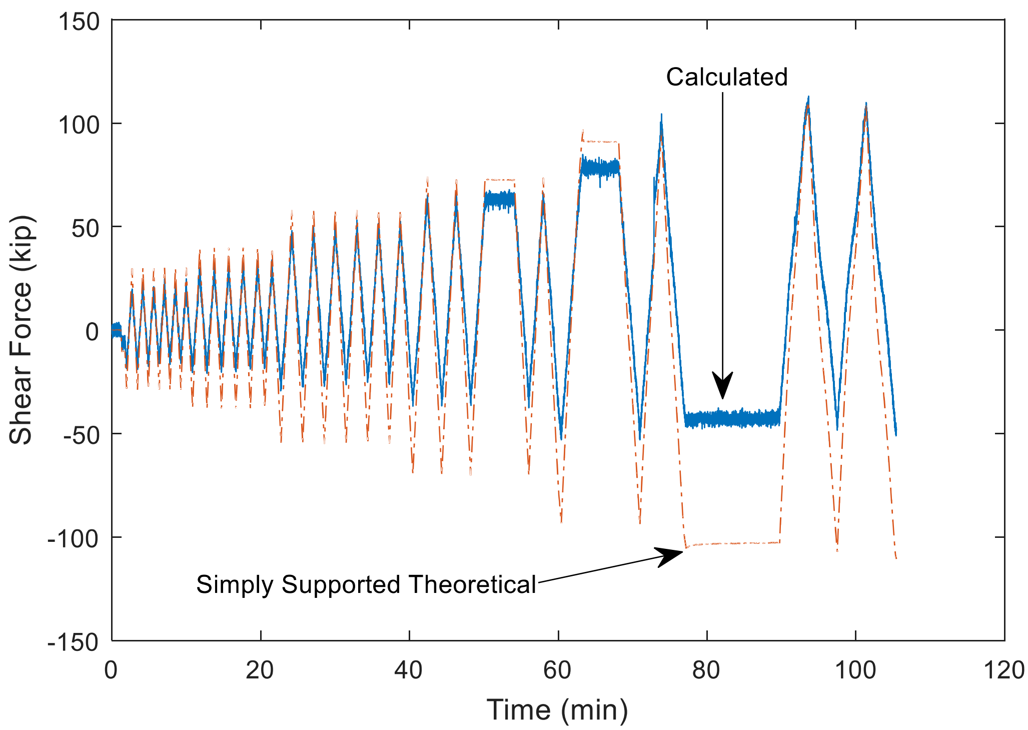


Figure U-15 Column Shear Force – South End

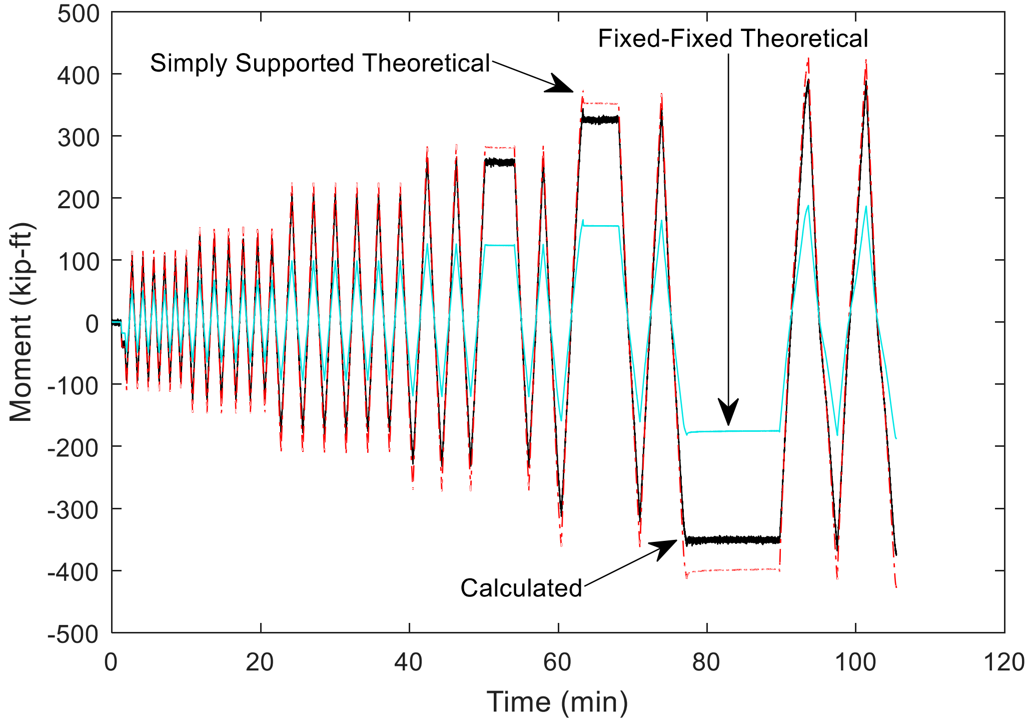


Figure U-16 Moment in Column at Location of Strain Gauges – North End

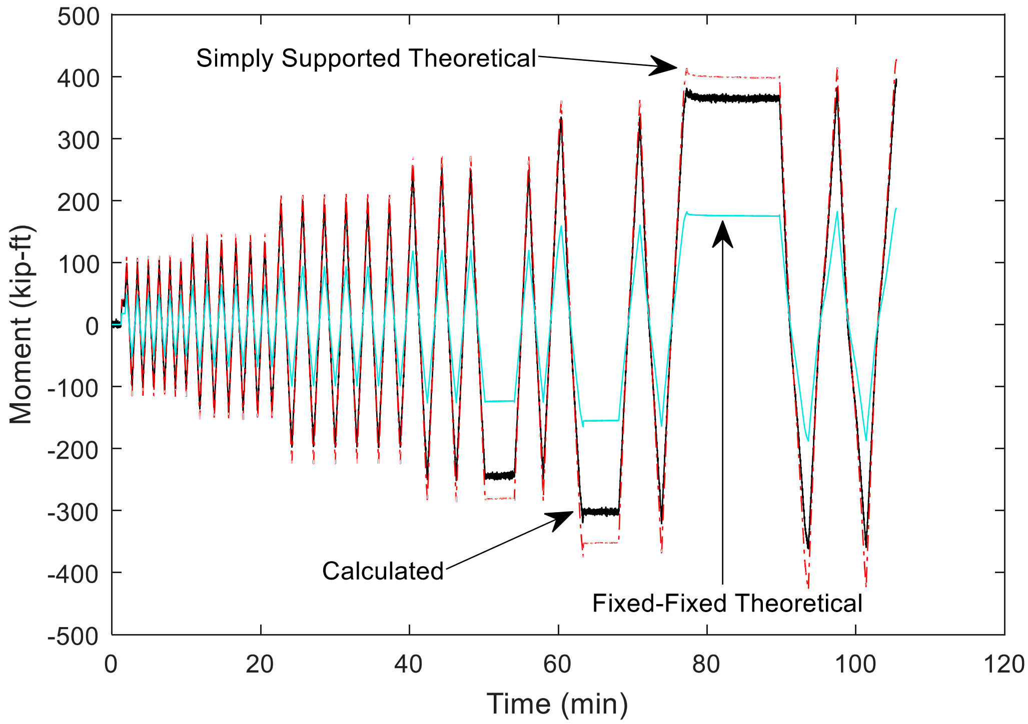


Figure U-17 Moment in Column at Location of Strain Gauges – South End

Instrumentation Data vs. Time

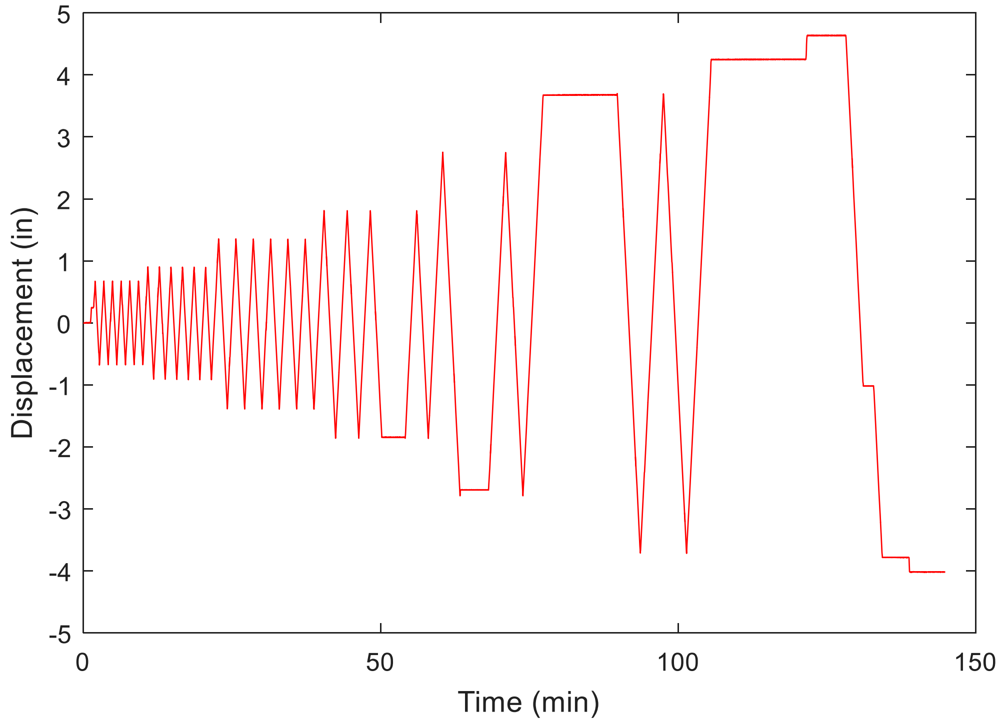


Figure U-18 SP01

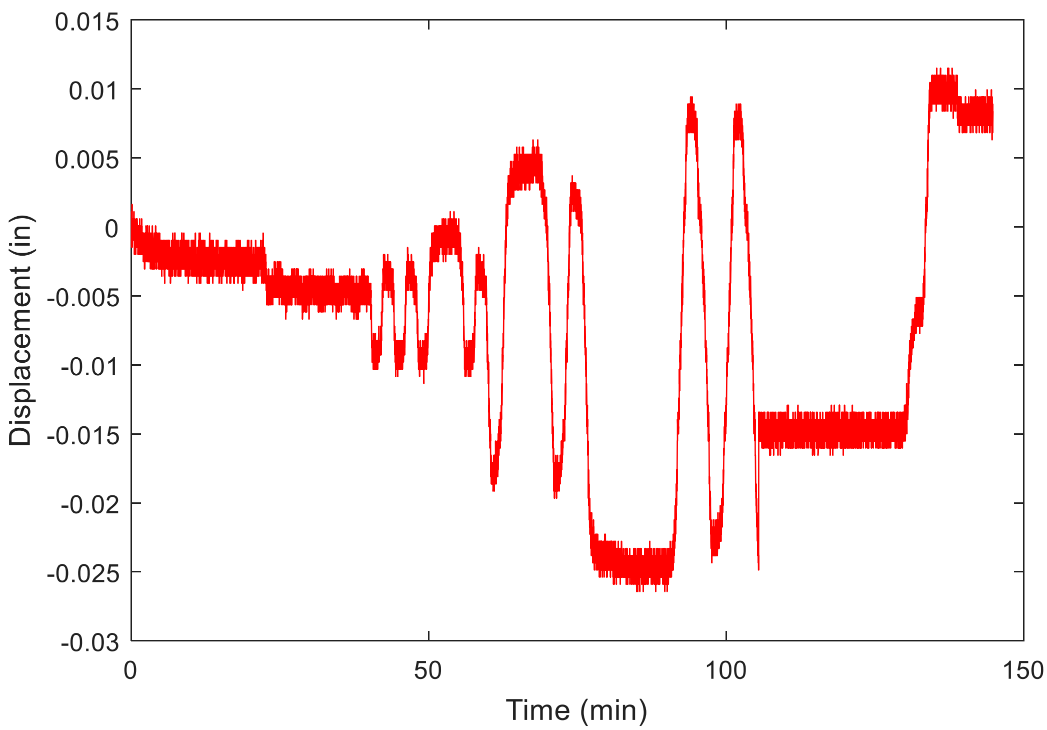


Figure U-19 SP02

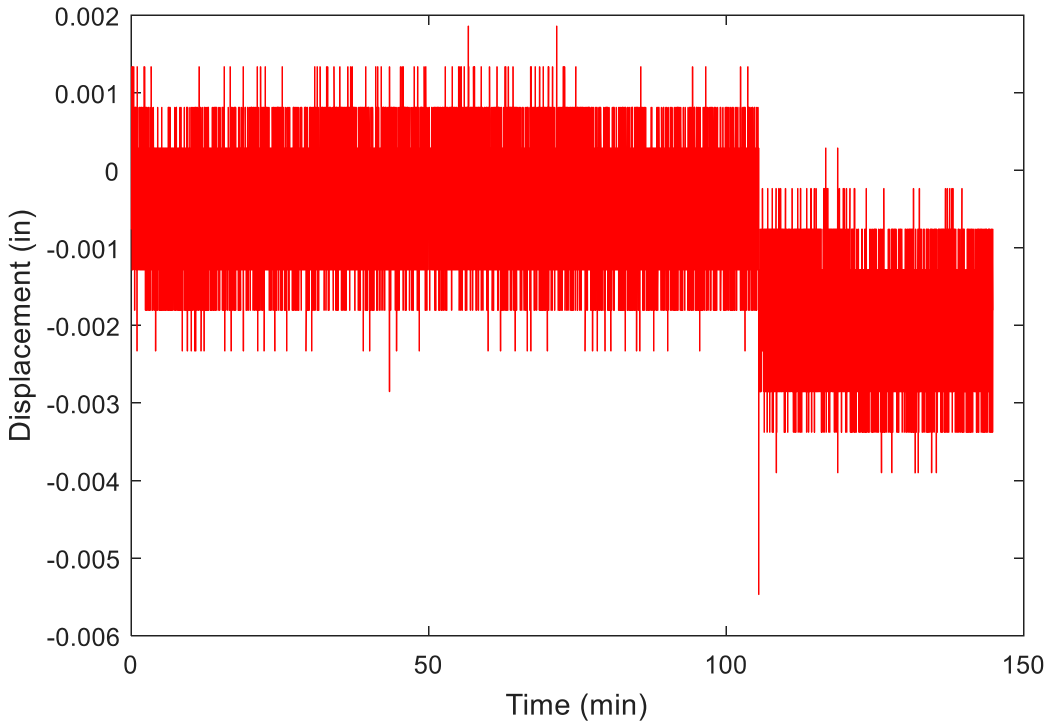


Figure U-20 SP03

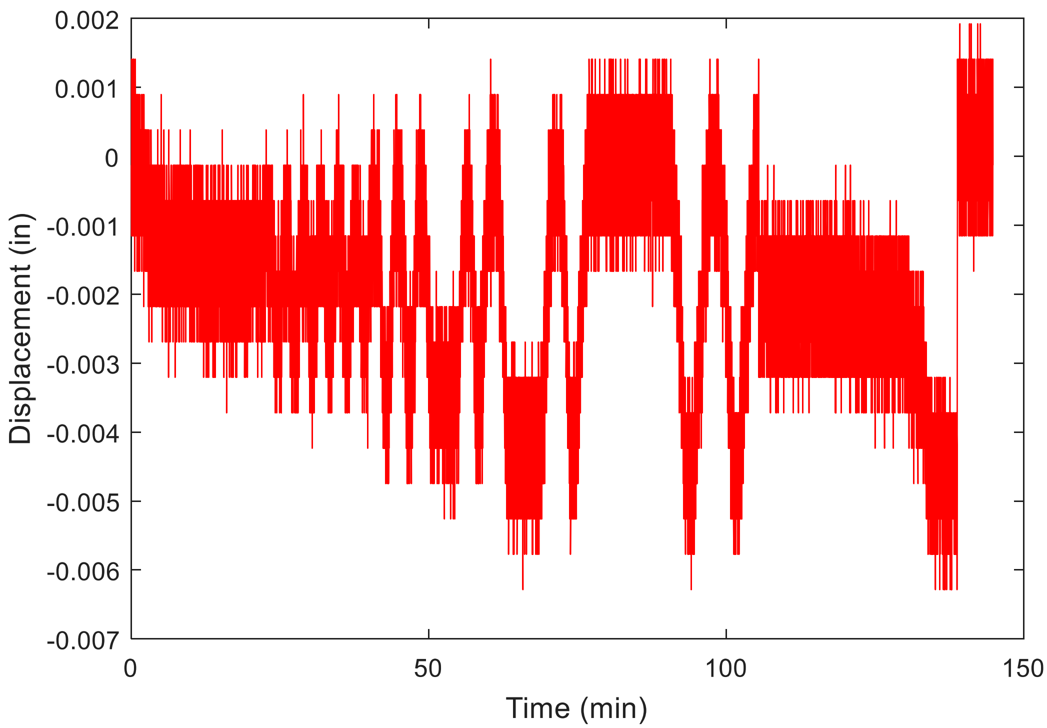


Figure U-21 SP04

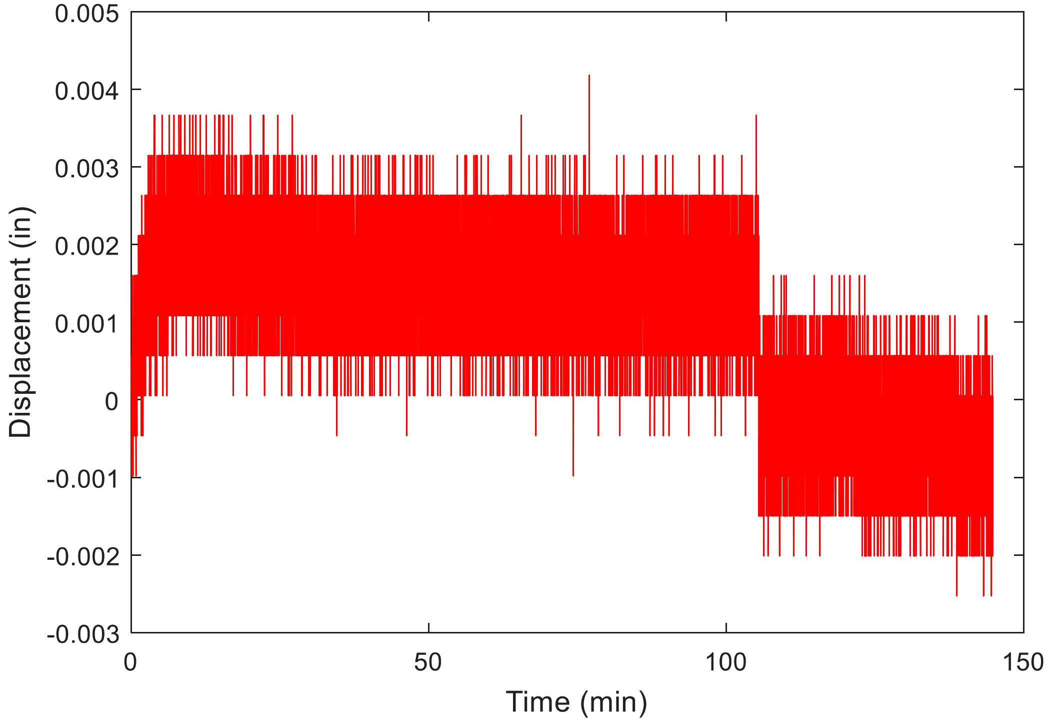


Figure U-22 SP05

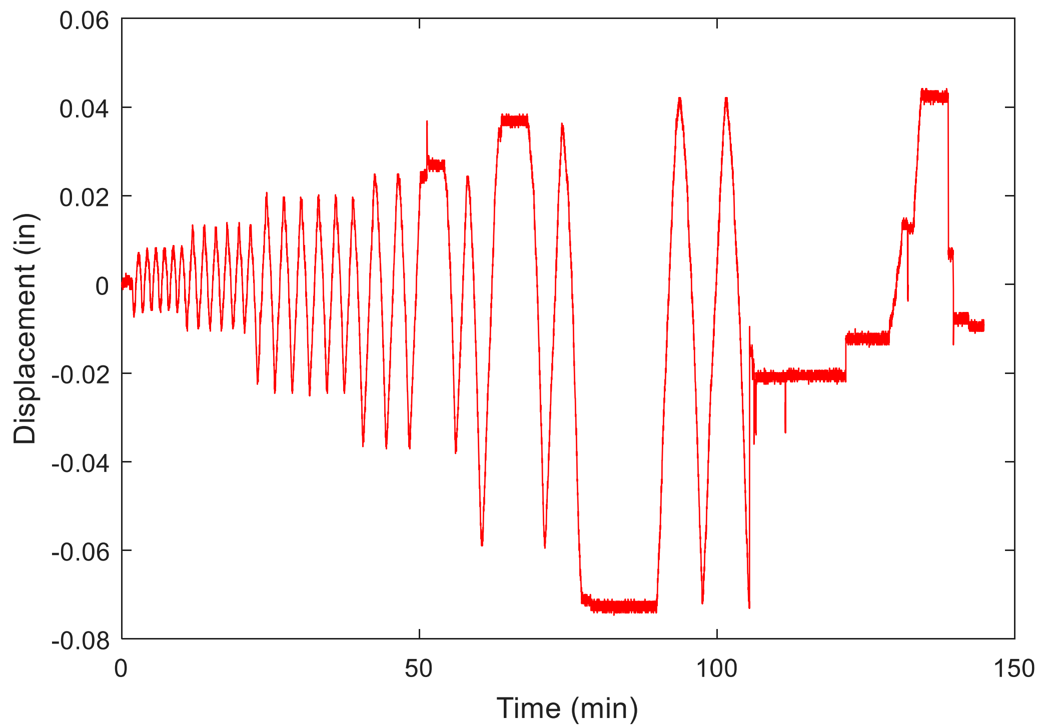


Figure U-23 SP06

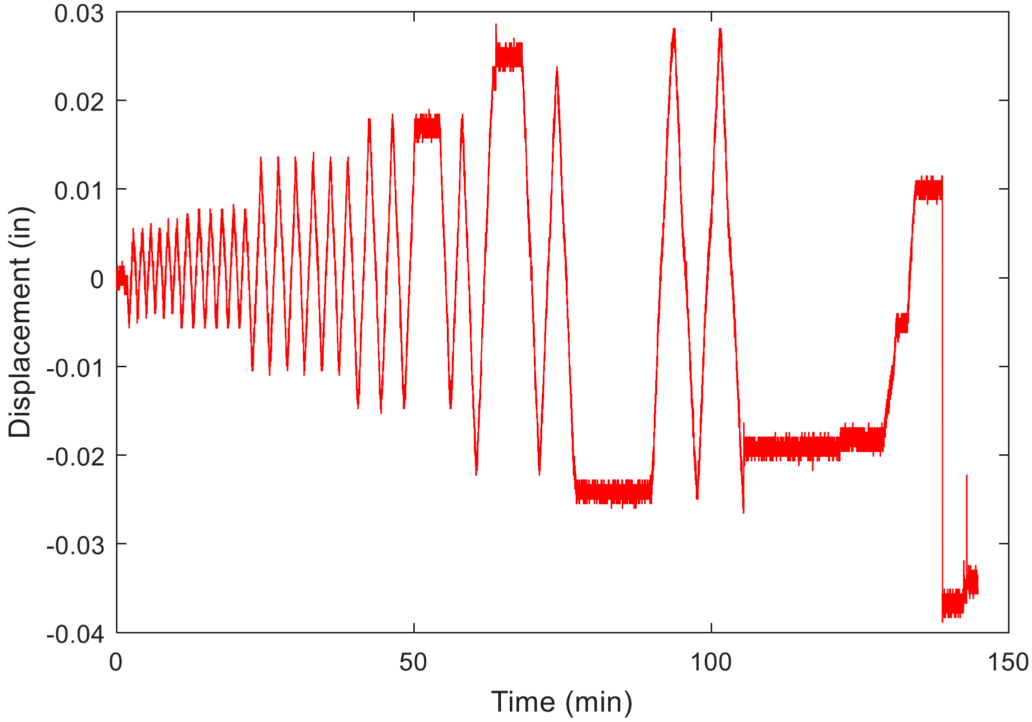


Figure U-24 SP07

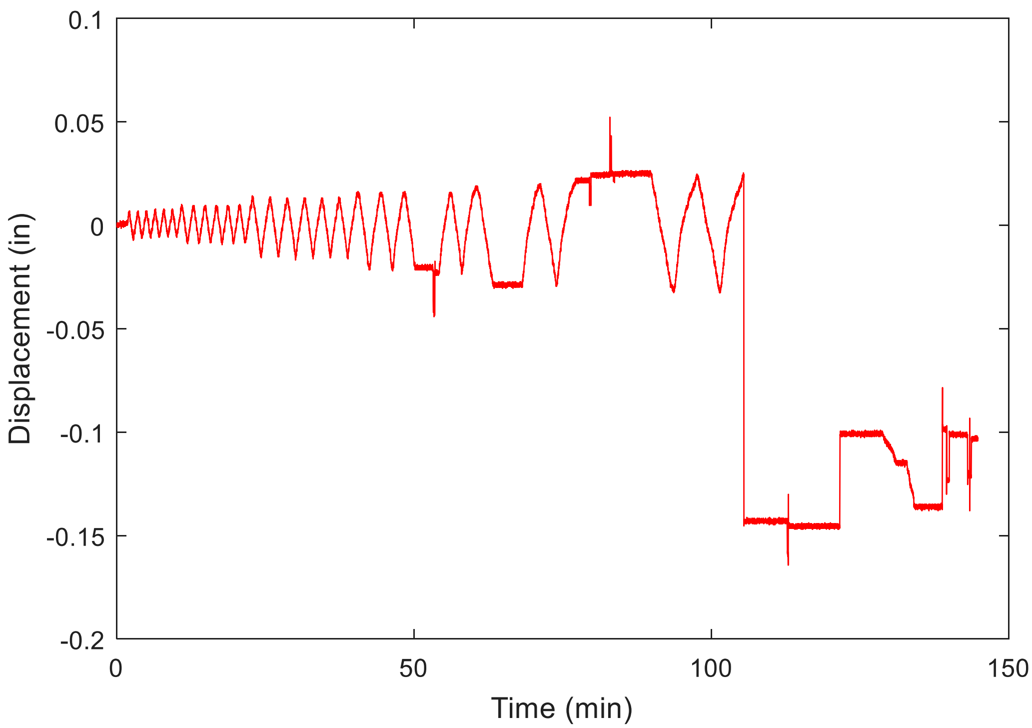


Figure U-25 SP08

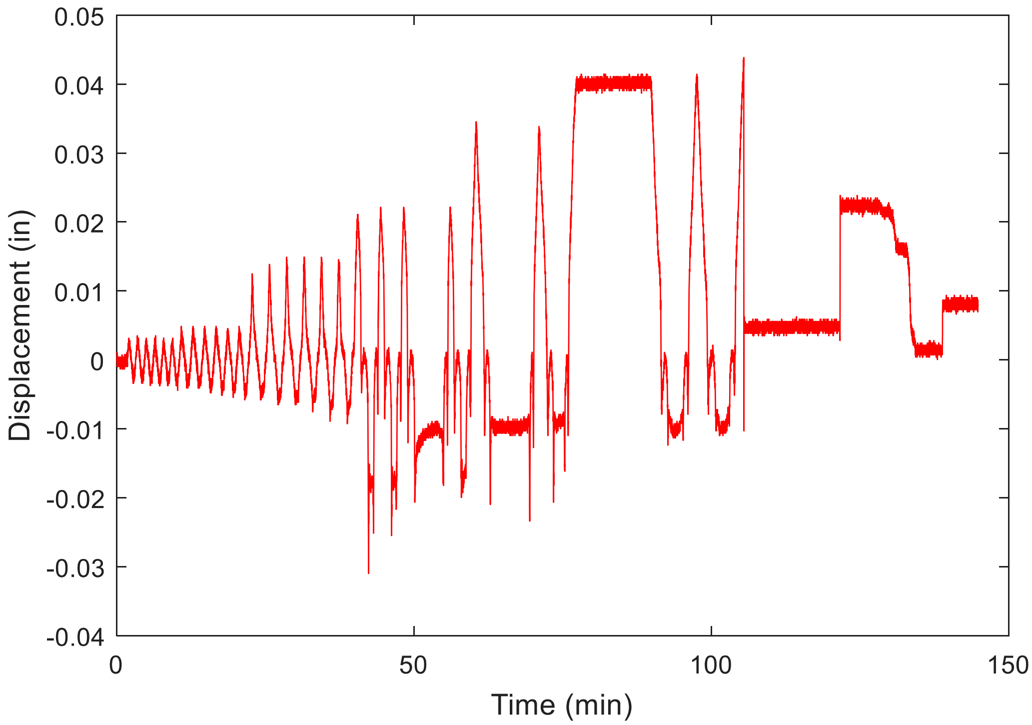


Figure U-26 SP09

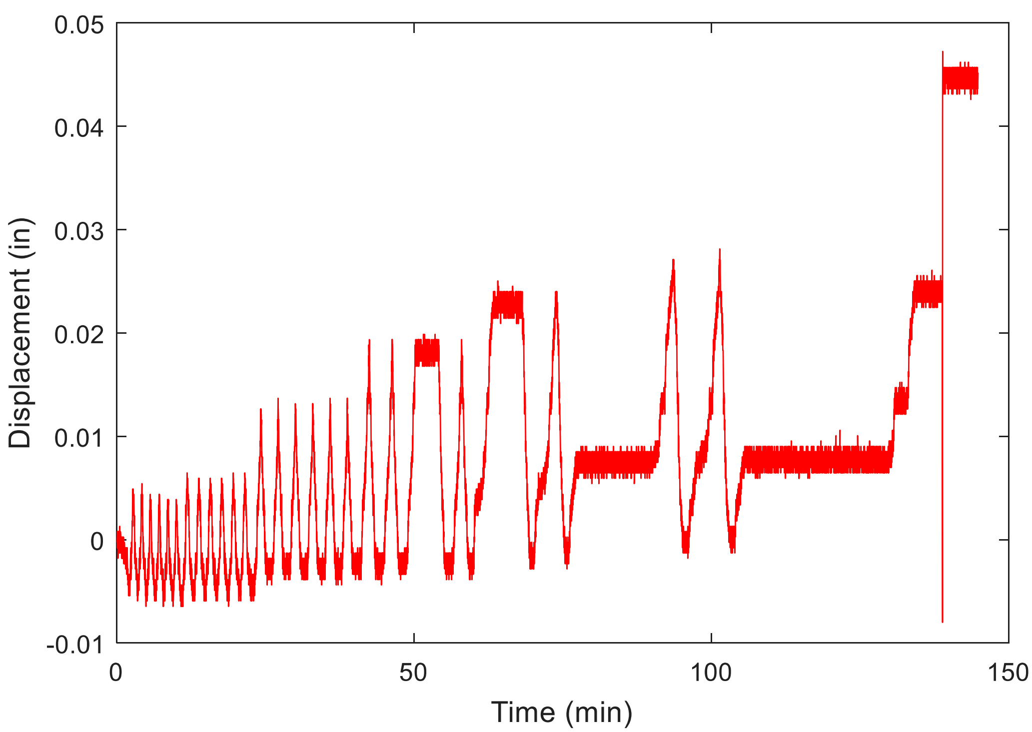


Figure U-27 SP10

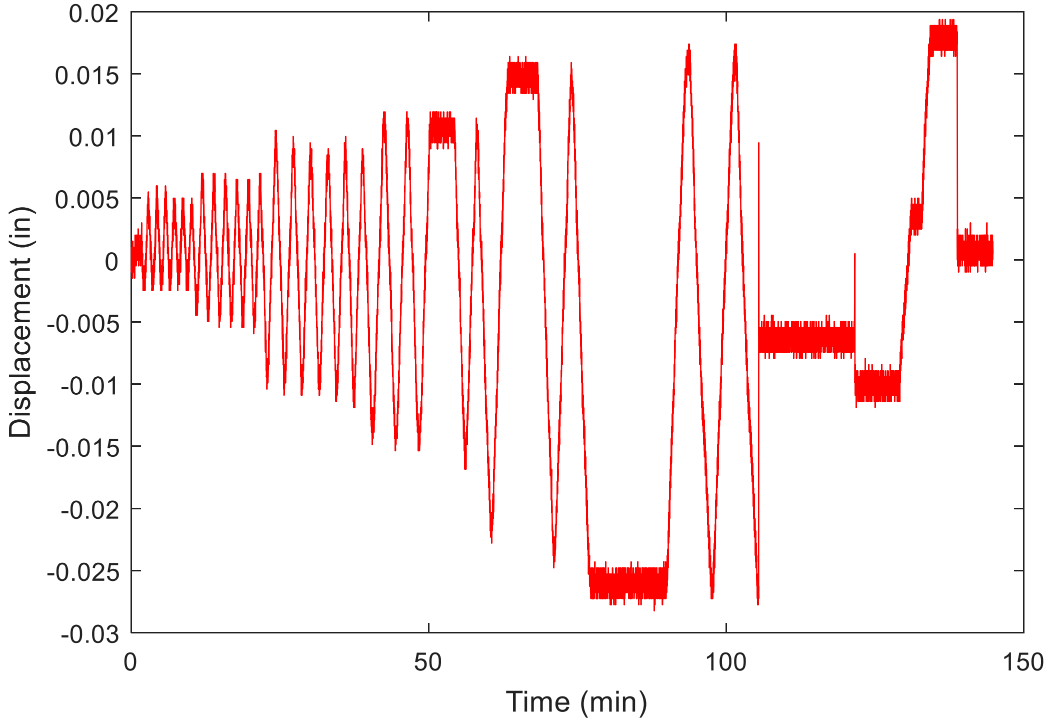


Figure U-28 SP11

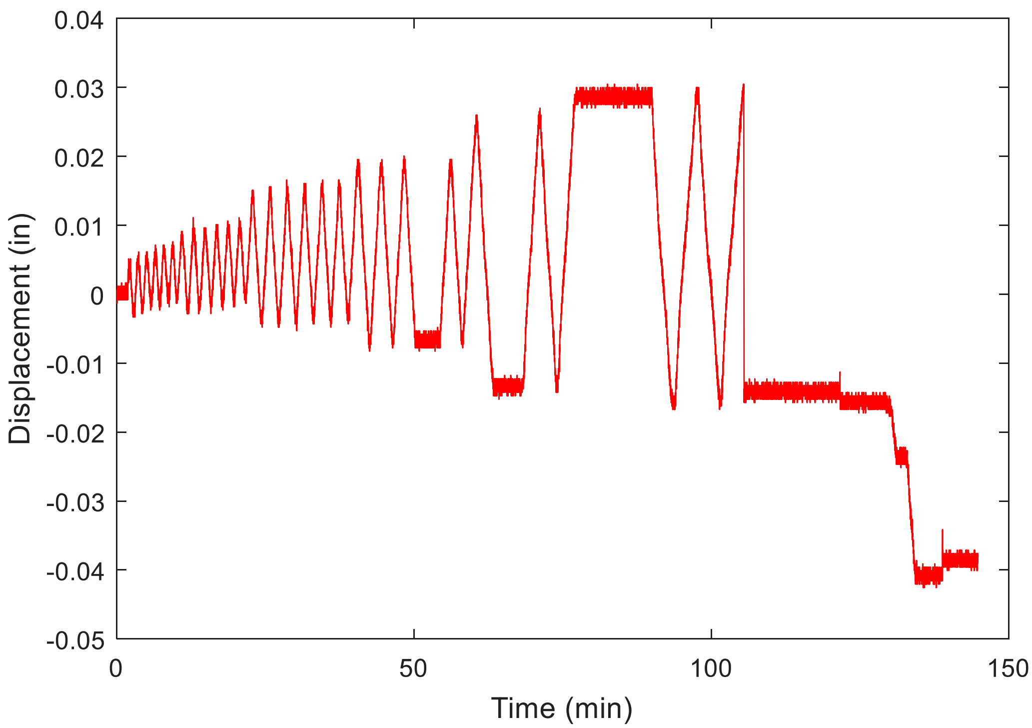


Figure U-29 SP12

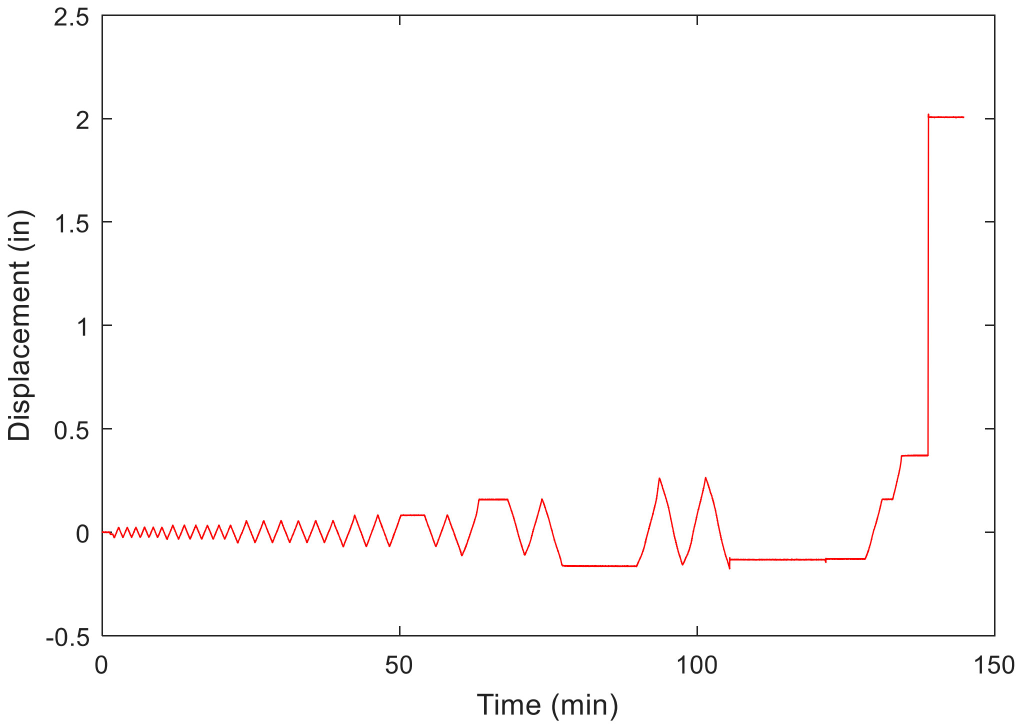


Figure U-30 SP13

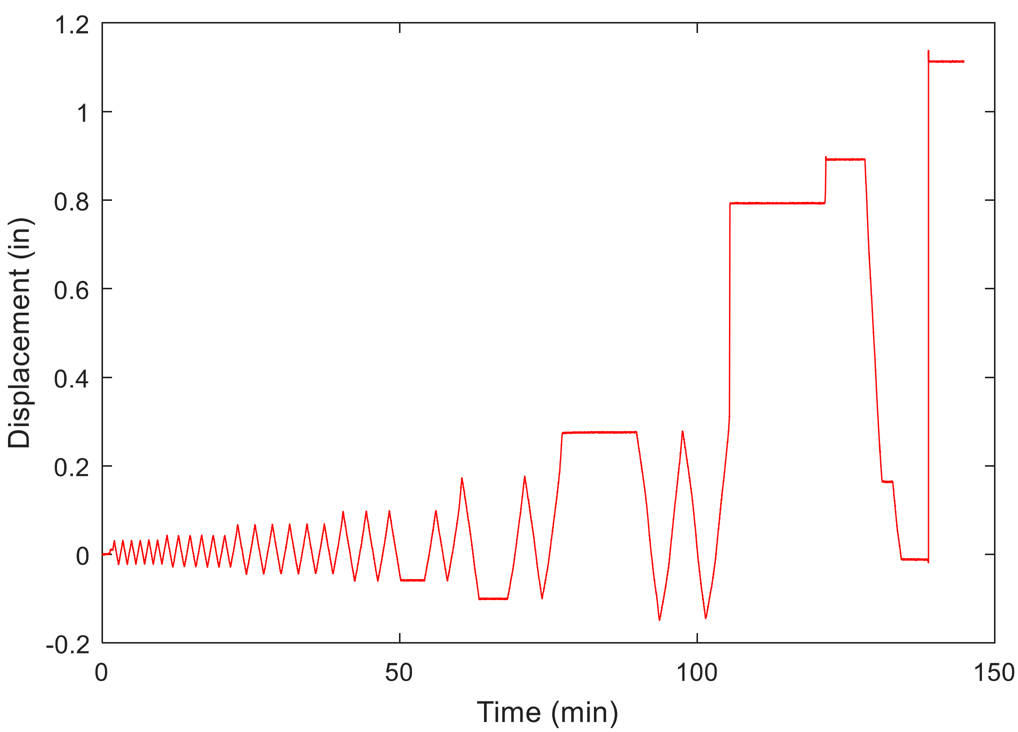


Figure U-31 SP14

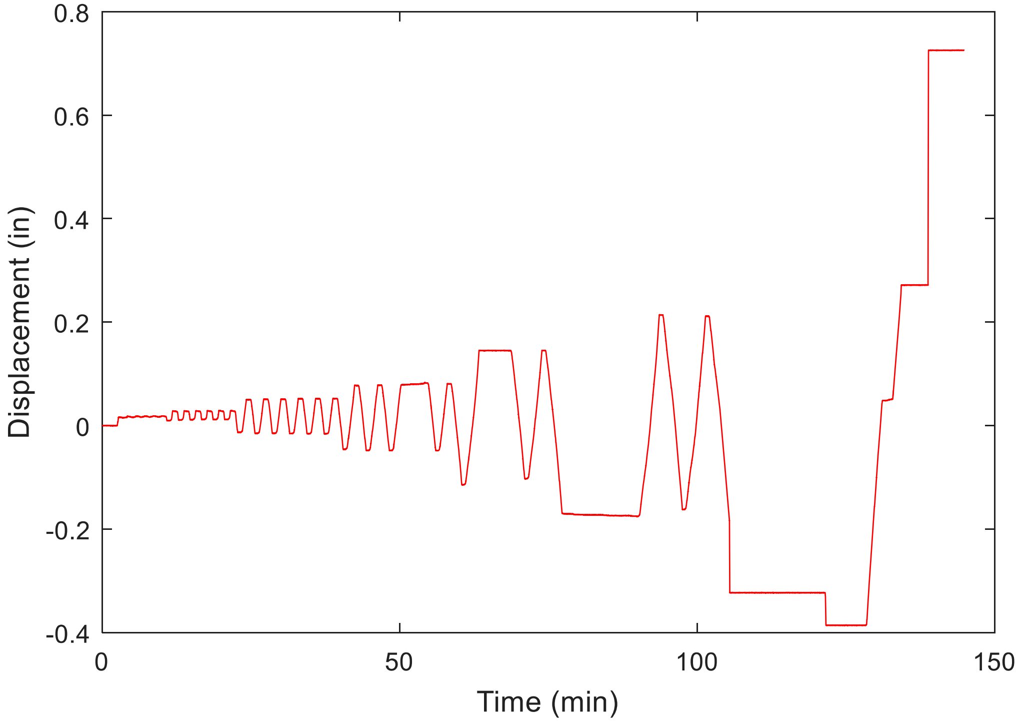


Figure U-32 LP02

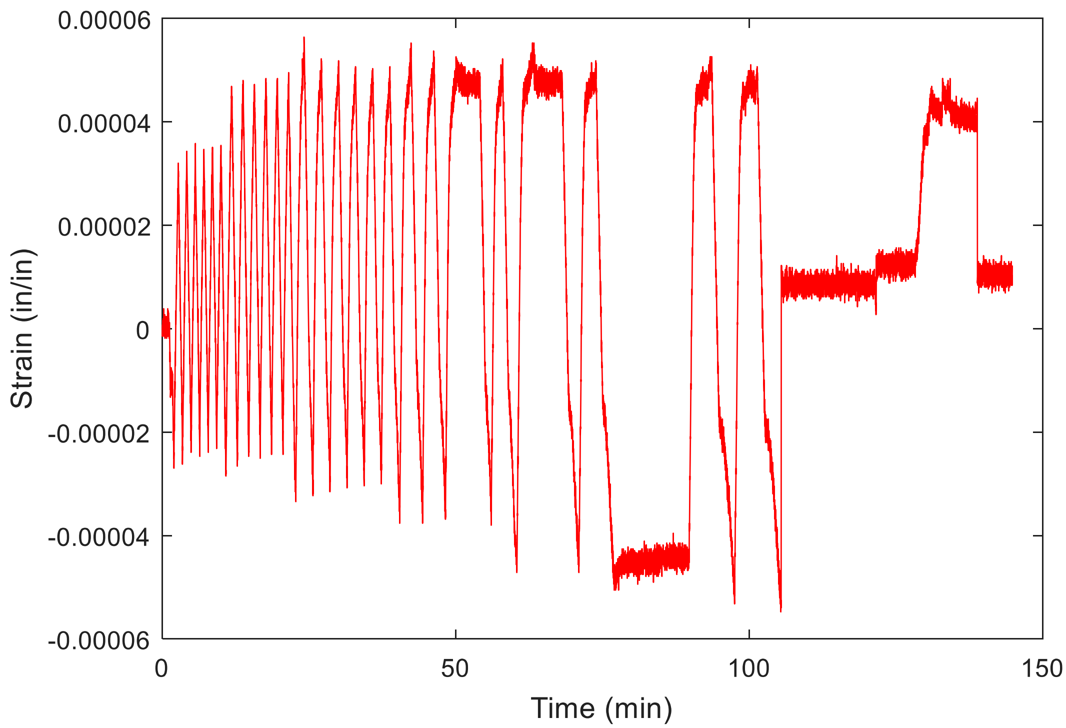


Figure U-33 SG01

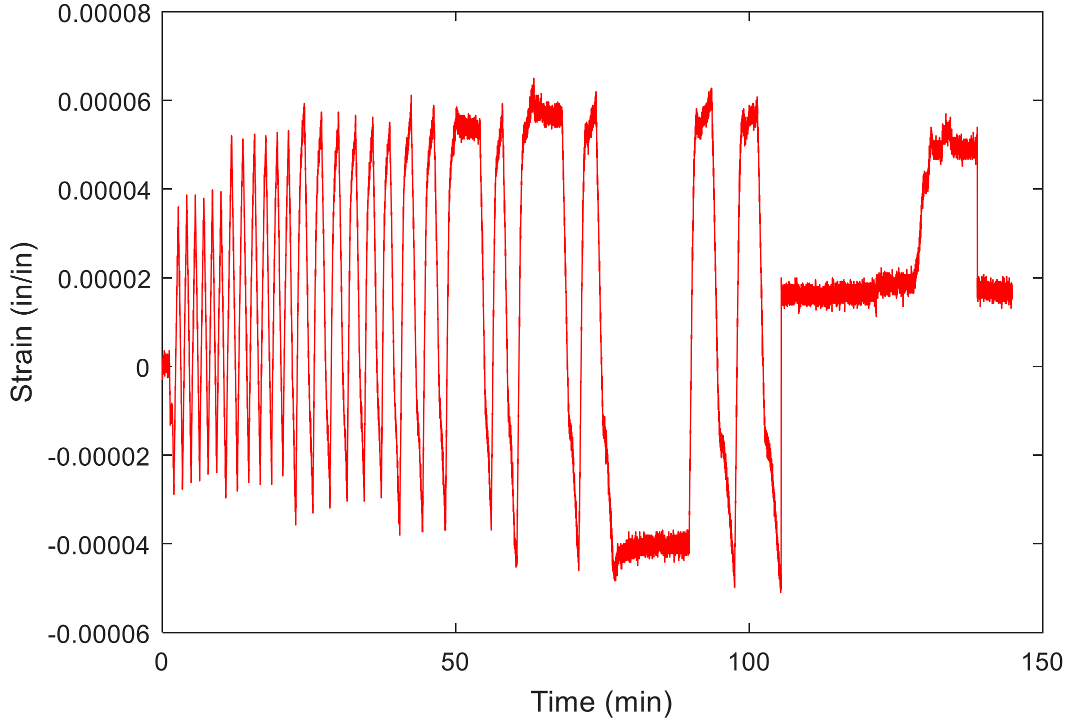


Figure U-34 SG02

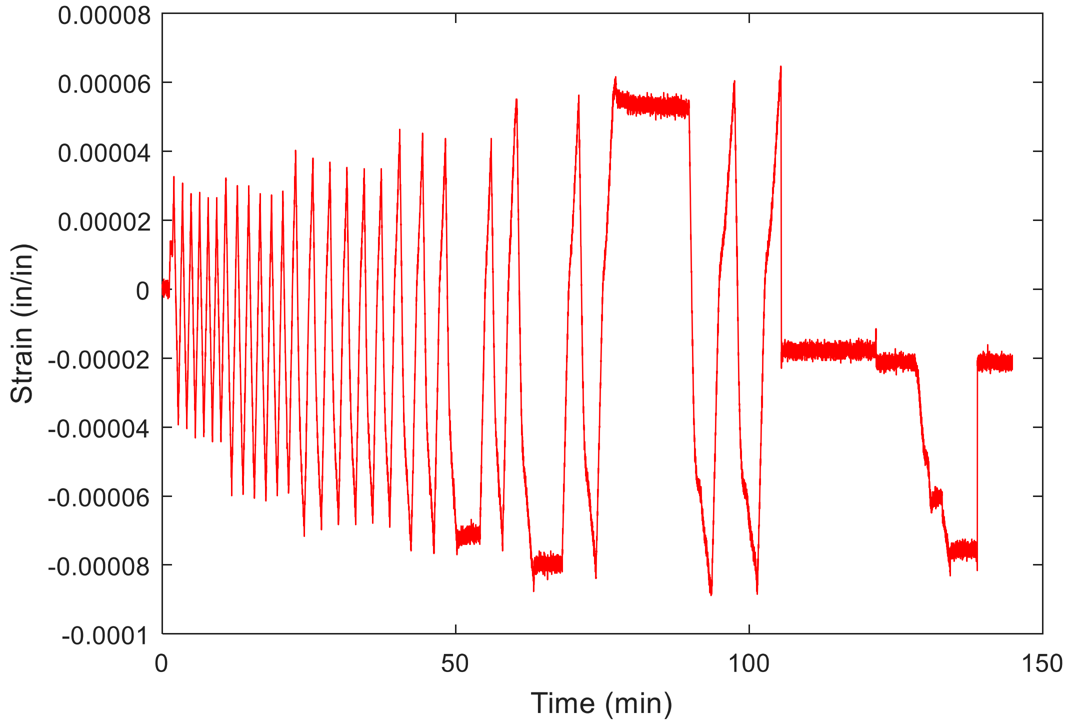


Figure U-35 SG03

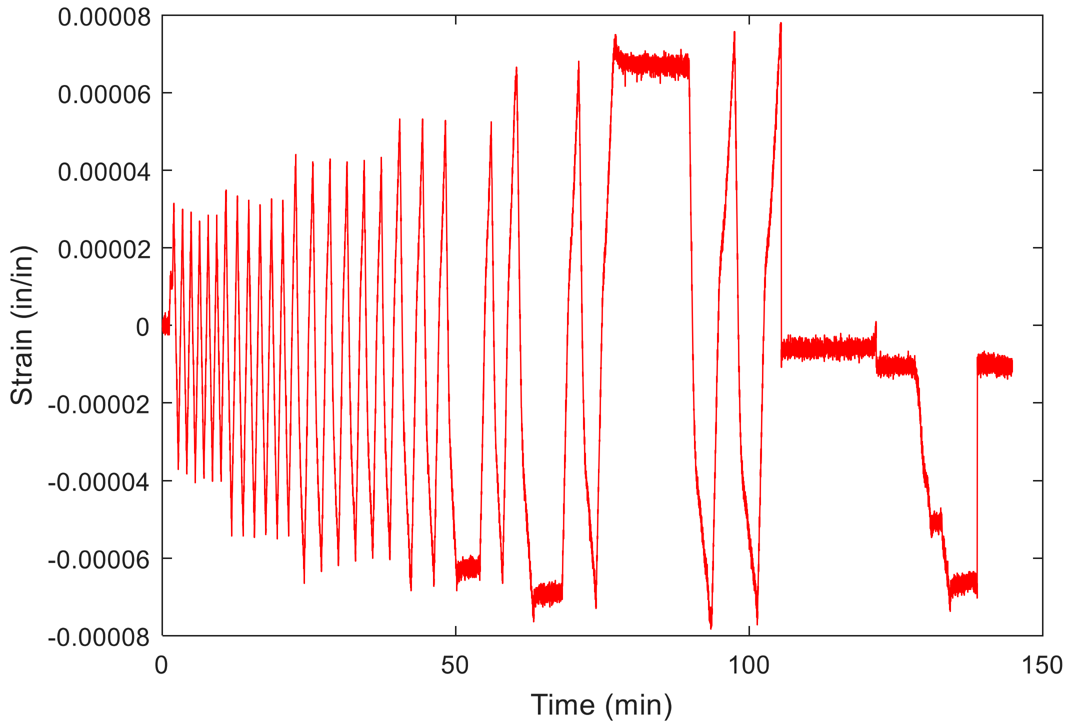


Figure U-36 SG04

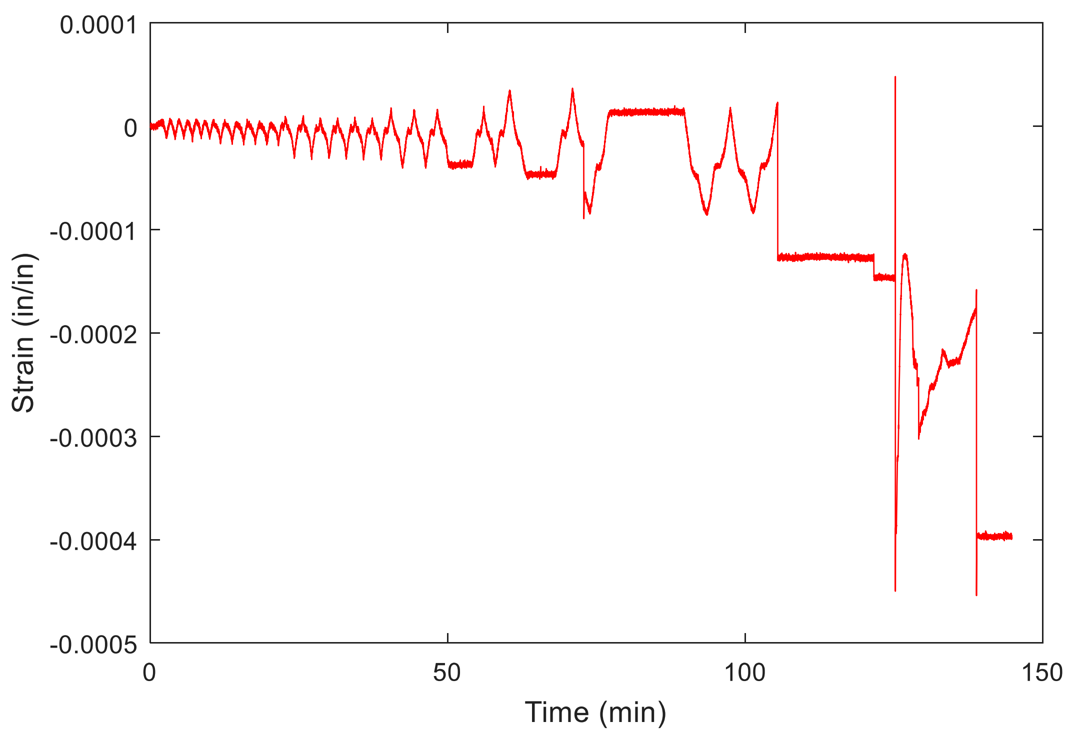


Figure U-37 SG05

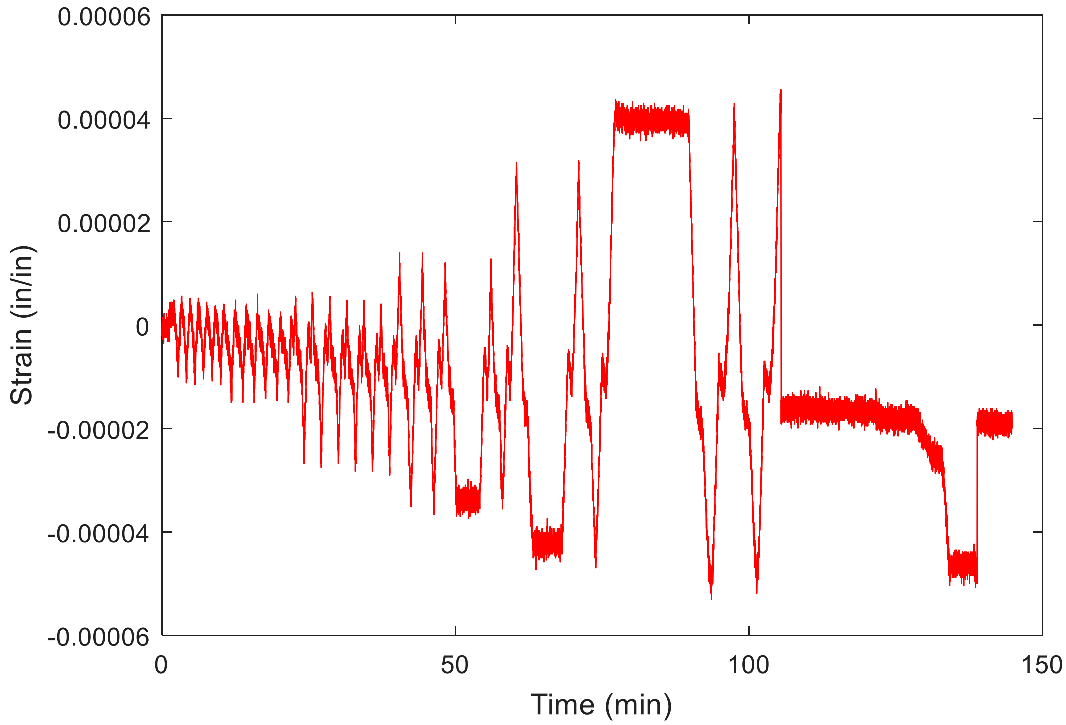


Figure U-38 SG06

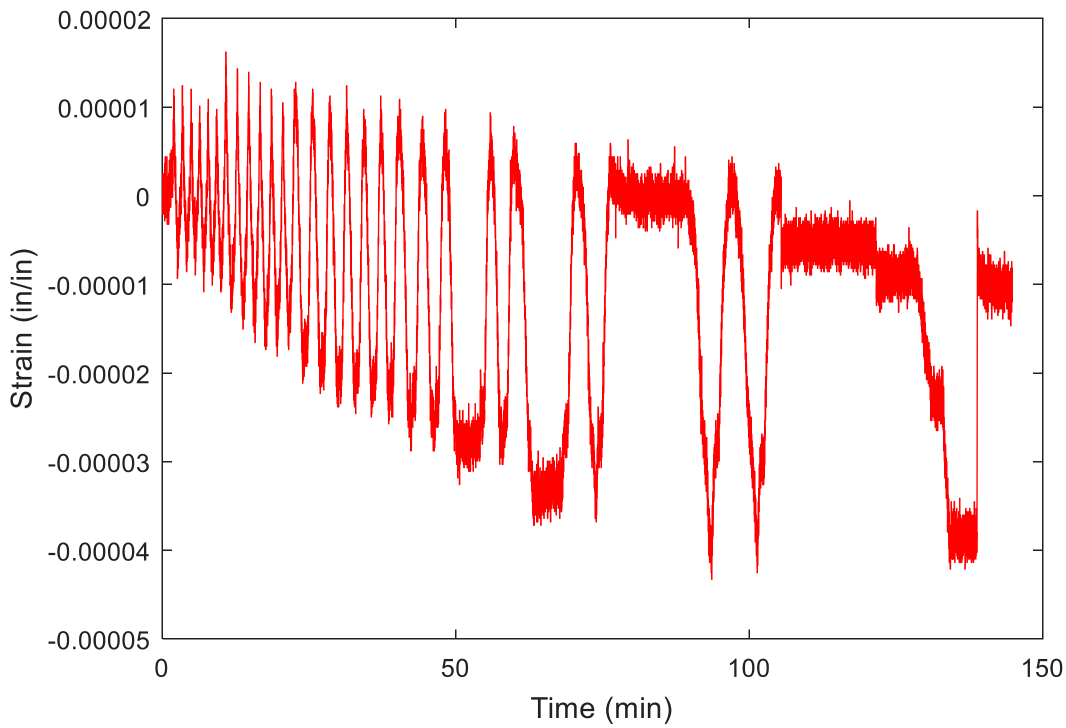


Figure U-39 SG07

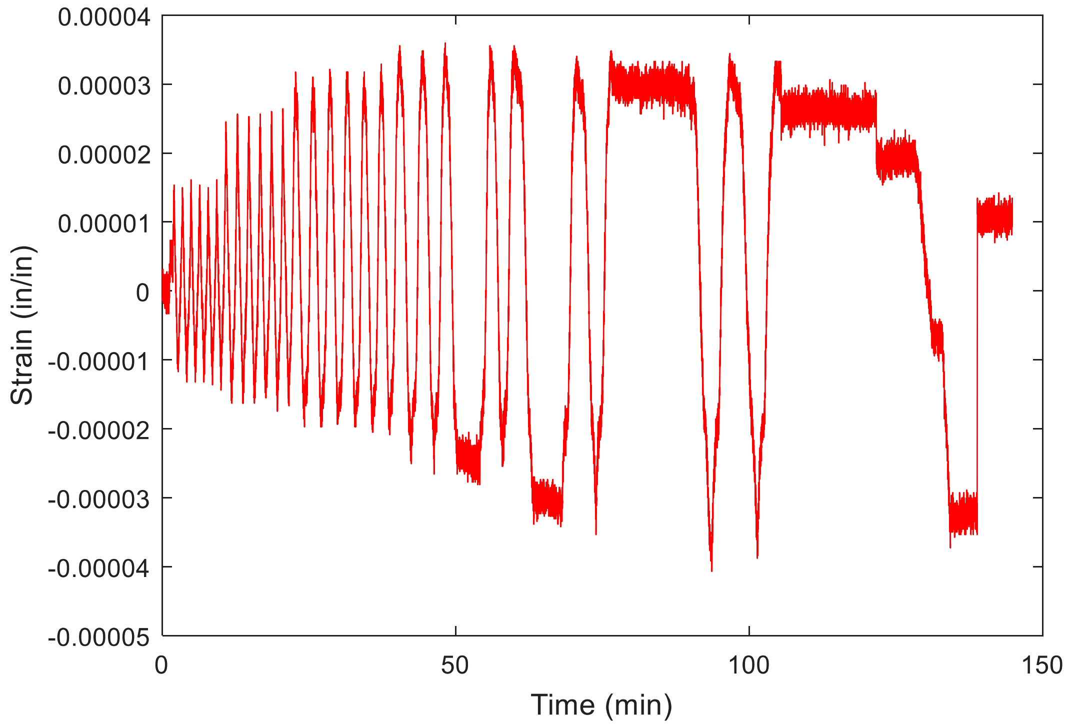


Figure U-40 SG08

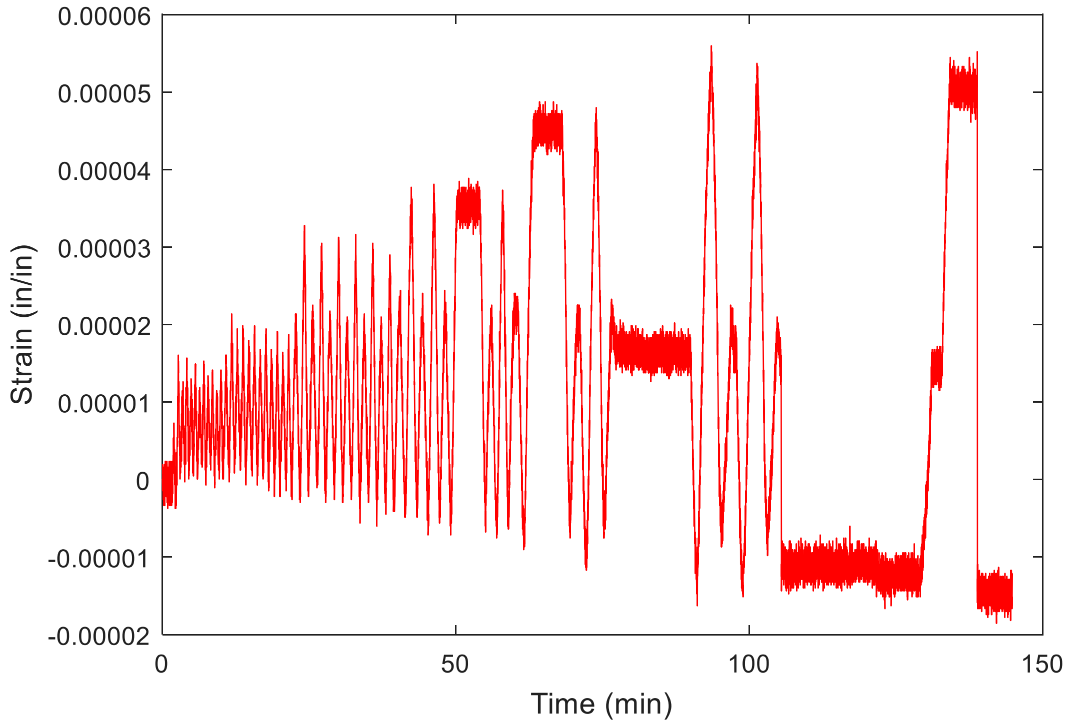


Figure U-41 SG09

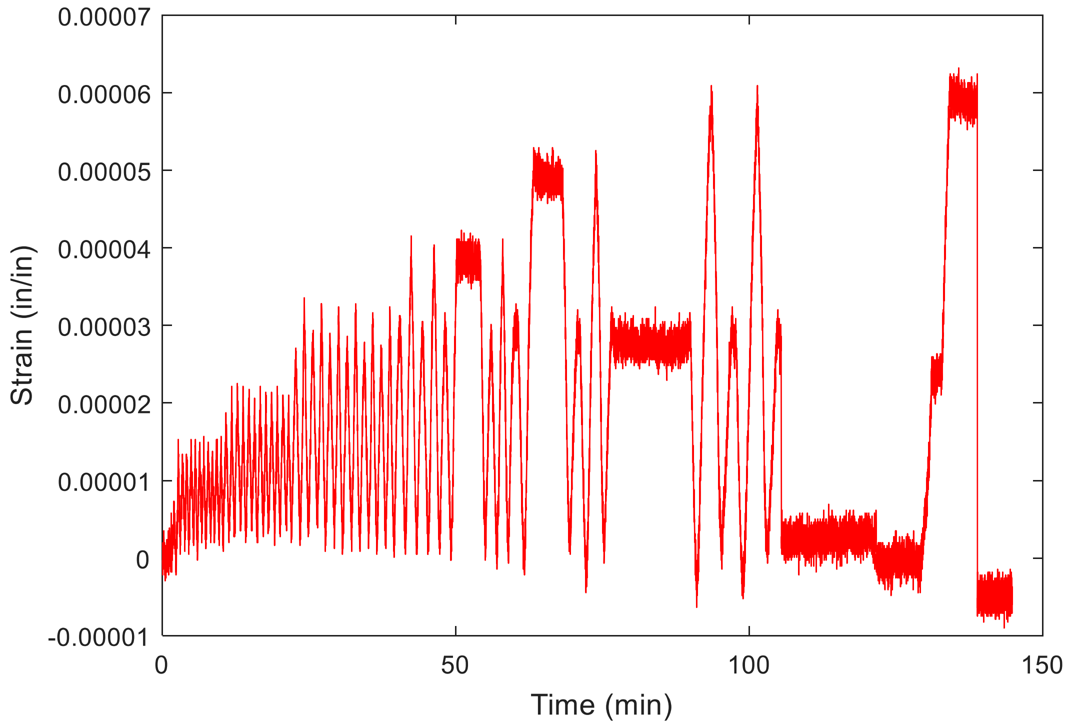


Figure U-42 SG10

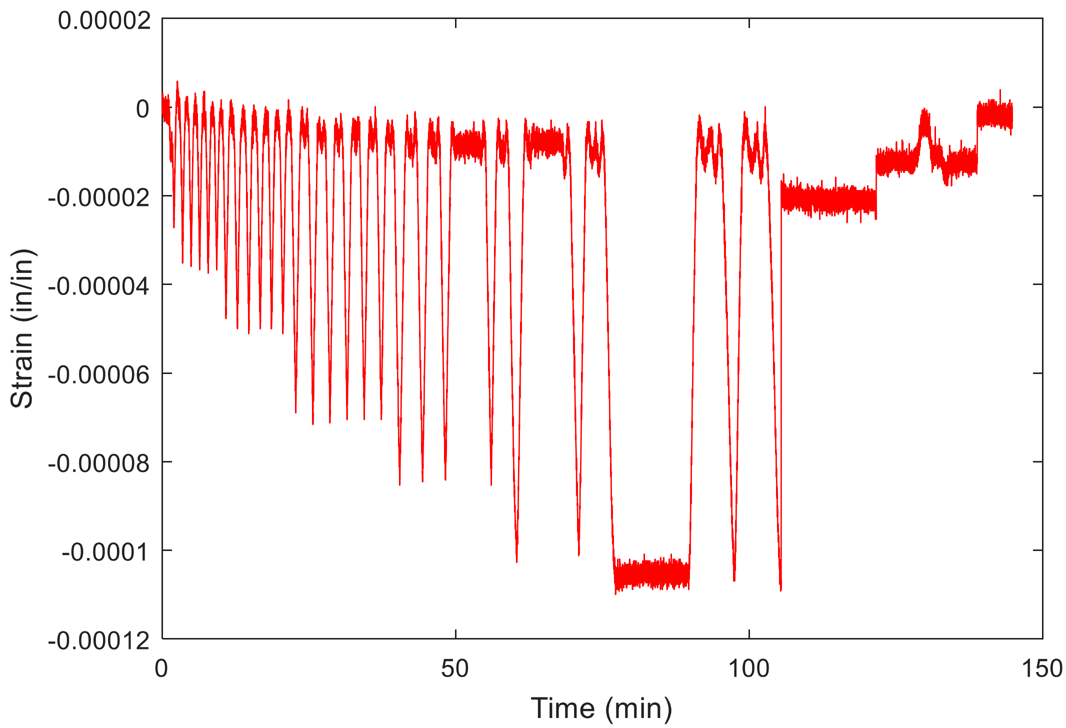


Figure U-43 SG11

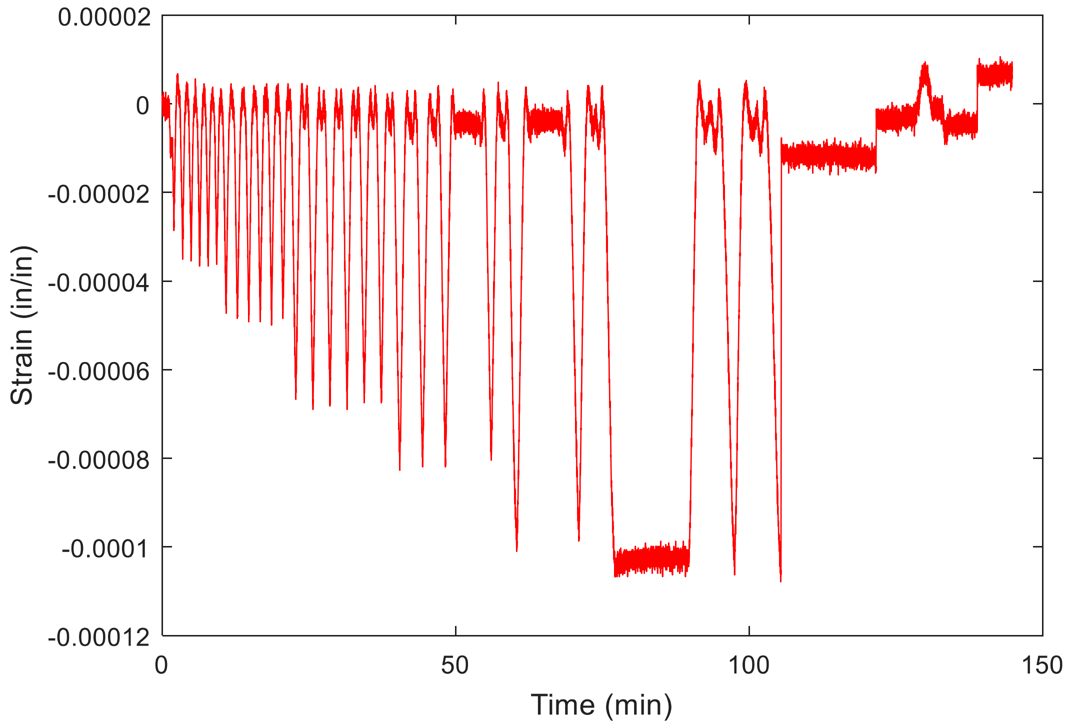


Figure U-44 SG12

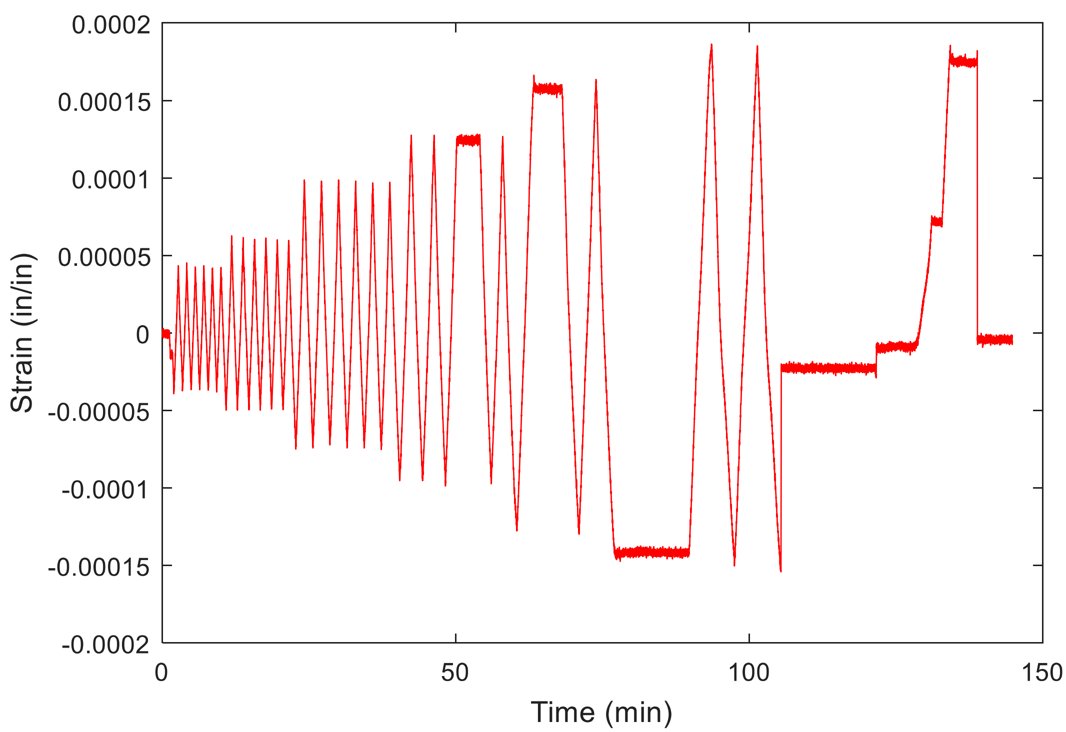


Figure U-45 SG13

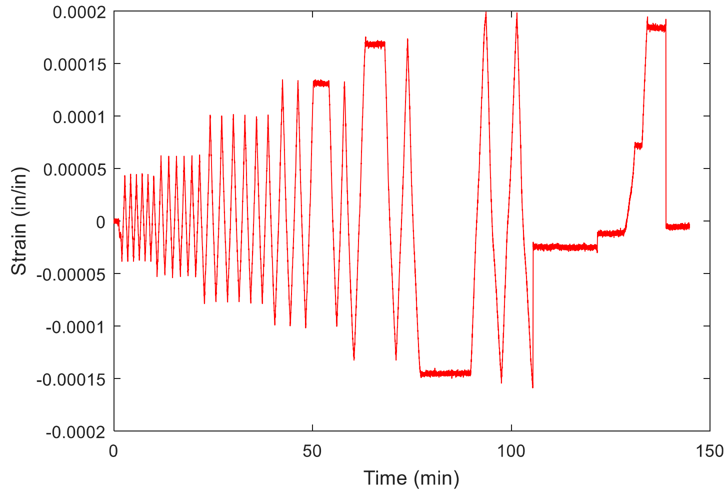


Figure U-46 SG14

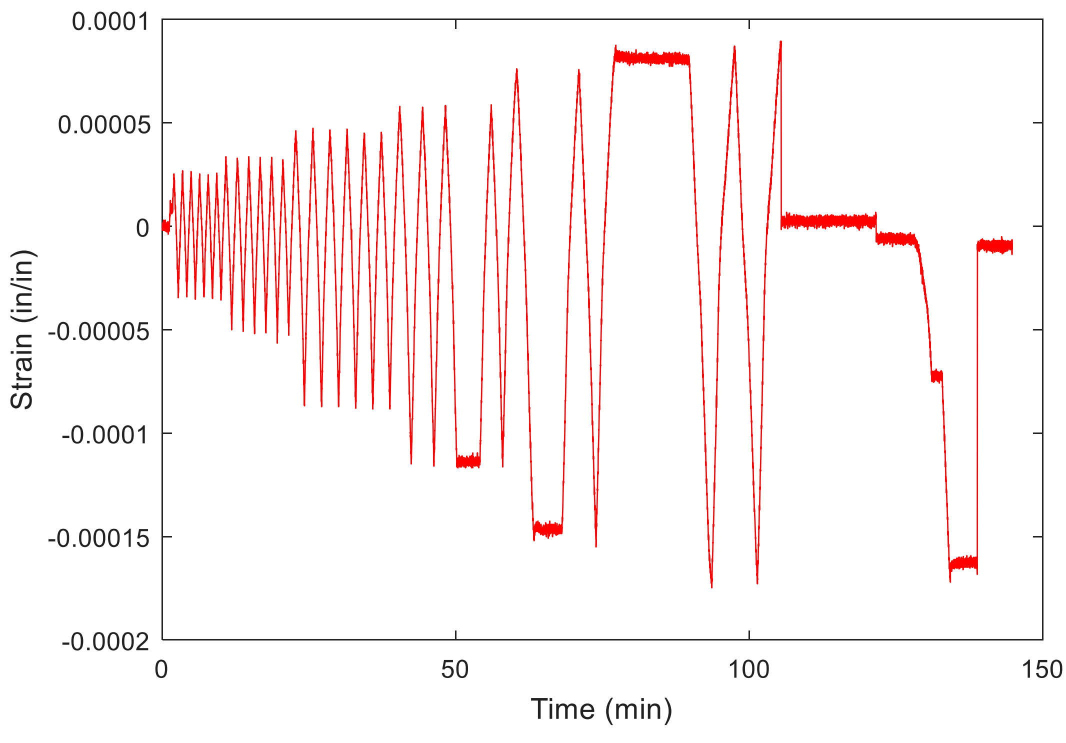


Figure U-47 SG15

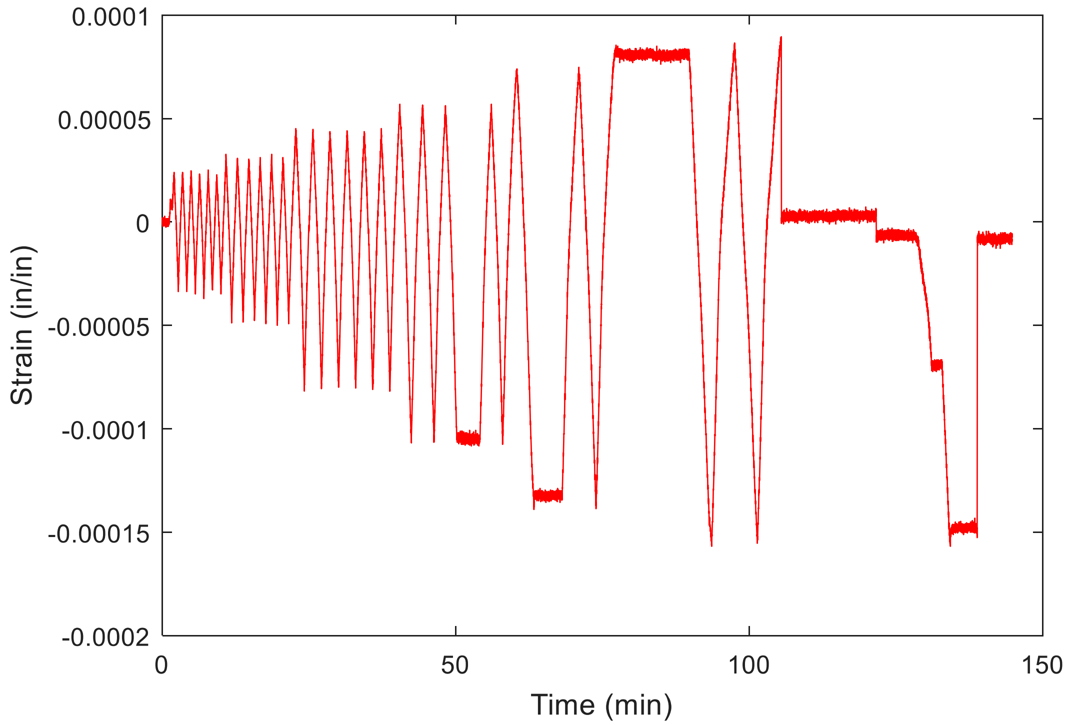


Figure U-48 SG16

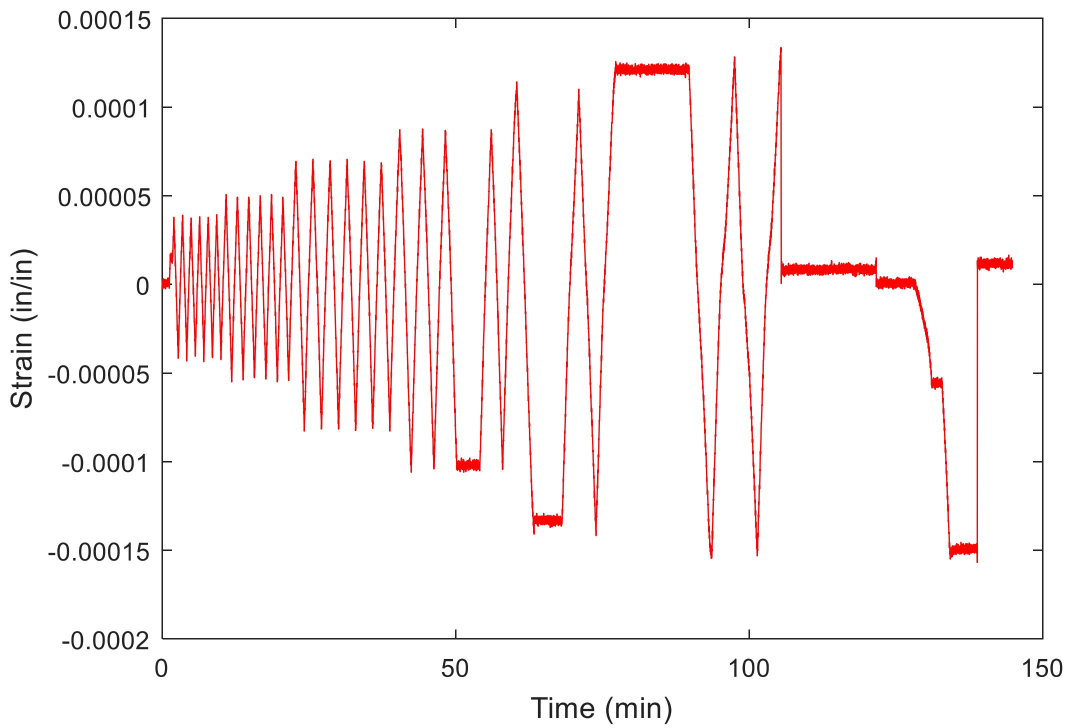


Figure U-49 SG17

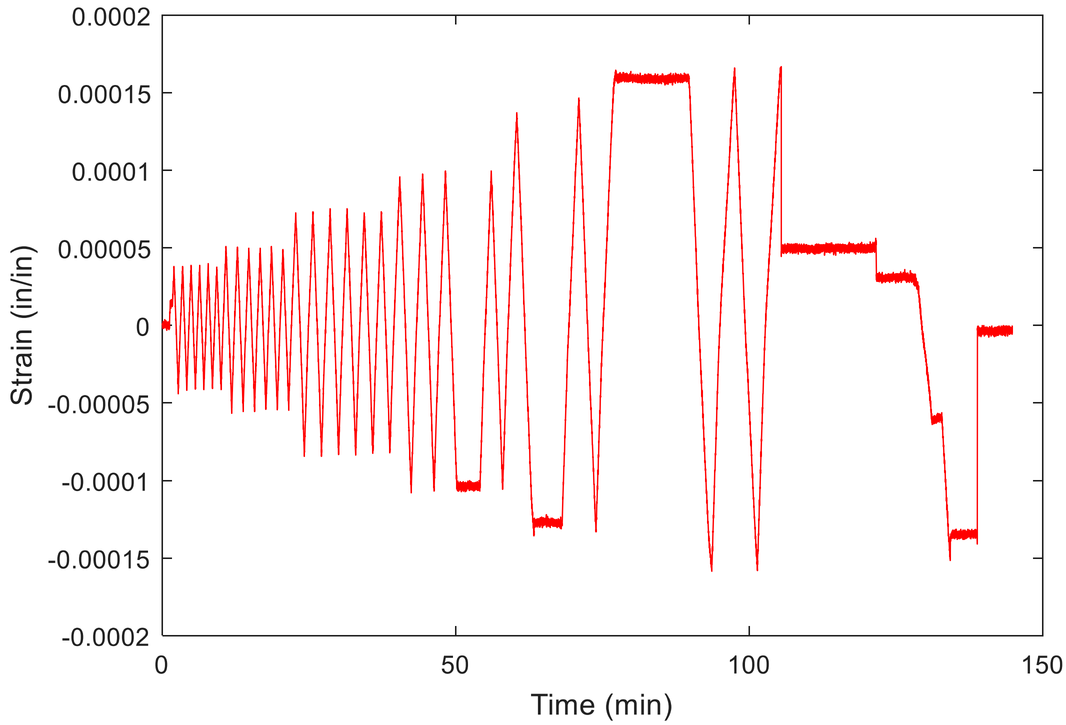


Figure U-50 SG18

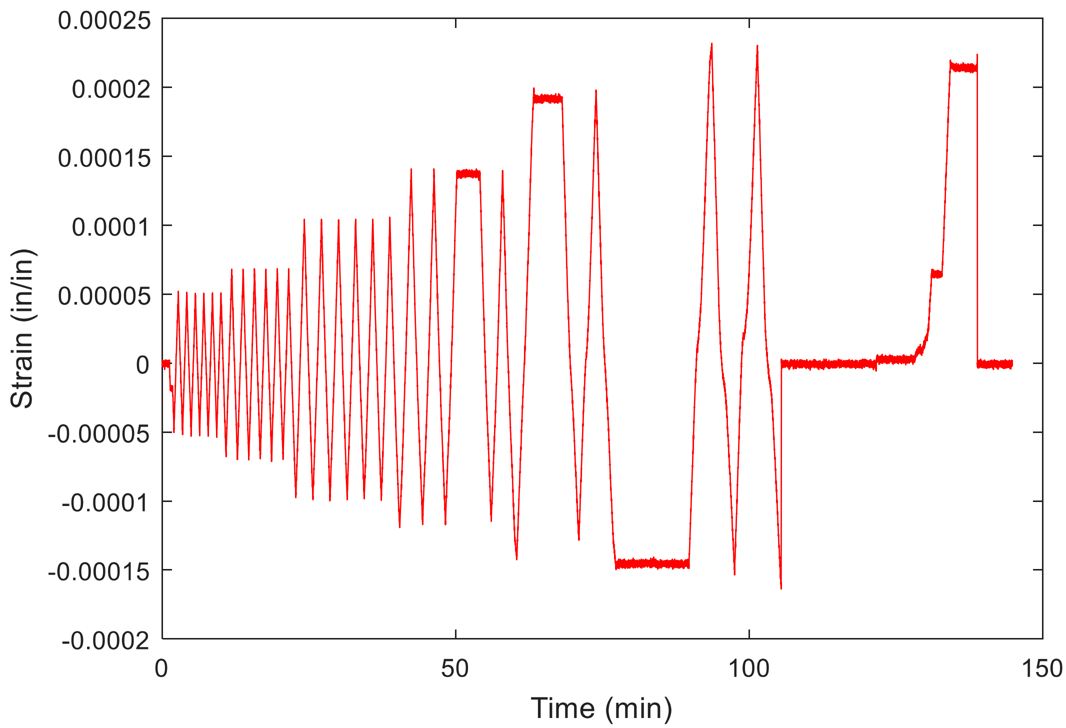


Figure U-51 SG19

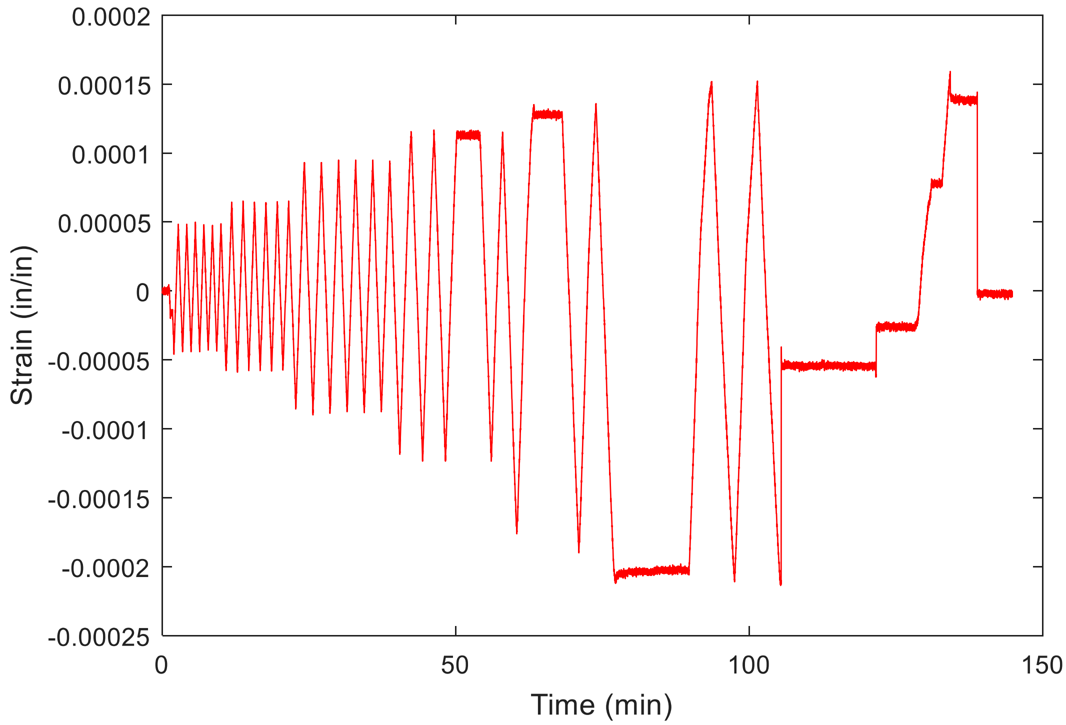


Figure U-52 SG20

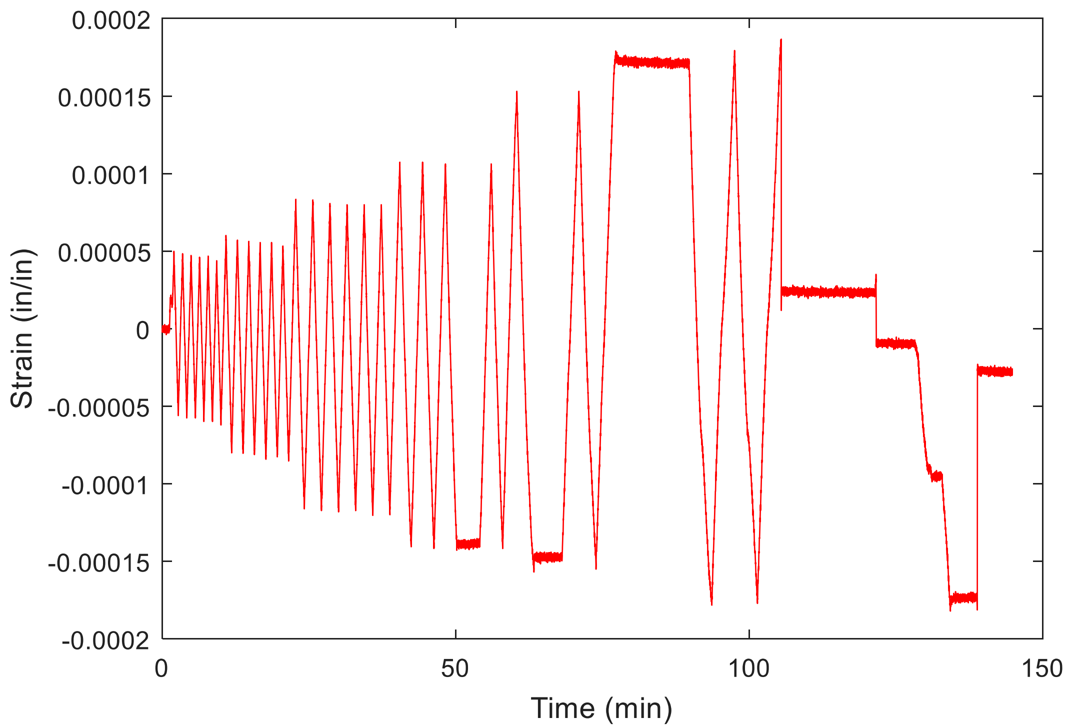


Figure U-53 SG21

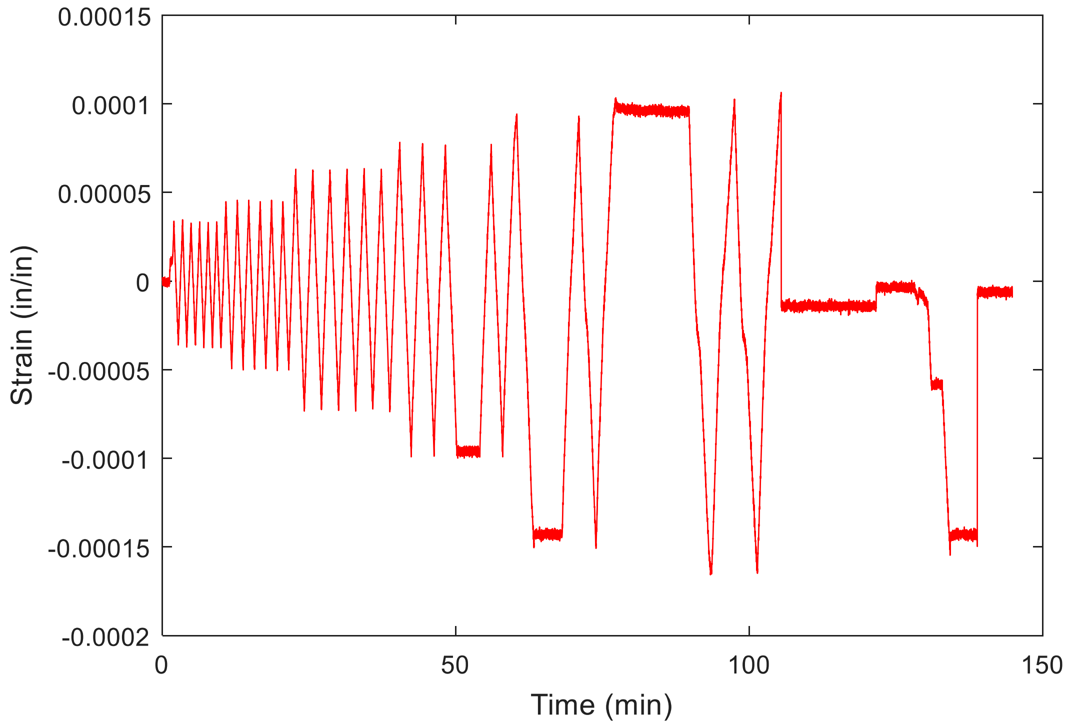


Figure U-54 SG22

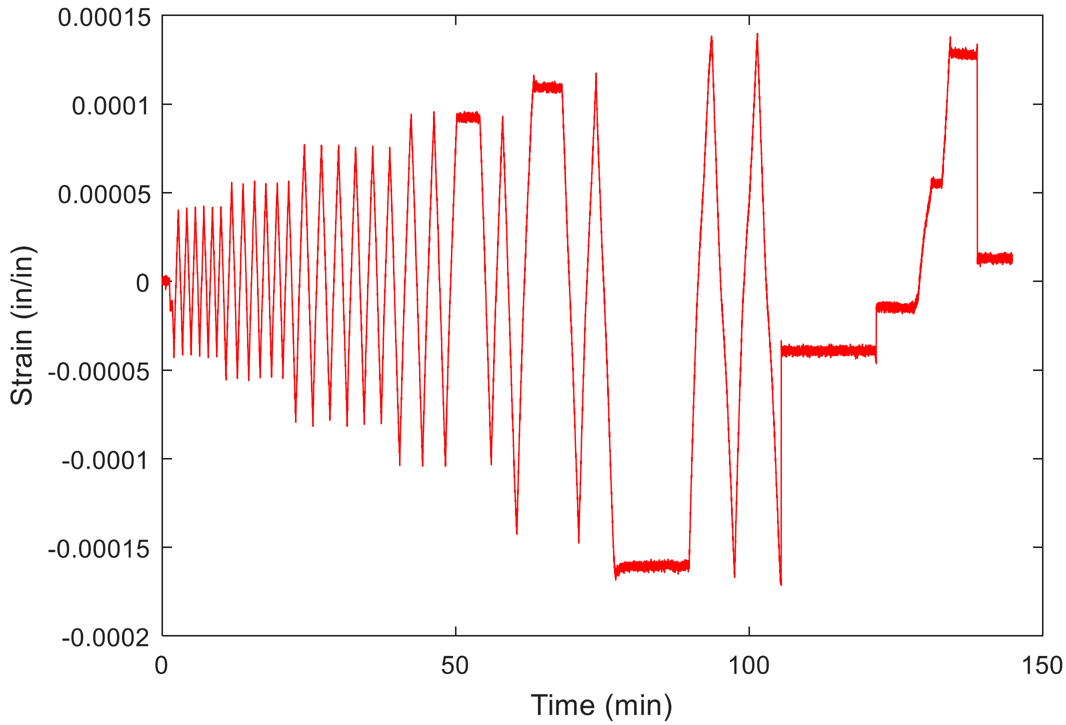


Figure U-55 SG23

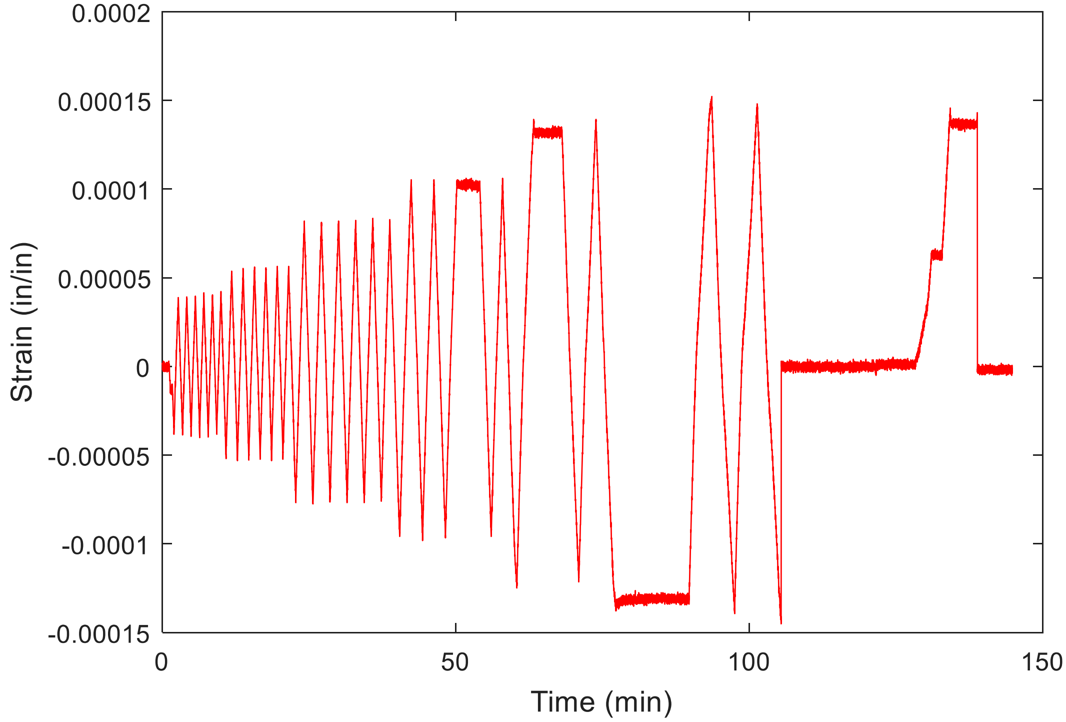


Figure U-56 SG24

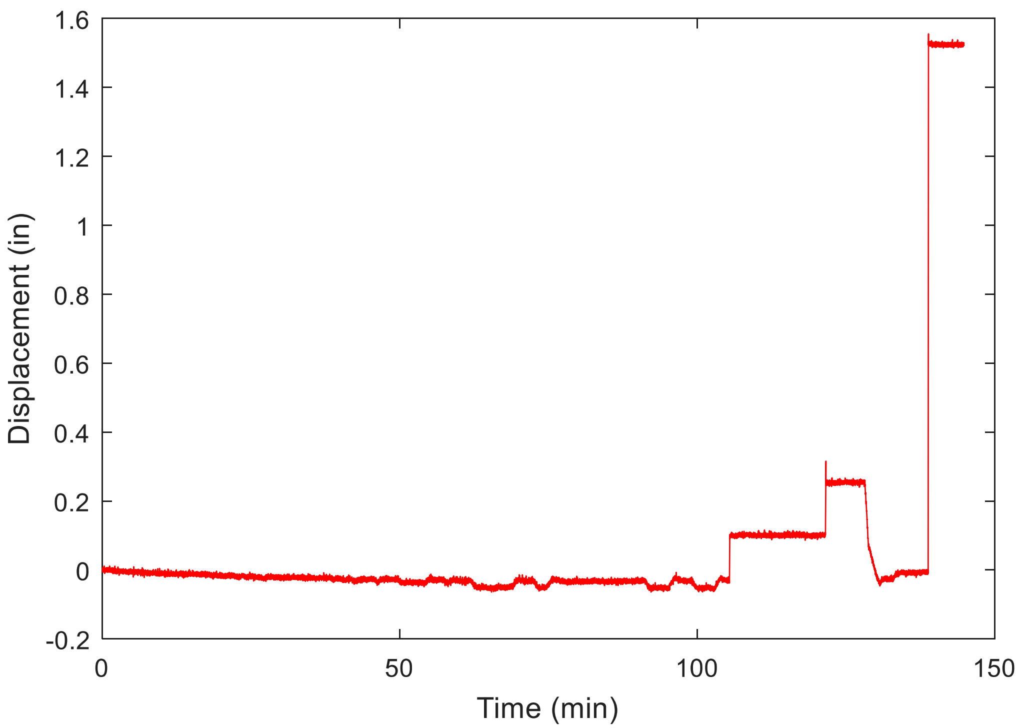


Figure U-57 CLP01

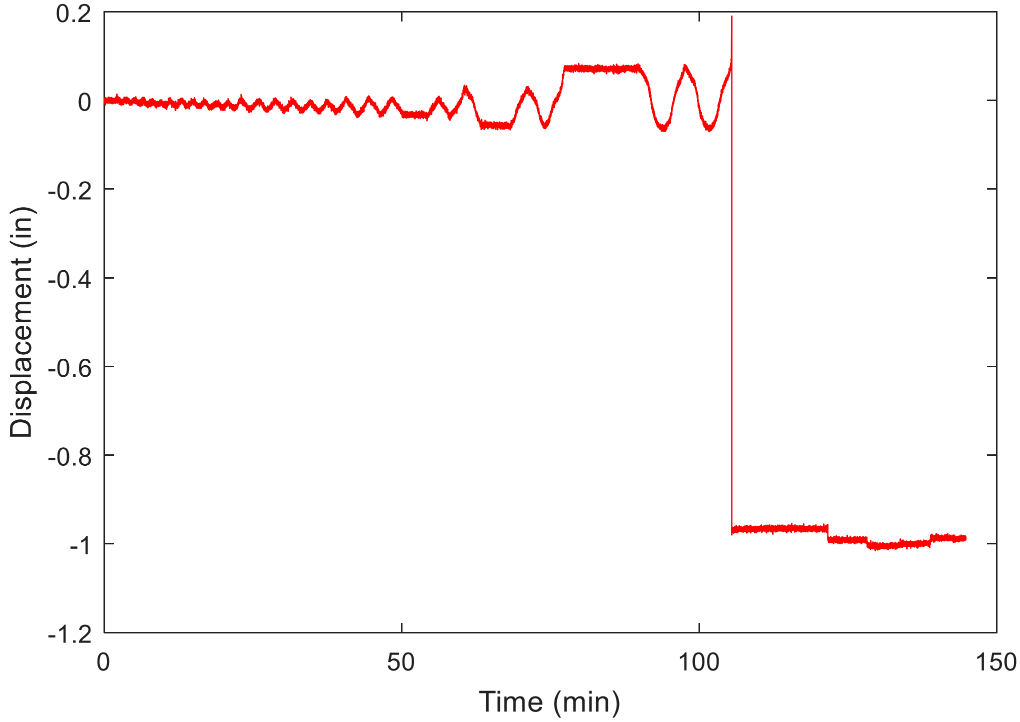


Figure U-58 CLP02

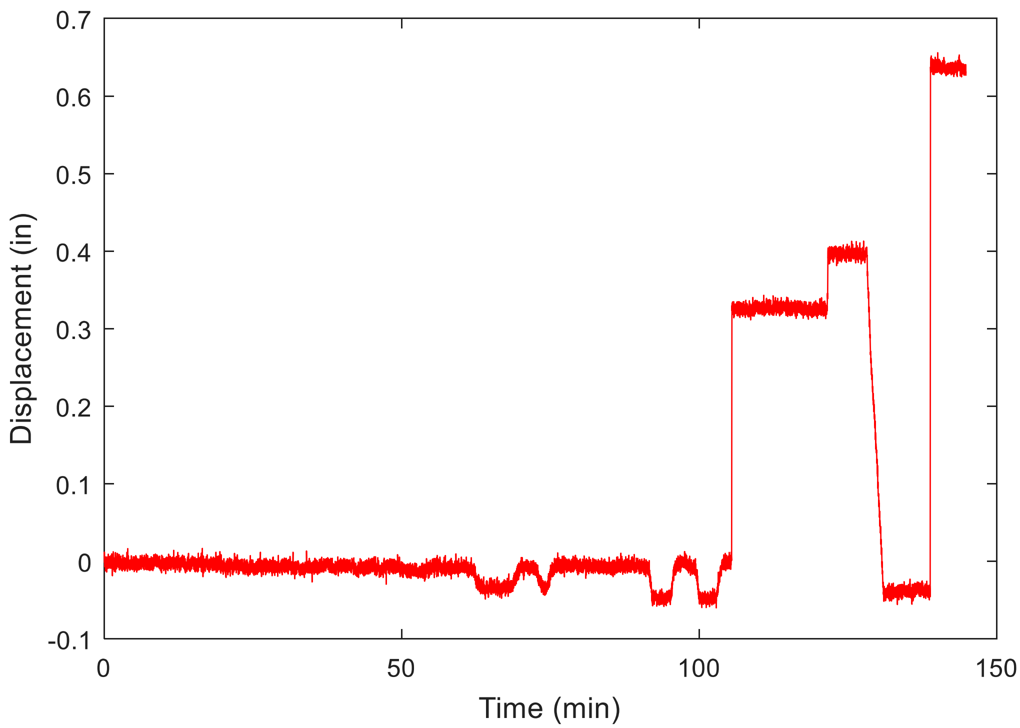


Figure U-59 CLP03

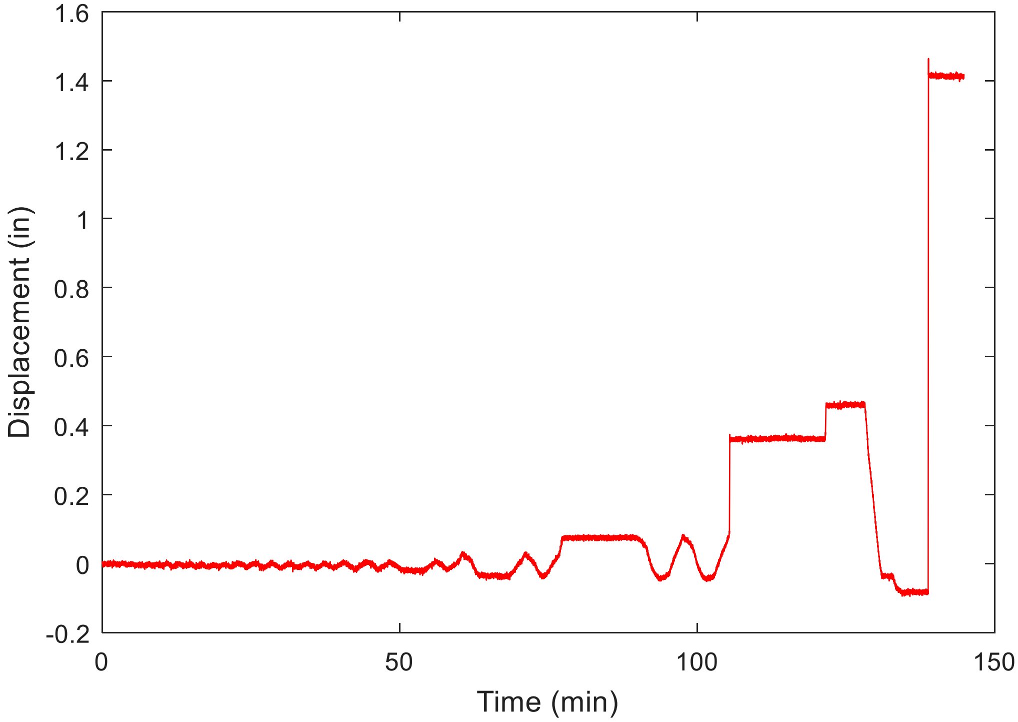


Figure U-60 CLP04

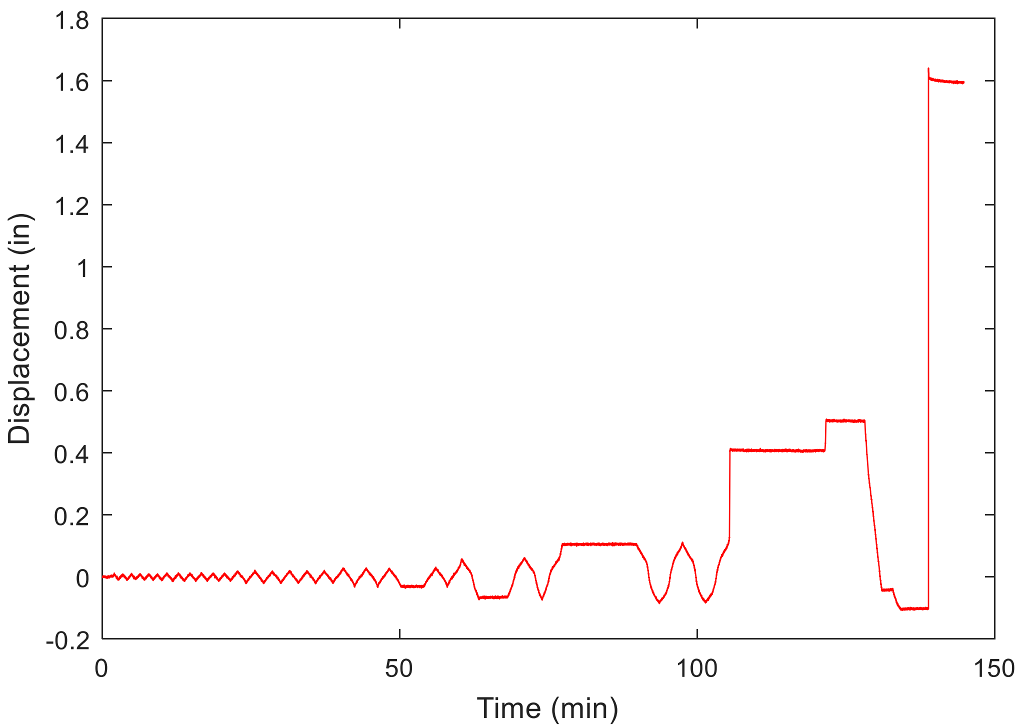


Figure U-61 CLP05

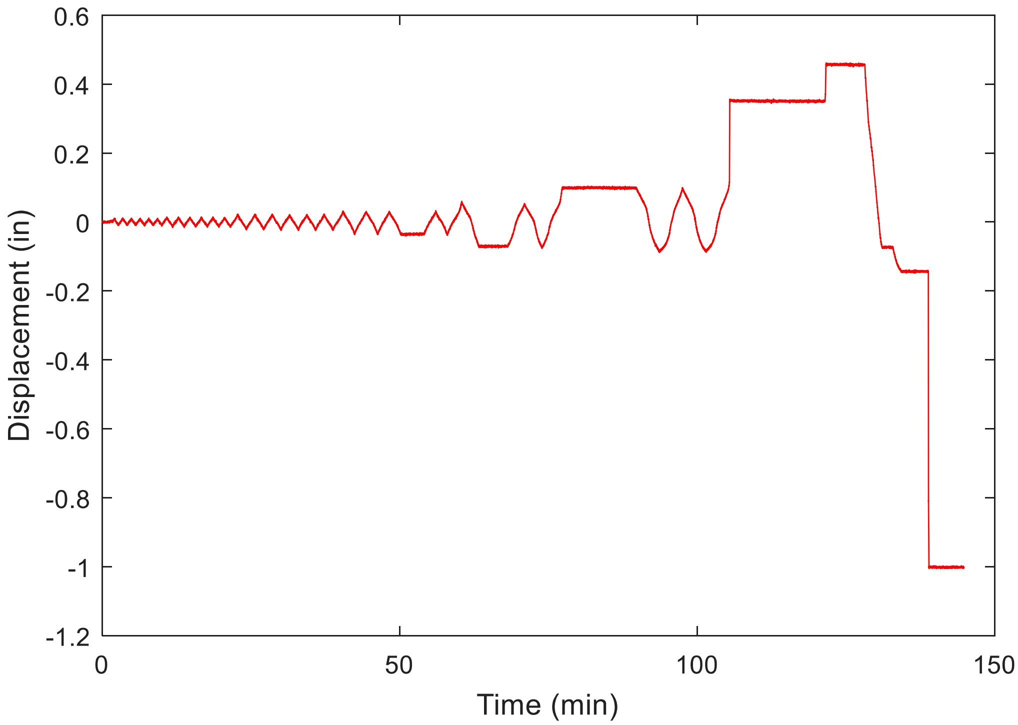


Figure U-62 CLP06

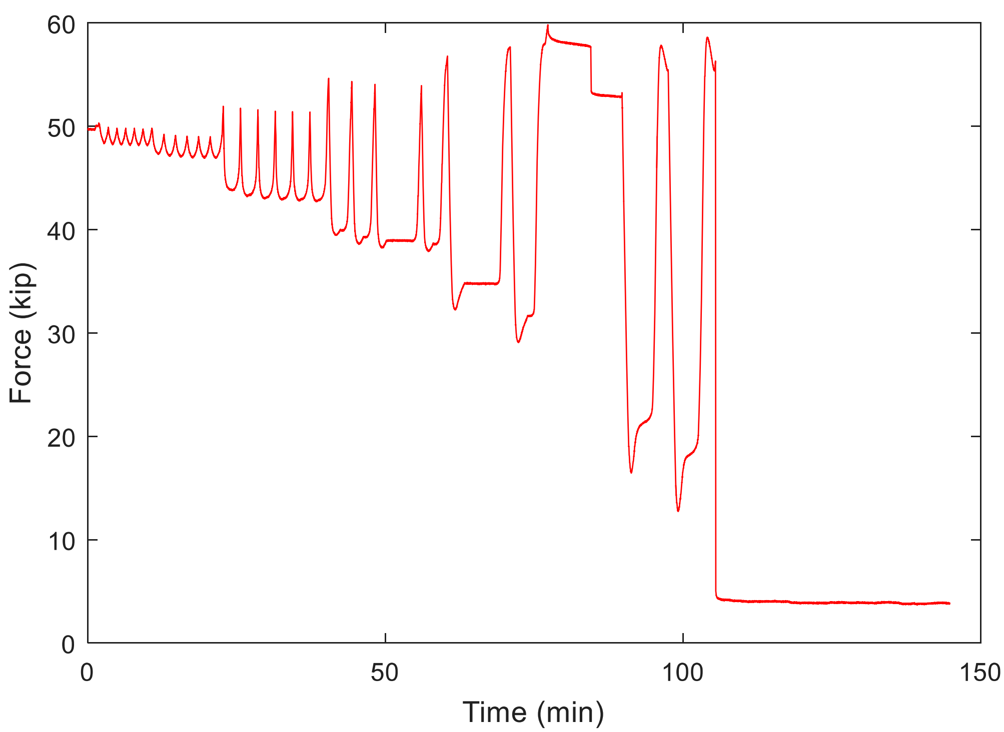


Figure U-63 Force in BSG01

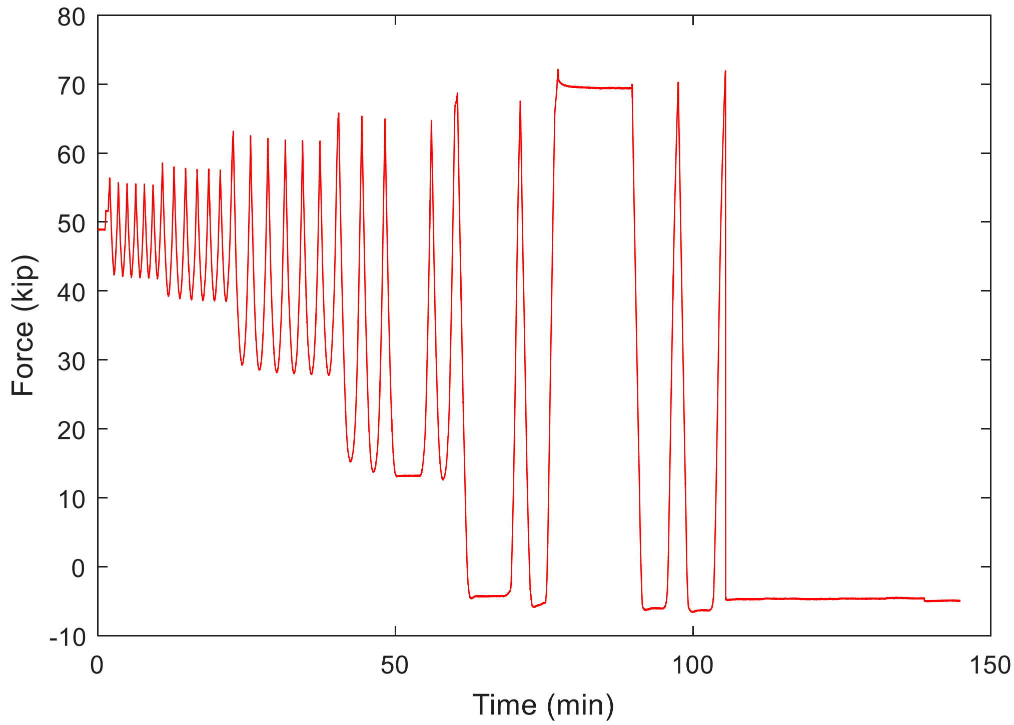


Figure U-64 Force in BSG02

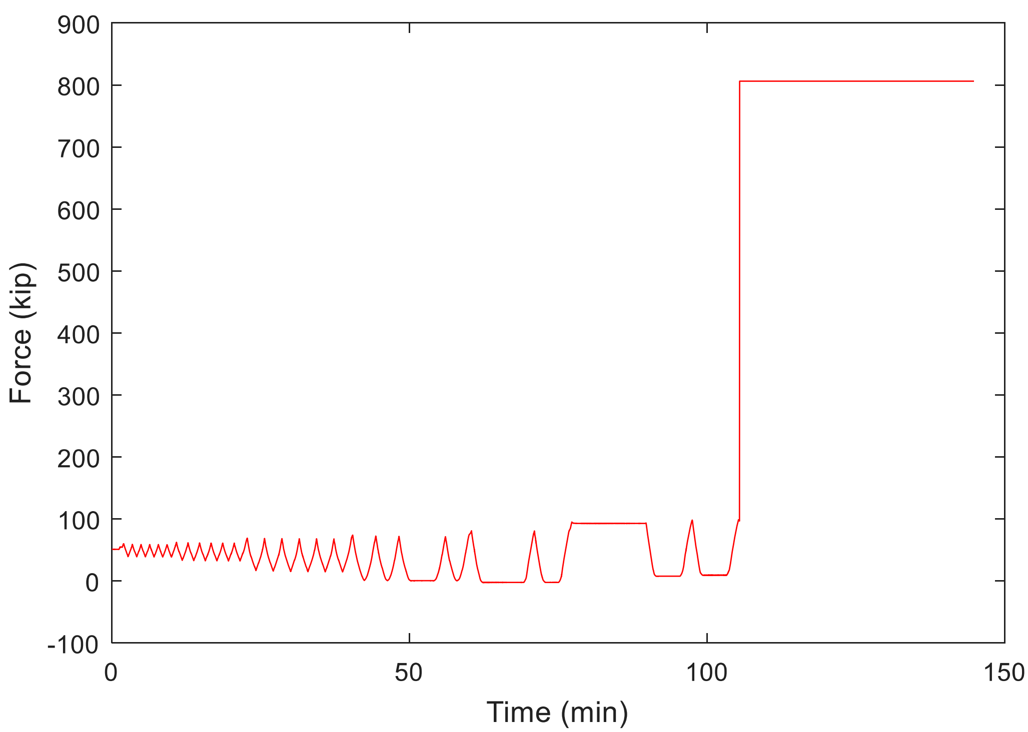


Figure U-65 Force in BSG03

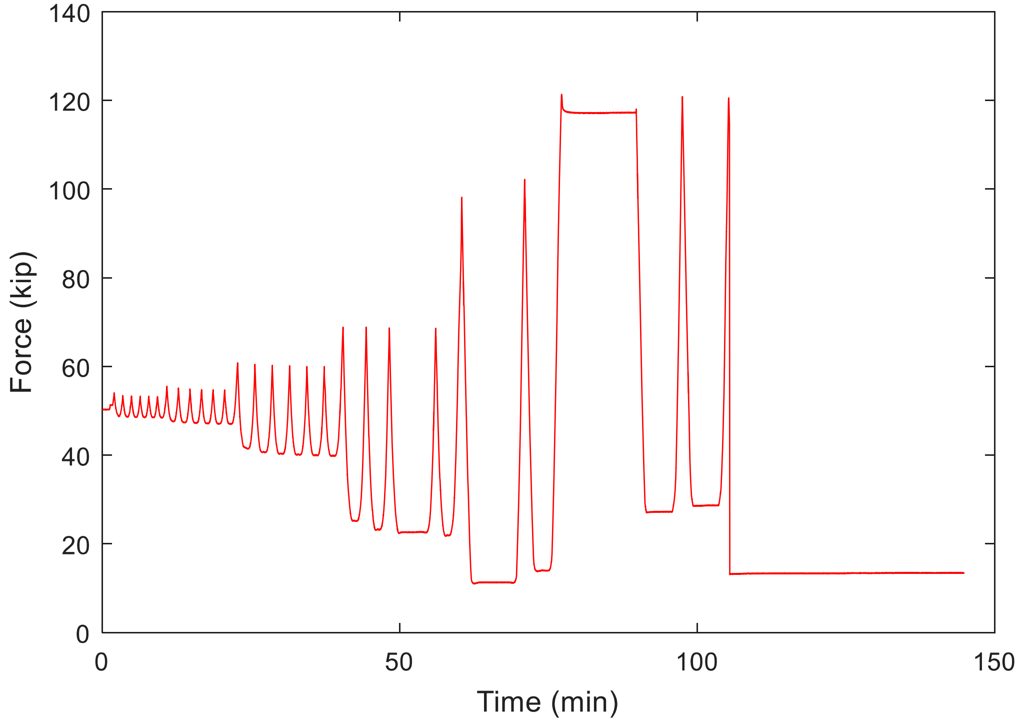


Figure U-66 Force in BSG04

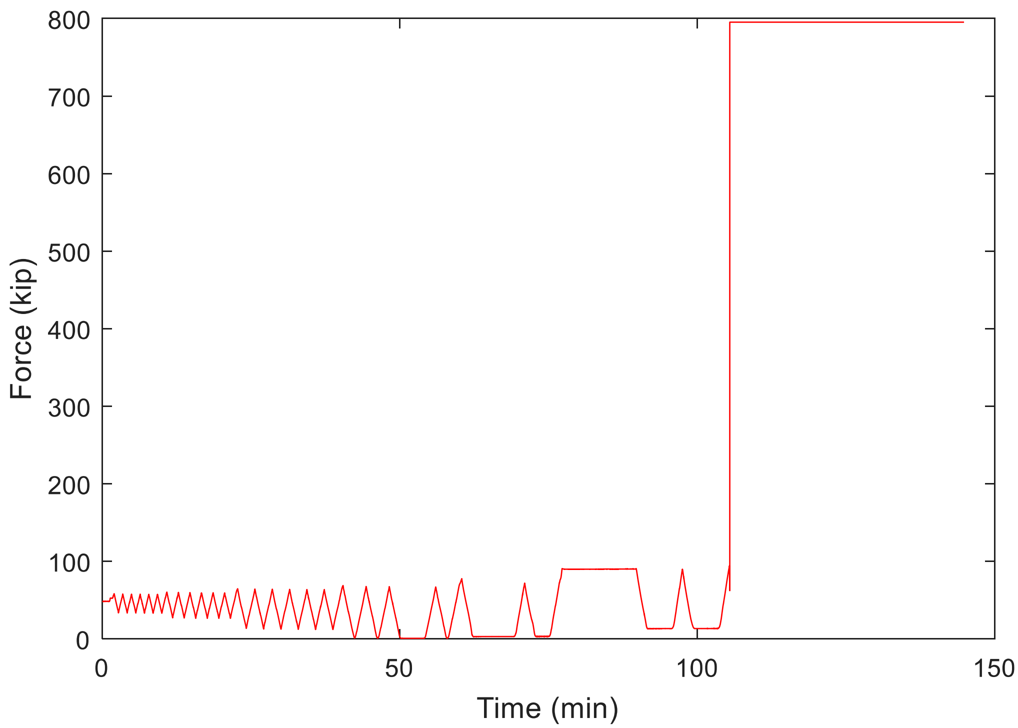


Figure U-67 Force in BSG05

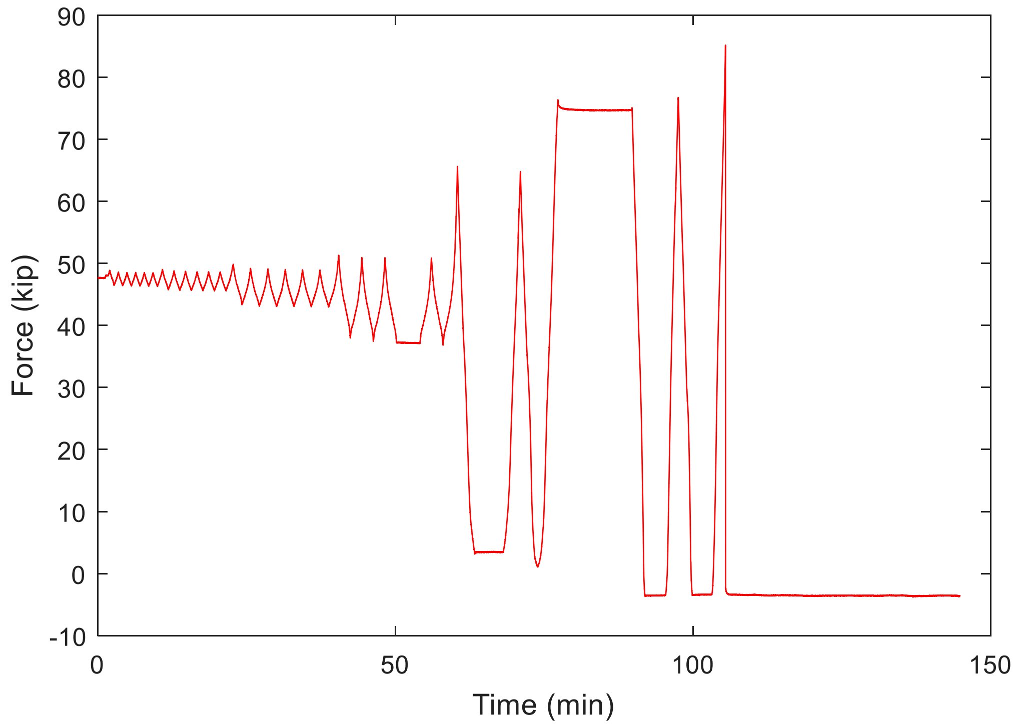


Figure U-68 Force in BSG06

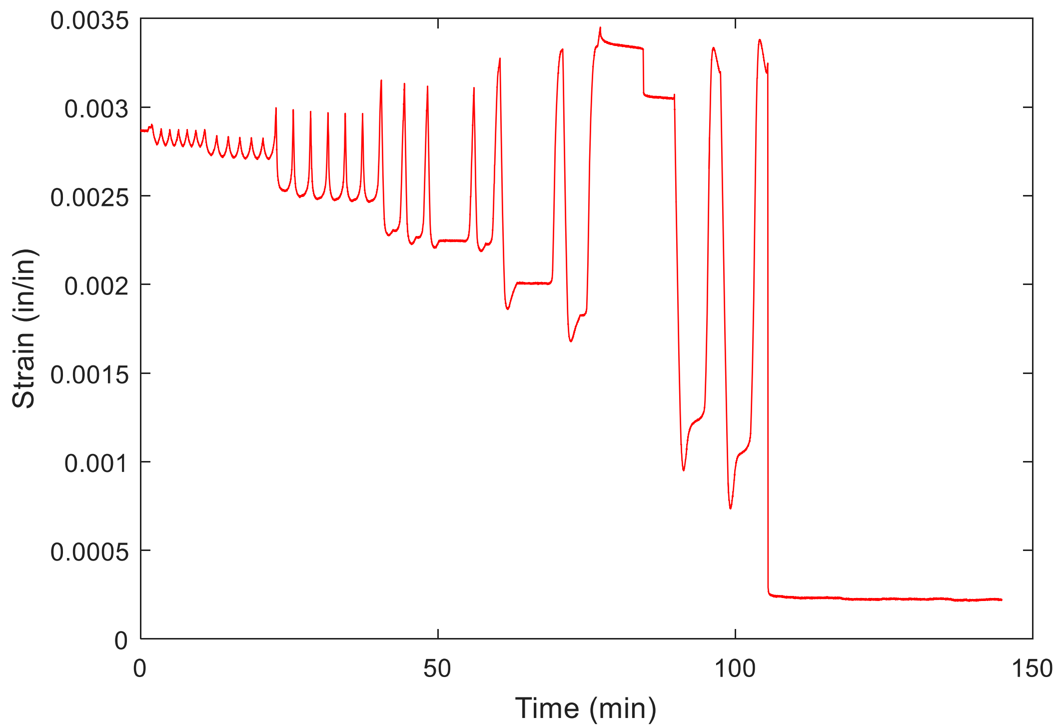


Figure U-69 Strain in BSG01

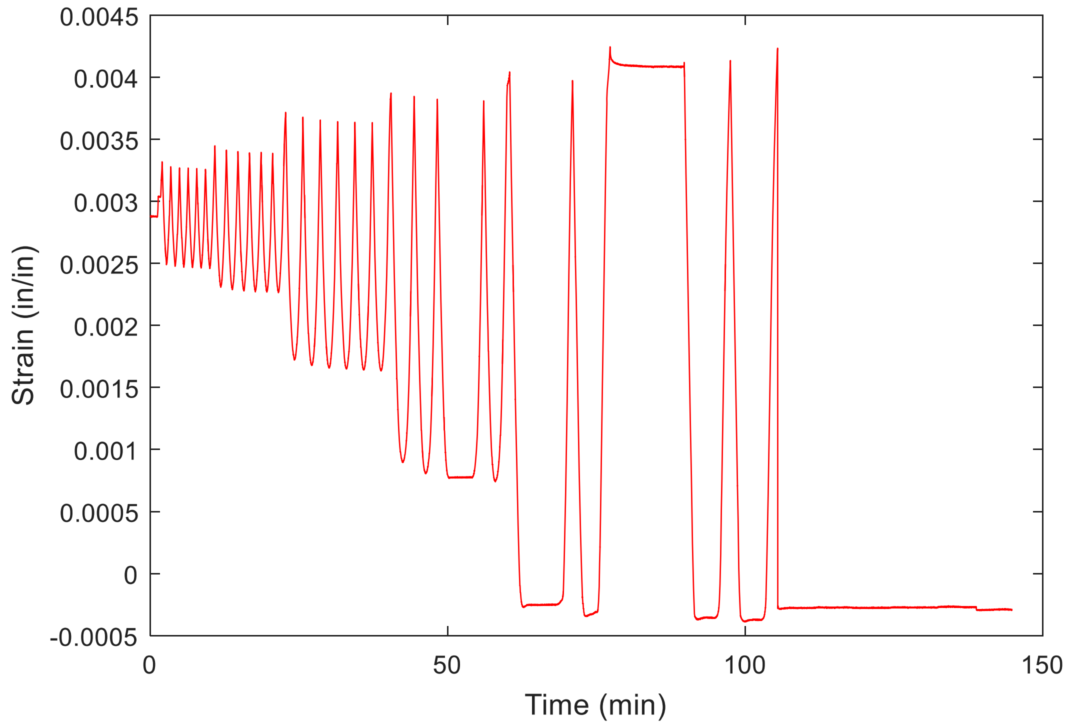


Figure U-70 Strain in BSG02

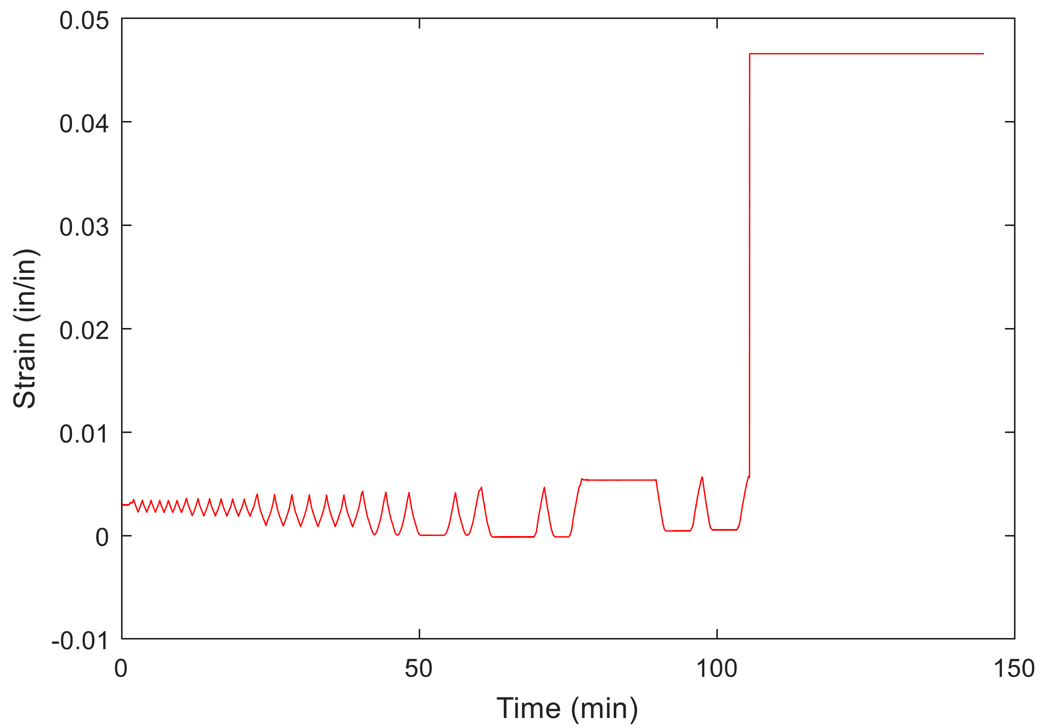


Figure U-71 Strain in BSG03

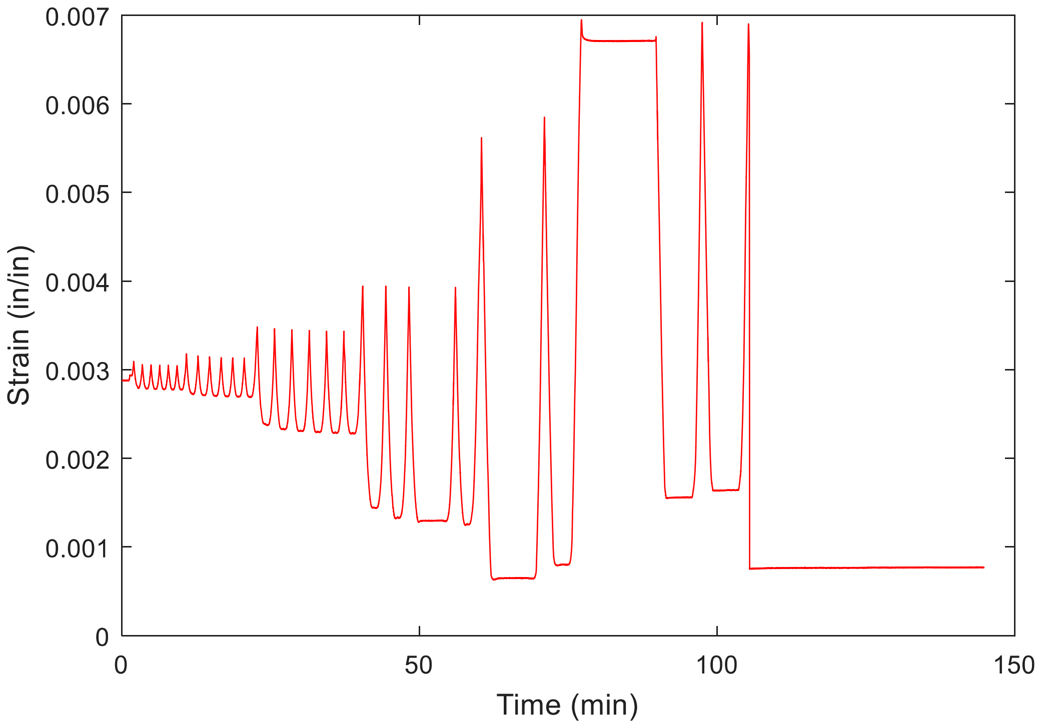


Figure U-72 Strain in BSG04

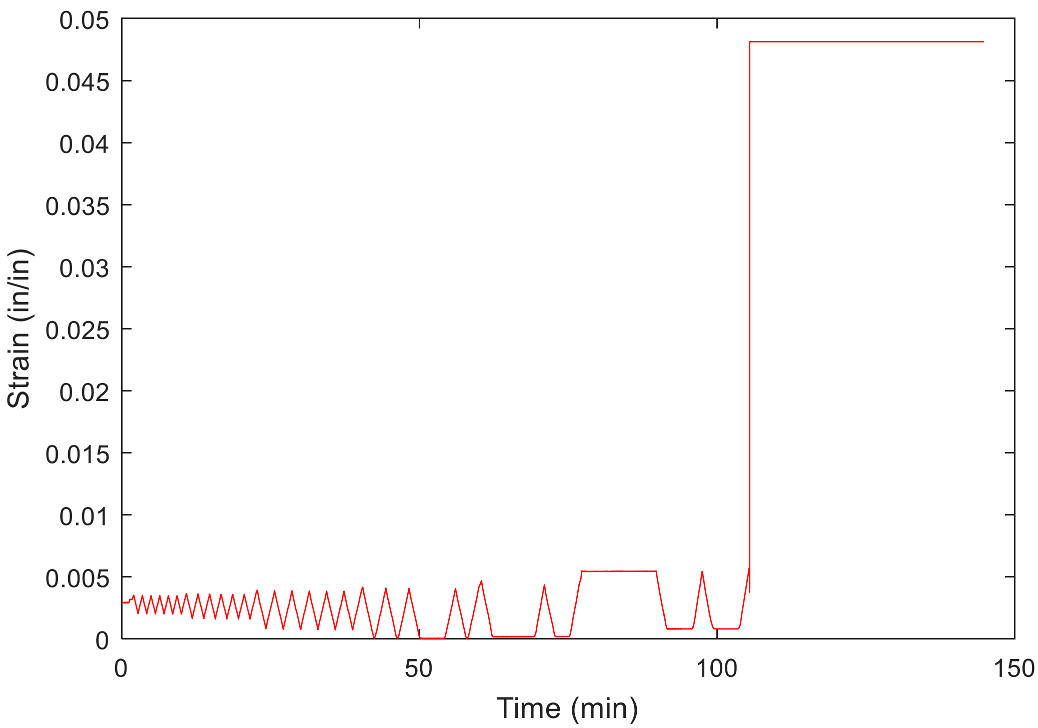


Figure U-73 Strain in BSG05

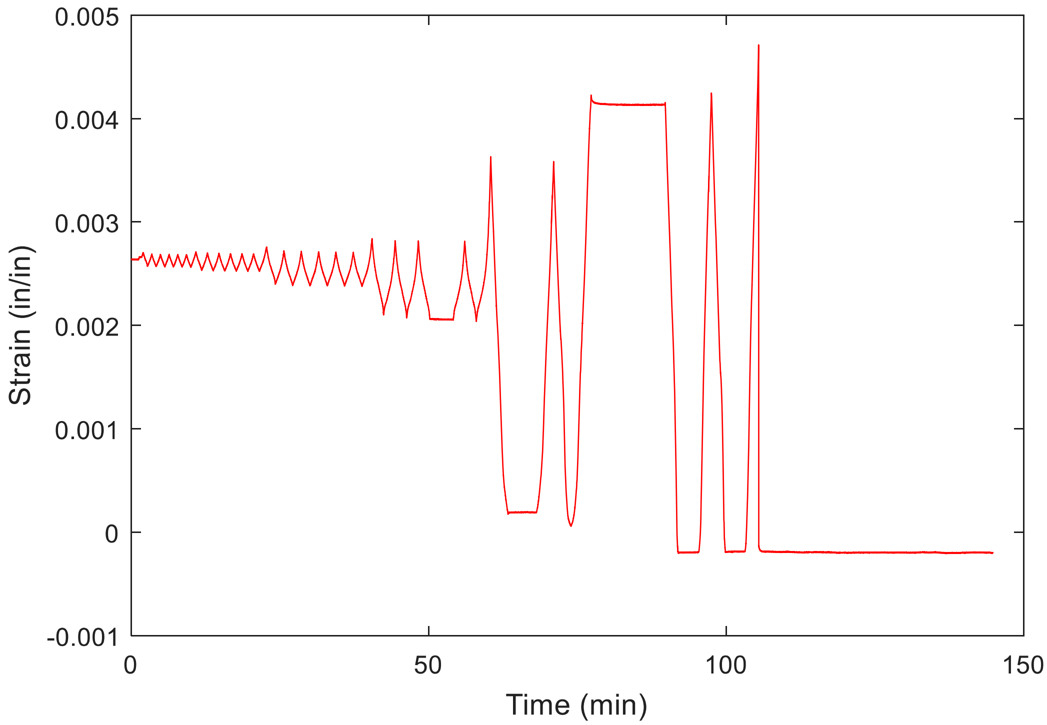


Figure U-74 Strain in BSG06

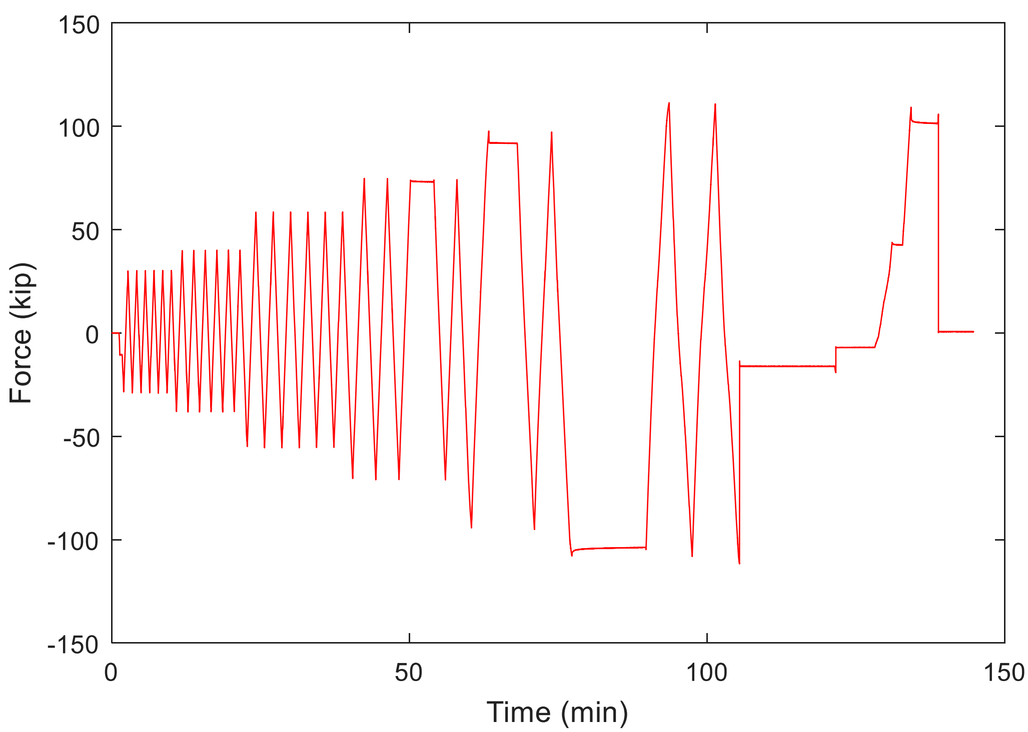


Figure U-75 Actuator Force

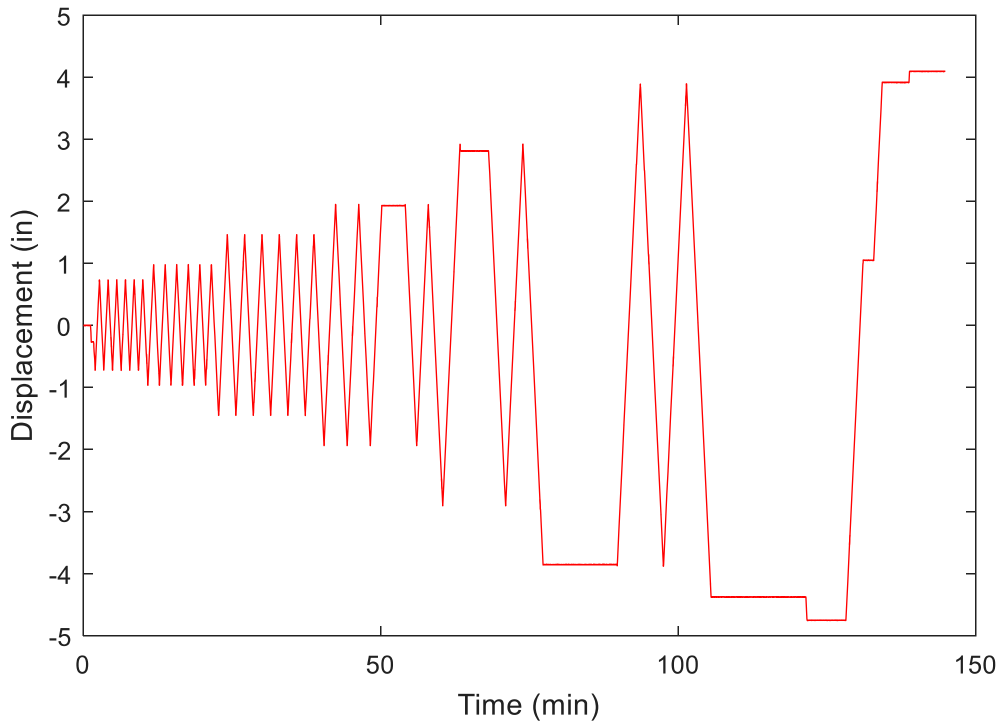


Figure U-76 Actuator Displacement

APPENDIX V ADDITIONAL PLOTS FOR SPECIMEN 12ES-1.125-1.25-24

Plots from the start until the beam fractured

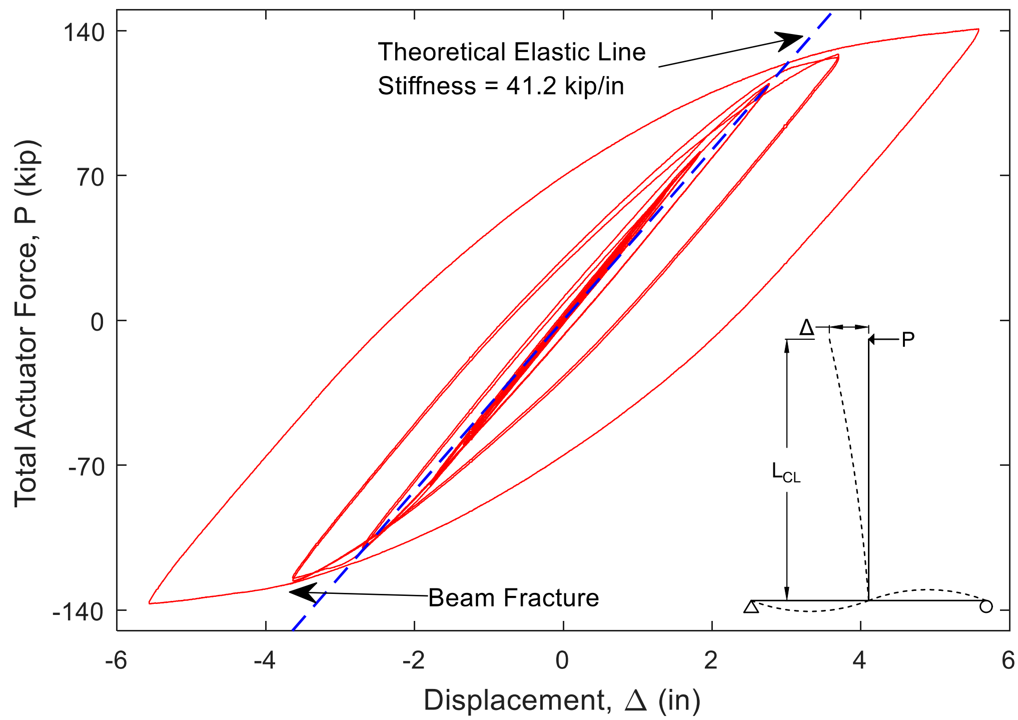


Figure V-1 Total Actuator Force vs. Applied Displacement at End of Beam

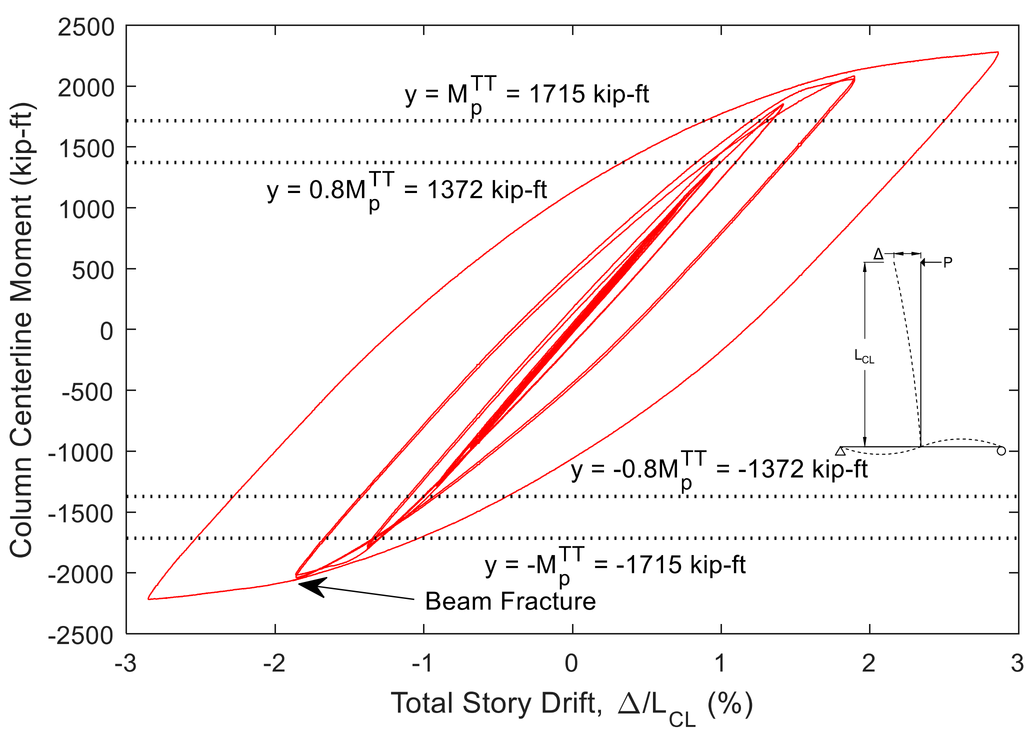


Figure V-2 Column Centerline Moment vs. Total Story Drift

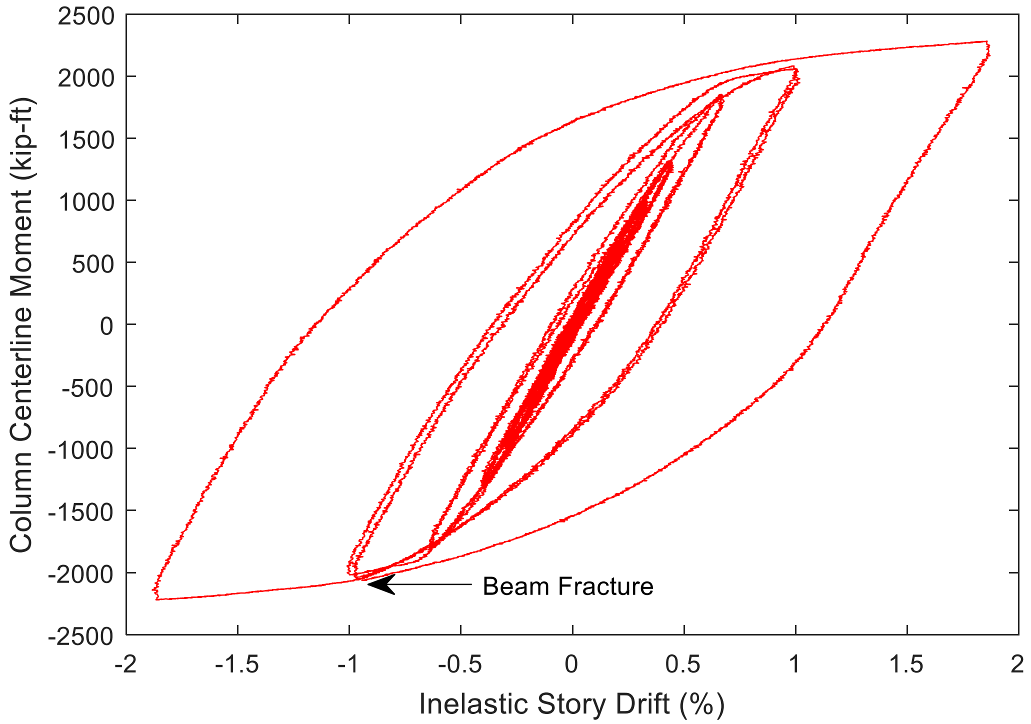


Figure V-3 Column Centerline Moment vs. Inelastic Portion of Story Drift

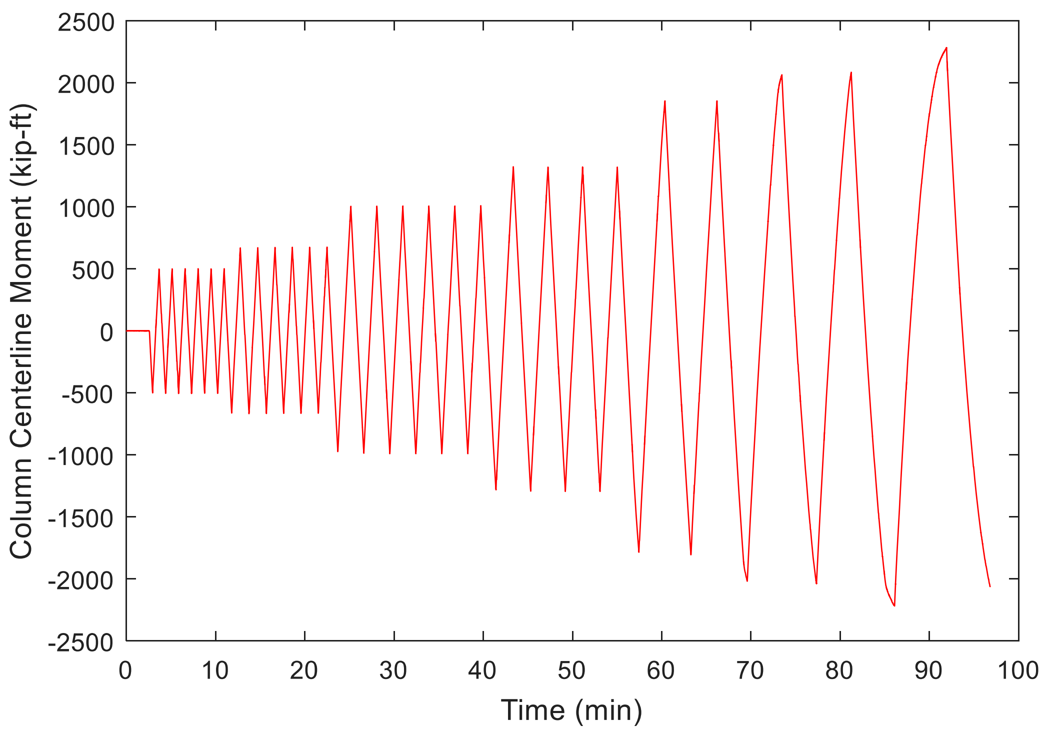


Figure V-4 Column Centerline Moment vs. Time

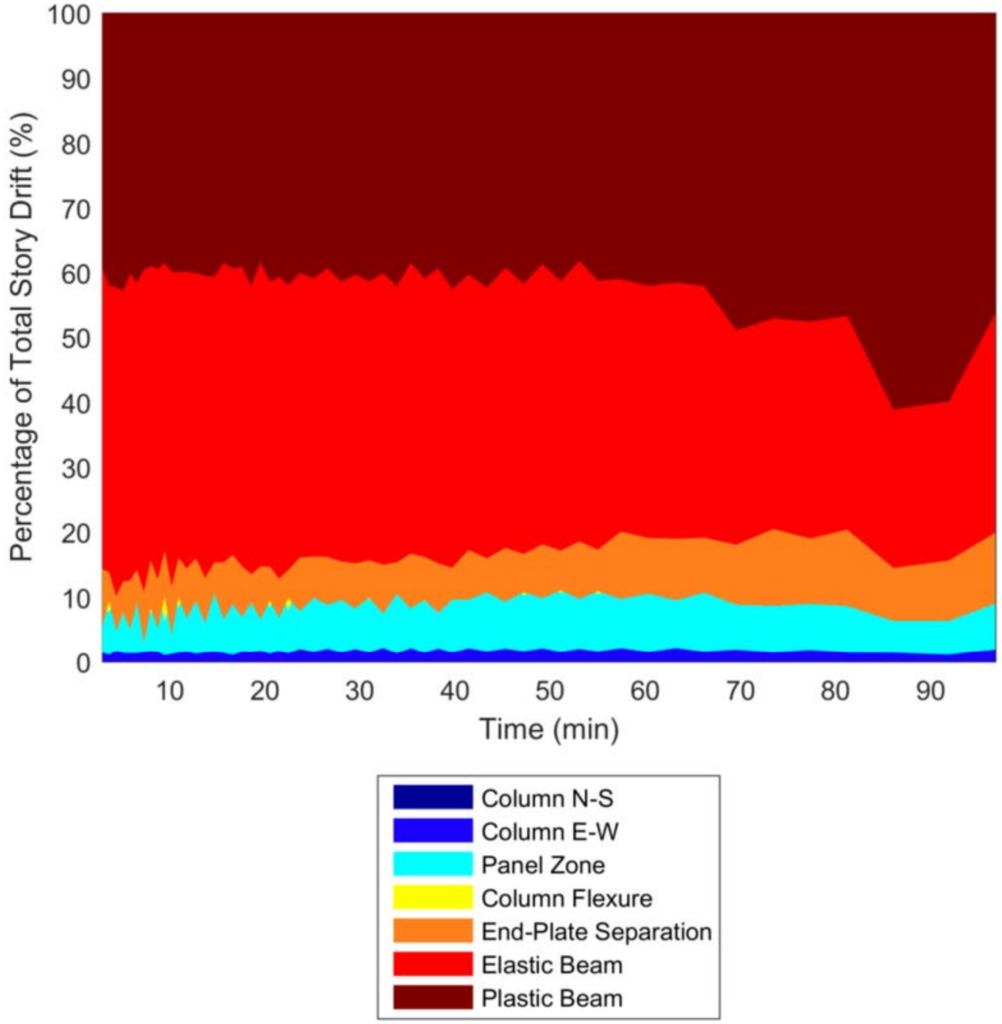


Figure V-5 Story Drift Components Percentage of Total Story Drift vs. Time

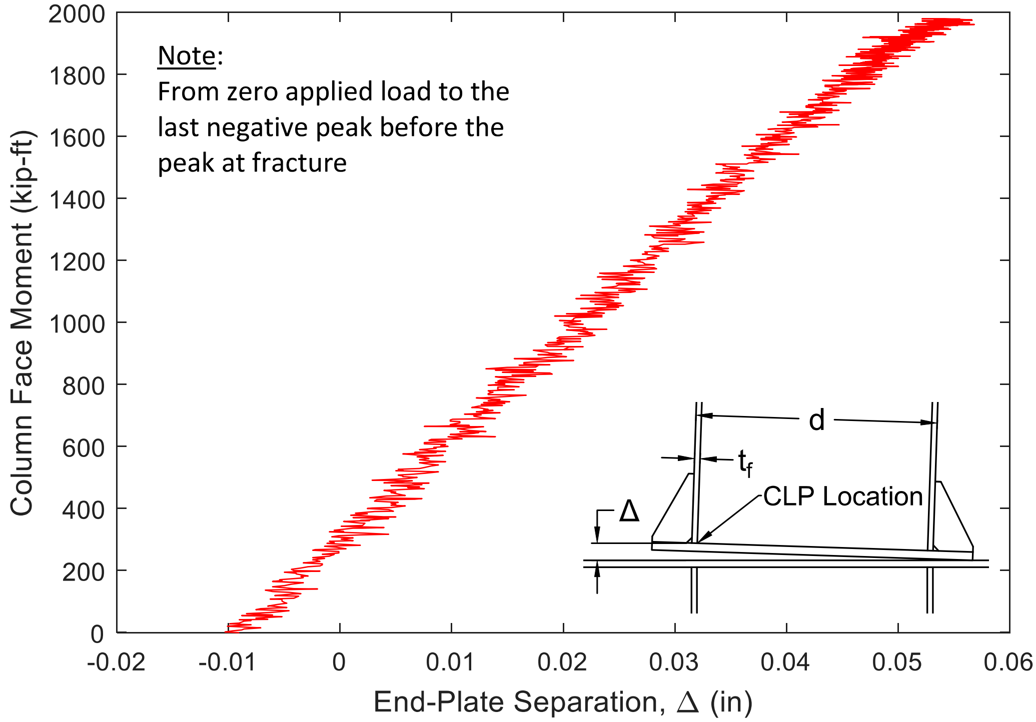


Figure V-6 Column Face Moment vs. End-Plate Separation

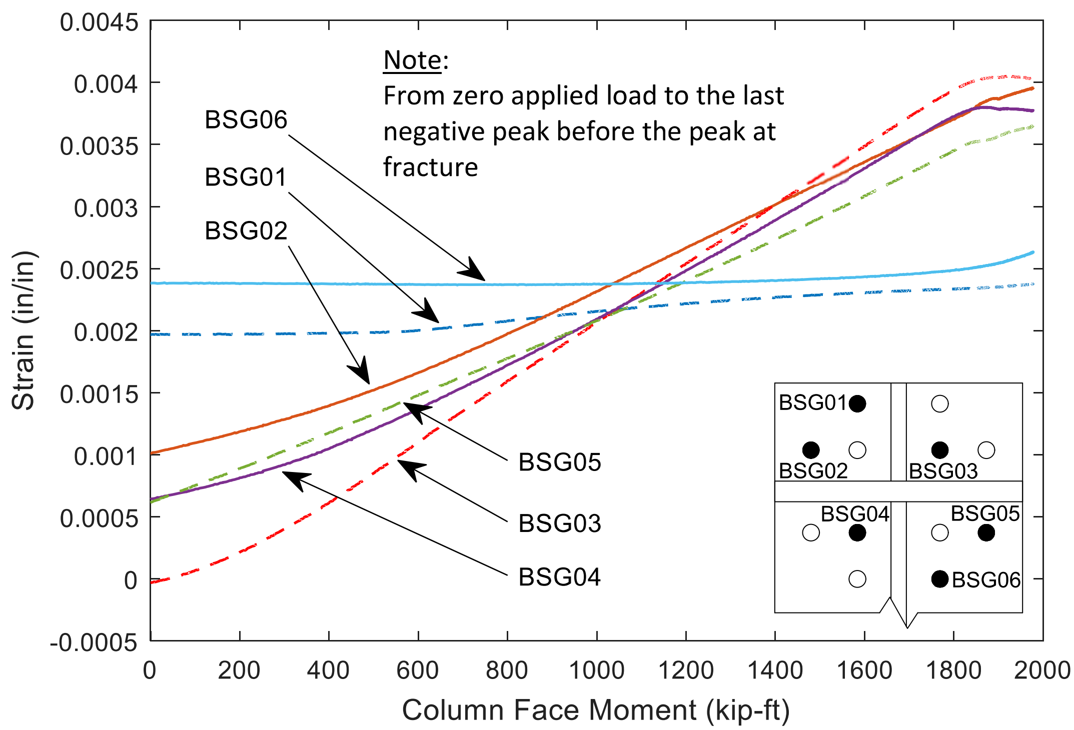


Figure V-7 Strain in Bolts vs. Column Face Moment

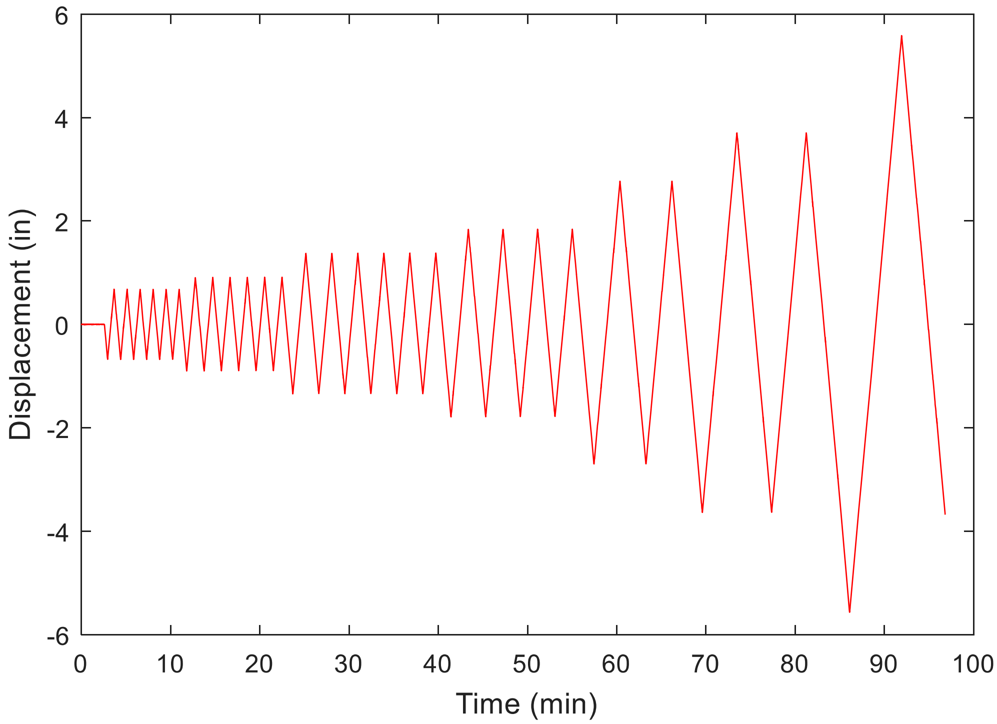


Figure V-8 Displacement at End of Beam vs. Time

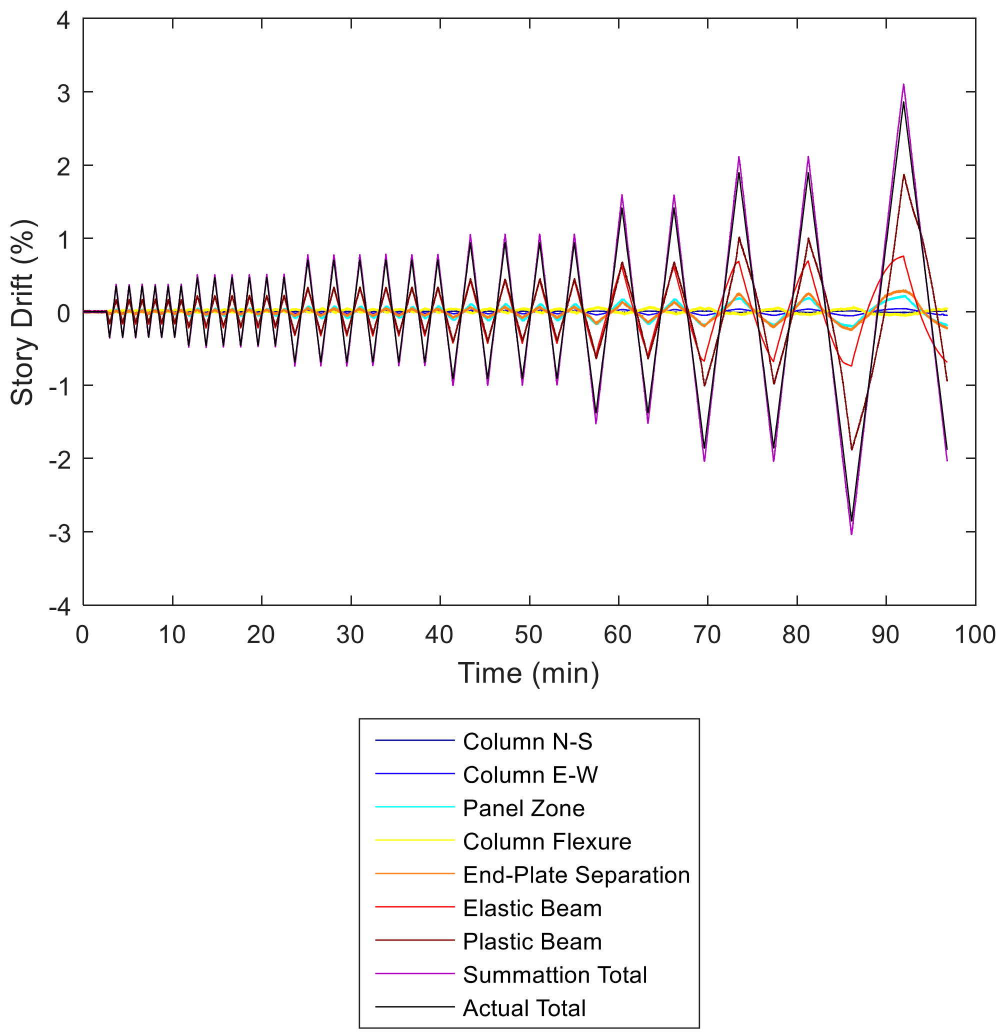


Figure V-9 Story Drift Components vs. Time

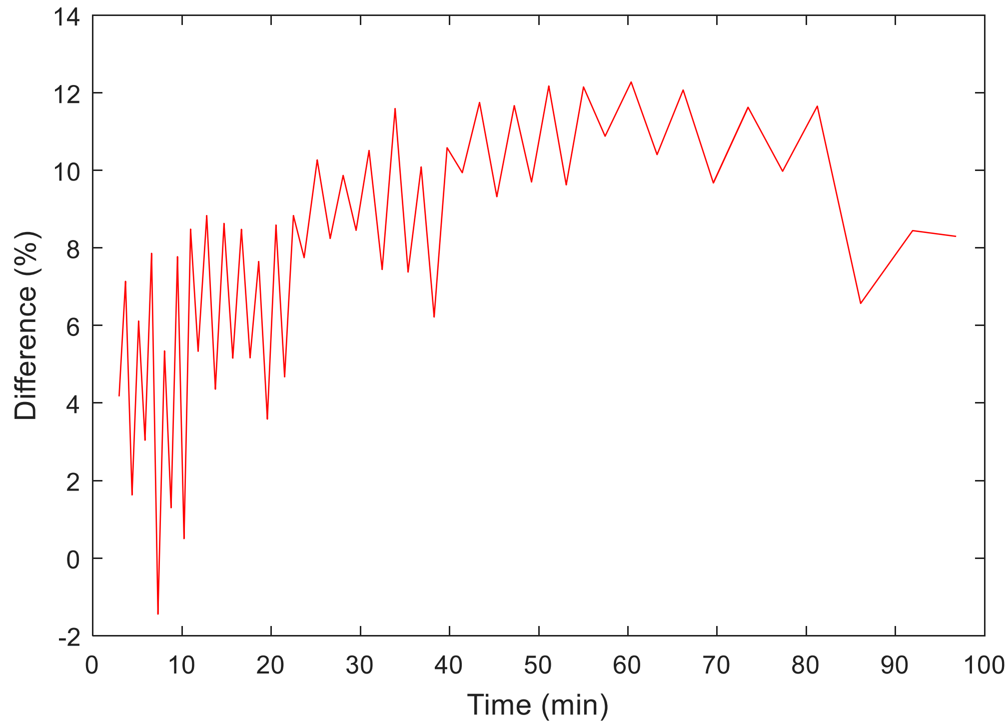


Figure V-10 Percent Difference Between Total Story Drift & Actual Story Drift at Peaks

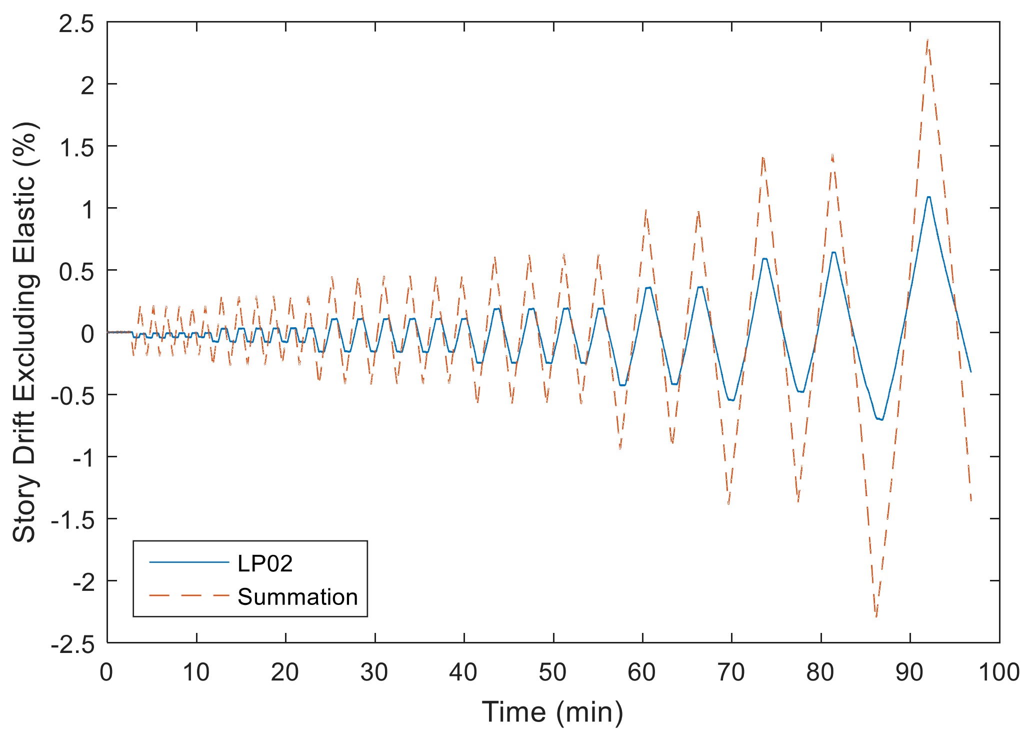


Figure V-11 Large Deformations Check

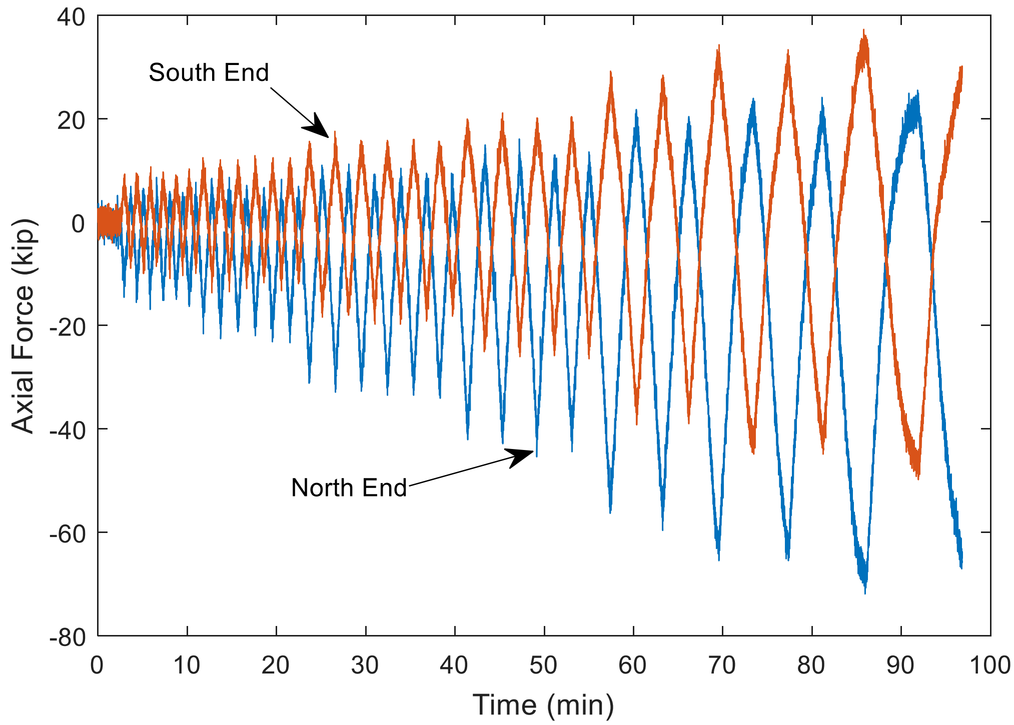


Figure V-12 Column Axial Force Calculated From Strain Gauges on Stub Sections

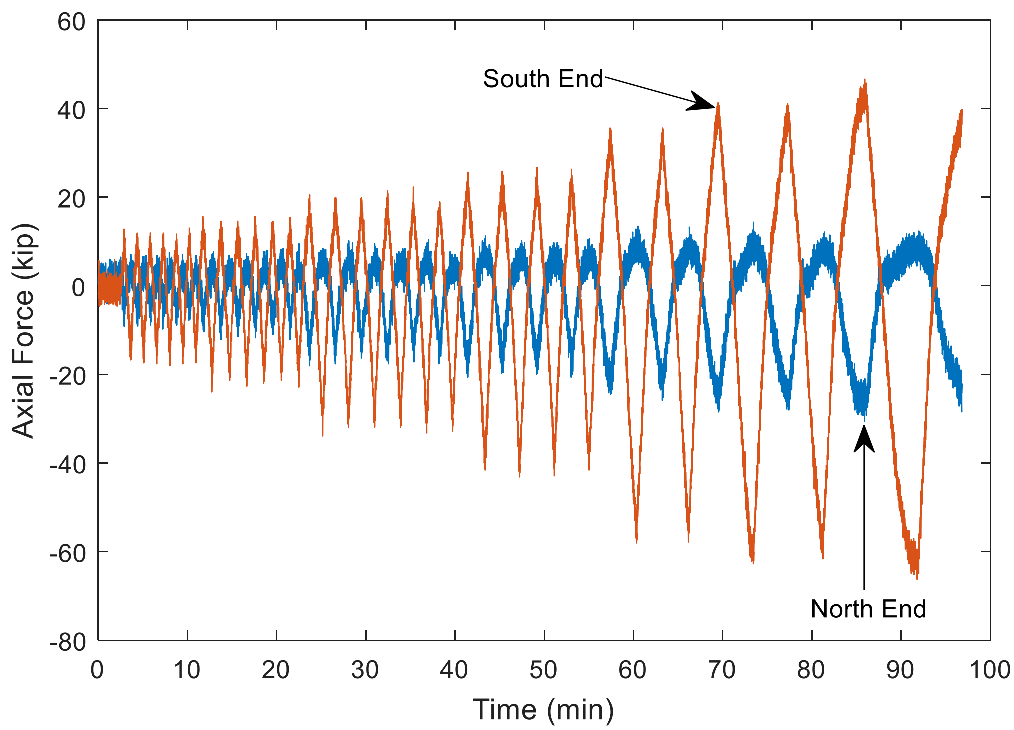


Figure V-13 Column Axial Force Calculated From Strain Gauges on Column

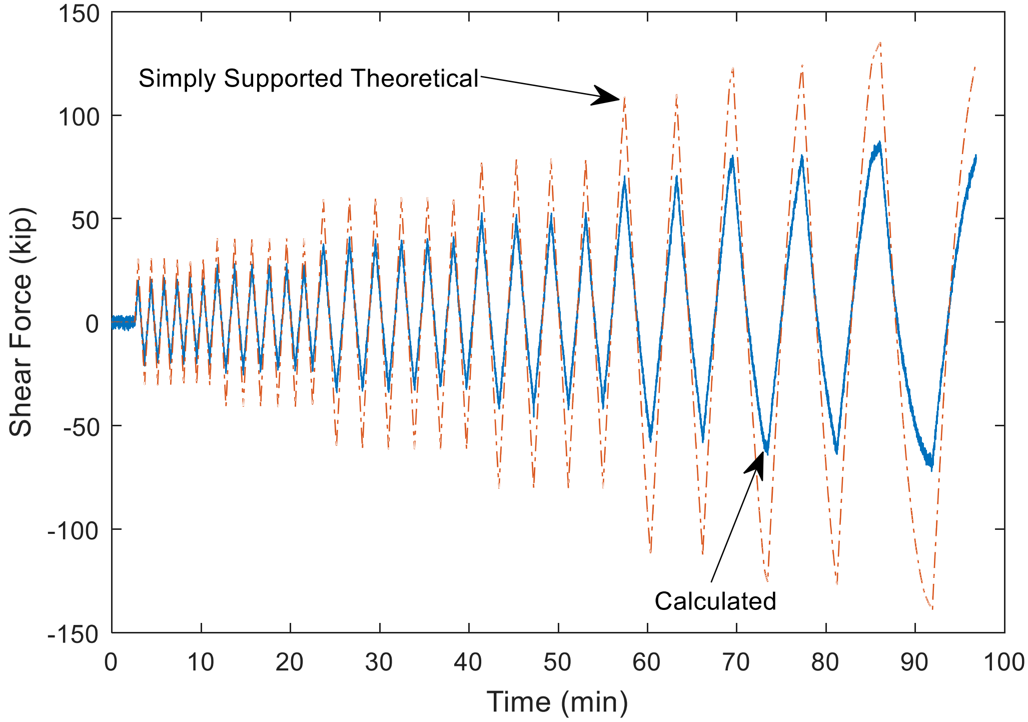


Figure V-14 Column Shear Force – North End

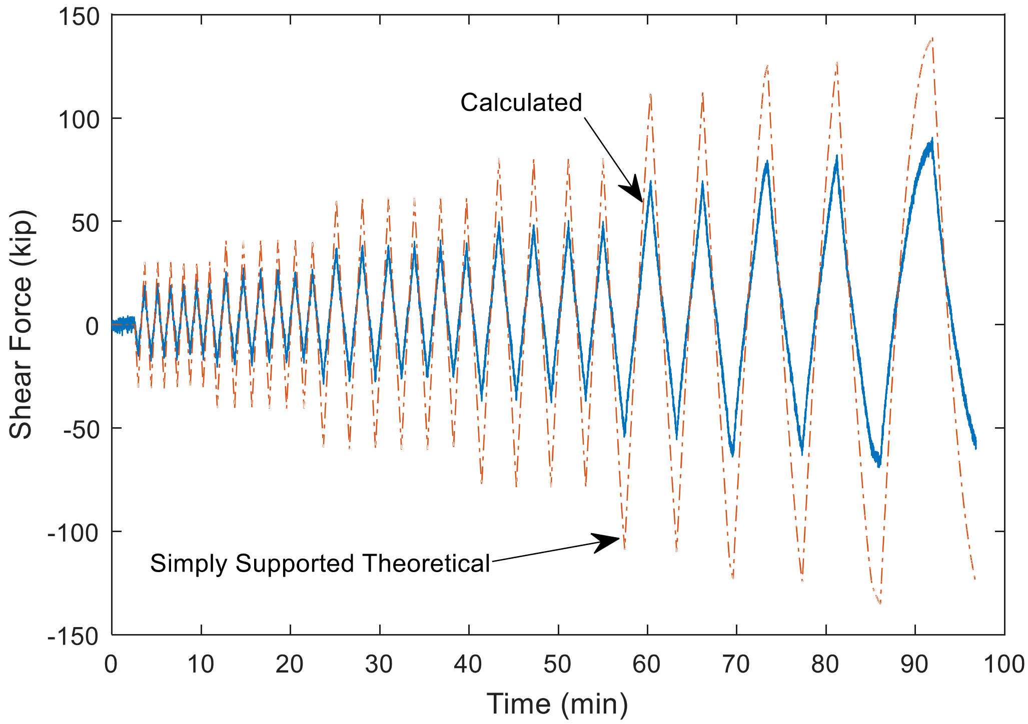


Figure V-15 Column Shear Force – South End

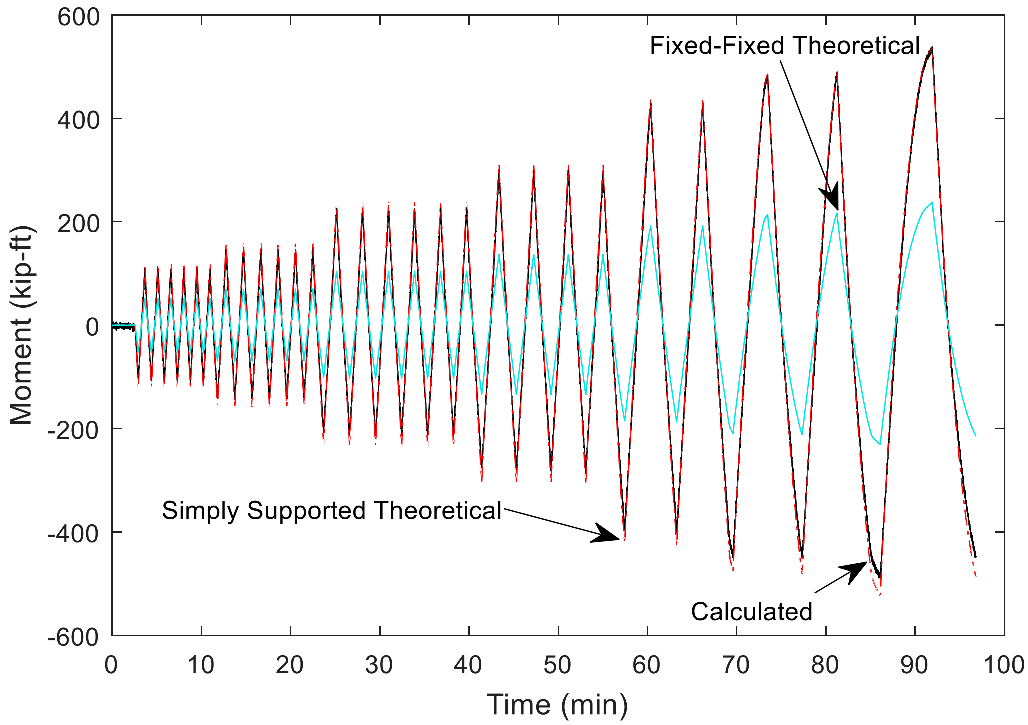


Figure V-16 Moment in Column at Location of Strain Gauges – North End

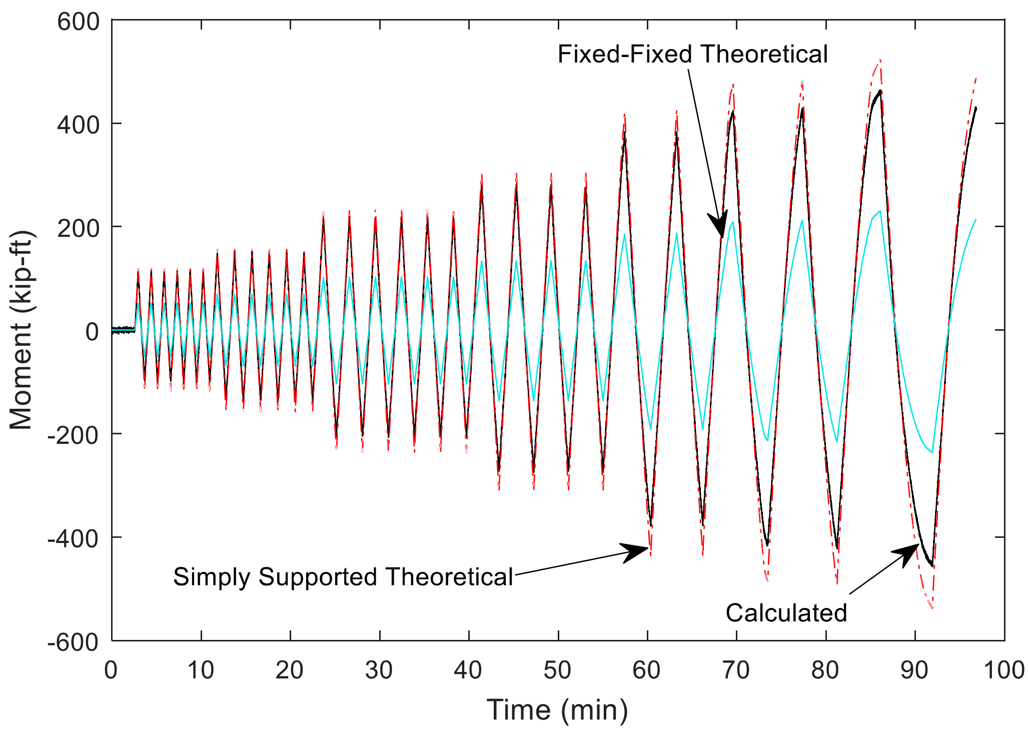


Figure V-17 Moment in Column at Location of Strain Gauges – South End

Instrumentation Data vs. Time

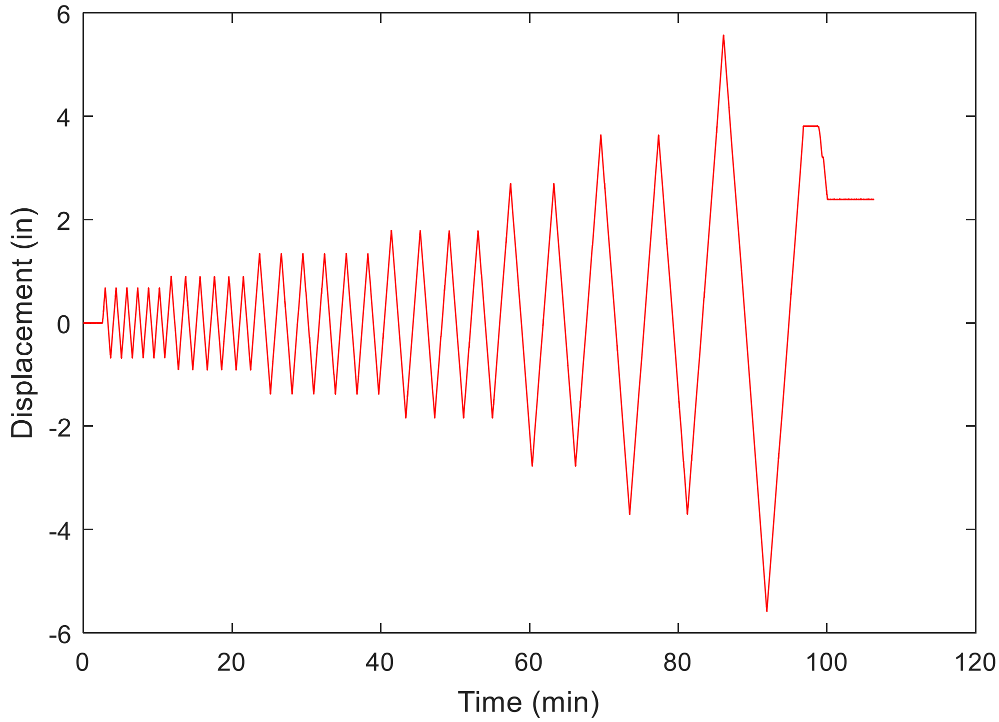


Figure V-18 SP01

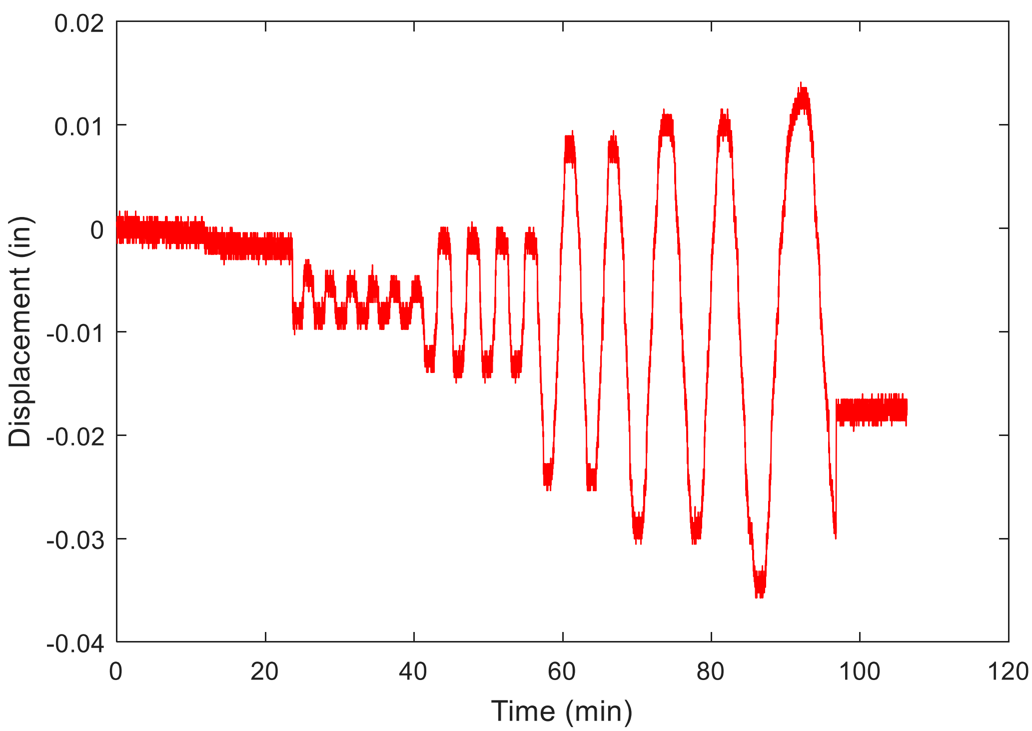


Figure V-19 SP02

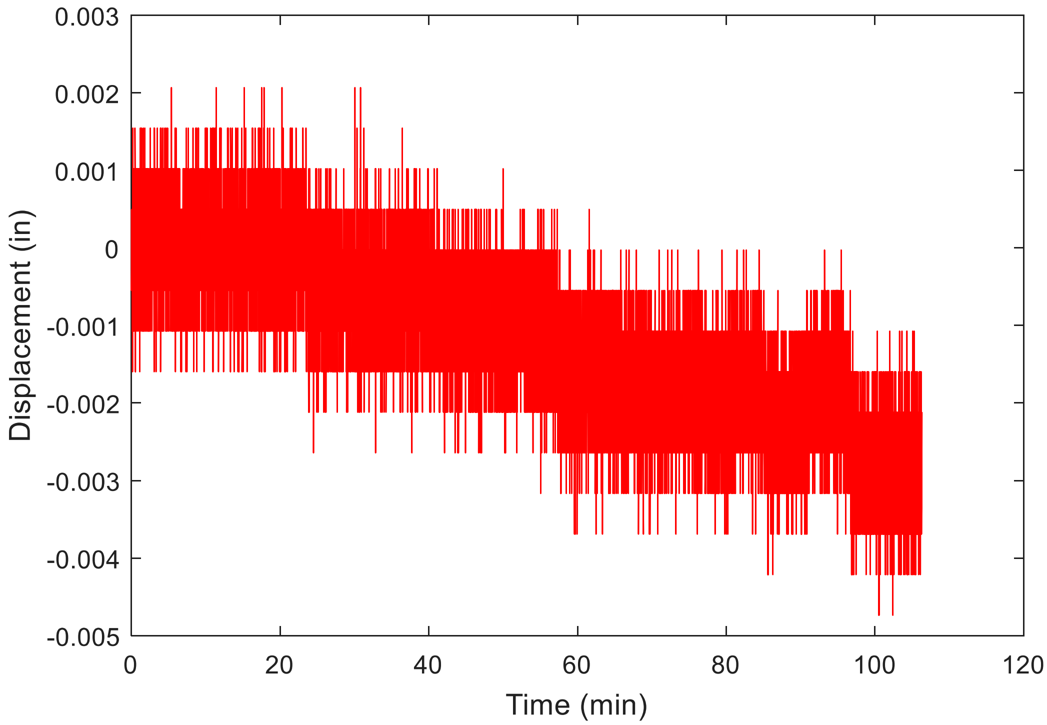


Figure V-20 SP03

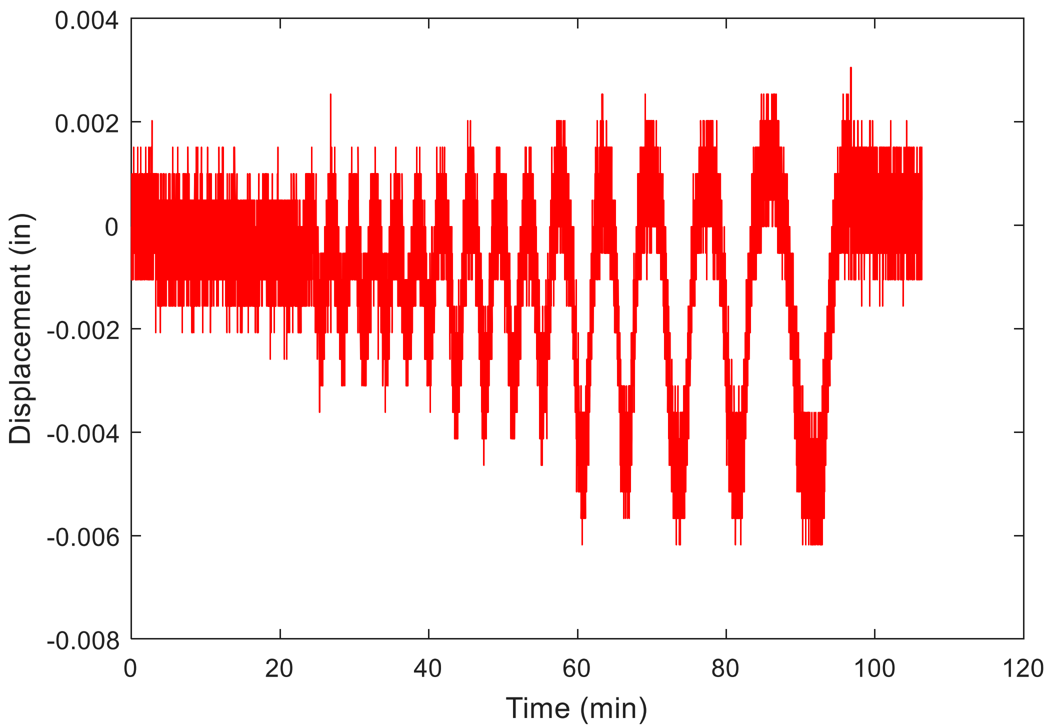


Figure V-21 SP04

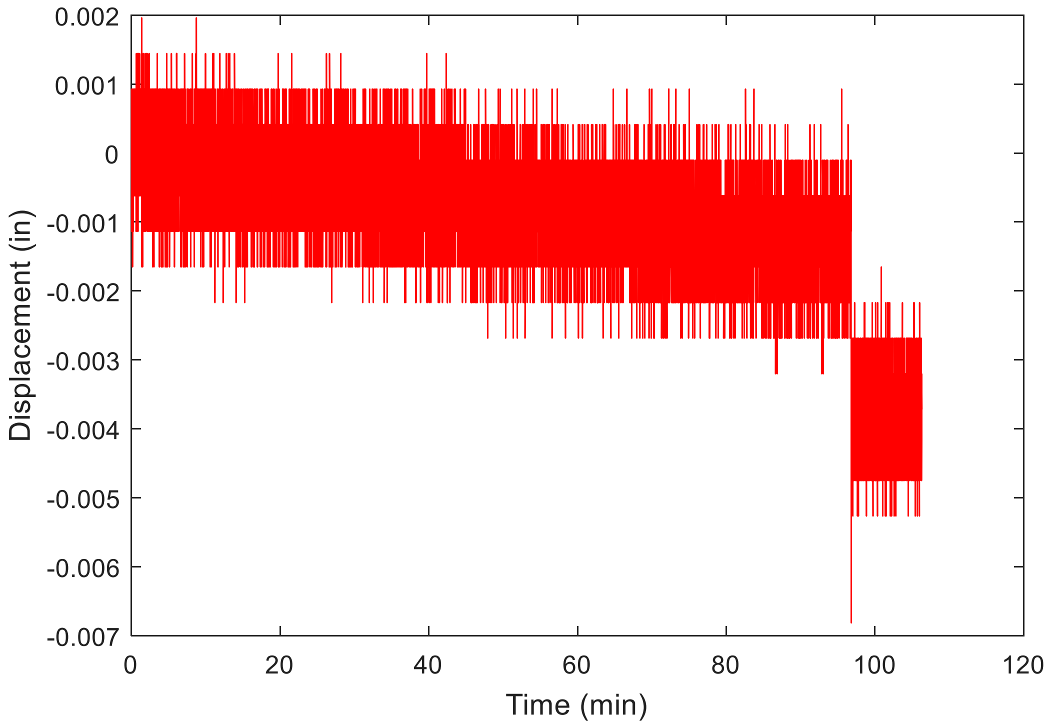


Figure V-22 SP05

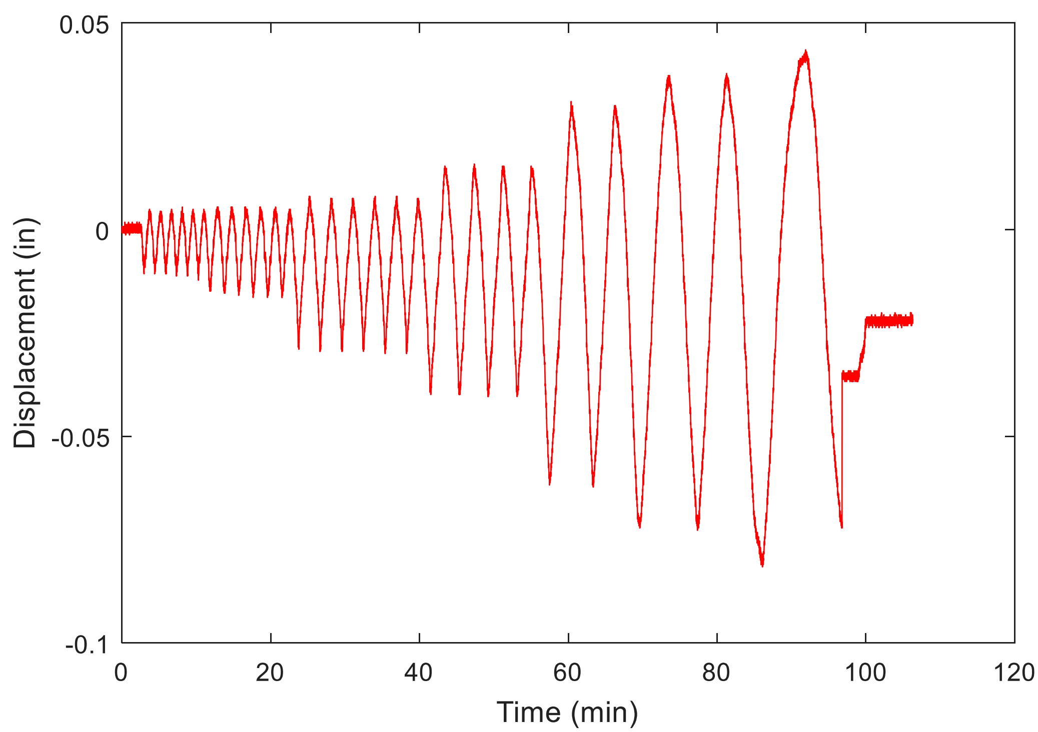


Figure V-23 SP06

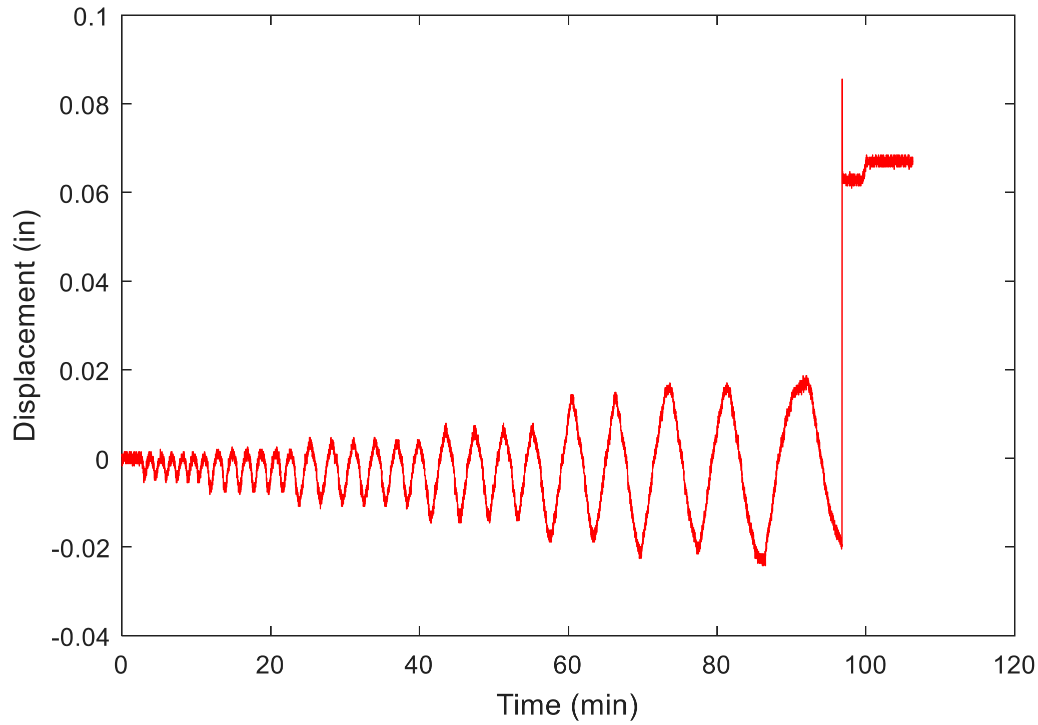


Figure V-24 SP07

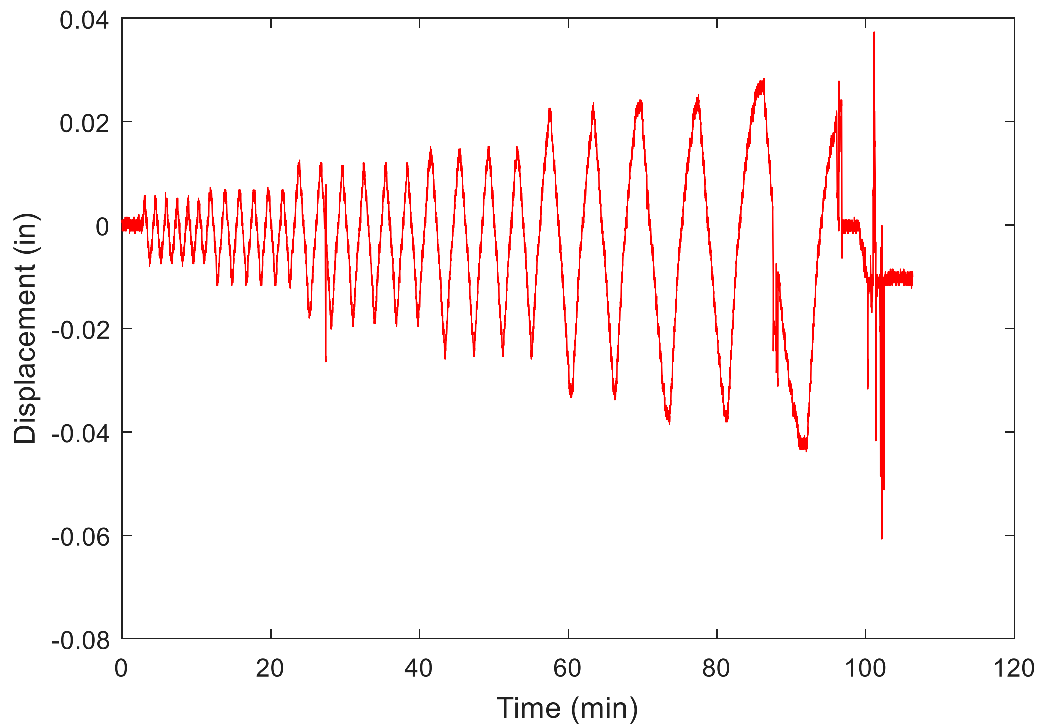


Figure V-25 SP08

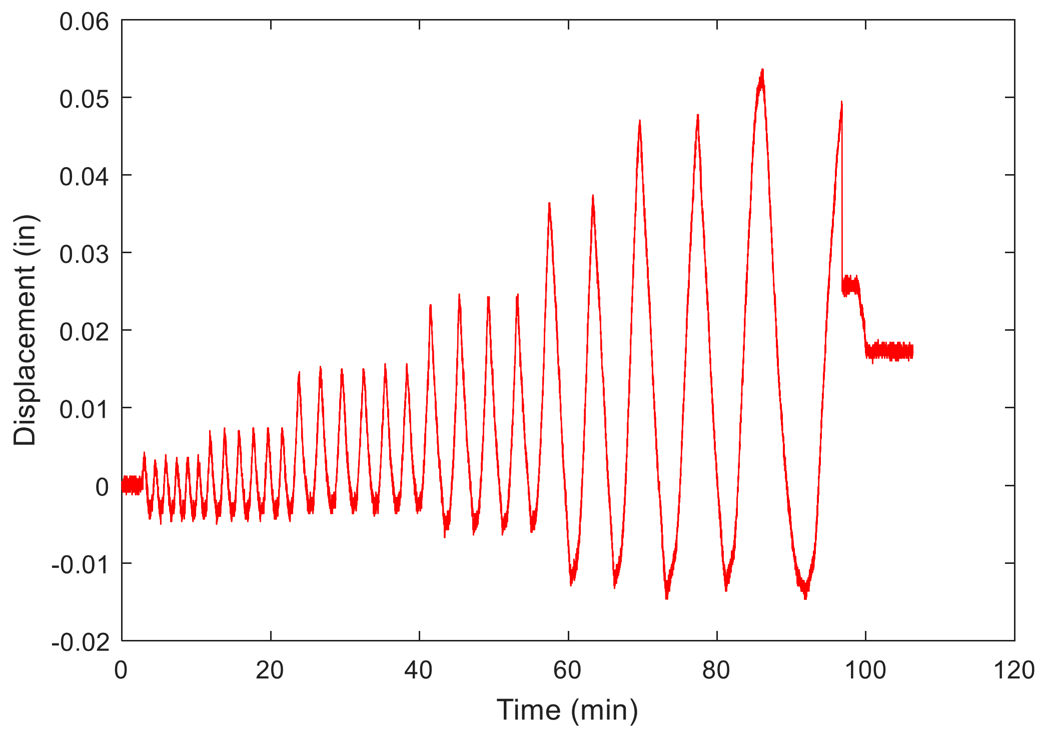


Figure V-26 SP09

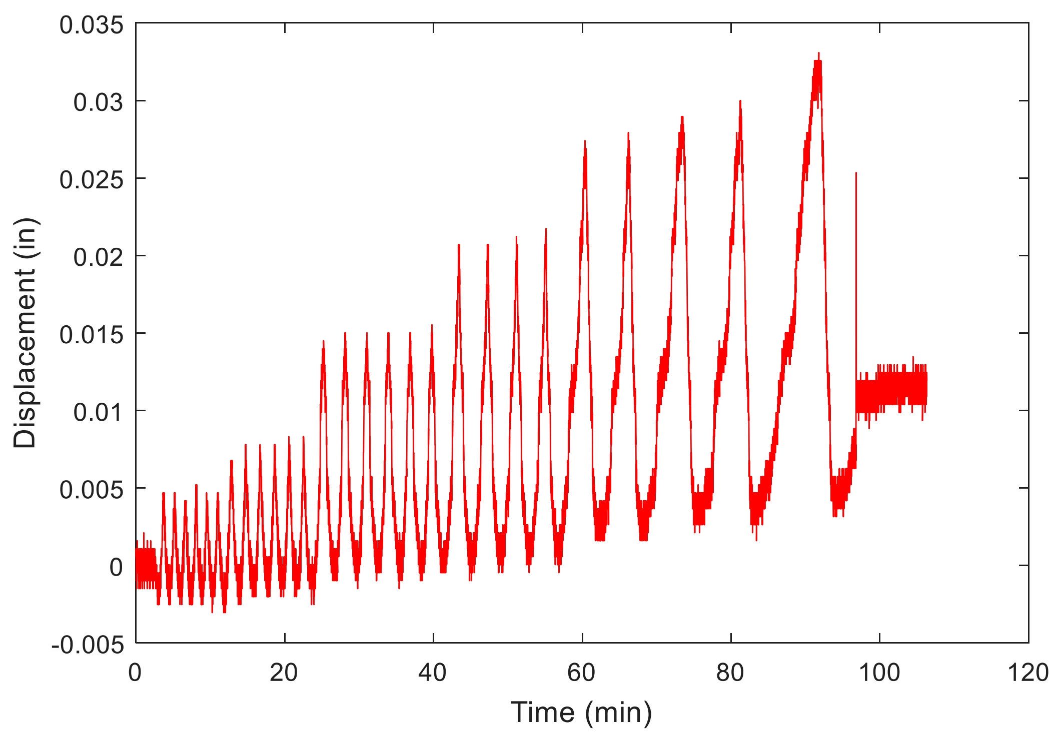


Figure V-27 SP10

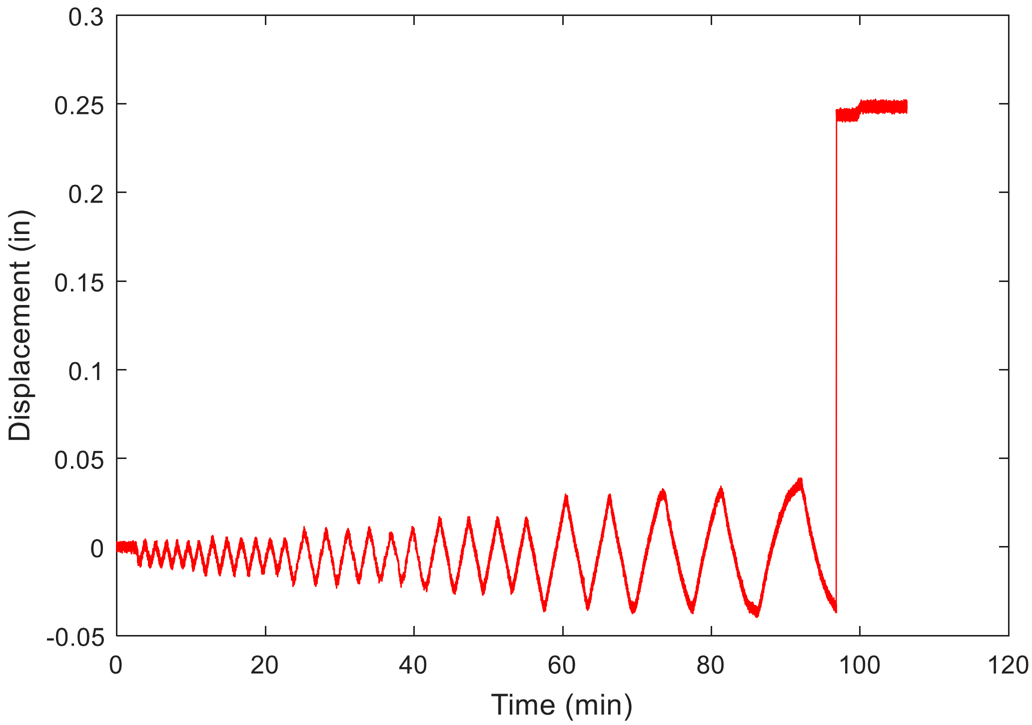


Figure V-28 SP11

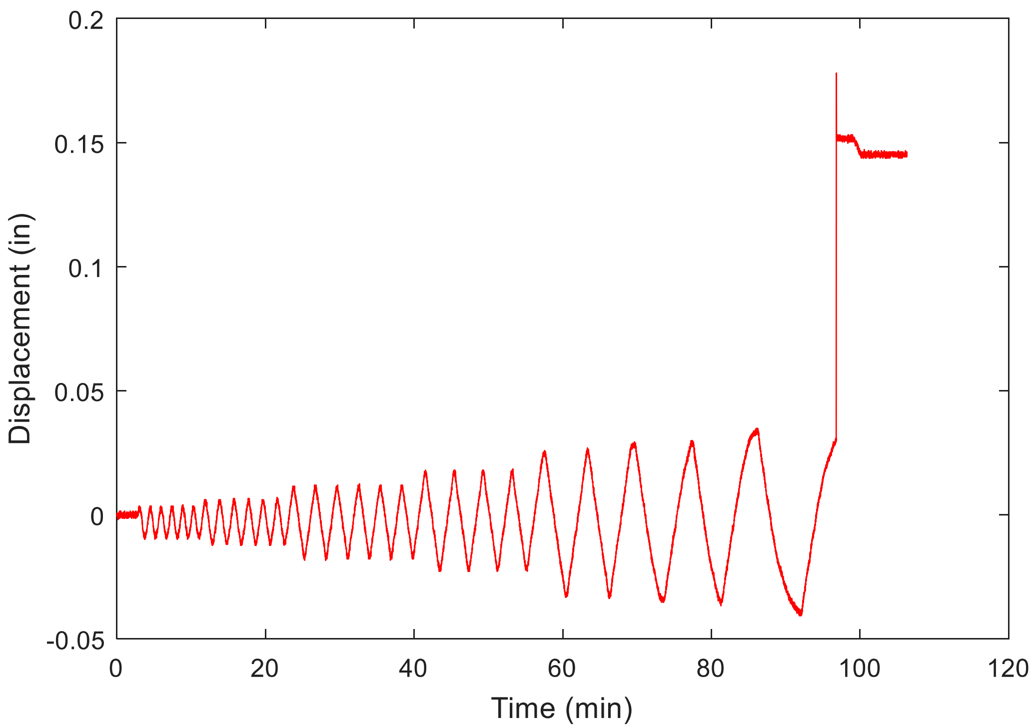


Figure V-29 SP12

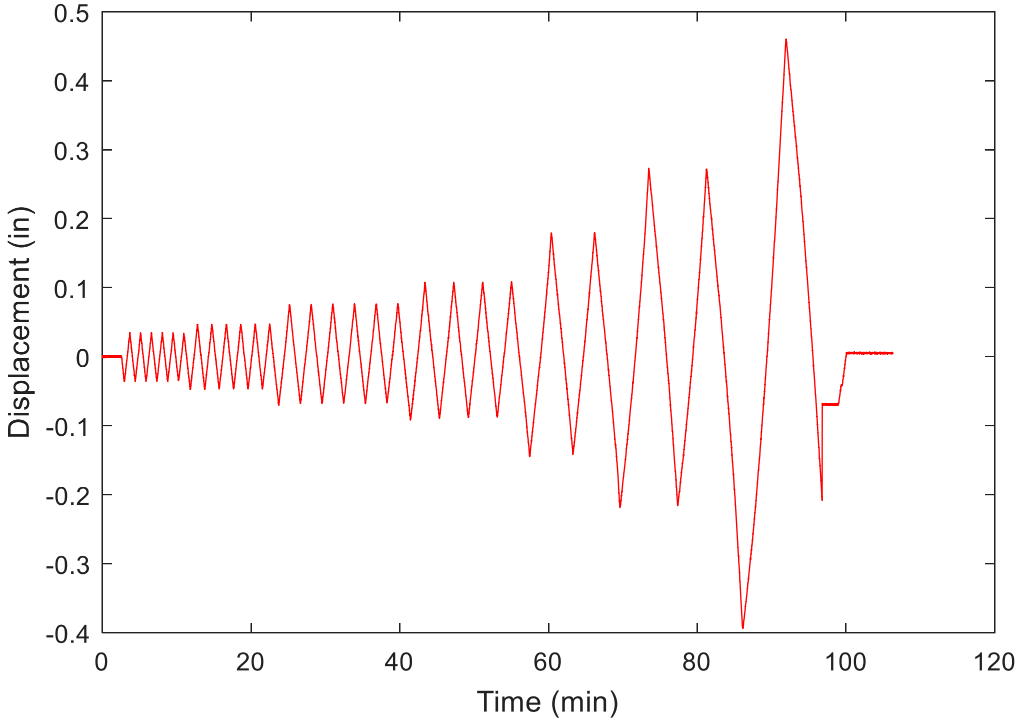


Figure V-30 SP13

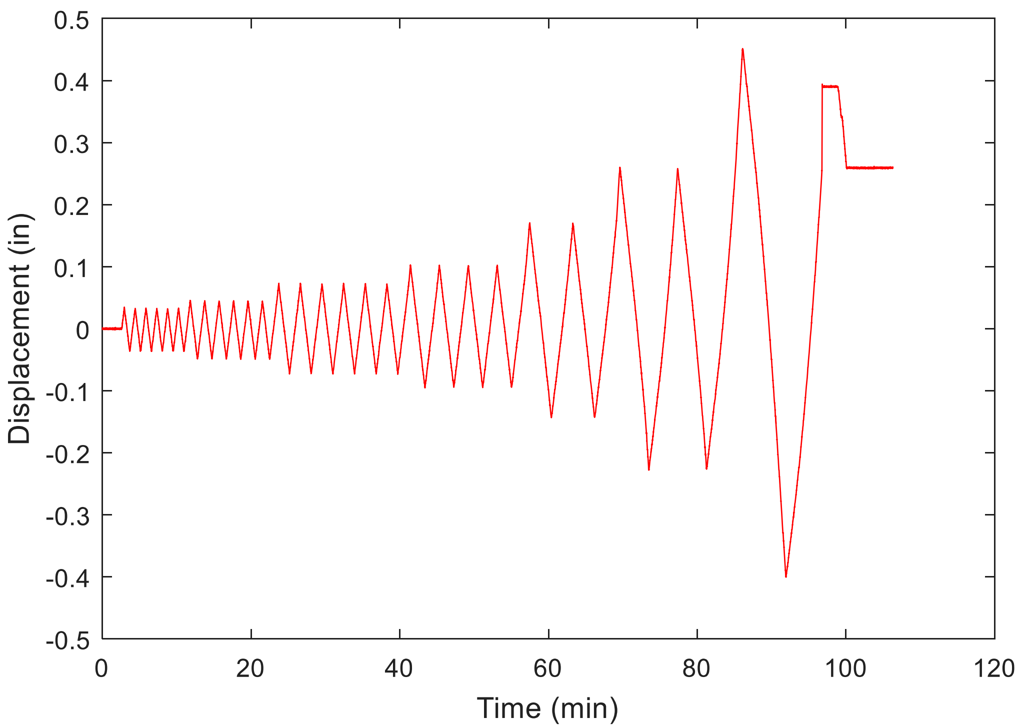


Figure V-31 SP14

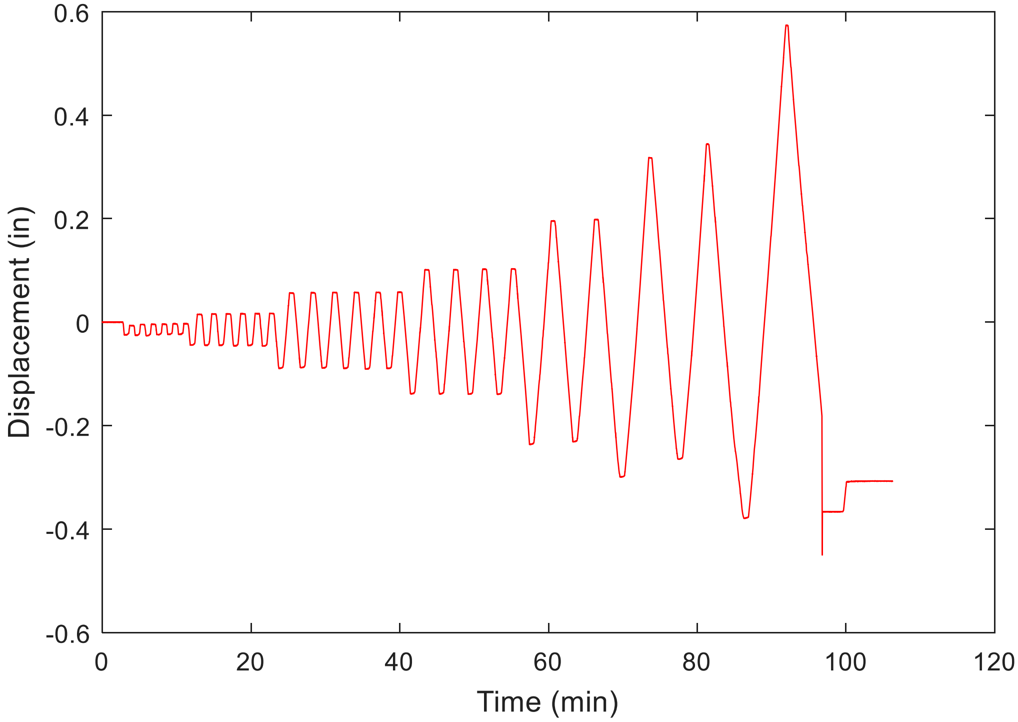


Figure V-32 LP02

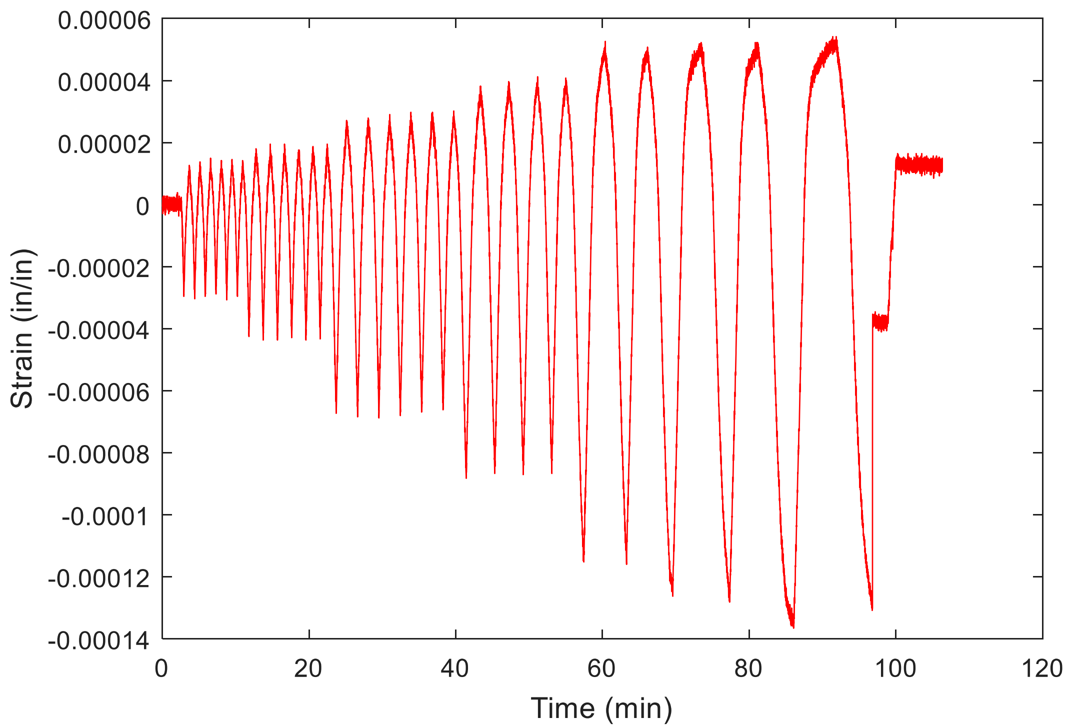


Figure V-33 SG01

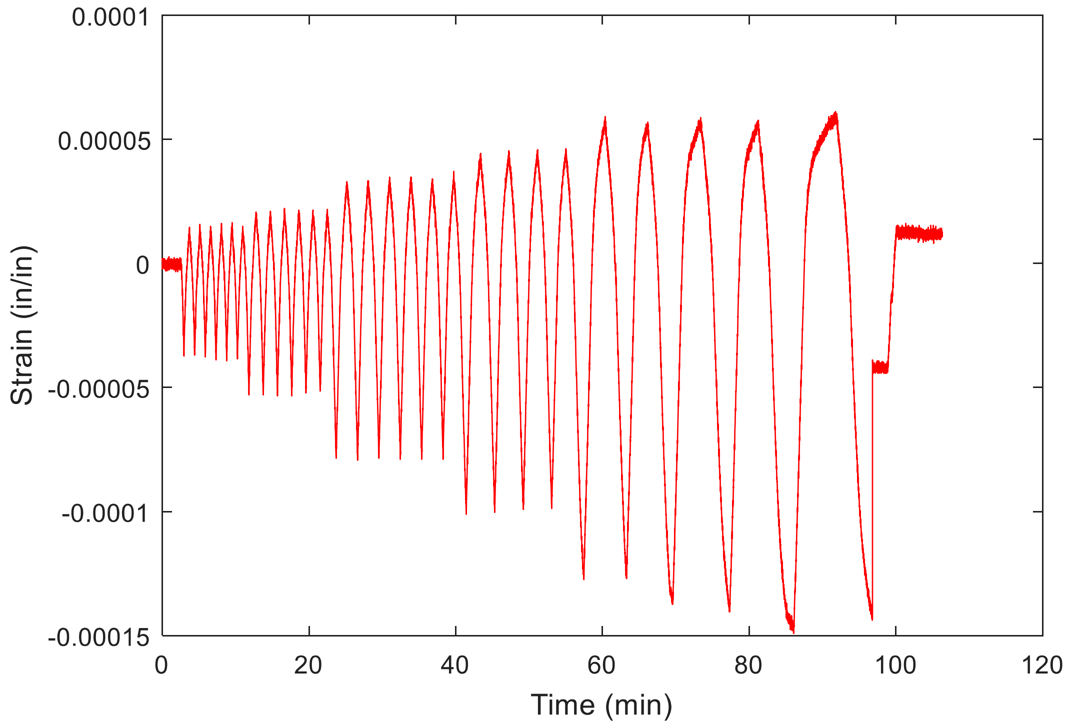


Figure V-34 SG02

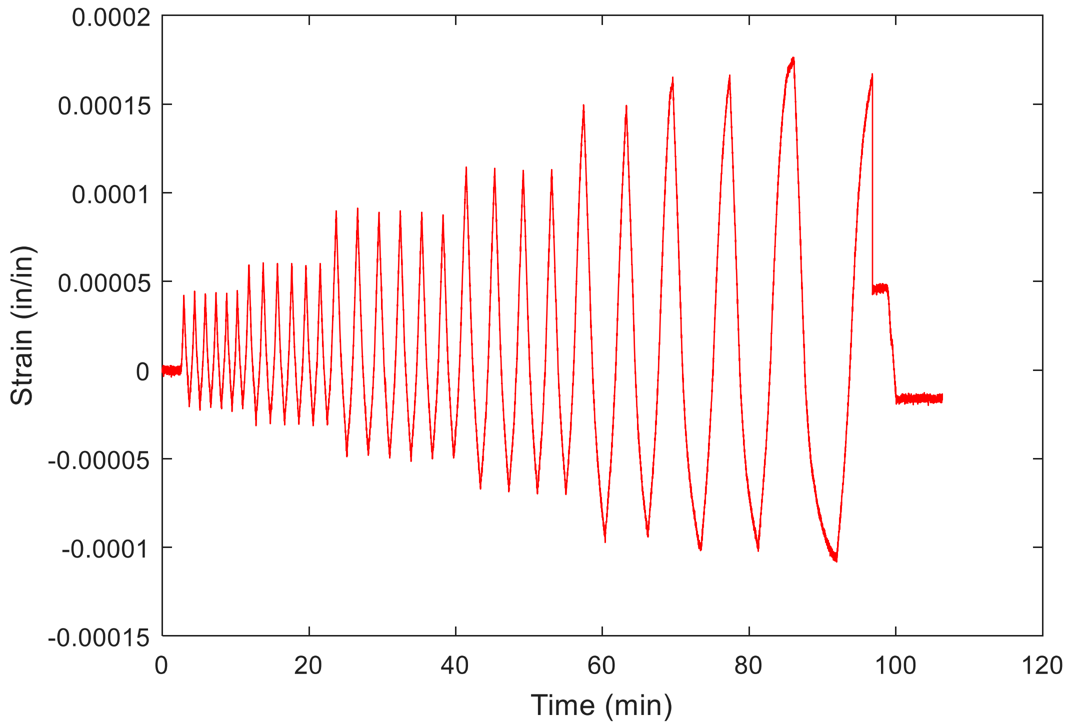


Figure V-35 SG03

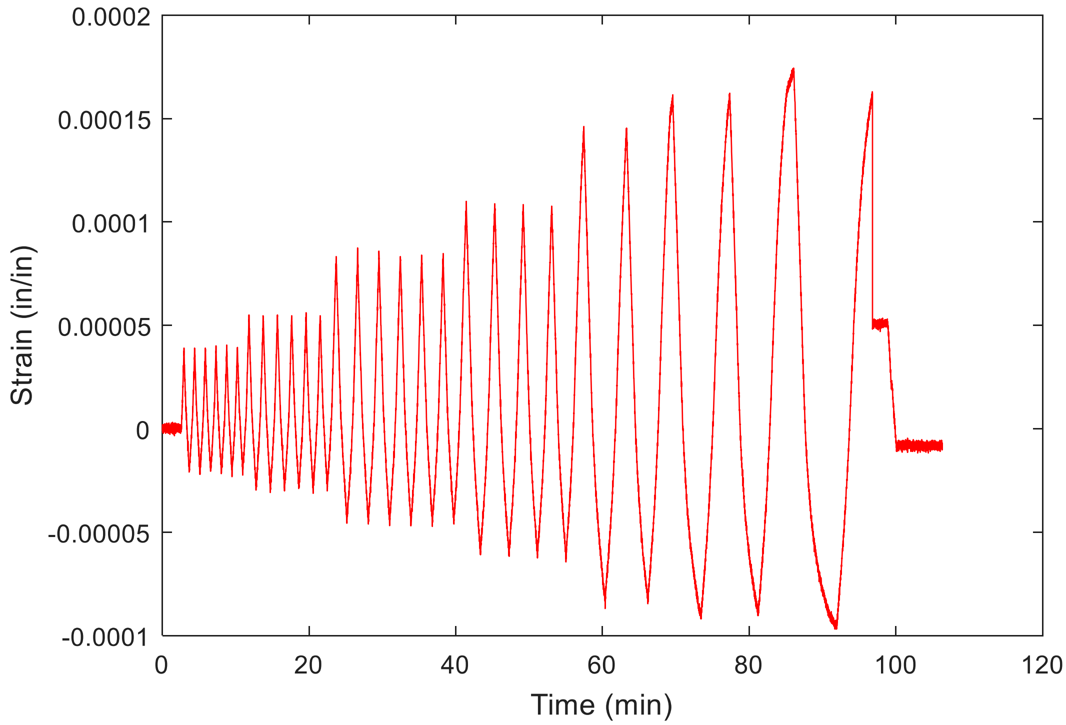


Figure V-36 SG04

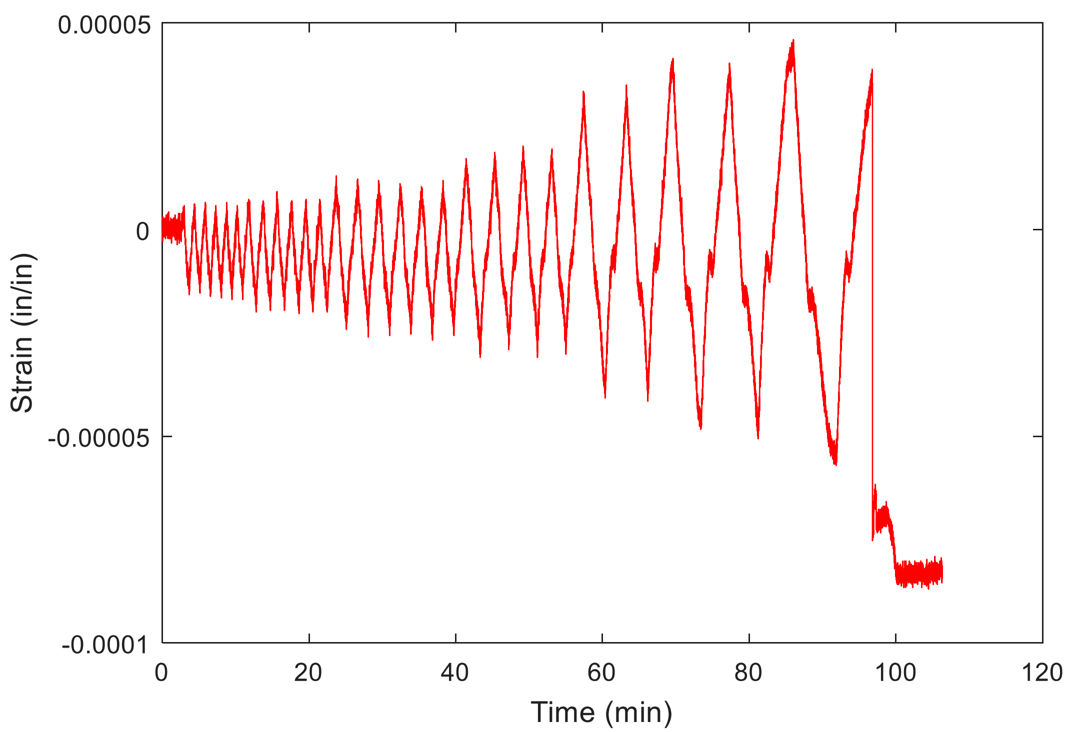


Figure V-37 SG05

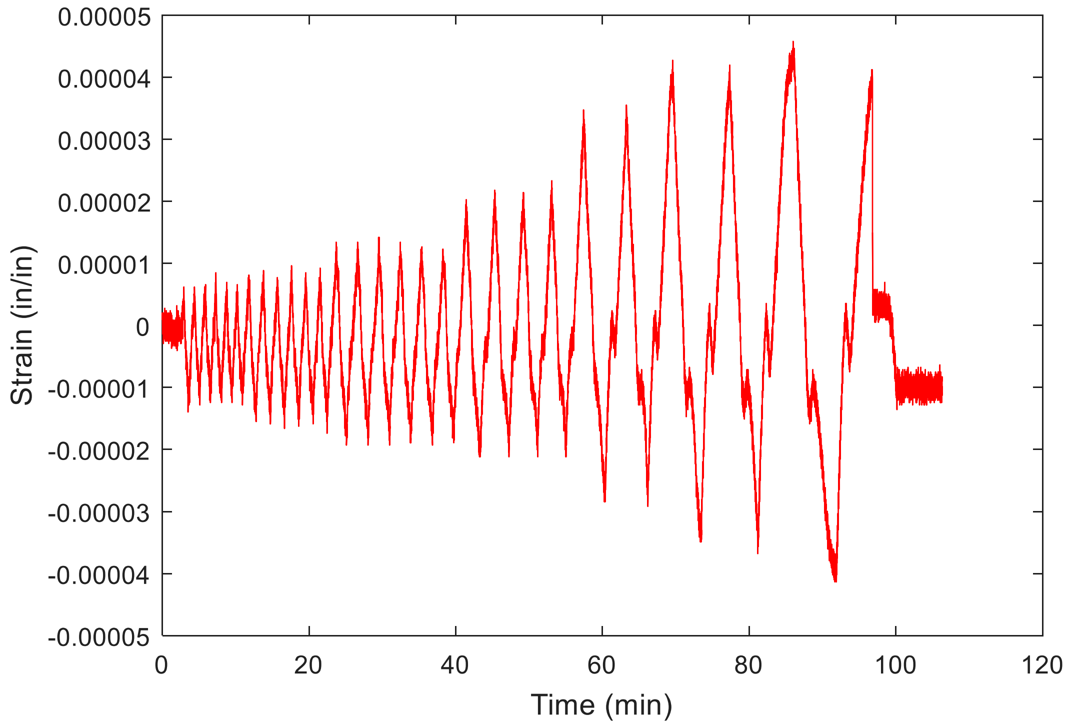


Figure V-38 SG06

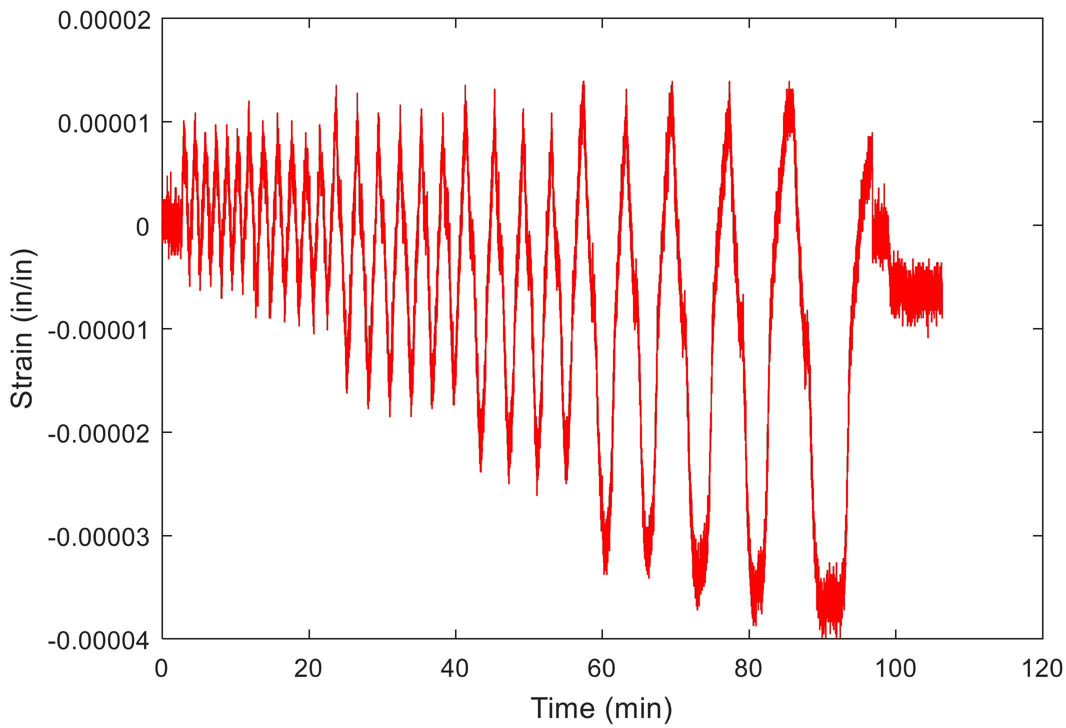


Figure V-39 SG07

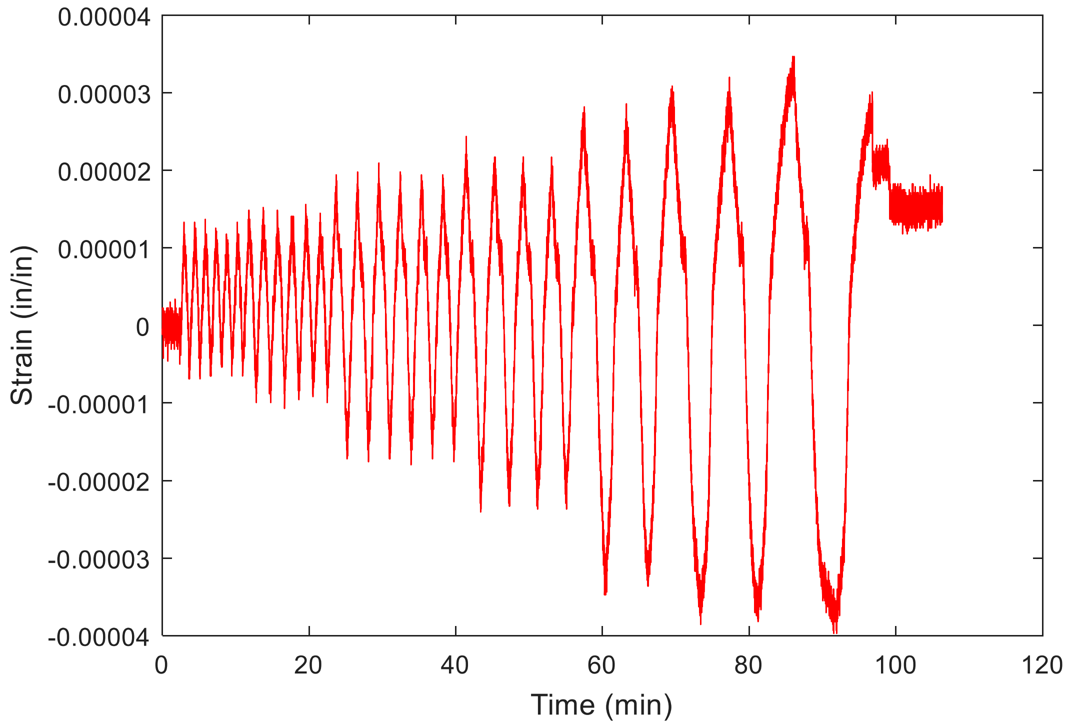


Figure V-40 SG08

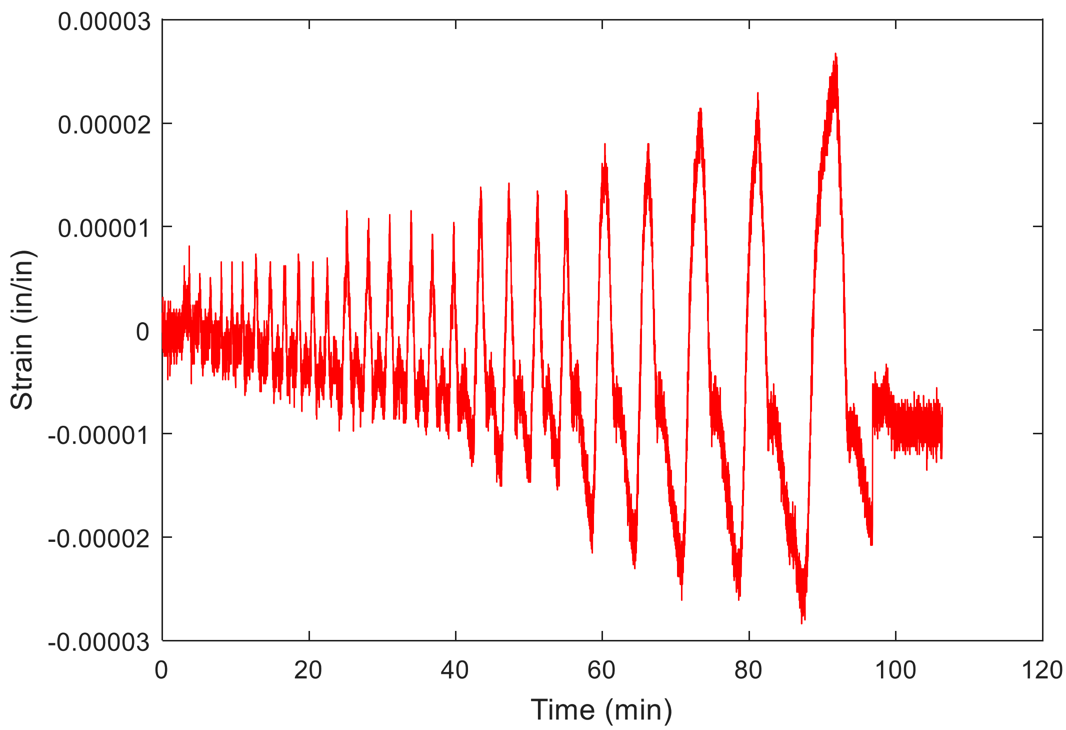


Figure V-41 SG09

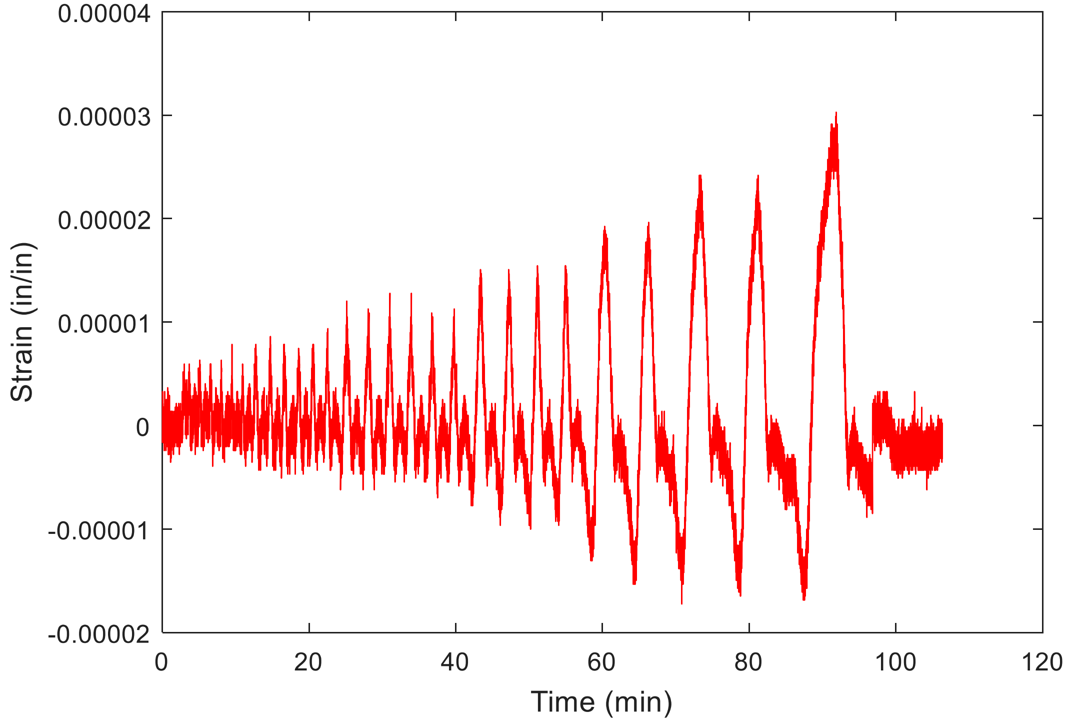


Figure V-42 SG10

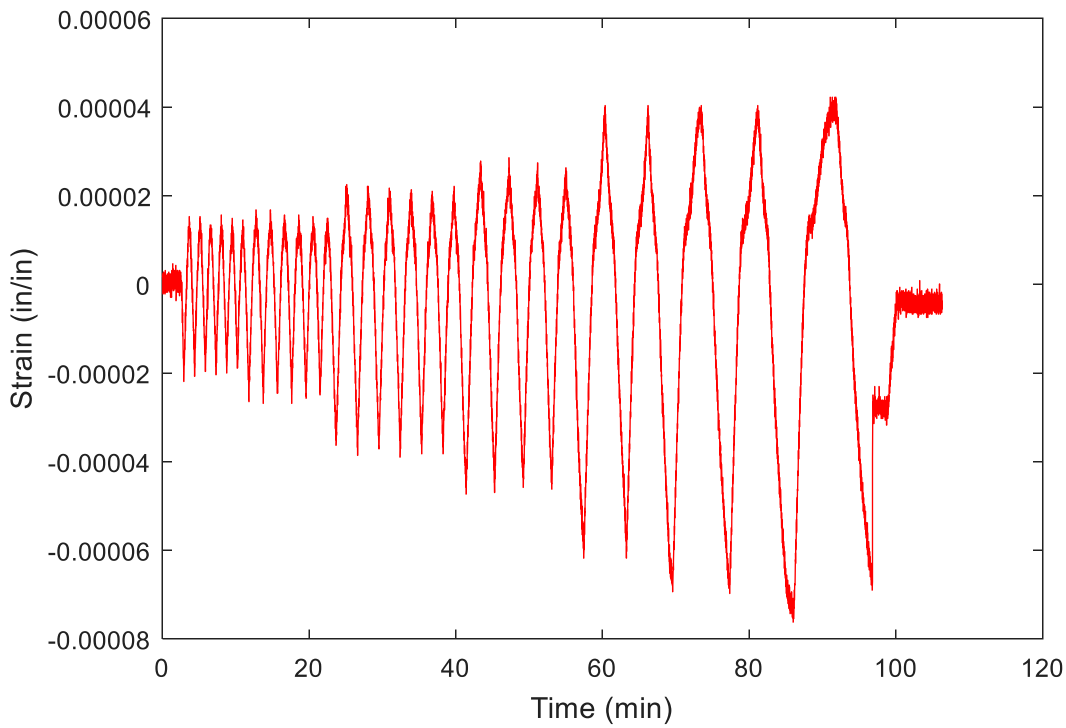


Figure V-43 SG11

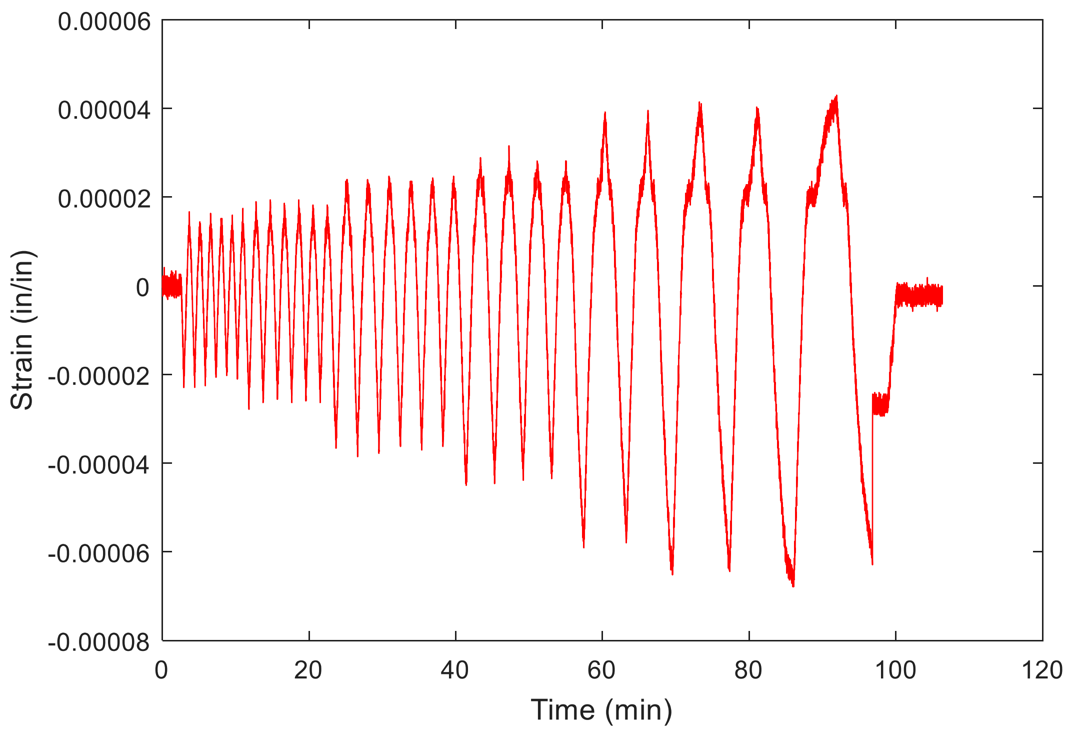


Figure V-44 SG12

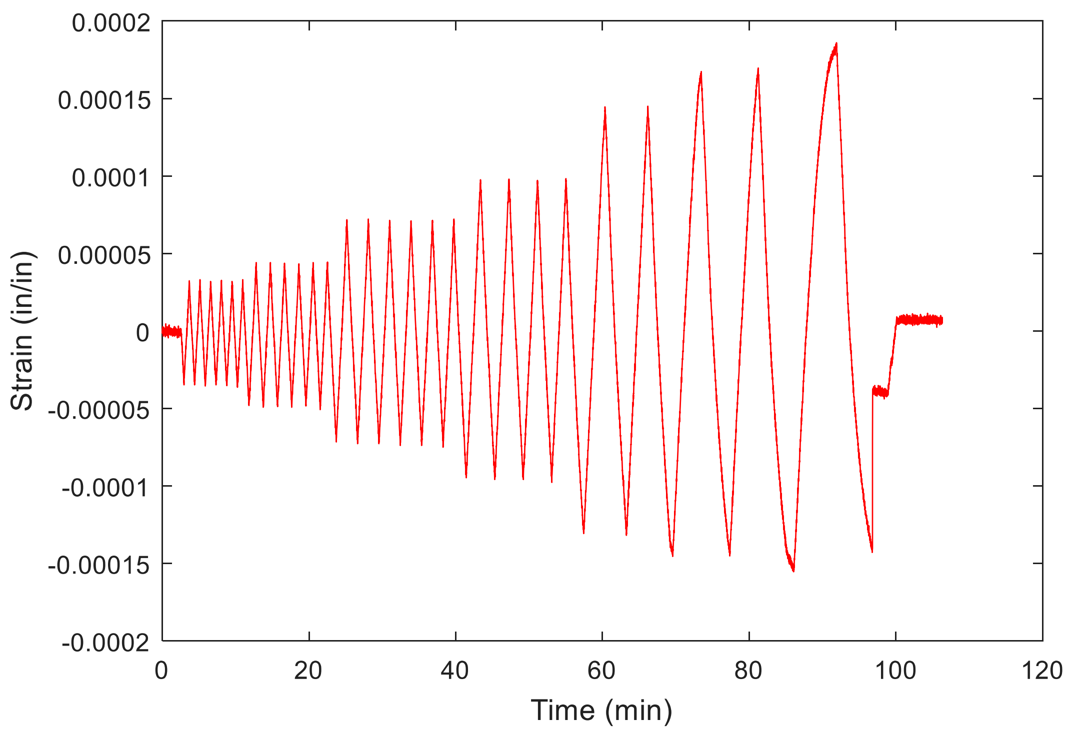


Figure V-45 SG13

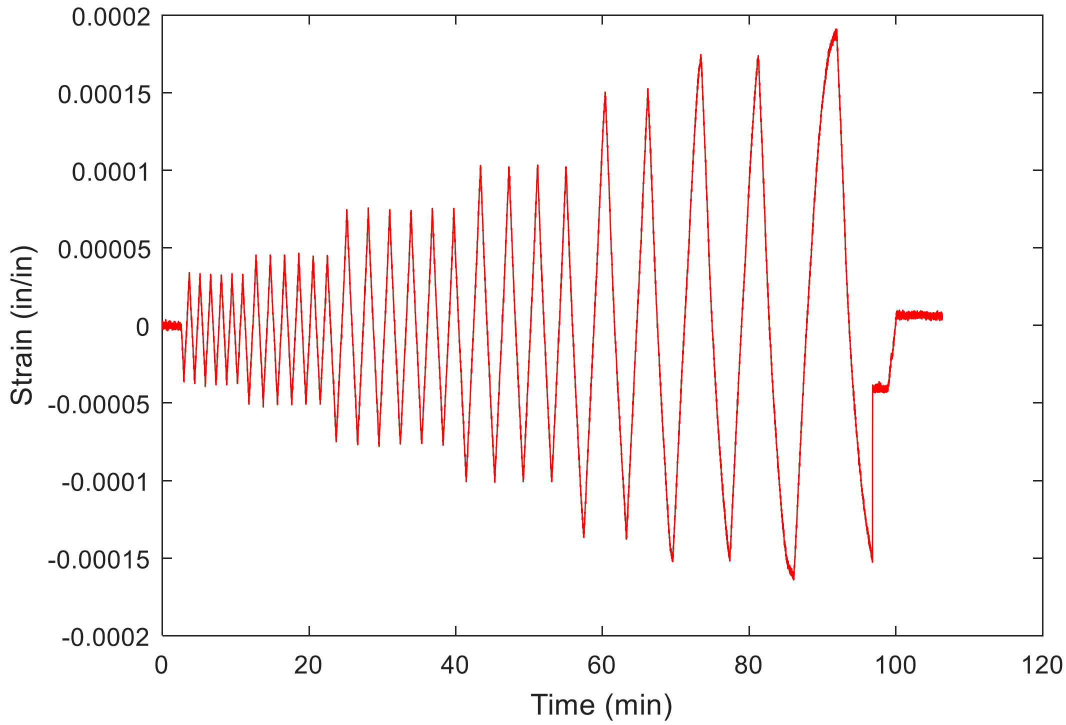


Figure V-46 SG14

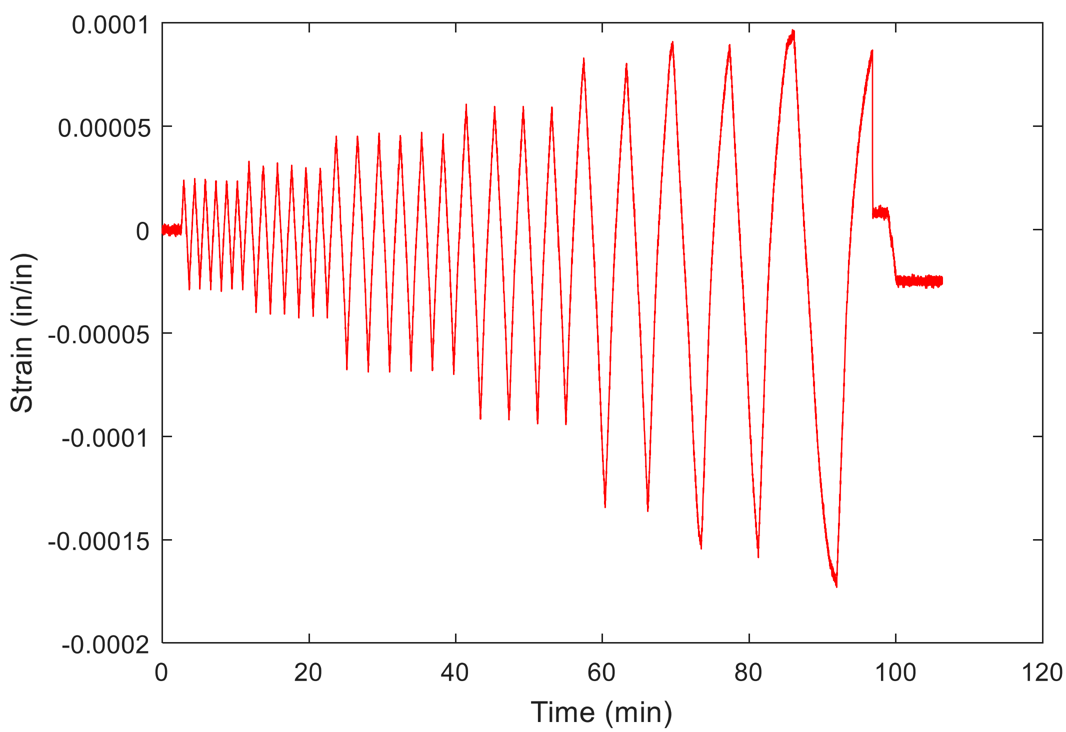


Figure V-47 SG15

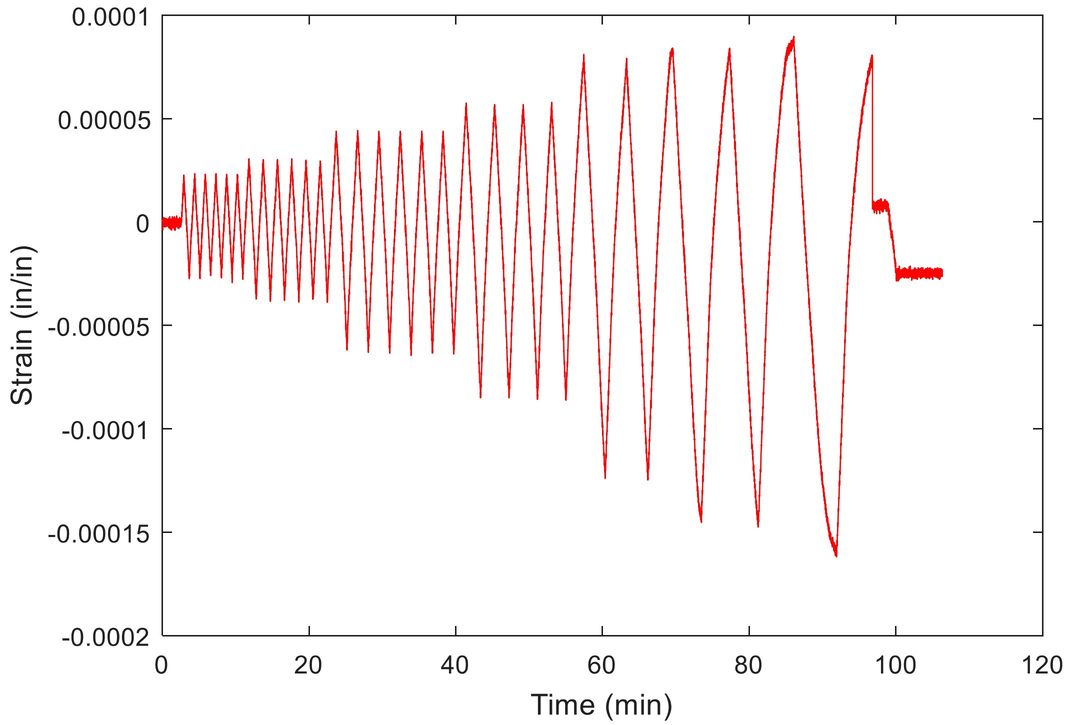


Figure V-48 SG16

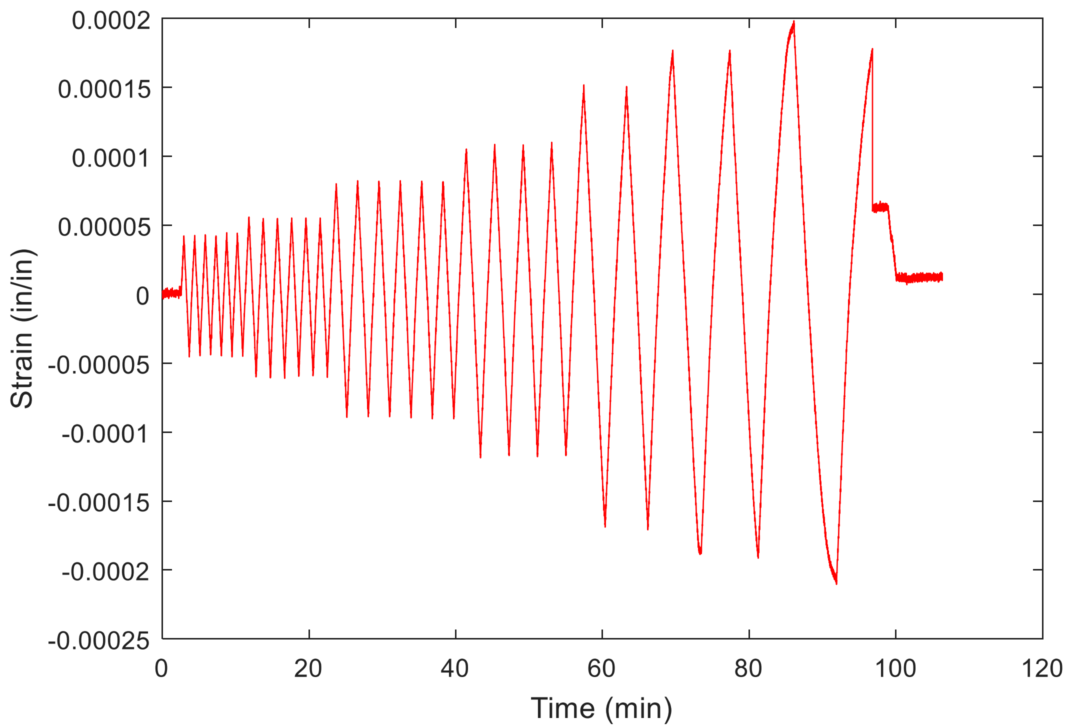


Figure V-49 SG17

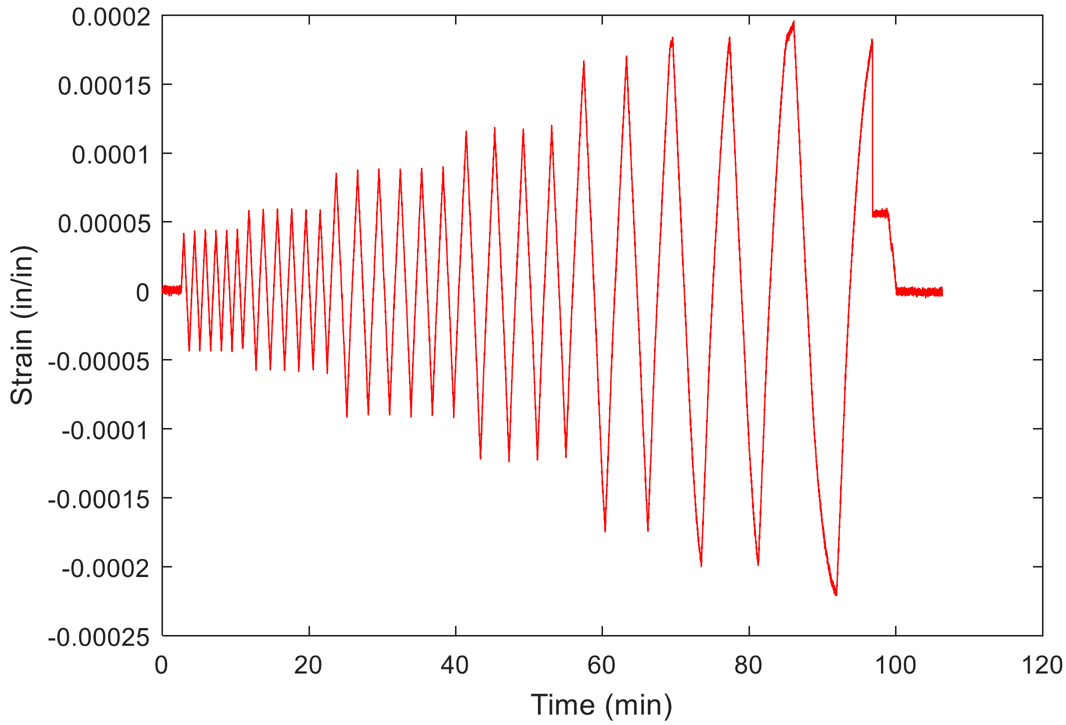


Figure V-50 SG18

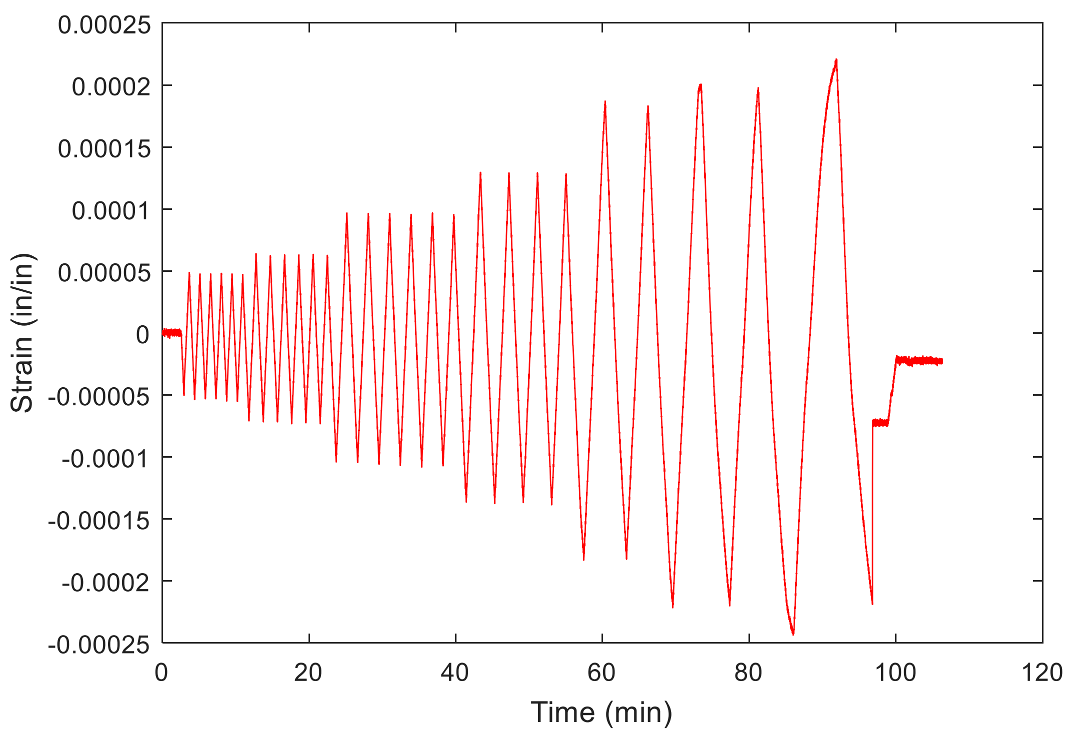


Figure V-51 SG19

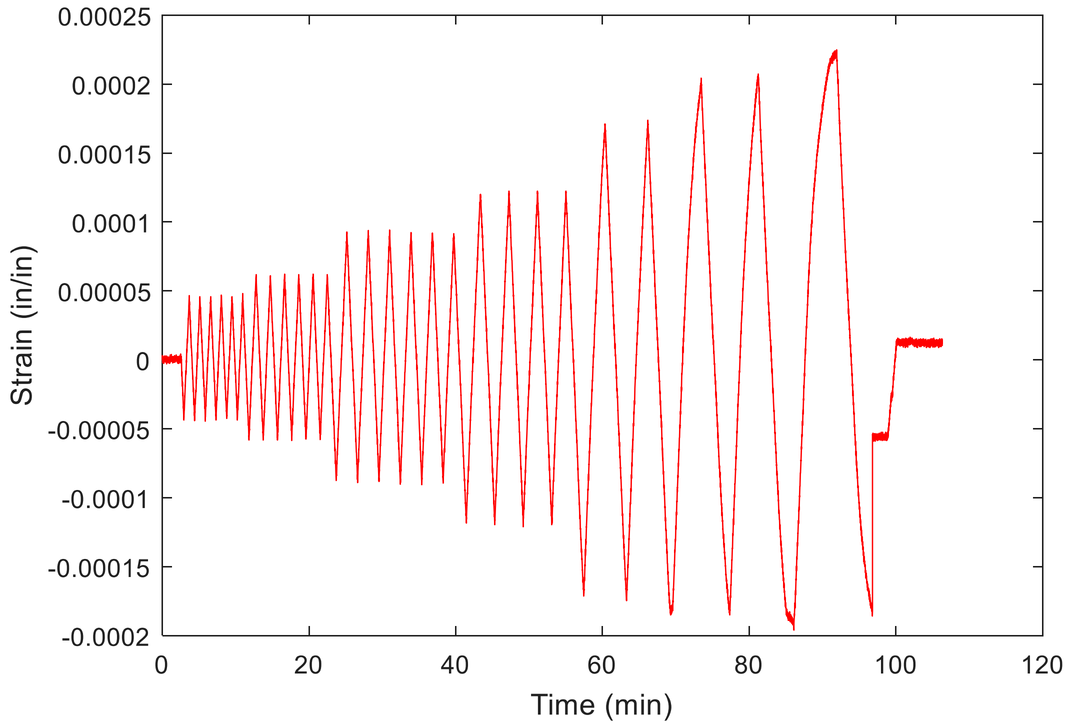


Figure V-52 SG20

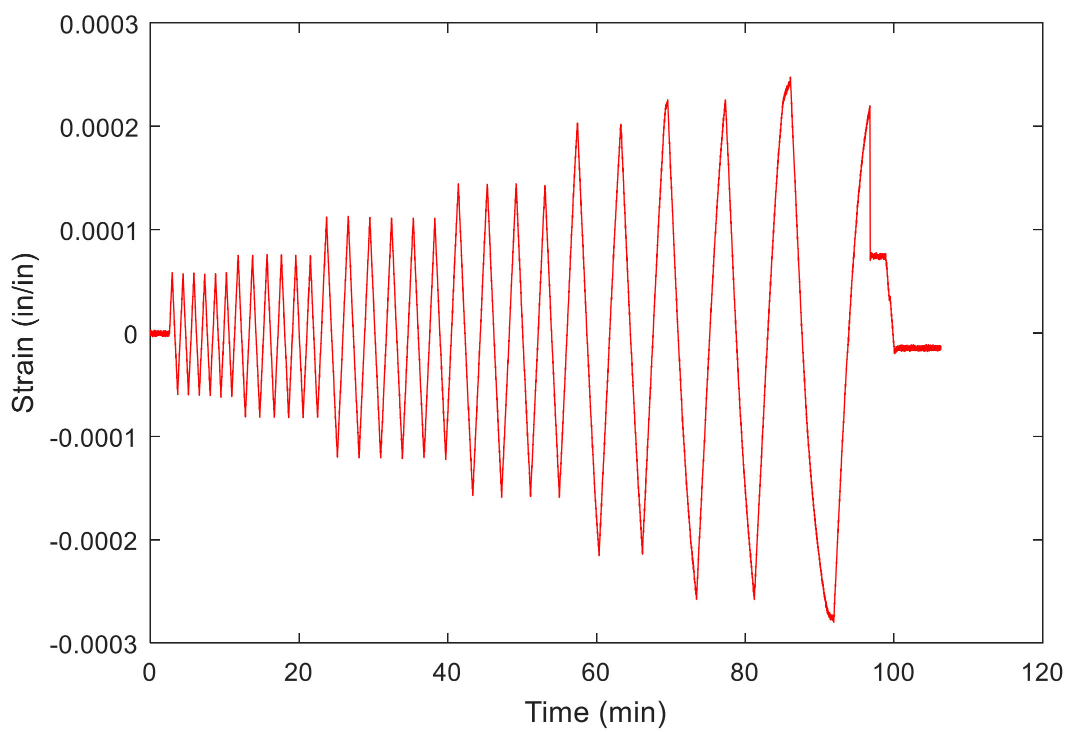


Figure V-53 SG21

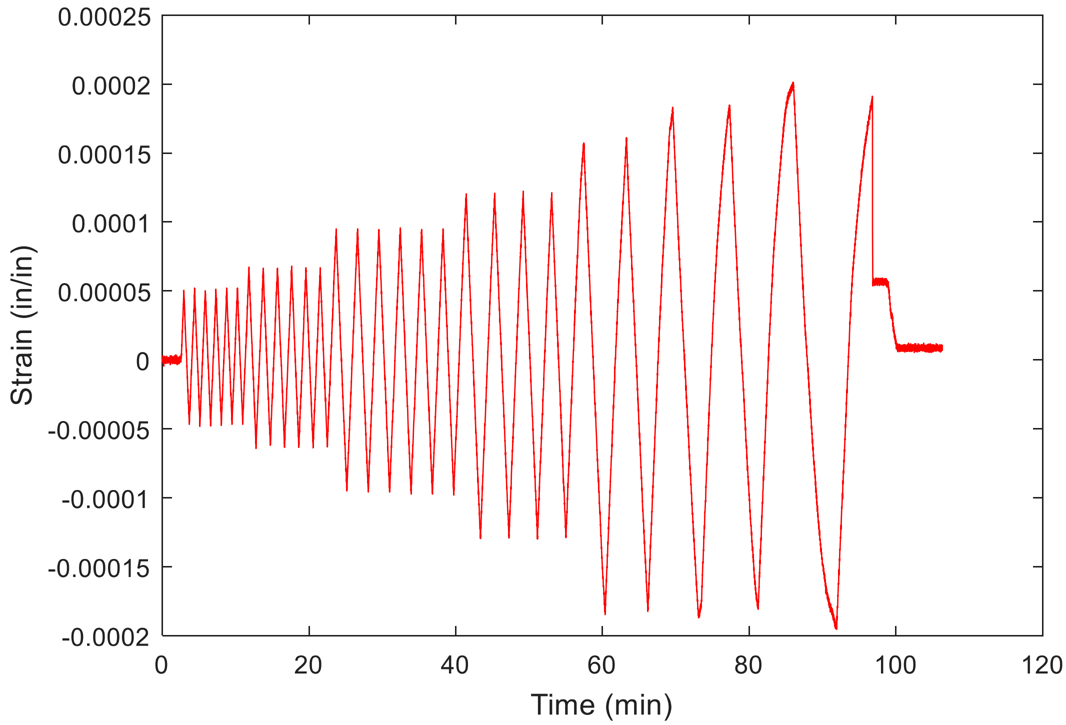


Figure V-54 SG22

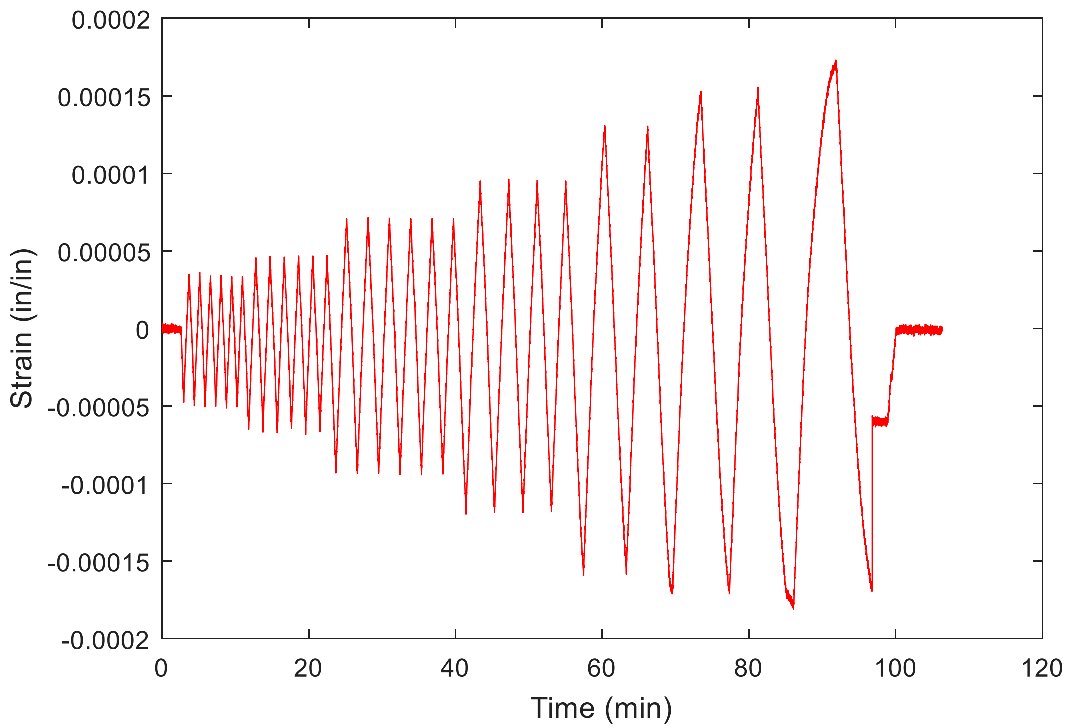


Figure V-55 SG23

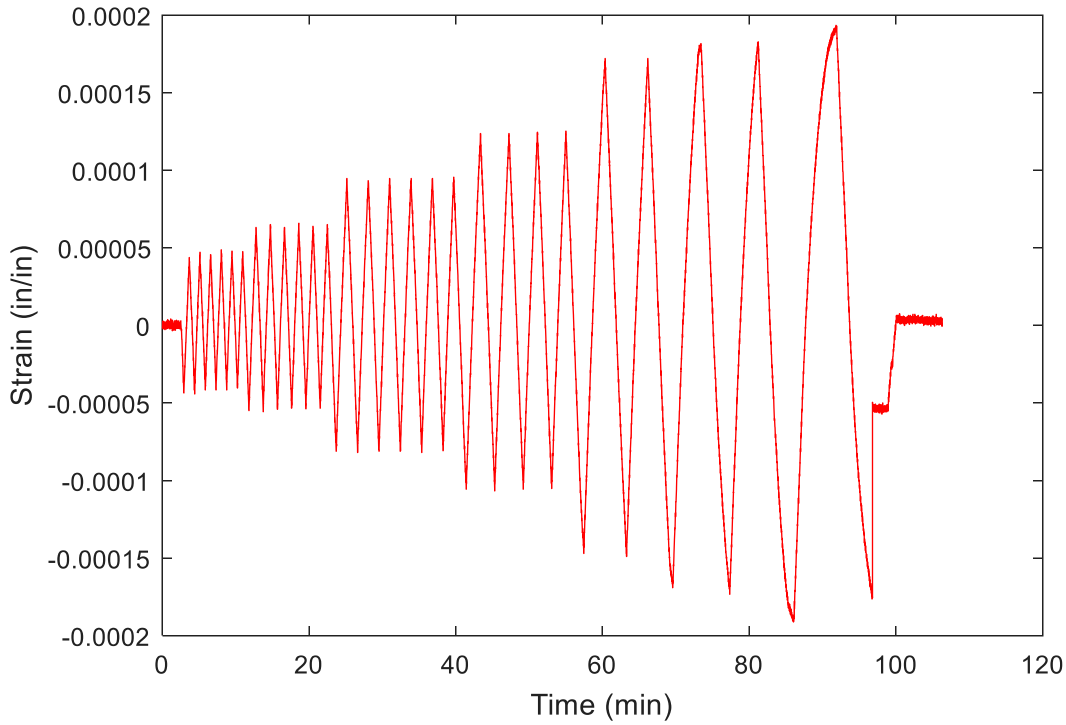


Figure V-56 SG24

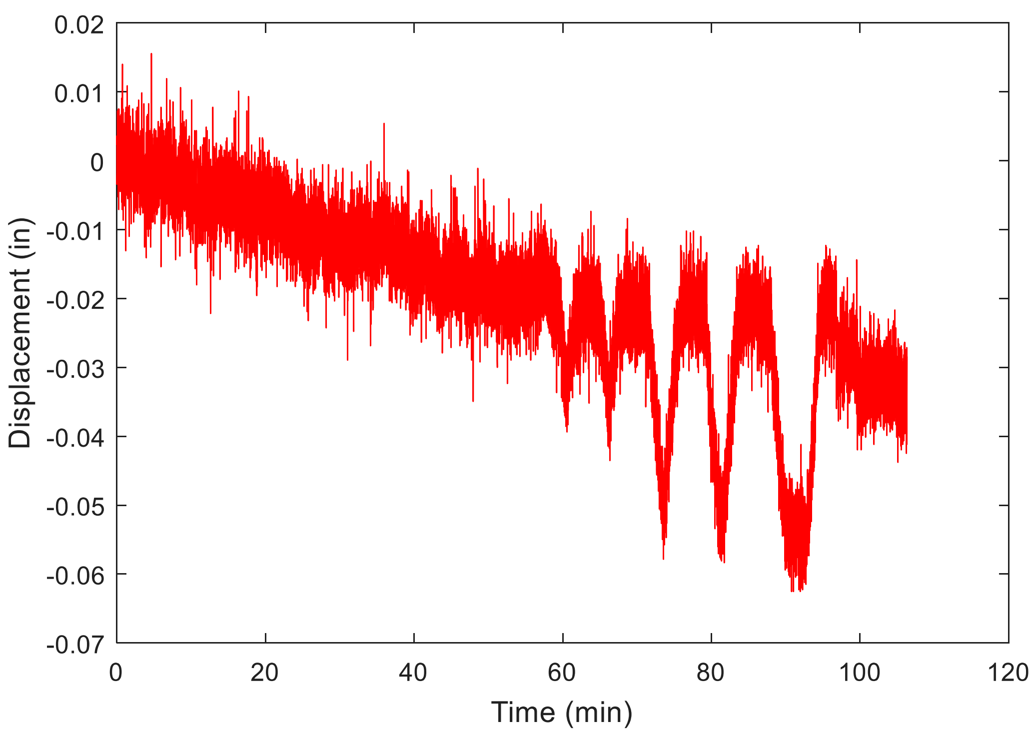


Figure V-57 CLP01

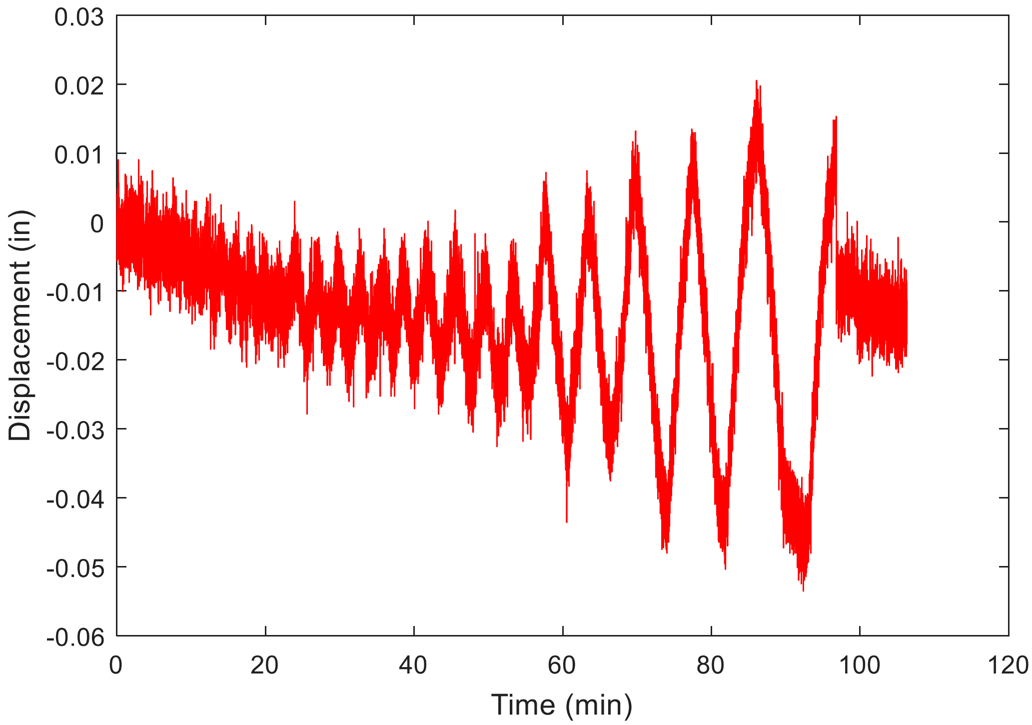


Figure V-58 CLP02

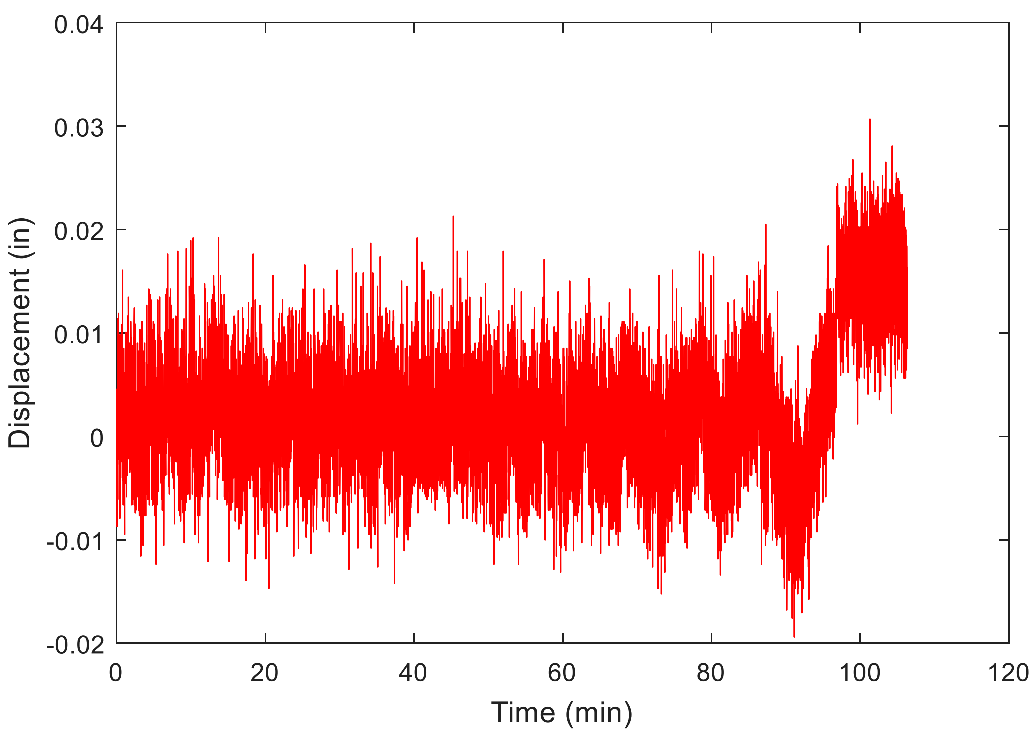


Figure V-59 CLP03

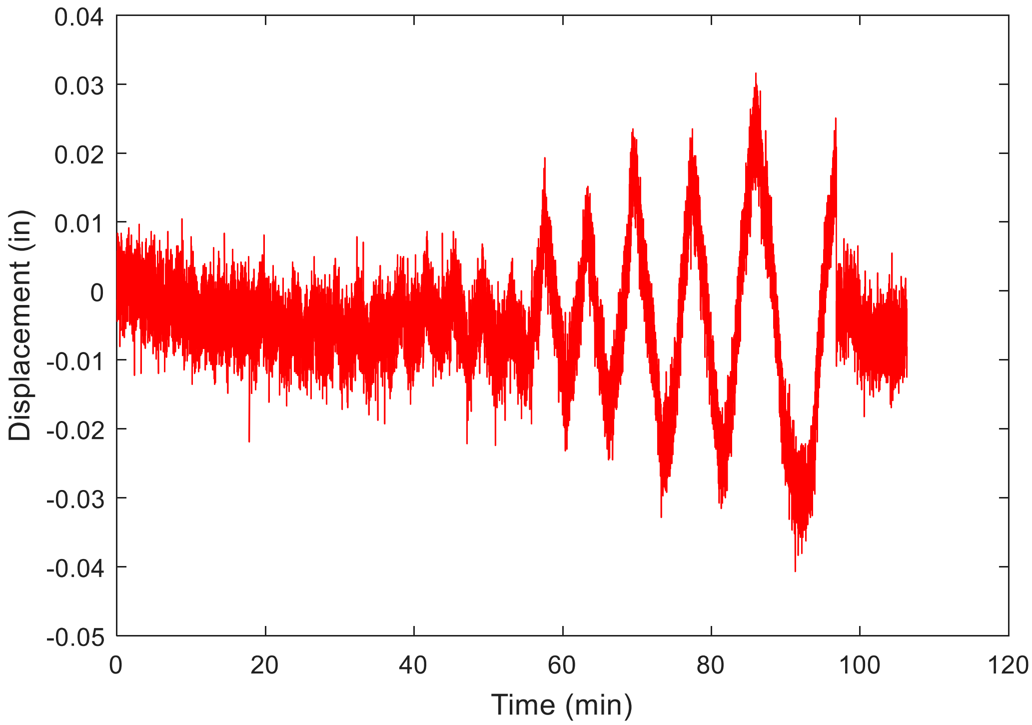


Figure V-60 CLP04

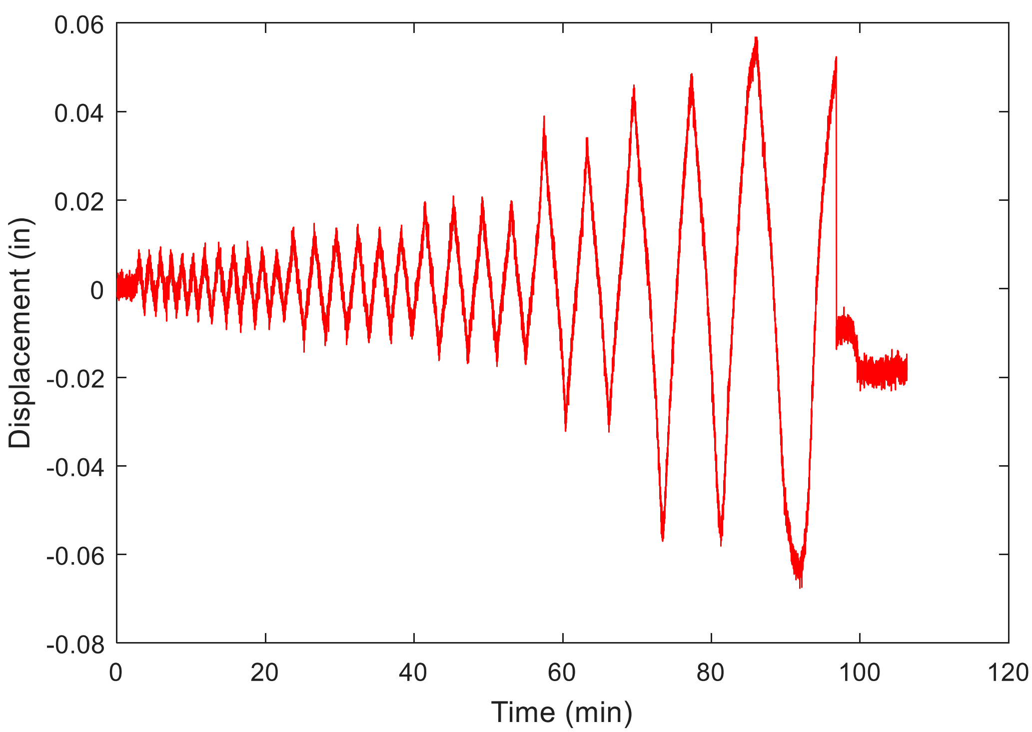


Figure V-61 CLP05

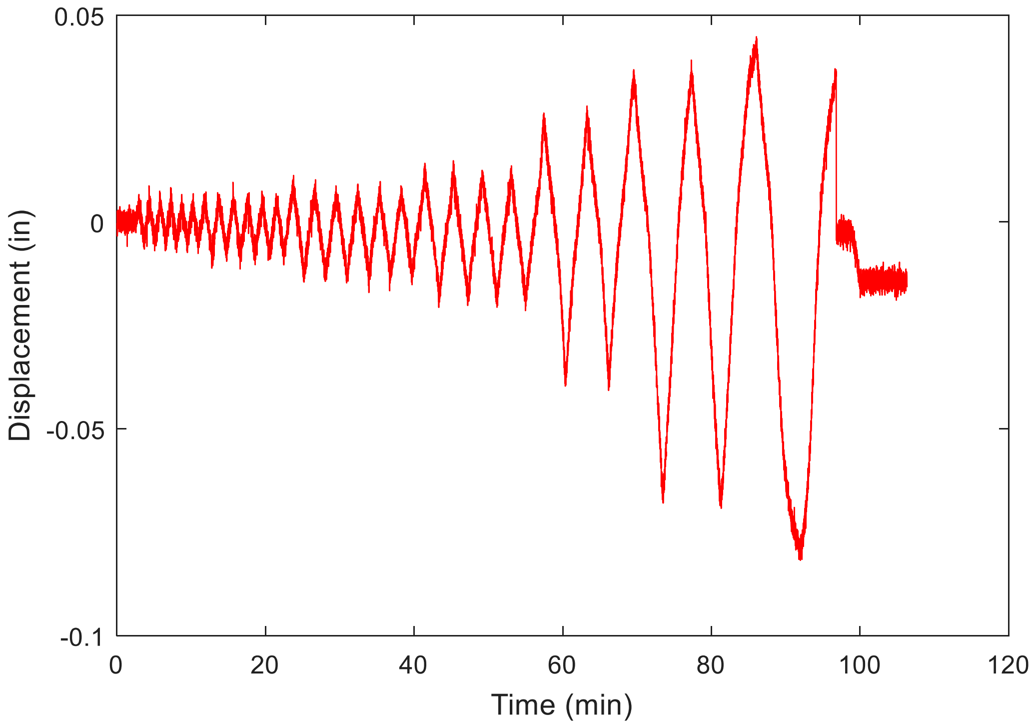


Figure V-62 CLP06

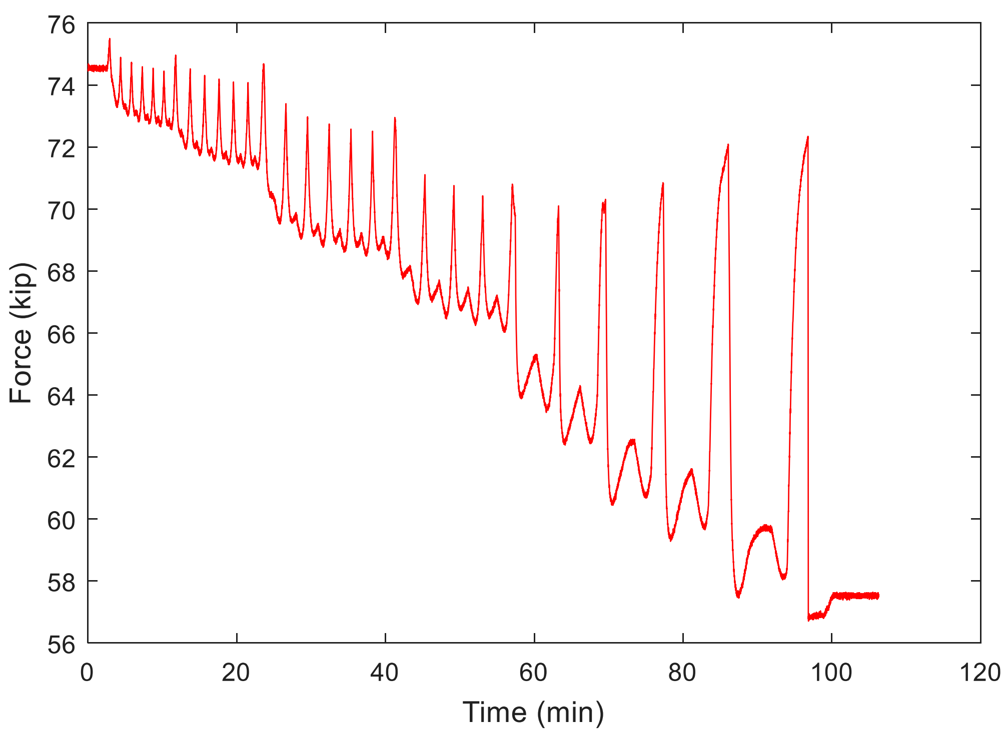


Figure V-63 Force in BSG01

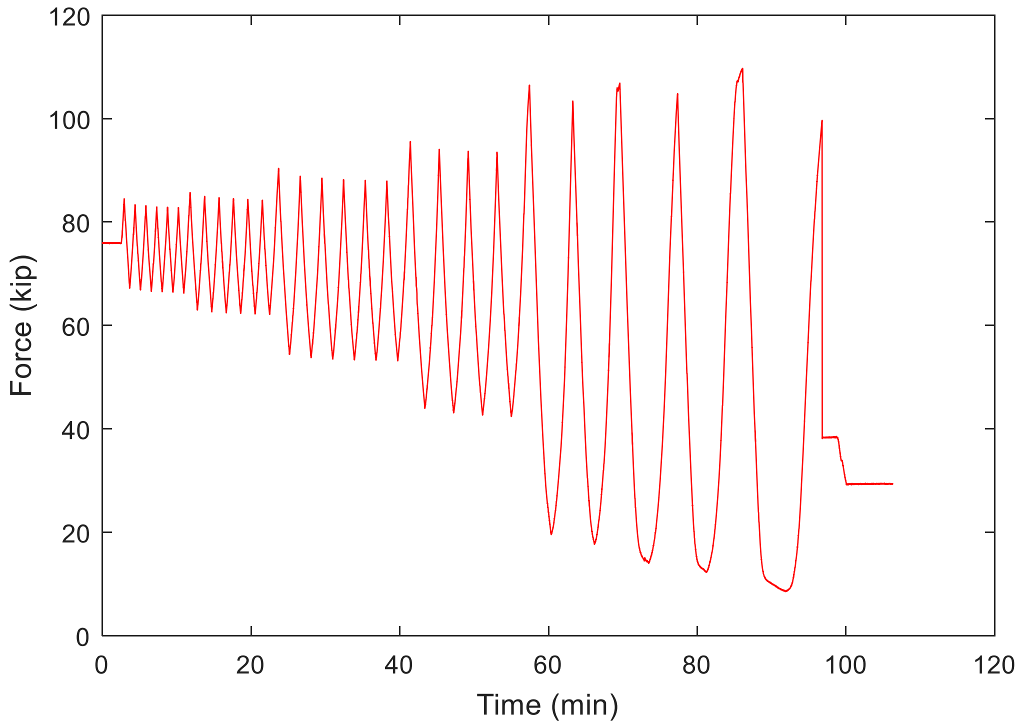


Figure V-64 Force in BSG02

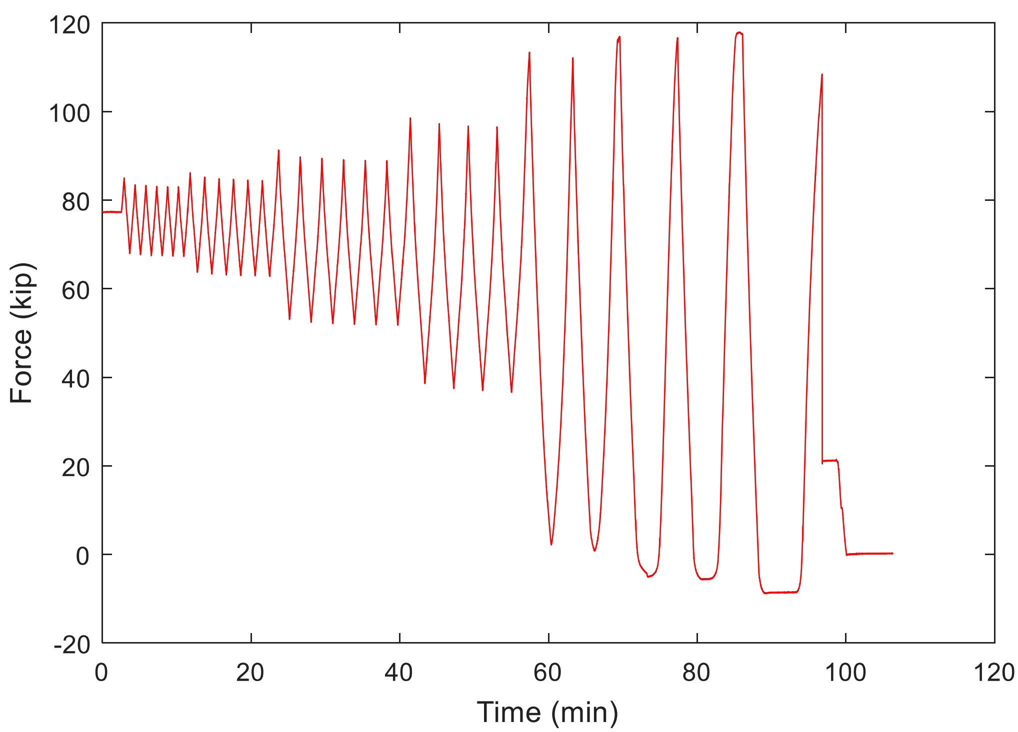


Figure V-65 Force in BSG03

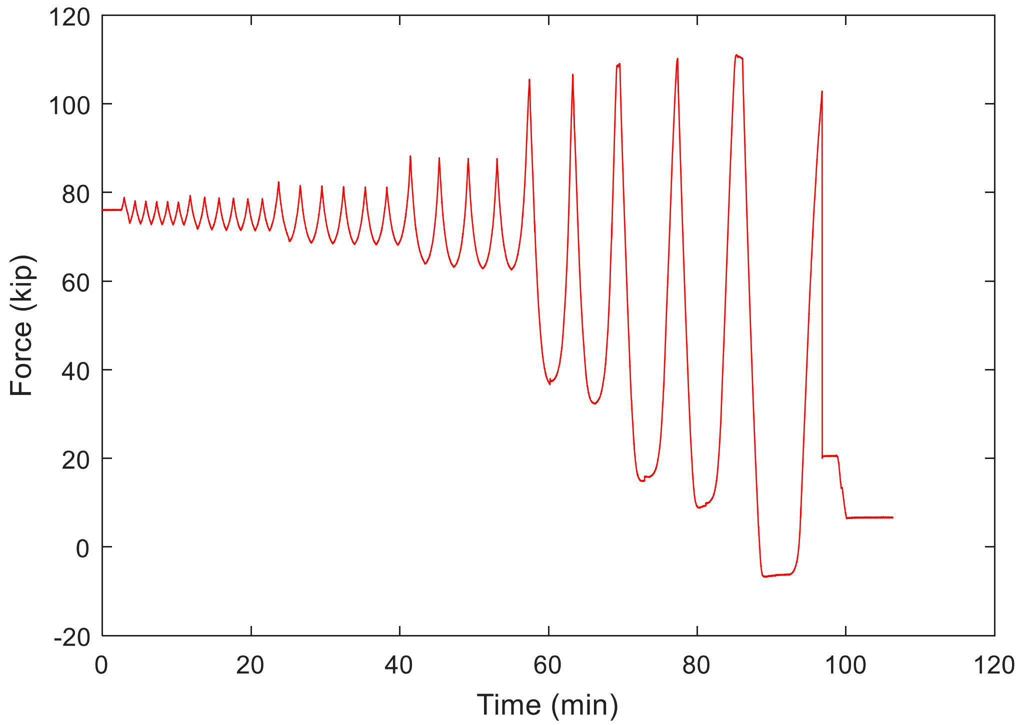


Figure V-66 Force in BSG04

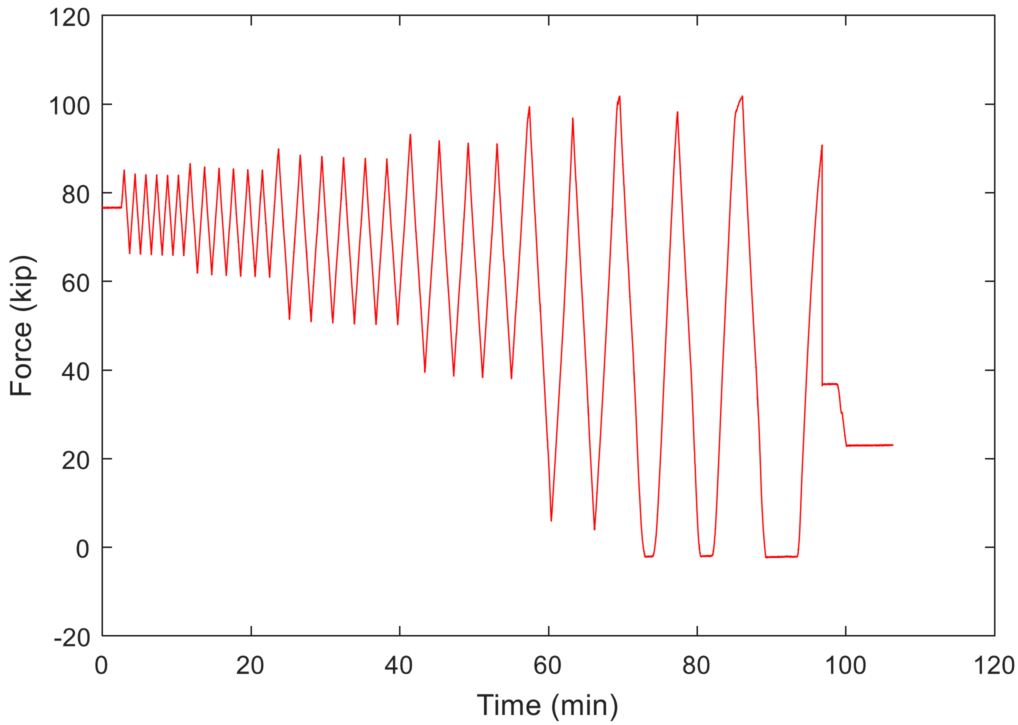


Figure V-67 Force in BSG05

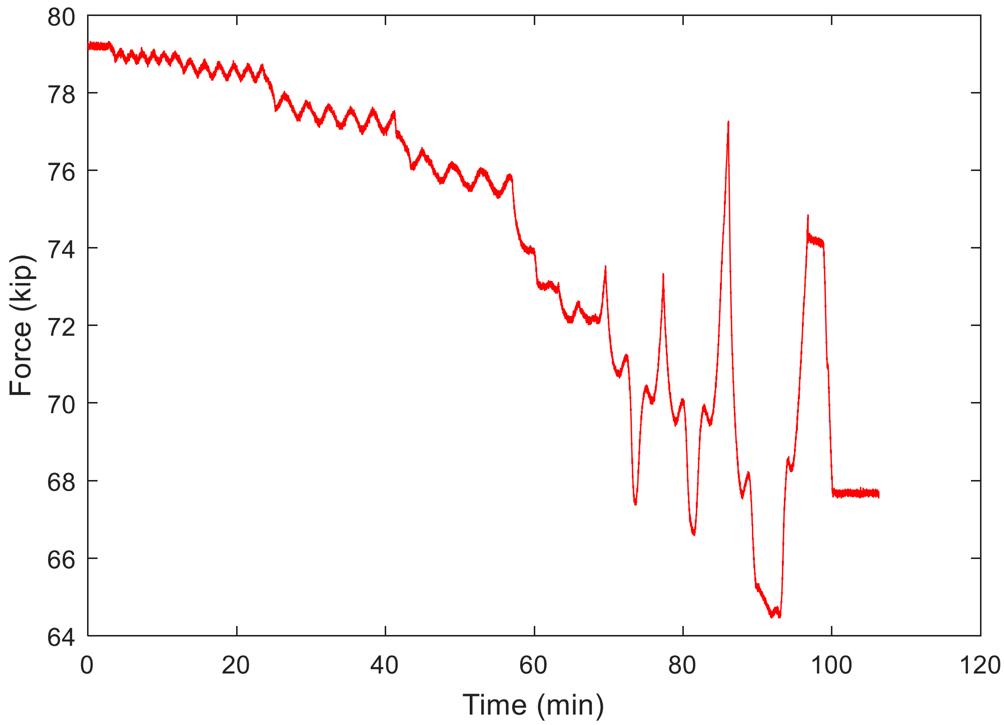


Figure V-68 Force in BSG06

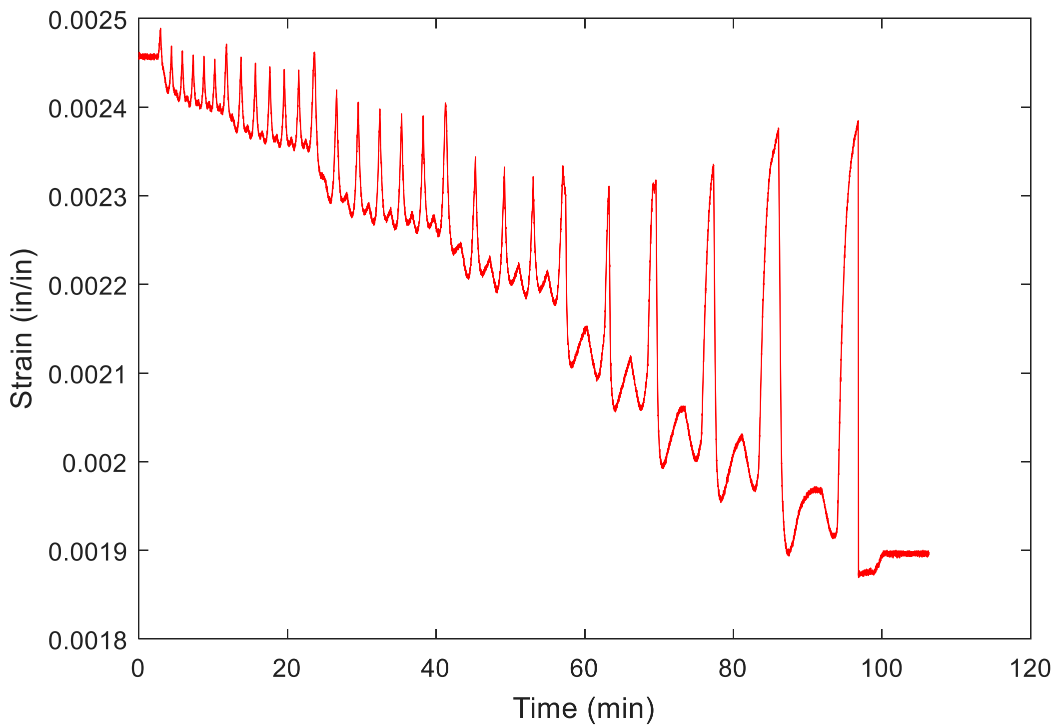


Figure V-69 Strain in BSG01

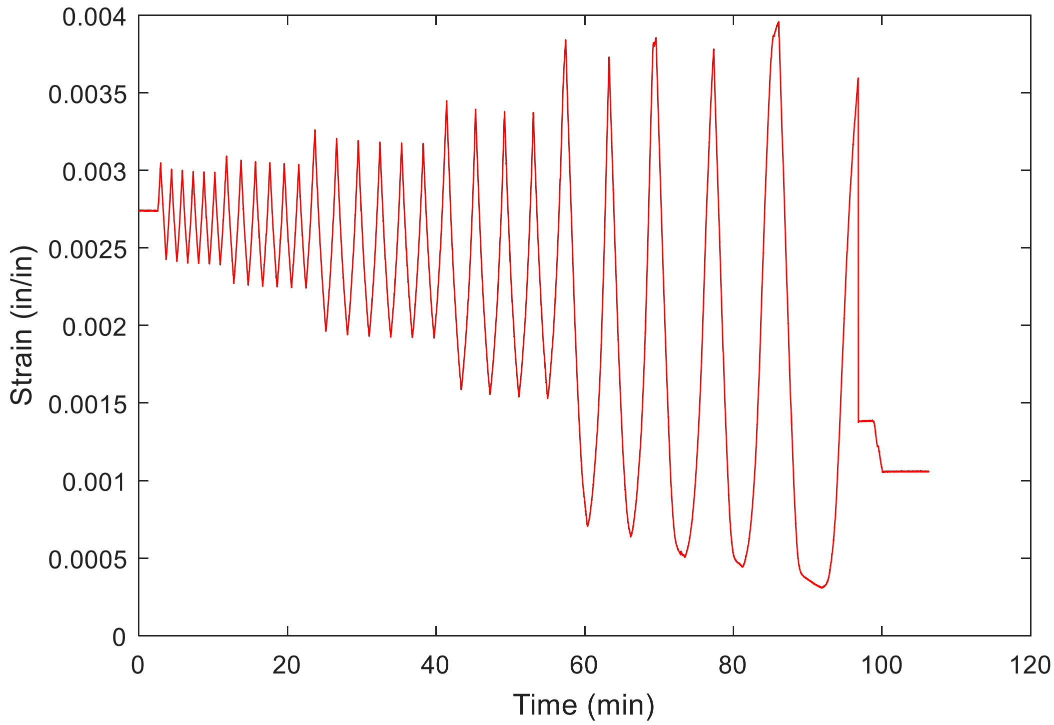


Figure V-70 Strain in BSG02

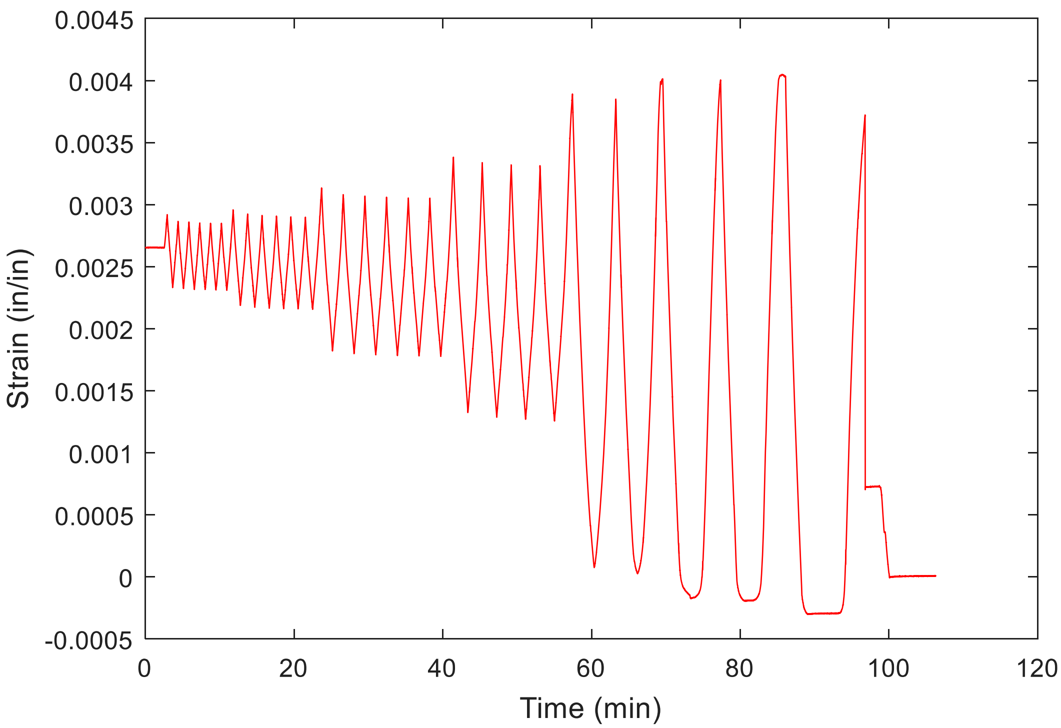


Figure V-71 Strain in BSG03

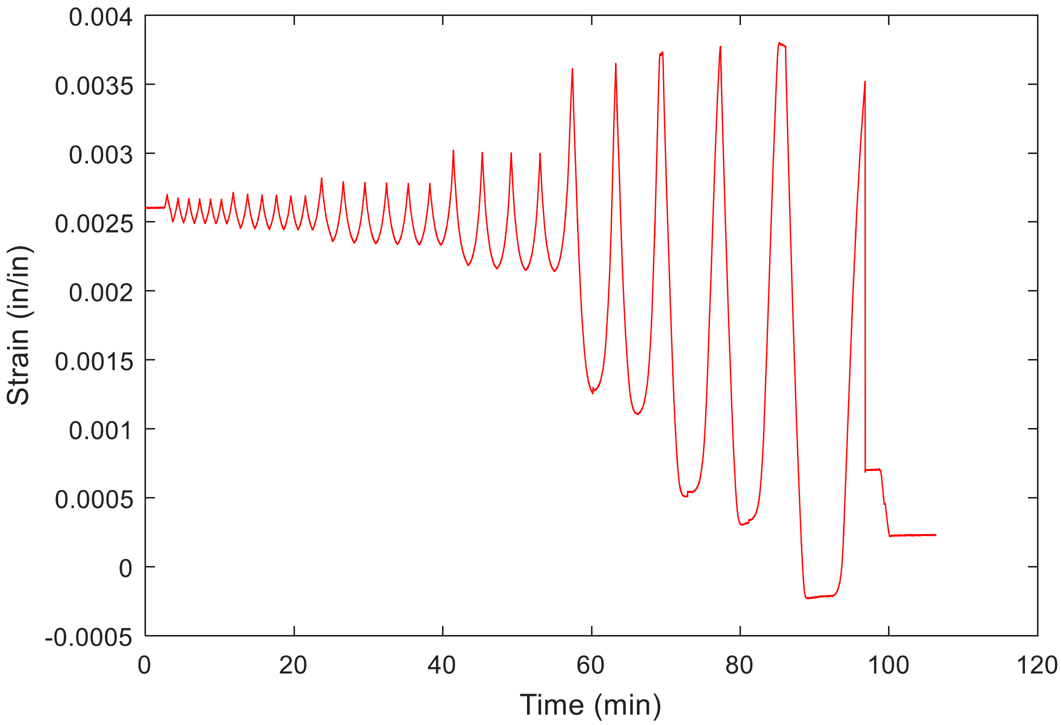


Figure V-72 Strain in BSG04

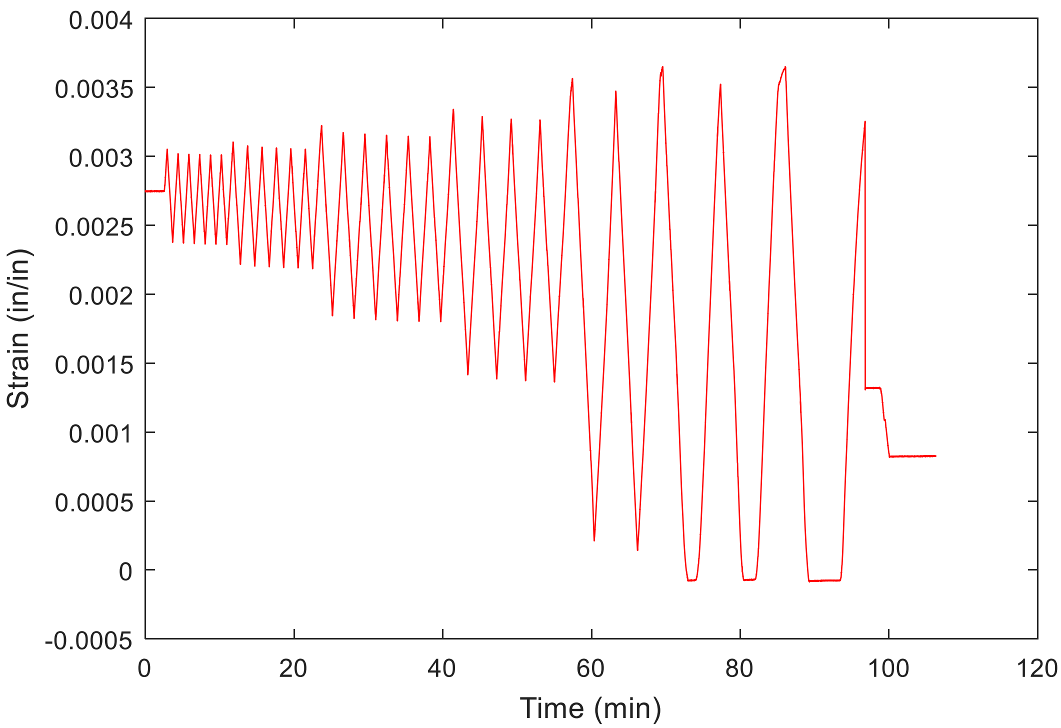


Figure V-73 Strain in BSG05

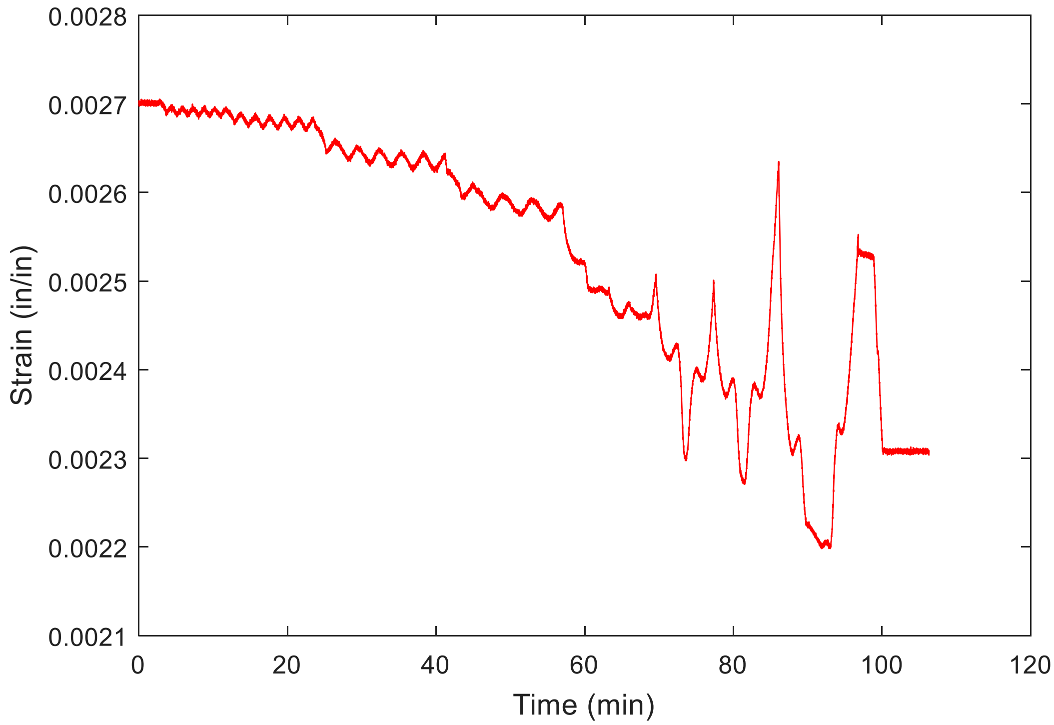


Figure V-74 Strain in BSG06

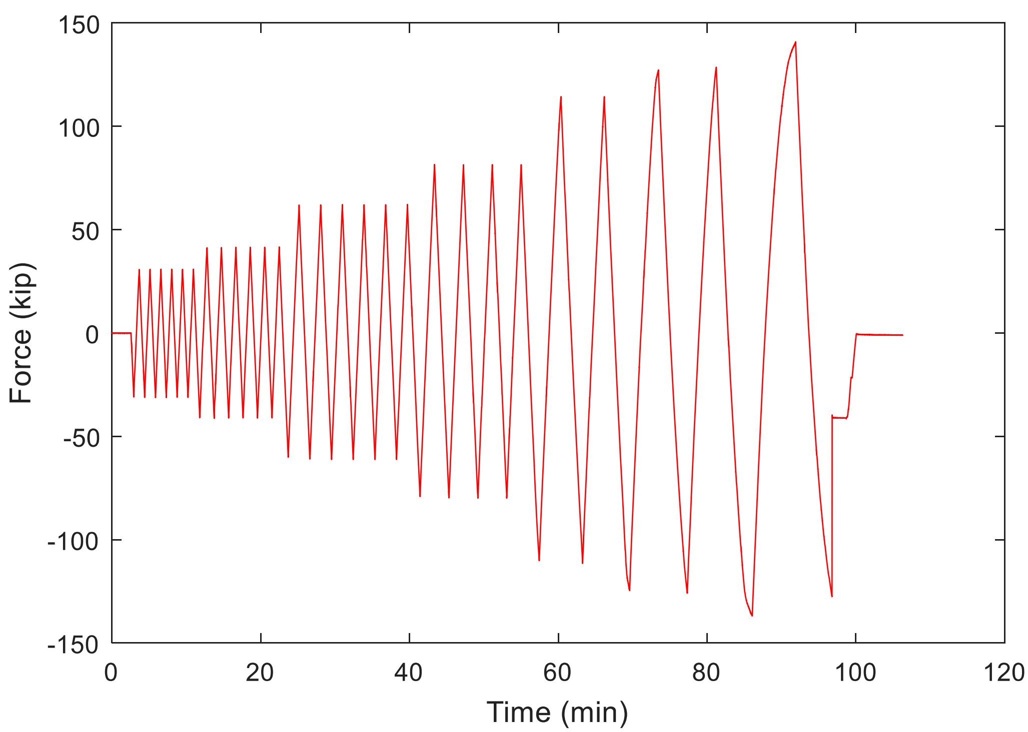


Figure V-75 Force in Actuator

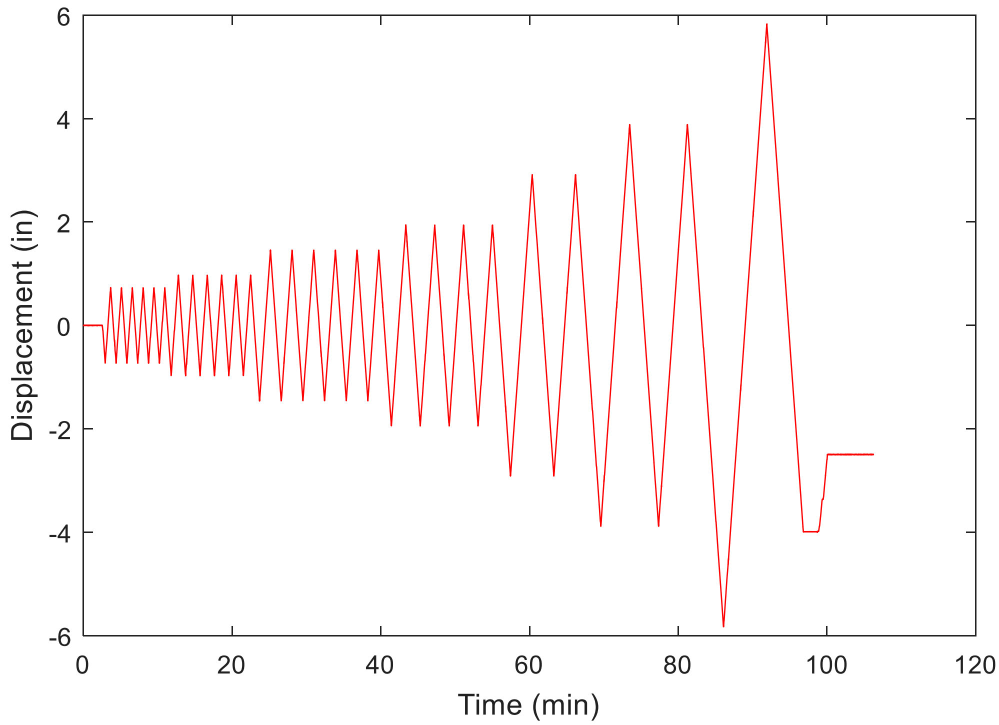


Figure V-76 Actuator Displacement