Columns and Beam-Columns in Submarine Modular Structures

A Review for Electric Boat of Groton, Connecticut

A Major Qualifying Project Report:
Submitted to the faculty of
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Abstract

Electric Boat's submarine modular structures may contain trusses designed using elastic analysis methods for dynamic underwater shock loading. Suitable member sizes are partly determined on buckling criteria usually intended for design of columns subject to static loads and may be overly conservative for dynamically loaded columns, which can translate into added module weight and cost. In an attempt to understand the degree of conservatism, tensile and compressive tests were performed to determine material properties for structural steel in column and truss configurations both statically and dynamically. Through analysis of results, it's recommended that the members not be loaded beyond their static yield strength and that consideration be given to reducing column effective length factors.

This project was sponsored and funded by Electric Boat, Groton, Connecticut. The conclusions and recommendations made in this report are those of WPI and do not necessarily reflect the opinions of Electric Boat.

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Authorship

This report was created through the equal contributions of all group members: Andrew Crouse and Timothy Manchester.

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Background

The following project, sponsored by Electric Boat (EB), was intended to better understand the behavior of columns and typical trusses members when subjected to static and dynamic loadings and to relate that behavior to the existing design methods for preventing column buckling in submarine modular structures. While a traditional analysis on the trusses in modular structures (based on design assumptions) concludes that the members are optimally designed, design methods may have unnecessary conservatism, which would lead to over-designed (costlier and heavier) members.

Certain large modular structures in a modern submarine are made of truss structures fabricated from square tubes. These tubes are often welded together to form Pratt-type Trusses, which are then used as the standard truss configuration for the modules. These modules must be designed to withstand rapidly-applied loadings due to underwater shock conditions.

Chapter H of the AISC Specifications (Chapter H, Steel Construction Manual, 2005) describes the design of members subjected to combined forces, in this case axial compression and dynamic bending. The design for combined forces requires both the allowable and actual strengths (i.e., axial and flexural bending), which are input into specific empirical interaction equations (i.e., Equation H1 of the AISC Specification). These equations are outlined and explained as follows:

$$For\ \frac{P_r}{P_c} \geq 0.2 \rightarrow$$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{[H1-1a]}$$

$$For\ \frac{P_r}{P_c} < 0.2 \rightarrow$$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \qquad \text{[H1-1b]}$$

In which:

P_r = Required axial strength (actual stress in axial loading),

P_c = Critical axial strength (allowable stress in axial loading),

M_r = Required flexural strength (actual stress in flexural bending) and

M_c = Critical flexural strength (allowable stress in flexural bending).

A major component in the above equation is the critical axial strength (P_c) which relies heavily on the slenderness ratio ($\frac{KL}{r}$). The slenderness ratio is the quotient of the length of the column (L) and its radius of gyration (r). The K factor (K) is a constant that depends on the end conditions of the column and the shape in which it buckles. Table C-C2.2 of the AISC Commentary on the Specification gives approximate values of the effective length factor (K) with respect to various end conditions. This table provides values ranging from 0.5 representing a column with both ends fixed, to a value of 1.0 for a column with both ends pinned and free to rotate.

The assumption made in the EB design methodology for columns in modern submarine modules involves using the rather conservative estimate of 1.0 for the K factor, which assumes pin ended connections despite the fact that the ends are welded in the frame. The question that needs to be asked is how conservative this assumption is and can the members be more optimally designed without compromising the strength of the submarine structure. If a more realistic effective length (KL) of each column in the basic truss configuration could be determined, this would reduce the slenderness ratio and allow for the use of members with smaller cross-sectional areas while still providing the necessary strength for the same loading conditions. Ultimately, relaxation of conservatism could save on materials and fabrication costs as well as decrease the overall weight of the structure.

The objective of this project is to perform a variety of experiments and compare the loads that precipitate actual failure with the design loadings using the AISC interaction equations. The comparison will enable a direct association between the experimental and design strengths of the frame modules and show how conservative the current design method may be.

Methodology

The following is a list of objectives that will be necessary for successful completion of this project:

- Determine the loading needed to elastically and plastically deform steel columns and the provided truss configuration in quasi-static tests.
- Determine the loading needed to elastically and plastically deform steel columns and the provided truss configuration in dynamic tests.
- Analyze the results of the quasi-static and dynamic tests using the AISC interaction equations and determine an appropriate compressive load strength based on safety factors acquired from a literature review.

These three objectives will be accomplished through the use of three specific tasks involving data collection and analysis. These tasks will consist of: procuring and fabricating the test structures; testing columns and trusses subjected to quasi-static loading; and testing columns and trusses subjected to dynamic loading. Each of these three tasks is explained in further detail below.

Before any major testing began, an introductory meeting was held between the group and representatives from Electric Boat. This initial meeting provided the group with the opportunity to learn about the basics of submarine construction and design in order to obtain a basic background of the field. Specifics have also been detailed by EB regarding the primary goals of this project as well as the creation of a reasonable time frame in which to complete it. Drawings, material specifications and welding fabrication specifications were provided by EB such that the test samples could be made in a manner that conformed to the manner actual submarine module trusses are built.

Accomplishing the above project objectives may allow EB to make less conservative design decisions which could reduce the cost of the submarine structure without detracting from the structure's strength and serviceability. The Gantt chart in Figure 1 gives a visual representation of the schedule of the project.

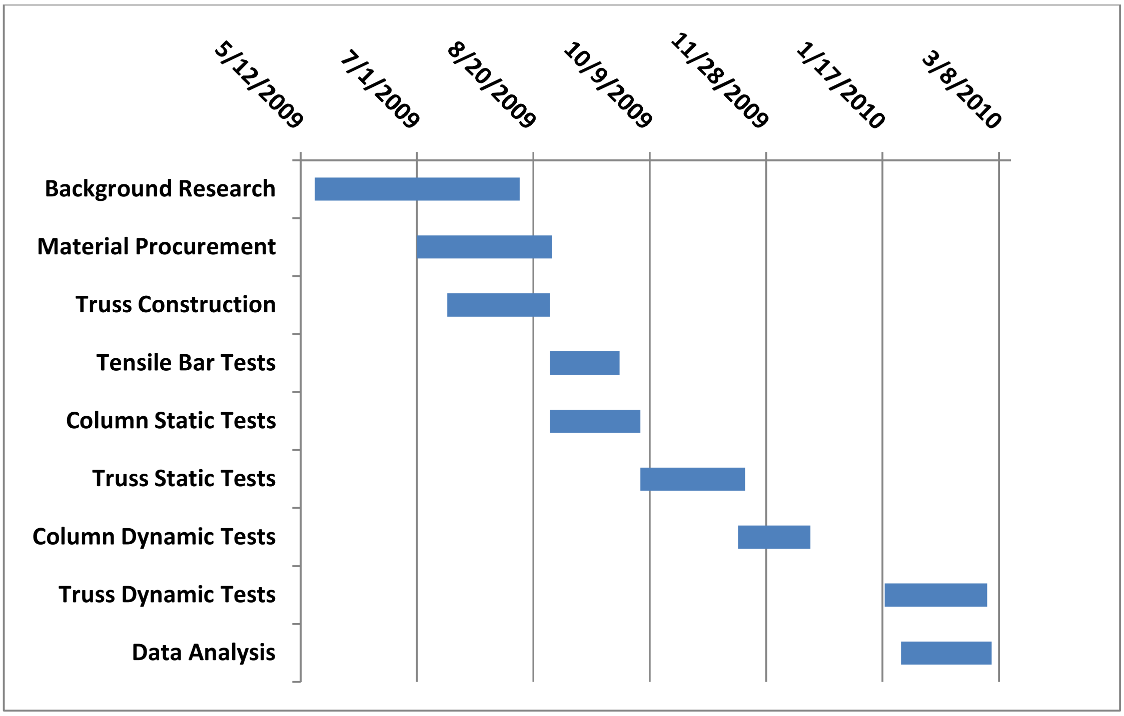

Figure 1: Gantt chart of Project Schedule.

Procurement and Fabrication

The construction of the experimental sample trusses will be one of the most time-consuming aspects of the entire project. The group will use instructions provided by Electric Boat to determine how to design the symmetric and asymmetric truss models that will be utilized in the testing procedures. Both frame designs are based on the file, "Sk WPI 01 Rev B 11-19-09.ppt" for all of the dimensions (see Figure 2 and Figure 3). The specimen frames will represent a scaled version of a portion of the truss for a large modular structure of a modern submarine. The specifications shown in the sketches provided by EB include material size and shape, truss configuration, and truss member lengths, based on EB design processes and the capabilities of the laboratory testing equipment at WPI.

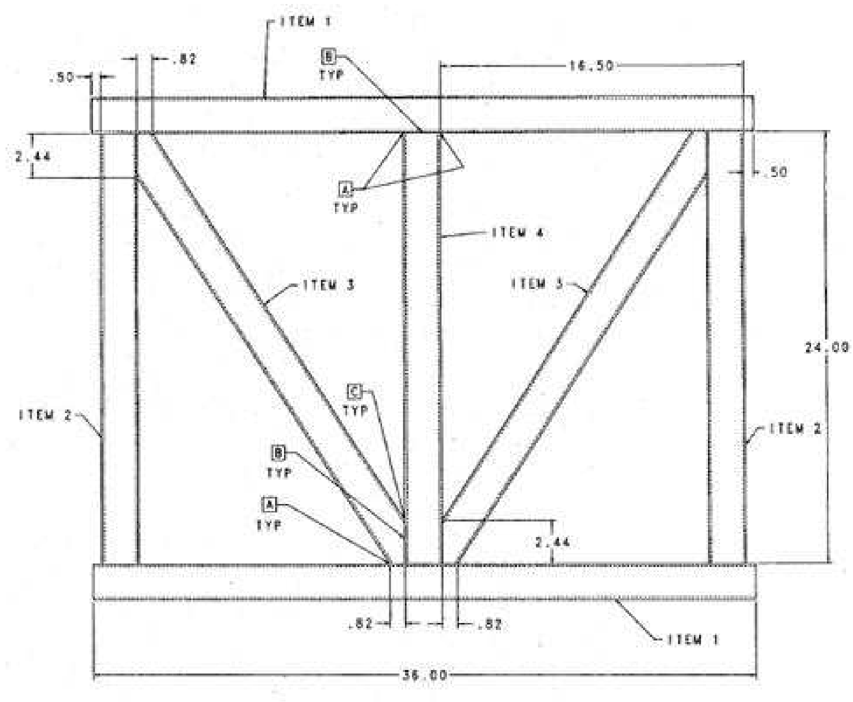

Figure 2: Symmetric truss configuration and dimensions for fabricated samples.

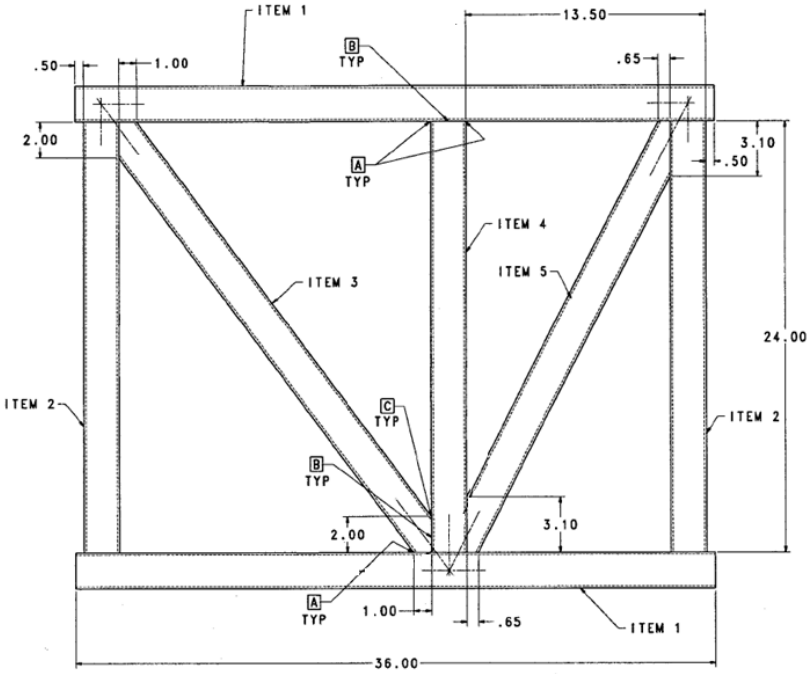

Figure 3: Asymmetric truss configuration and dimensions for fabricated samples.

Material for the first round of testing will be procured based on ASTM and Electric Boat specifications. ASTM A500 steel will be used to construct the frames. HSS 2” x 2” x 1/8” steel tubes will be purchased from Peterson Steel in Worcester, Massachusetts in 24-foot lengths. The total procurement process will provide enough material to make 15 truss samples for testing. The steel will be cut into lengths in the WPI CEE machine shop as specified in the project description and roughly assembled to make sure that all members fit properly. Diagonal members are especially important to consider, as the ends need to be cut perfectly in order for the truss to fit together correctly.

Cut pieces of steel tubing will be welded based on the information provided in the file, “Sketch: WPI-01 Rev B.11-19-09” from Electric Boat (see **Appendix A**). Due to the nature of the welding specifications, the steel members will be welded by Worcester County Welding of Worcester, Massachusetts. The final products will be checked by Worcester County Welding through visual inspection for flaws or discrepancies, approval documentation will be obtained, and the trusses will be returned to WPI for testing.

Tensile Bar Testing

Before the truss testing began, material properties for the steel columns needed to be obtained. Some of the more important properties of interest when performing this test were the yielding stress, fracture stress and ultimate stress. Sections of the HSS steel tubes used to fabricate the trusses were cut into steel “dog bone” shaped bars that were 15.5” long, 3/4” wide on the ends, 1/2” wide in the middle, and 1/8” in thickness as mandated by ASTM E8 specifications (ASTM International, 2008). The fabrication of the bars was outsourced to HydroCutter Co. of North Oxford, MA in order to reduce both heating and curving of the samples which is typical when using a standard milling machine to perform the same operation. These bars were sent out to be punched by the Mass Materials Research Company in order to create reference points with which to measure elongation following the testing. The bars were subjected to tensile forces using the Tinius Olsen Testing Machine until they broke in half.

The Tinius Olsen Testing Machine (Tinius Olsen) will be used for all tensile bar testing and quasi-static testing of both the frames and columns. It is a large hydraulically-driven tensile and compressive testing machine which can apply a compressive force on an object that sits between the main table and the head of the machine or apply a tensile force on an object that is held between the head of the machine and the upper stationary head. The main table also has the ability to measure the force being applied by the head when under compression. The machine is connected to a computer which measures and records the amount of force being applied to the object in the machine as well as the stress-strain curve. The Tinius Olsen Testing Machine and attached computer module can be seen in Figure 4.

Figure 4: Tinius Olsen Testing Machine workstation for tensile testing and quasi-static testing.

A tensile extensometer was used in order to measure the tensile distance so that a stress-strain curve of the elastic deformation during the testing process could be obtained. Computer software and the punch marks made on the bars helped to determine the properties for the steel that were mentioned above, which will be important in future testing and analysis. A detailed drawing of a tensile bar can be seen in Figure 5.

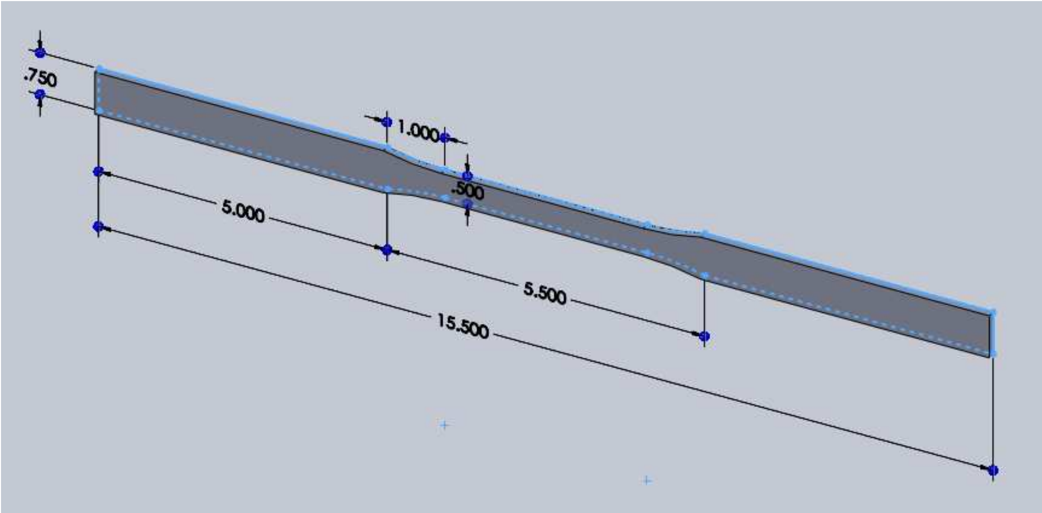

Figure 5: Detailed model of a tensile bar.

Quasi-Static Testing

The quasi-static testing process will be divided into two parts: axial compression of a single member (i.e., 24" in length from stock material) and compression along the top chord of a truss. Both tests will be performed multiple times to obtain consistent loading and stress-strain data. Important pieces of information that will need to be observed include the overall forces and deformation of the frame.

For axial compression on a single column, the member will be placed vertically in the Tinius Olsen with the top end free to rotate and the bottom end held in place by a ½" thick sheet of plywood with a 2" x 2" square cut in the center to keep the column from sliding along the main table or kicking out during testing. The quasi-static column test setup can be seen in Figure 6.

Figure 6: Setup for quasi-static loading on a column sample.

For testing of the truss samples, two steel I-beams will be placed side-by-side on the main table of the Tinius Olsen. Two steel half rounds with a 2” radius simulating roller supports will be placed 32” apart on opposite ends of the I-beams and the truss under load will sit atop them so it is elevated during testing to allow for deflection when forces are applied; both half rounds will run perpendicular to the length of the I-beams, will create point loads on the bottom chord of the truss, and will have negligible deflection.

Loading situations will consist of both distributed (using solid steel stock across the top chord) and point loaded (using a roller) configurations, as well as “strain-gauged” and non-“strain-gauged” samples. Samples will be both symmetric and asymmetric. Symmetric trusses will be tested for buckling stresses on the center column while asymmetric trusses will look more into the moment that is created about the center column due to central loading along the top chord and values of strain on the top chord on either side of the center column. The quasi-static distributed and point load truss test setups can be seen in Figure 7 and Figure 8, respectively.

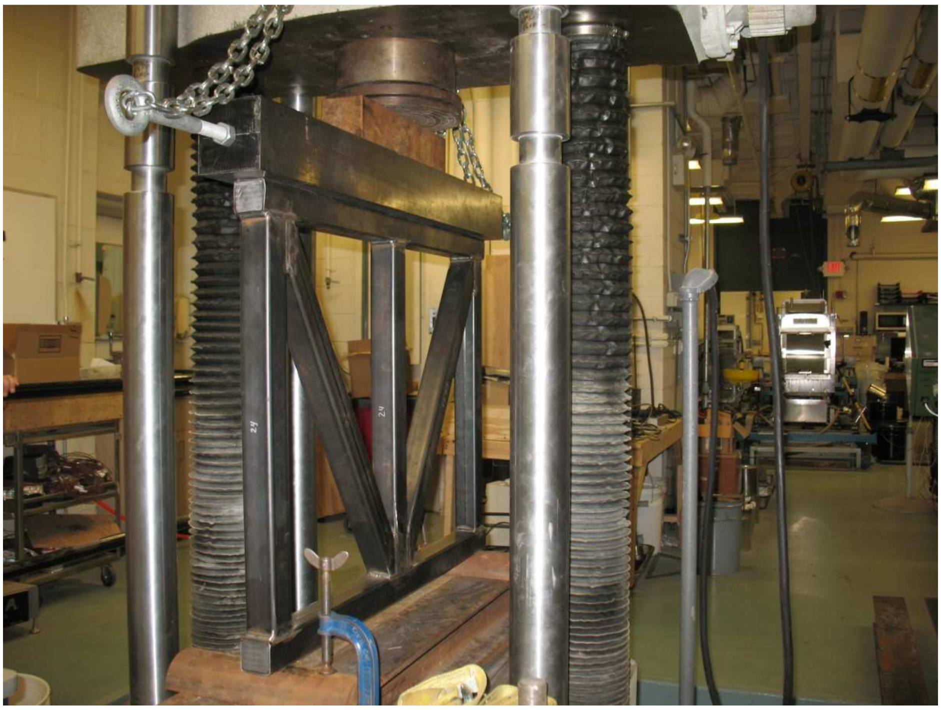

Figure 7: Setup for quasi-static distributed loading on a truss sample.

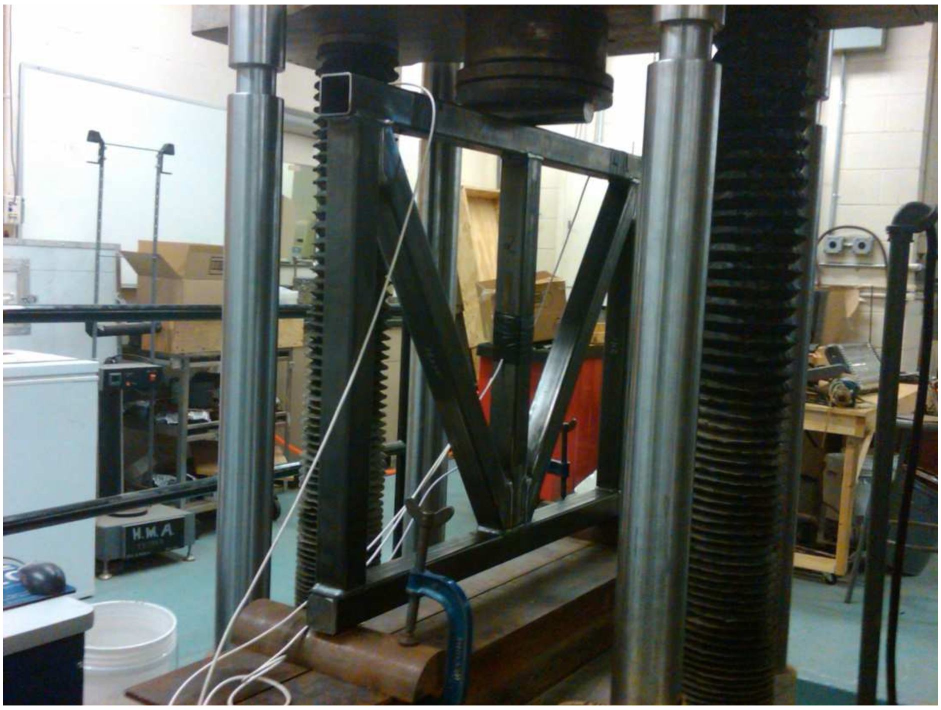

Figure 8: Setup for quasi-static point loading on a truss sample.

Dynamic Testing

Like the quasi-static testing, the dynamic testing will be broken into two stages: axial impact of a single member 24” in length from stock material and a point-load impact along the top chord of a truss. Both tests will be performed multiple times to obtain consistent impact data. Important pieces of information that will need to be documented include initial height of the impact source from the test subject, acceleration, and deformation.

A drop tower with a maximum drop height from the subject of five feet will be used for all dynamic testing. Compressive springs are also available to increase the drop velocity if necessary. The mass of the drop can be adjusted with the addition or subtraction of steel plates to the impact head which is connected to the tower by the four vertical columns. The drop tower and attached computer module can be seen in Figure 9.

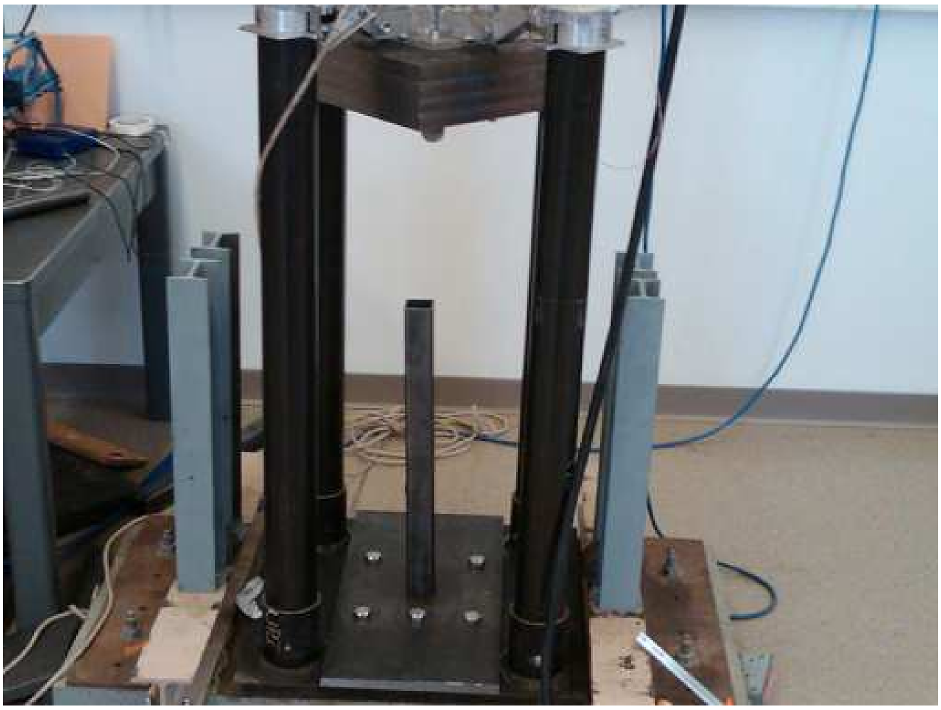

Figure 10: Setup for dynamic loading on a column sample.

The drop tower has two parallel I-beams attached to the bottom of the device. In order to test the frames dynamically, two-inch radius solid steel rollers will be placed on either I-beam to act as point loads along the bottom chord of the truss. The truss will be situated on top of the two rollers, running perpendicular to the two I-beams. Two 24” tall aluminum I-beams will be clamped vertically to each of the horizontal I-beams with a clearance of 2.5” between each pair. The I-beams will have tabs welded onto the end in order to allow clamping. This setup will hold the sample straight up and prevent swaying of the truss. Rubber padding will be added to the insides of each pair of vertical I-beams and on each roller support. Tests will be performed both with and without padding.

Impact experiments will consist of point loaded (using a roller) configurations, as well as “strain-gauged” and non-“strain-gauged” samples. Samples will be symmetric only and will be tested for impact stresses on the center column. In order to dampen the signal of the reading as well as extend the duration of impact for better graphical results, ½” rubber padding will be used in thicknesses of ½” and 1”. Calibration tests will be performed for impacts with no rubber padding, ½” rubber padding, and 1” rubber padding to decide on appropriate dampening. The dynamic truss test setup can be seen in Figure 11.

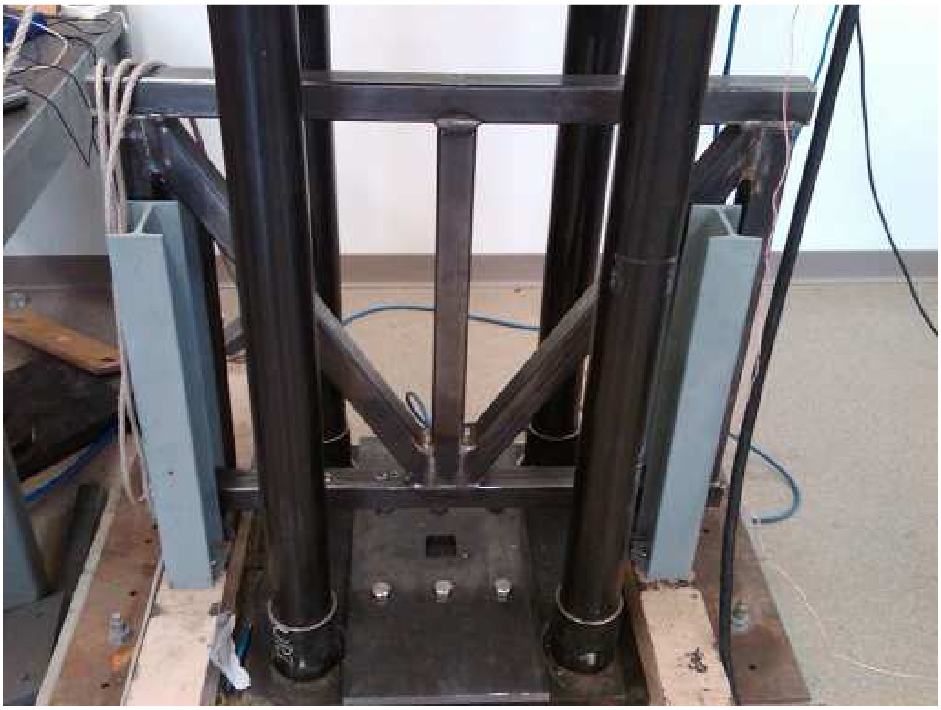

Figure 11: Setup for dynamic loading on a truss sample.

Instrumentation

Proper documentation of the tests is the most important part of performing the following experiments. Throughout the entire testing procedure, the following instruments will be used to collect data for documentation:

- High-speed video camera capable of capturing up to 4,500 picture frames per second.

- 10,000-g piezoelectric shock accelerometer placed on the impact head.
- Strain gauges attached to select columns and truss members which are wired to a computer module.
- Digital photography of the failure patterns.

The high-speed video camera to be used is a FASTCAM camera capable of capturing up to 4,500 frames per second with attached lighting that allows the user to capture the pattern in which the columns and frames fail with respect to the mass dropped at every instant of the test in very slow motion. The camera will be used only for dynamic testing. The setup of the high-speed camera along with the lighting configuration can be seen in Figure 12.

Figure 12: Setup of the high-speed video camera.

The 10,000-g accelerometer will be placed on the impact head of the drop tower during dynamic tests in order to record the mass's acceleration history with respect to time and to calculate the load applied to the columns and frames using energy. An important note about accelerometers is that they measure acceleration in G's (g-force or acceleration due to free fall). This is important because the measurement being made is instantaneous acceleration which does not rely on gravity alone, but results from other forces on the object such as stresses and strains in different directions (impact stress and strain in our case). As the sensor is unable to record constant accelerations for times greater than its time constant, data can only be recorded from the time of contact to the time of rebound. The data collected will then have the noise and frequency filtered using a MATLAB program employing an SAE type signal processing filter. The setup for the shock accelerometer can be observed in Figure 13.

Figure 13: 10,000-g piezoelectric shock accelerometer on impact head

Lastly, for the purpose of collecting strain data on specific members of the trusses and the single columns being tested in order to calculate stress and make conclusions regarding the buckling behavior of the members, strain gauges were employed. The strain gauges used were procured from Vishay Electric and have a resistance of 120 Ohms with leads attached to simplify the implementation. These gauges were attached to the steel members using epoxy, allowed to dry overnight, and then soldered to wires which were connected to a National Instruments Signal Conditioner. A setup of a strain gauge attached to a truss and the wiring configuration connecting the strain gauge to the signaling conditioner can be seen in Figure 14 and Figure 15, respectively.

Figure 14: Strain gauge setup on a truss.

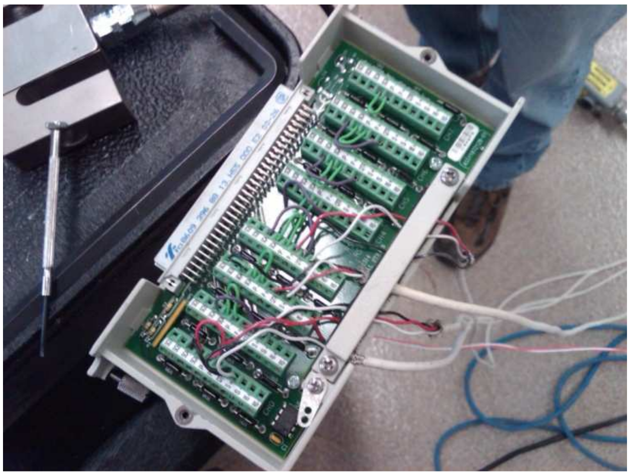

Figure 15: Wiring of strain gauge to signaling conditioner.

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These devices were interfaced with the accelerometers using the software National Instruments LabView. The interface was governed by the use of a LabView Virtual Instrument, and screenshots of the logical block diagram governing this Virtual Instrument can be observed in Figure 16. This particular example consists of five inputs: four for the strain gauges and one for the accelerometer. The four strain gauge values are multiplied by 1,000,000 to convert measurements in micro-strain. Both strain and acceleration data is outputted to a waveform graph and are recorded to an Excel spreadsheet file for easy access after the test. For dynamic tests, this data is filtered using MatLab.

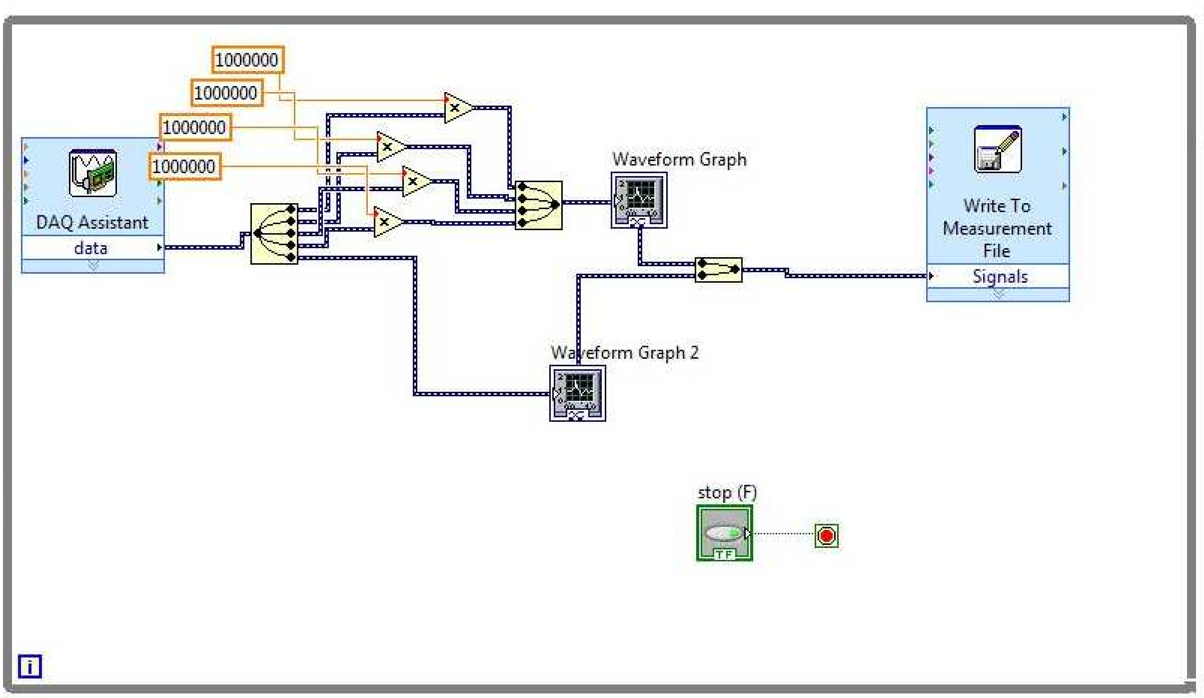


Figure 16: LabView block diagram for acceleration.

Results

Tensile Bar Testing

Quasi-static tensile tests were carried out in accordance with ASTM Specification E8 (ASTM International, 2008) in order to investigate the properties of the A500 steel used in the HSS members. The five inch long grips were each placed into the tensile portion of the Tinius Olsen Testing Machine with the reduced area visible. With an extensometer placed on the reduced section the samples were loaded until they began to yield. The extensometer was then removed and specimen was loaded until it failed. Minimum properties for this material as stated by ASTM for ASTM A500 Grade B steel are shown in Table 1.

Table 1: Material properties for ASTM A500 Grade B steel.

Material Properties for ASTM A500 Grade B Steel	
Ultimate Tensile Strength	58,000 psi
Yield Strength	46,000 psi
Minimum Elongation	23%

Tensile Bar #1 Test

The dimensions of the first sample's reduced area were measured by caliper to be 0.506 inches by 0.119 inches resulting in an area of 0.0602 square inches that was subjected to tensile forces in the test. The sample was subjected to a load of 400 pounds per minute until failure. The material properties determined from the experiment are summarized in Table 2.

Table 2: Material properties obtained in Tensile Test #1.

Summary of Properties: Tensile Test #1	
Peak Load	3,961 pounds
Ultimate Tensile Strength	65,797 psi
Yield Strength	57,490 psi
Elongation	32.1%

The stress-strain curve for the portion of the test measured with the extensometer (measuring yielding in the elastic region alone) is given in

Figure 17.

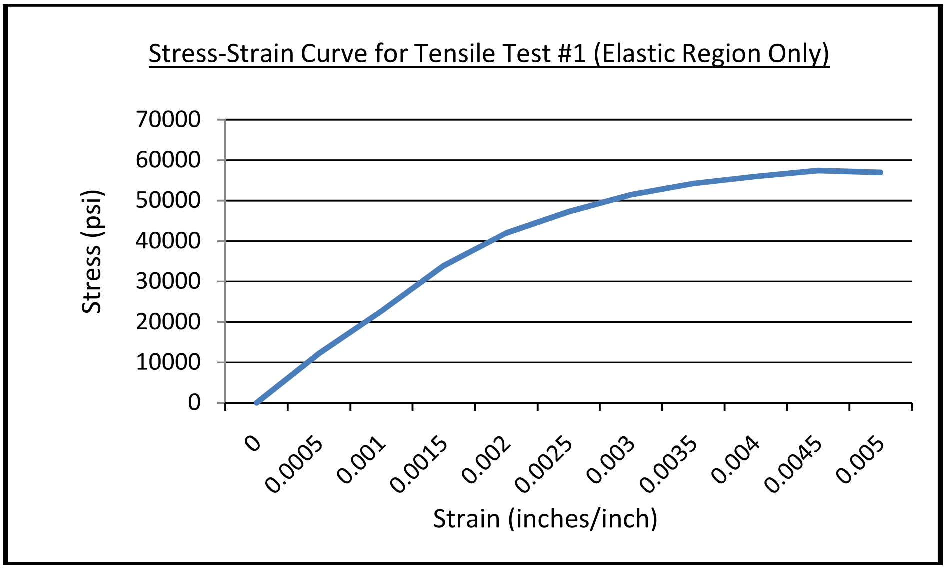


Figure 17: Stress-strain curve for Tensile Test #1.

The load-position curve for this test is given in

Figure 18.

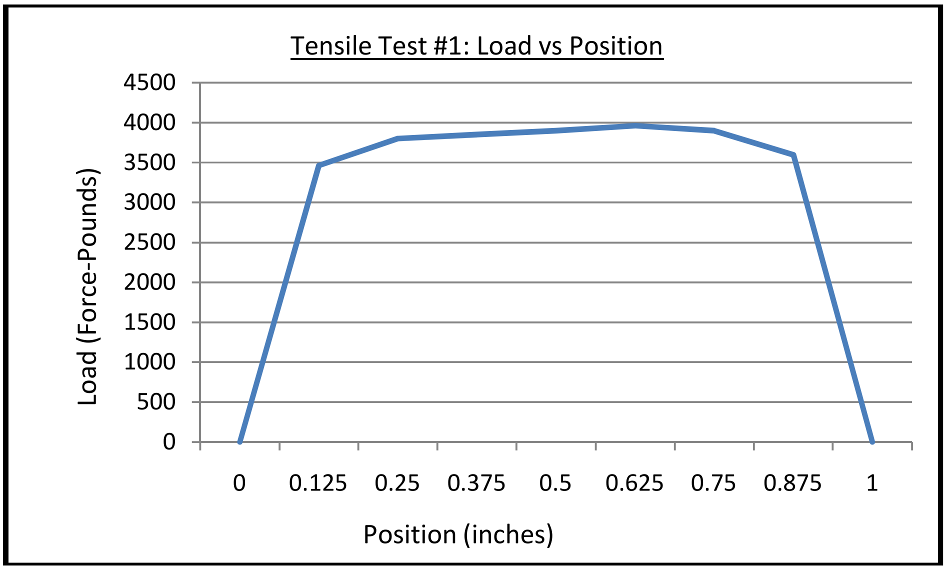


Figure 18: Load versus position graph for Tensile Test #1.

The fracture pattern of tensile bar #1 as well as a close-up of the break can be seen in Figure 19 and Figure 20, respectively.

Figure 19: Tensile bar #1 after testing.

Figure 20: Close-up of tensile bar #1 after testing.

Tensile Bar #2 Test

The dimensions of the first sample's reduced area were measured by caliper to be 0.503 inches by 0.115 inches resulting in an area of 0.0578 square inches that was subjected to tensile forces in the test. The sample was subjected to a load of 400 pounds per minute until failure. The material properties determined from the experiment are summarized in Table 3.

Table 3: Material properties obtained in Tensile Test #2.

Summary of Properties: Tensile Test #2	
Peak Load	3,905 pounds
Ultimate Tensile Strength	67,561 psi
Yield Strength	58,010 psi
Elongation	29.4%

The stress-strain curve for the portion of the test measured with the extensometer (measuring yielding in the elastic region alone) is given in Figure 21.

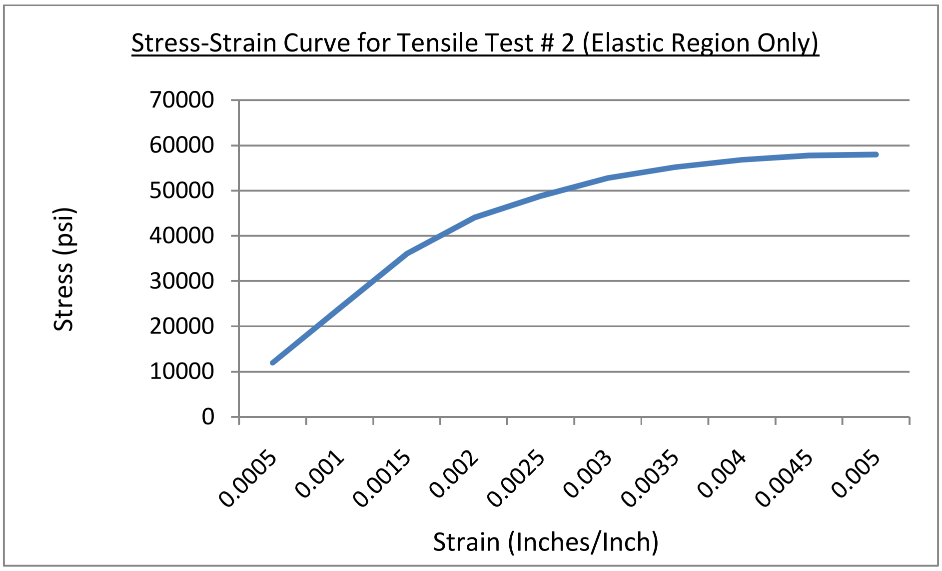


Figure 21: Stress-strain curve for Tensile Test #2.

The load-position curve for this test is given in Figure 22.

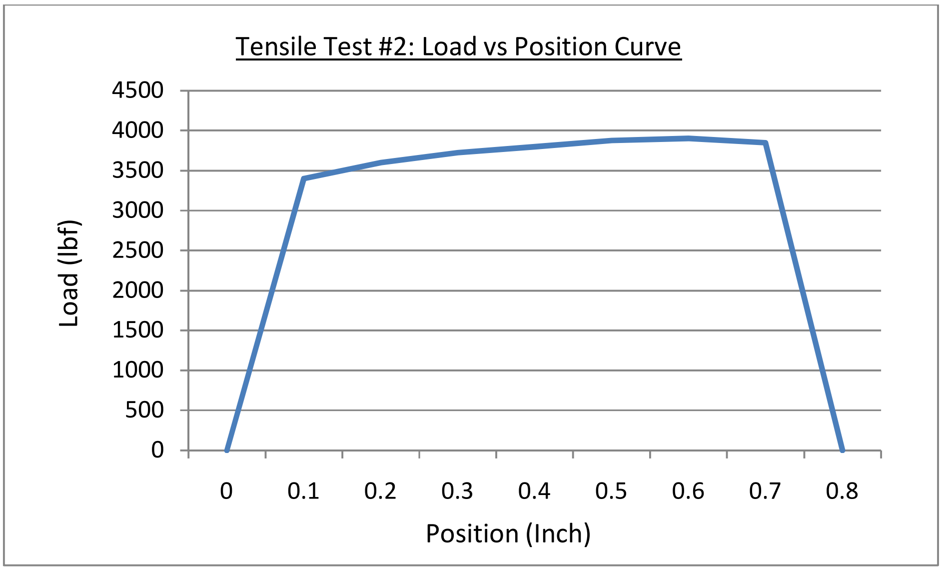


Figure 22: Load versus position graph for Tensile Test #2.

The fracture pattern of tensile bar #2 as well as a close-up of the break can be seen in Figure 23 and Figure 24, respectively.

Figure 23: Tensile bar #2 after testing.

Figure 24: Close-up of tensile bar #2 after testing.

Tensile Bar #3 Test

The dimensions of the first sample's reduced area were measured by caliper to be 0.503 inches by 0.115 inches resulting in an area of 0.0578 square inches that was subjected to tensile forces in the test. The sample was subjected to a load of 400 pounds per minute until failure. The material properties determined from the experiment are summarized in Table 4.

Table 4: Material properties obtained in Tensile Test #3.

Summary of Properties: Tensile Test #3	
Peak Load	3,873 pounds
Ultimate Tensile Strength	60,610 psi
Yield Strength	53,600 psi
Elongation	29.9%

The stress-strain curve for the portion of the test measured with the extensometer (measuring yielding in the elastic region alone) is given in Figure 25.

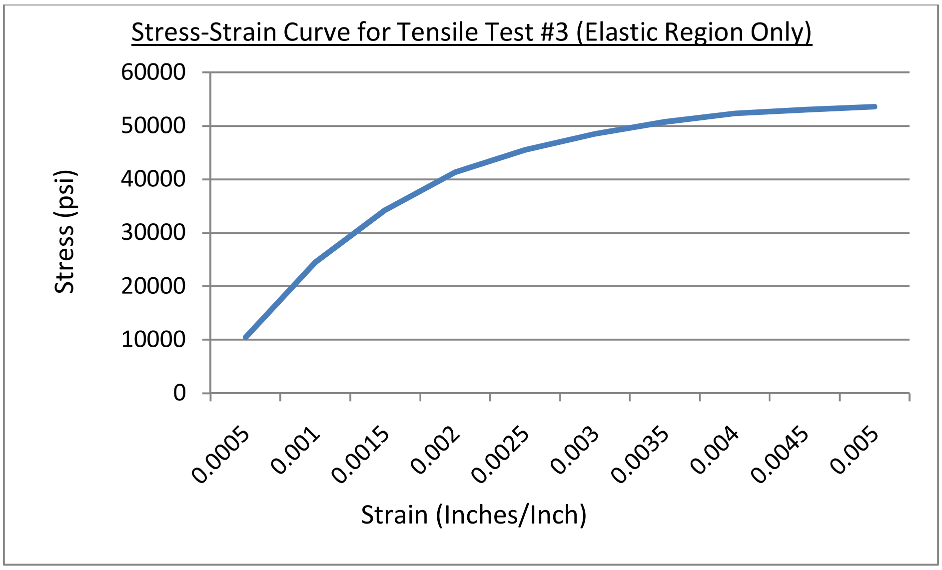


Figure 25: Stress-strain curve for Tensile Test #3.

The load-position curve for this test is given in Figure 26.

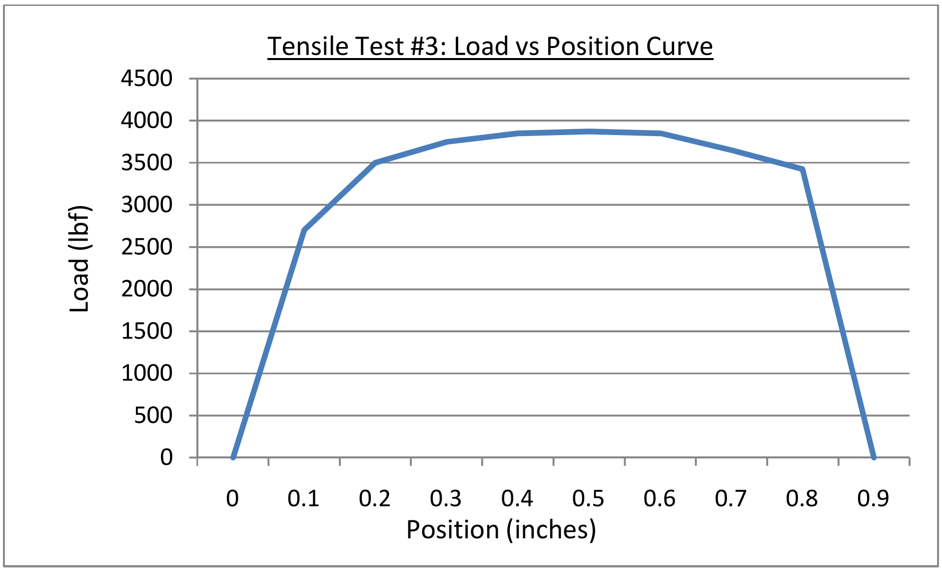


Figure 26: Load versus position graph for Tensile Test #3.

The fracture pattern of tensile bar #3 as well as a close-up of the break can be seen in Figure 27 and Figure 28, respectively.

Figure 27: Tensile bar #3 after testing.

Figure 28: Close-up of tensile bar #3 after testing.

Summary of Tensile Bar Results

A summary of the testing results for all three tensile tests is presented below in Table 5.

Table 5: Summary of Tensile Test results.

Property	Minimum by ASTM Spec	Average Result
Tensile Strength (psi)	58,000	64,656
Yielding Strength (psi)	46,000	56,367
Elongation (%)	23	30.5

As shown in Table 5, the material is considerably stronger than the minimum allowable stresses specified by ASTM E8 Material Testing Specification. The average yield strength in particular was over 56,000 psi as opposed to the minimum yield stress of 46,000 psi (i.e., more than 20 percent higher) shows that the tubes are made of adequate material.

Quasi-Static Testing

Quasi-static testing was performed in order to obtain relevant information regarding yield stress and buckling points in trusses as well as to prepare for the initial condition of dynamic testing, which governs the design of submarines. All quasi-static tests were performed in the Tinius Olsen Testing Machine. Any changes to the typical test setup are documented in the respective sections below.

Column Static Test

Quasi-static testing began with an axial compression test on a single 24-inch long column in order to determine the buckling load of a column and to ensure all necessary data could be obtained using the test setup before beginning the tests on the frames. The AISC equations governing buckling stress are Equations E3-2 and E3-3 (Chapter E, Steel Construction Maunal, 2005). The slenderness ratio of a 24 inch long HSS 2 x 2 x 1/8 is 31.5, which puts it in the criteria of AISC Equation E3-2. A summary of theoretical calculations is shown below in Table 6, using the average yielding stress from the tensile bar tests (F_y=56,367 psi from Table 5).

Table 6: Theoretical calculations of tested steel members.

Property	Calculated Value (AISC)
Critical Buckling Stress (F_{cr})	51,929.48 psi
Nominal Strength (P_n)	43,620.76 lbs
Factored Allowable Load ($\varphi_c P_n$)	39,258.68 lbs

The member was placed vertically with the top end free to rotate and the bottom end held place by a ½” thick sheet of plywood with a 2” x 2” square cut in the center to keep the column from sliding along the main table or kicking out during testing, these end conditions correspond to a theoretical K value of 0.7, but based on the EB design criteria a more conservative K value of 1.0 was used. The sample was then loaded axially at a compression rate of 4,000 lbs per minute until the column buckled. Results of this test are given in Table 7.

Table 7: Column static test data.

Mechanical Property	***Test Result***
Peak Load (lbf)	51,233
Compressive Strength (psi)	57,097
Initial Length of Column (in)	24
Final Length of Column (in)	23.75

The standard yielding stress for the material used in the columns (ASTM A500 Grade B) is 46,000 psi, and using a conservative effective length factor (K) of 1.0 and the AISC equations for column buckling (Equation E3-2), this yielding stress would provide a buckling load of 36,138 lbs for a 24 inch column. Using the yield stress found in the tensile tests (F_y=56,367 psi) the buckling load would be calculated as 43,620.76 lbs for a 24 inch column. So using the theoretical yielding stress of 46,000 psi, this column turned out to take 15,095 more lbs (30% more weight). Using the experimental yielding stress of 56,367 psi, the column took 7,612 lbs more (15% more weight). The load versus position diagram for the test is presented in Figure 29.

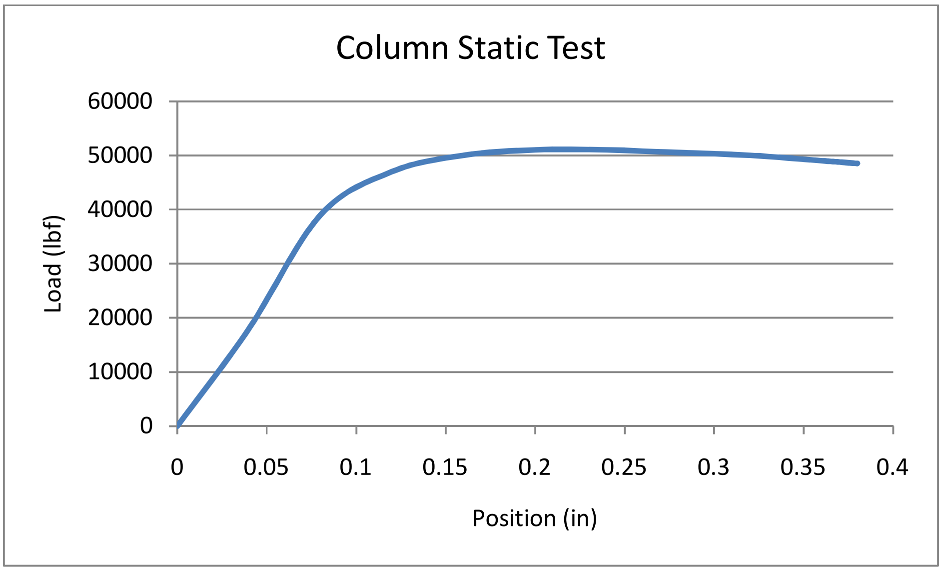


Figure 29: Load versus position graph for column static test.

Figure 30 is a picture of the single column after the test showing the deformation in the column.

Figure 30: Result from column static test.

The deformation in the column was very pronounced along with reasonably well-defined inflection points a couple of inches from each end, which showed some promise with the end condition theory for buckling. The loading was also reassuring because the compressive strength was only slightly below the ultimate failure strength for the material, thus the failure was due to mechanical failure in buckling rather than material failure. Lastly, this test produced useful information for the quasi-static testing of the frames since it provided an available peak load that each column can withstand and displayed success in the test setup to obtain the necessary data in the static testing of the frames. The actual buckling load taken by the column in this test was at least 15% greater than one would expect through theoretical calculations of buckling loads.

Column Static Test (Longer)

A second column of 36" in length was subjected to axial compression in order to compare the behavior of it to the 24" column. The AISC equations governing buckling stress are labeled by Equations E3-2 and E3-3 in the background. The slenderness ratio of a 36 inch long HSS 2 x 2 x 1/8 is 47.31, which puts it in the criteria of AISC Equation E3-2. A summary of theoretical calculations is shown below in Table 8, using the average yielding stress from the tensile bar tests (F_y=56,367 psi from Table 5).

Table 8: Theoretical calculations of tested steel members (long).

Property	**Calculated Value (AISC)**
Critical Buckling Stress (F_{cr})	39.570 psi
Nominal Strength (P_n)	33,239.1 lbs
Factored Allowable Load ($\varphi_c P_n$)	35,434.2 lbs

The member was placed vertically with the top end free to rotate and the bottom end held in place by a ½" thick sheet of plywood with a 2" x 2" square cut in the center to keep the column from sliding along the main table or kicking out during testing. The sample was then loaded axially at a compression rate of 2,000 lbs per minute until the column buckled. Results of this test are given in Table 9.

Table 9: Column (Long) static test data.

Mechanical Property	***Test Result***
Peak Load (lbf)	52,282
Compressive Strength (psi)	58,266

Initial Length of Column (in)	36
Final Length of Column (in)	35.5

The standard minimum yield strength for the material used in the columns (ASTM A500 Grade B Steel) is 46,000 psi along with an ultimate strength of 58,000 psi. The load versus position diagram for the test is presented in Figure 31.

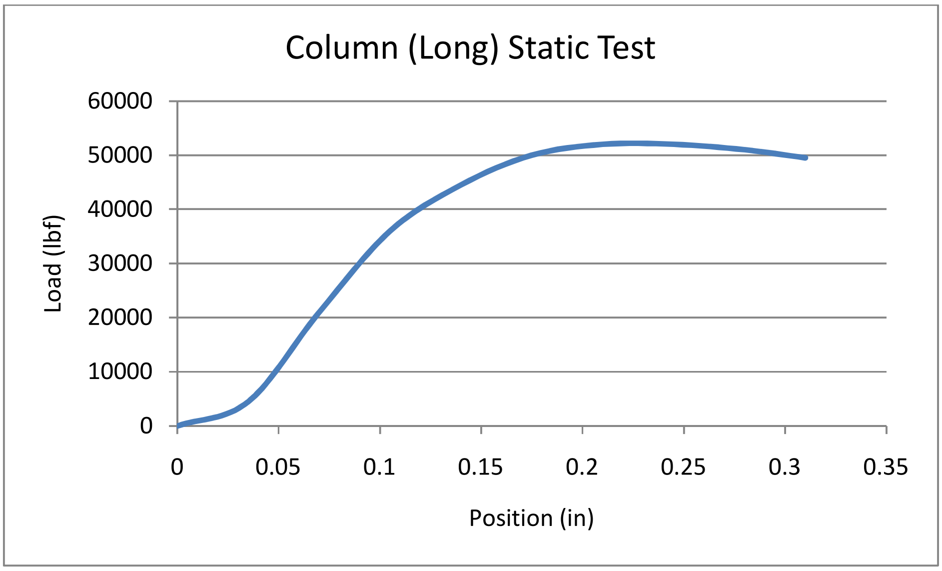


Figure 31: Load versus position graph for column (long) static test.

Figure 32 is a picture of the longer member after the test showing the deformation in the column.

Figure 32: Result from column (long) static test.

The deformation experienced in the longer column was also very noticeable along with reasonably well-defined inflection points a couple of inches from each end, which showed some promise with the end condition theory for buckling. The test reinforced the conclusions made during the first column test although a strange observation was made in that the load the column took before buckling is actually higher than that of the 24” column. By increasing the length of the column, one can normally assume that the axial compressive load that the member can take before buckling should decrease. Although no extensive research was performed on the reasoning, one factor that did change between both tests was the load rate (4,000 lb/min for the 24” column, 2,000 lb/min for the 36” column). This decrease in loading rate may have helped the column surpass the expected buckling strength.

Truss #1 Static Test (Symmetric)

The first frame was tested with the sample resting on roller supports on either end of the bottom chord centered underneath the outer vertical members. The truss was oriented such that the two diagonal members ran from the bottom corners to the top center of the frame. A steel I-beam was placed on the top chord in order to distribute the applied force. A 2”x2” piece of solid steel stock was then placed on top of the I-beam in order to aid in the distribution of the load. This set up and loading is summarized in Figure 33 along with number labels for each member.

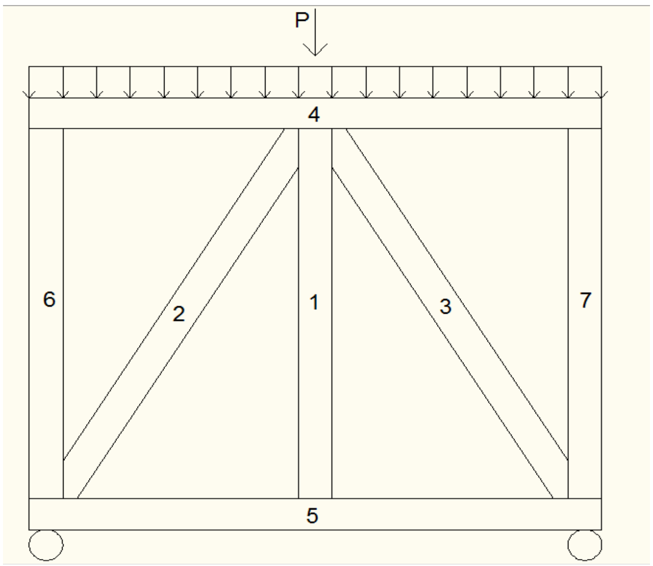


Figure 33: Truss setup for Static Test #1.

The sample was loaded on the center of the 2”x2” steel stock at a compression rate of 4,000 lbs per minute until the frame plastically deformed. The load versus position diagram for the test is presented in Figure 34.

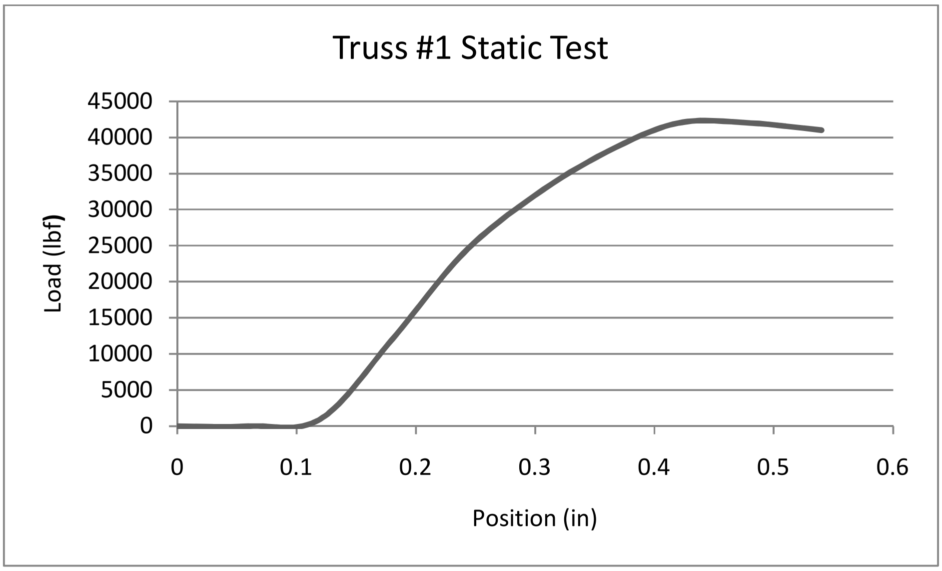


Figure 34: Load versus position graph for static test of Truss #1.

Figure 35 is a picture of part of the truss frame that was held up by a support roller after the test showing the local deformation that occurred.

Figure 35: Result from Truss #1 static test.

The first truss tested ended up failing at a much lower load than anticipated (i.e., the peak load was 42,611 pounds). The group was expecting to see deformation in the overall truss but instead, local buckling occurred first. The force applied to the truss was so large that the reactive point load forces experienced from the two roller supports locally deformed the specimen's bottom chord on both ends. The result was both unexpected and disconcerting, as a new setup had to be experimented with for future static truss tests. Aside from the local collapse at the loading points, the members of the truss remained unbent after the test and the sample was reused for a second trial, although it is labeled as "Truss #2" in this paper.

Truss #2 Static Test (Symmetric)

The second frame was tested with the sample resting on roller supports on either end of the bottom chord directly underneath the outer vertical members. The truss was oriented such that the two diagonal members ran from the top corners to the bottom center of the frame. The local deformation issue was addressed by inserting a solid piece of steel stock (1.75"x1.75"x4", henceforth known as a "plug") into either end of the bottom chord directly over the roller supports. By doing so, local deformation due to the reactive forces of the supports would be eliminated. A steel I-beam was placed on the top chord in order to distribute the applied force. A 2"x2" piece of solid steel stock was then placed on top of the I-beam in order to aid in the distribution of weight. This set up and loading is summarized in Figure 36 along with number labels for each member.

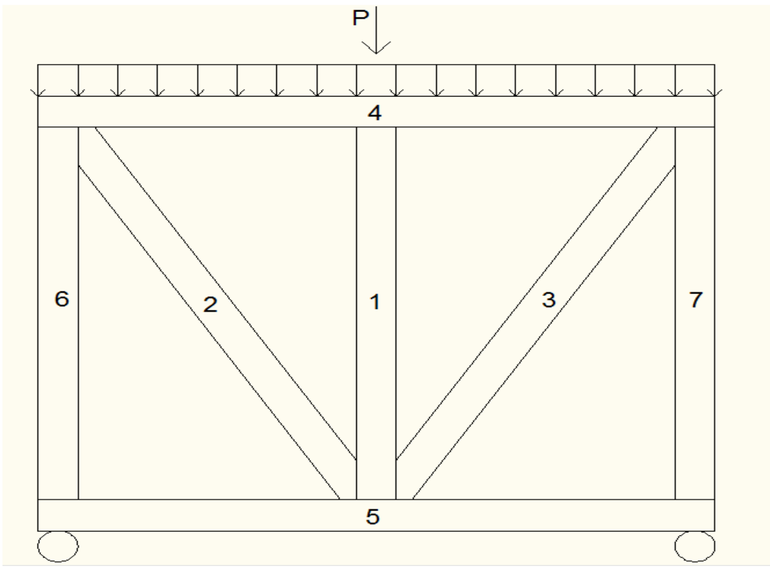


Figure 36: Truss setup for Static Test #2.

The sample was loaded on the center of the 2”x2” steel stock at a compression rate of 4,000 lbs per minute until the frame plastically deformed. The load versus position diagram for the test is presented in Figure 37.

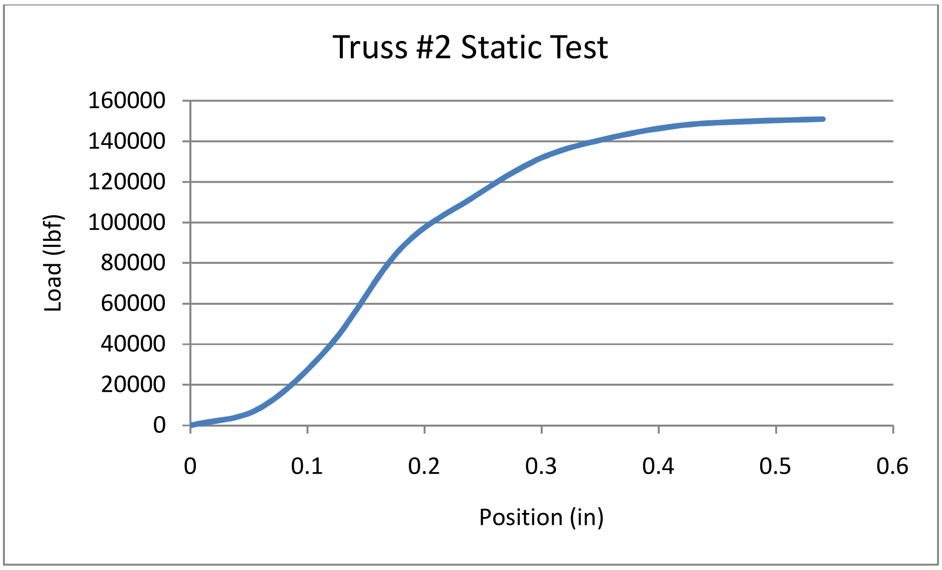


Figure 37: Load versus position graph for static test of Truss #2.

Figure 38 is a picture of the top center of the truss frame showing the local deformation that occurred.

Figure 38: Result from Truss #2 static test.

The truss was able to withstand much more static load with the plugs inserted inside the bottom chord. However, whether it was from the large amount of pressure being exerted or the fact that the top chord was previously deformed from the first test, the distributed load along the sample began to locally deform the horizontal top chord in the center of the truss after a peak loading of 76,382 pounds. Member buckling did not occur during this experiment which meant that the current setup of the test needed to be adjusted to take into account local deformation in the top chord.

Truss #3 Static Test (Symmetric)

The third frame was tested with the sample resting on roller supports on either end of the bottom chord directly underneath the outer vertical members. The truss was oriented such that the two diagonal members ran from the bottom corners to the top center of the frame. The local deformation issue was addressed by inserting three more plugs into either end of the top chord and one directly in the center of the member along with two in the bottom chord directly over the supports. By doing so, local deformation due to the reactive forces of the supports as well as deformation from the force applied by the applied load would be severely limited. A steel I-beam was placed on the top chord in order to distribute the applied force. A 2”x2” piece of solid steel stock was then placed on top of the I-beam in order to aid in the distribution of weight. This set up and loading is summarized in Figure 39 along with number labels for each member:

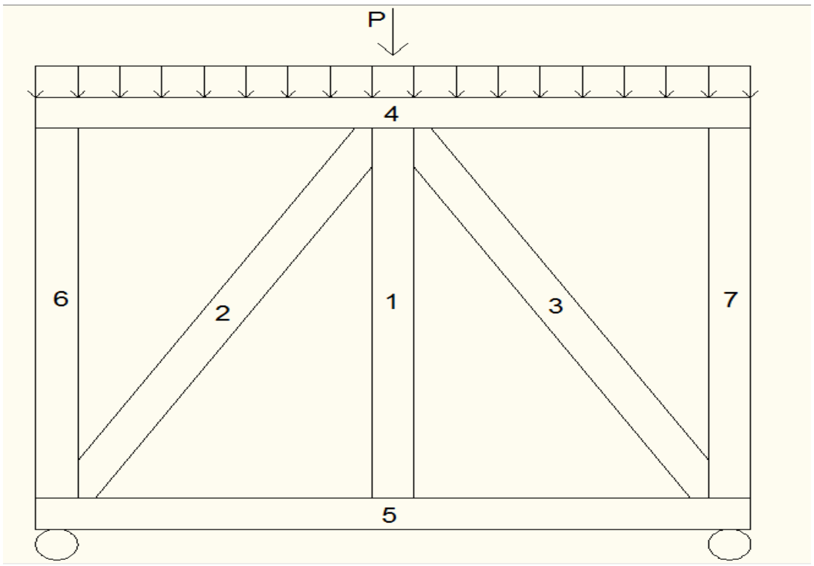


Figure 39: Truss setup for Static Test #3.

The sample was loaded on the center of the 2”x2” steel stock at a compression rate of 4,000 lbs per minute until the frame plastically deformed. Results of this test based on standard static truss analysis and further study of individual member forces are given in Table 10.

Table 10: Truss #3 static test data.

Member	***Resulting Force (Pounds)***	***Type of Axial Force***
Peak Load (for frame)	151,617	
1	0	N/A
2	47,380.31	Tension
3	47,380.31	Tension
4	0	N/A
5	28,428.19	Compression
6	37,904.25	Compression
7	37,904.25	Compression

The load versus position diagram for the test is presented in Figure 40.

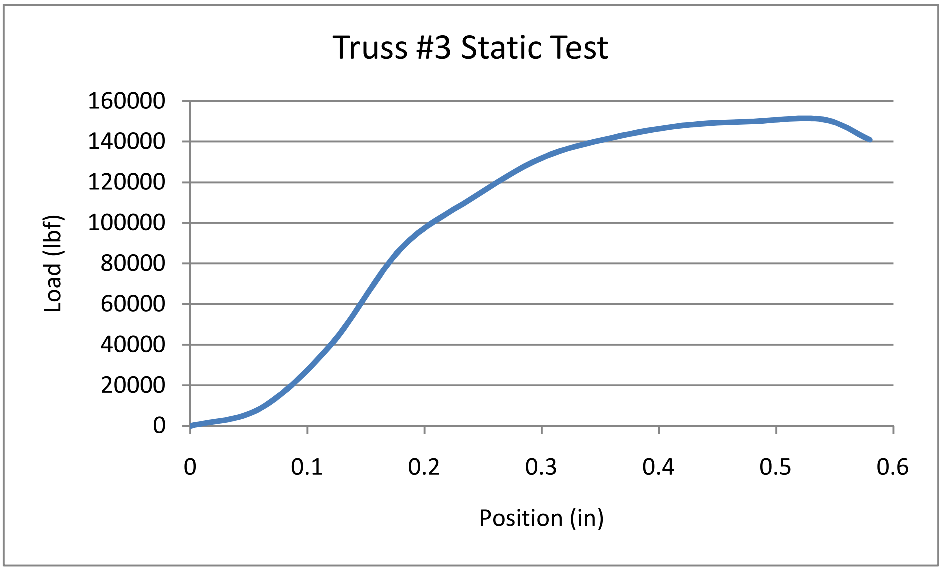


Figure 40: Load versus position graph for static test of Truss #3.

The third static truss test was the first of the truss tests to be labeled as a success. One of the outer vertical members buckled under the force with a final deformed length of about 16”. Both diagonal members were also slightly deformed, though not enough to make proper calculations on the pieces. All five plugs prevented local deformation; although slight dents were made into the steel, the plug prevented the cross-section from collapsing. A final image of the third static truss was not available, as it was considered scrap following testing and it was torn apart to retrieve the metal plugs in the top and bottom chords

Truss #4 Static Test (Symmetric)

The fourth frame was tested with the sample resting on roller supports on either end of the bottom chord directly underneath the outer vertical members. The truss was oriented such that the two diagonal members ran from the top corners to the bottom center of the frame. The local deformation issue was addressed by inserting plugs into the top and bottom chords, as described earlier. The steel I-beam distributing the load on the top chord was replaced with a 4”x4” piece of 1018-grade steel stock because the applied weight from previous tests began to deform the I-beam. A 2”x2” piece of solid steel stock was then placed on top of the 4”x4” steel stock in order to aid in the distribution of weight. This set up and loading is summarized in Figure 41 along with number labels for each member.

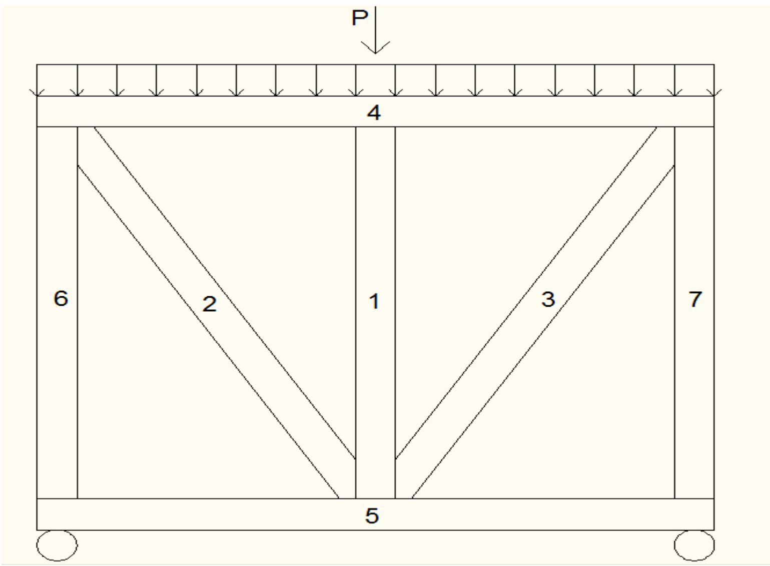


Figure 41: Truss setup for Static Test #4.

The sample was loaded on the center of the 2”x2” steel stock at a compression rate of 4,000 lbs per minute until the frame plastically deformed. Results of this test based on standard static truss analysis and further study of individual member forces are given in Table 11.

Table 11: Truss #4 static test data.

Member	***Resulting Force (Pounds)***	***Type of Axial Force***
Peak Load (for frame)	106,549	
1	53,274.5	Compression
2	33,296.56	Tension
3	33,296.56	Tension
4	19,977.94	Compression
5	0	N/A
6	53,274.5	Compression
7	53.274.5	Compression

The load versus position diagram for the test is presented in Figure 42.

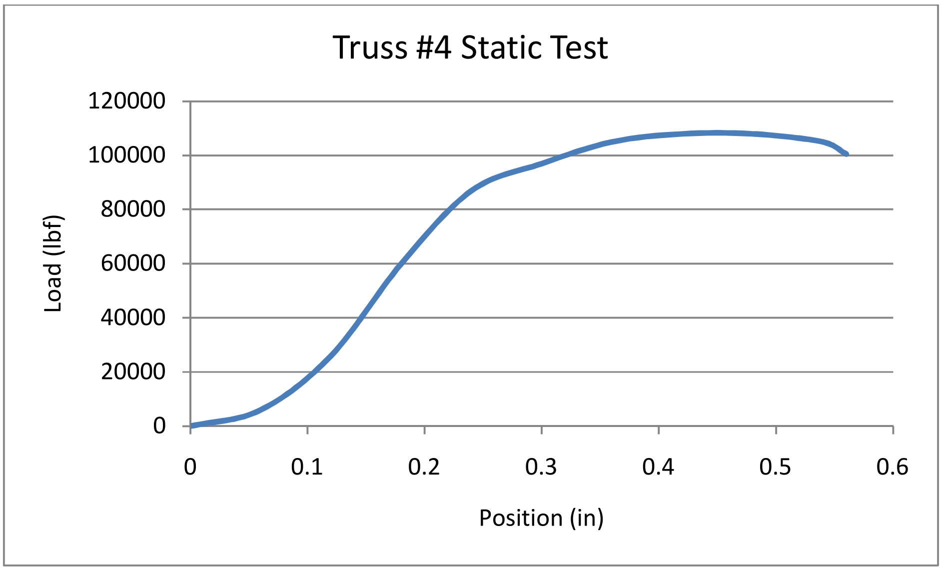


Figure 42: Load versus position graph for static test of Truss #4.

Figure 43 is a picture of the truss frame after the test showing the out-of-plane deformation that occurred.

Figure 43: Result from Truss #4 static test.

The first run of the fourth static test ended up in a failure of the I-beam, which acted as the distribution beam. The fact that the truss did not deform during this initial test allowed for the group to reconfigure the test setup while using the same truss. The I-beam was replaced with the 4”x4” 1018-grade steel bar of comparable length (as explained earlier) and the same truss was tested a second time with successful results. Buckling occurred slightly in the center column while one of the outer vertical members experienced the most plastic deformation.

Truss #5 Static Test (Symmetric)

The fifth frame was tested with the sample resting on roller supports on either end of the bottom chord directly underneath the outer vertical members. Two strain gauges were placed on opposite sides of the central vertical member to measure strain values and determine deformation locations. The truss was oriented such that the two diagonal members ran from the top corners to the bottom center of the frame. The local deformation issue was addressed by inserting one plug into the top chord in the center and two plugs in the bottom chord above each roller support. A 1" diameter roller was placed on the top chord approximately 2" from the center of the member such that it was lying down and running perpendicular to the top of the frame in order to simulate a moment. The sample was loaded on the roller at a compression rate of 4,000 lbs per minute until the force reached 30,000 lbf and strain values were obtained. Deformation did not occur.

The roller was then moved to the center of the truss where the loading was applied once again. This set up and loading is summarized in Figure 44 along with number labels for each member:

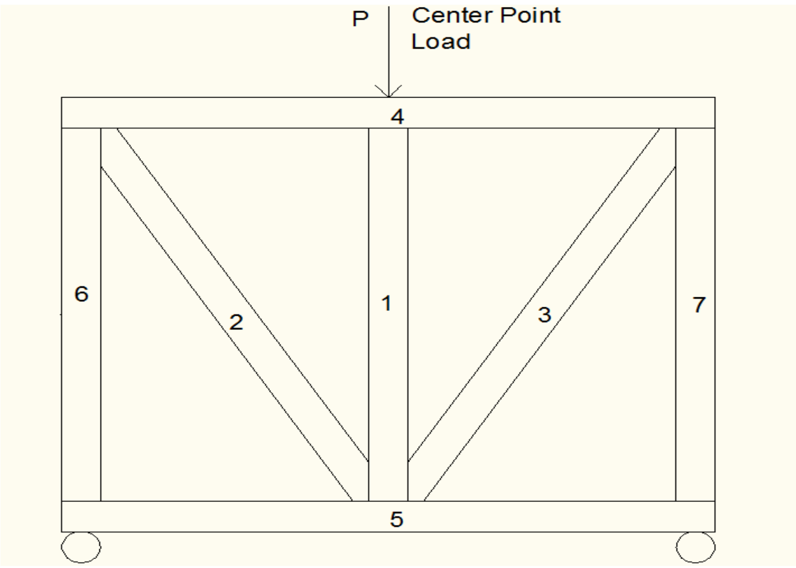


Figure 44: Truss setup for Static Test #5.

Figure 45 shows the placements of the strain gauges for the test.

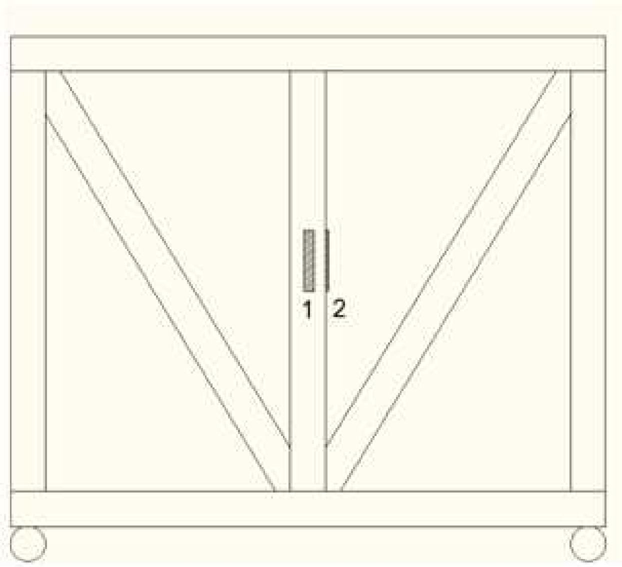


Figure 45: Strain gauge placement on Static Truss #6.

The sample was loaded on the roller at a compression rate of 4,000 lbs per minute until the frame plastically deformed. Results of this test based on standard static truss analysis and further study of individual member forces are given in Table 12.

Table 12: Truss #5 static test data.

Member	***Resulting Force (Pounds)***	***Type of Axial Force***
Peak Load (for frame)	53,278	
1	53,278	Compression
2	33,298.75	Tension
3	33,298.75	Tension
4	19,979.25	Compression
5	0	N/A

6	26,639	Compression
7	26,639	Compression

It is important to note that the center column (24 inches in length) is expected to buckle at 43,620.76 lbs calculated previously in the Static Column section based on the AISC buckling equations and the average yielding stress of 56,367 psi found in the tensile tests. The load that the column withstood in this test as part of the truss system was 53,278 lbs before buckling. This is 18% higher than expected.

The load versus position diagram for the test is presented in Figure 46.

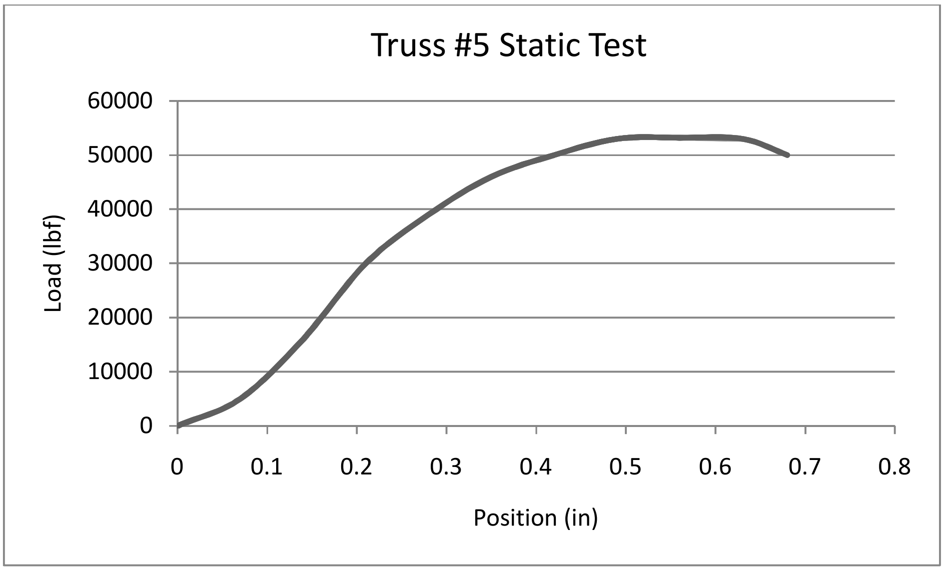


Figure 46: Load versus position graph for static test of Truss #5.

Figure 47 is a picture of the truss frame after the test showing the deformation that occurred.

Figure 47: Result from Truss #5 static test.

A graph of the strain gauge readings can be seen in Figure 48.

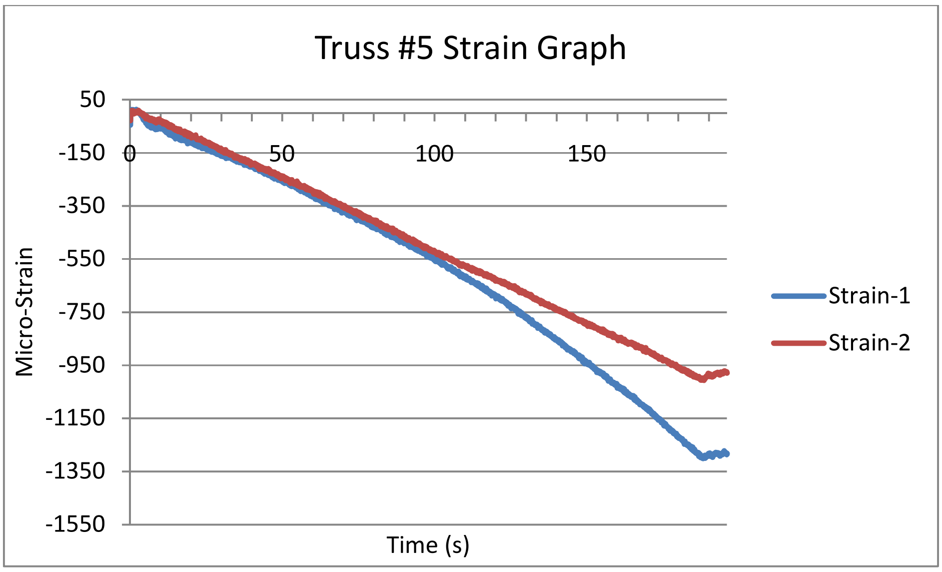


Figure 48: Micro-strain versus time graph for static test of Truss #5.

This strain gauge data is correlated with the stresses of the member by use of Hooke's Law to calculate the peak stress in each member face in Table 13 below as referenced in Figure 45.

Table 13: Peak stress on members based on strain gauge measurements.

Strain Gauge Position	**Stress (psi)**
1	36,975
2	28,275

Every test up until this point had involved distributed loads so the group wanted to experiment with how the truss would act under a point load. Not only would the setup provide information on static moments on the center column due to the 2" offset described earlier, but it also would set a benchmark to aim for when the dynamic point load tests began. Overall, the test led to successful results with local deformation occurring in both the top chord and buckling in the center column. The strain gauges appear to have worked very well and provided reasonable values given that some of the force is being supported by the two outer members (though very little) and through deflection of the top chord.

Truss #6 Static Test (Symmetric)

The sixth frame was tested with the sample resting on roller supports on either end of the bottom chord directly underneath the outer vertical members. Four strain gauges were placed on the truss: two on opposite sides of the central vertical member and two on the top of either side of the top chord, all to measure strain values and determine deformation locations. The truss was oriented such that the two diagonal members ran from the top corners to the bottom center of the frame. The local deformation issue was addressed by inserting one plug into the top chord in the center and two plugs in the bottom chord above each roller support. A 1" diameter roller was placed in the center of the top chord such that it was lying down and running perpendicular to the top of the frame. This set up and loading is summarized in Figure 49 along with number labels for each member:

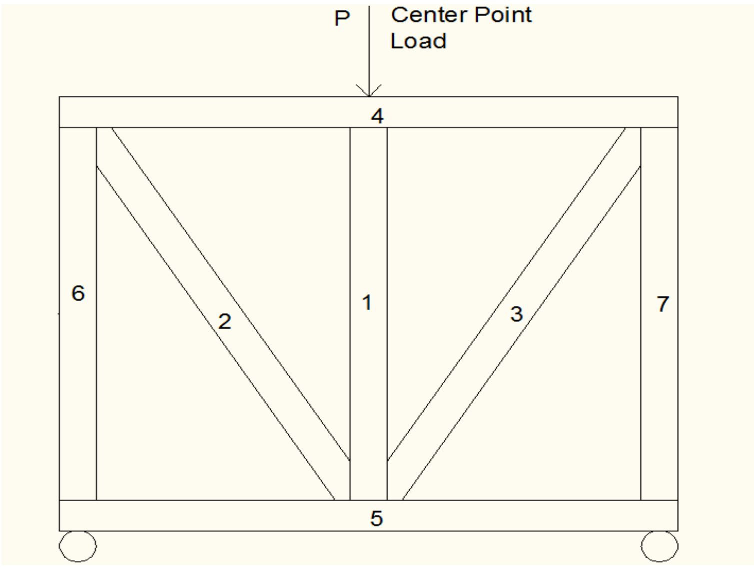


Figure 49: Truss setup for Static Test #6.

Figure 50 shows the placements of the strain gauges for the test.

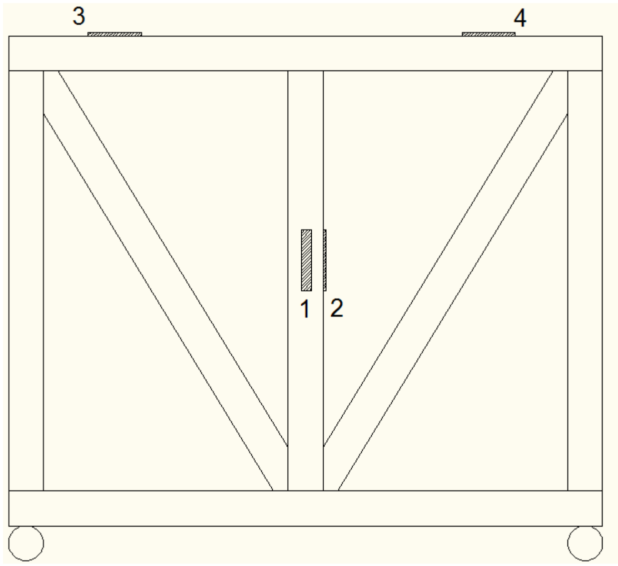


Figure 50: Strain gauge placement on Static Truss #6.

The sample was loaded on the roller at a compression rate of 4,000 lbs per minute until the center vertical member of the frame plastically deformed. Results of this test based on standard static truss analysis and further study of individual member forces are given in Table 14.

Table 14: Truss #6 static test data.

Member	***Resulting Force (Pounds)***	***Type of Axial Force***
Peak Load (for frame)	57,126	
1	57,126	Compression
2	35,703.75	Tension
3	35,703.75	Tension
4	21,422.25	Compression
5	0	N/A

6	28,563	Compression
7	28,563	Compression

The load versus position diagram for the test is presented in Figure 51.

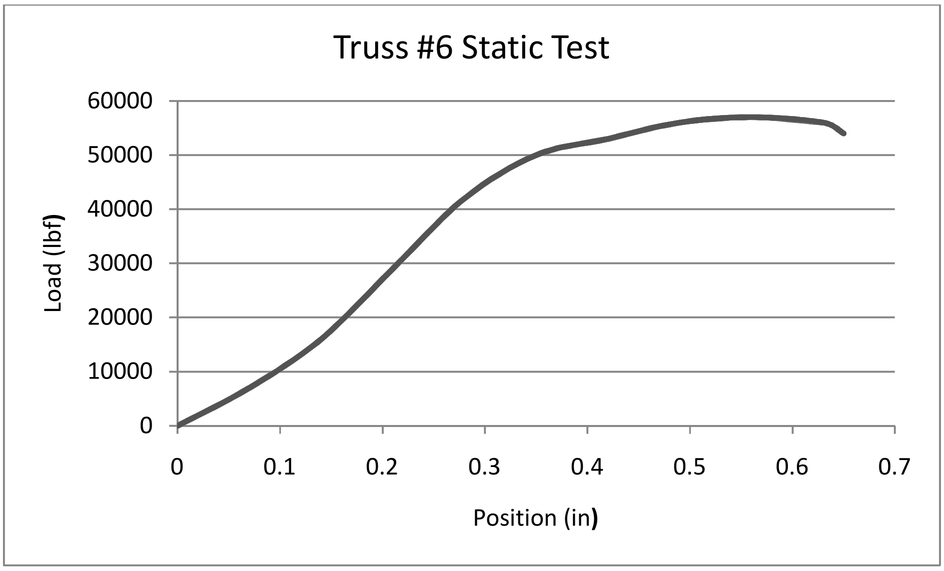


Figure 51: Load versus position graph for static test of Truss #6.

Figure 52 is a picture of the truss frame after the test showing the deformation that occurred.

Figure 52: Result from Truss #6 static test.

Similar to the previous sample, the test was ultimately very successful with deformation occurring in both the top chord and the central column. A graph of the strain gauge readings can be seen in Figure 53.

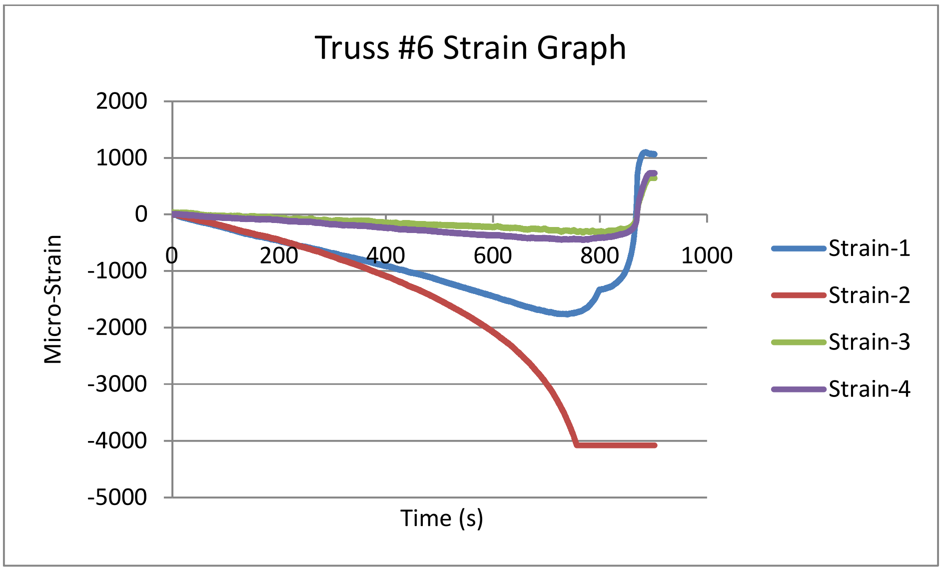


Figure 53: Micro-strain versus time graph for static test of Truss #6.

This strain gauge data is correlated with the stresses of the member by use of Hooke's Law to calculate the peak stress in each member face in Table 15 below as referenced in Figure 50.

Table 15: Peak stress on members based on strain gauge measurements

<u>**Strain Gauge Position**</u>	<u>**Stress (psi)**</u>
1	52,200
2	118,900
3	20,300
4	21,025

The peak load for the center column of the overall truss was calculated to be 57,126 lbs shown in Table 14, which correlates to a peak stress of 75,166 psi. For that column, strain gauges 1 and 2 were placed and upon failure, measured strains correlating to 50,750 and 116,000 psi, respectively. While the data from strain gauge 2 was considerably skewed due to failure of the strain gauge itself during testing, the jump in strain for the internal face of the member (internal meaning not on the plane of the truss sketch) displays important properties of the buckling behavior. What this shows is that while theory would tell us that the load delivers 75,166 psi to the center column itself, the column compressed on the face of strain gauge 2 during buckling and compensated for the decreased stress on the surface of strain gauge 1. Strain gauge 1 displayed less stress than expected and went directly into tension, showing two things: one that the columns buckled out towards this surface, and two that the face of the column that buckles out does not receive as much of the buckling load as the compression face.

The top chord displays expected results, as the top chord itself did not fail each half of the chord measured stresses of approximately 14,500 psi. This shows that while the truss failed in the center column, the top chord still took on more than calculated, as the total stress in the chord turns out to be around 29,000 psi, this is extremely close to the value that you would expect in the top chord of 28,187 psi (based on the load of 21,422.25 lbs and the area of 0.76 square inches).

This correlates with the failure mode of the truss shown in Figure 52, because the center column is the column that buckled the most, and it buckled in two directions. This makes sense as it compressed the face with strain gauge 2 and put the side with strain gauge 1 into tension as it buckled.

Truss #7 Static Test (Asymmetric)

The seventh frame was tested with the sample resting on roller supports on either end of the bottom chord directly underneath the outer vertical members. Two strain gauges were placed on the truss on opposite sides of the central vertical member to measure strain values and determine deformation locations. The truss was oriented such that the two diagonal members ran from the top corners to the bottom center of the frame. The local deformation issue was addressed by inserting one plug into the top chord in the center and two plugs in the bottom chord above each roller support. A 1” diameter roller was placed above the middle vertical member along the top chord such that it was lying down and running perpendicular to the top of the frame. This set up and loading is summarized in Figure 54 along with number labels for each member:

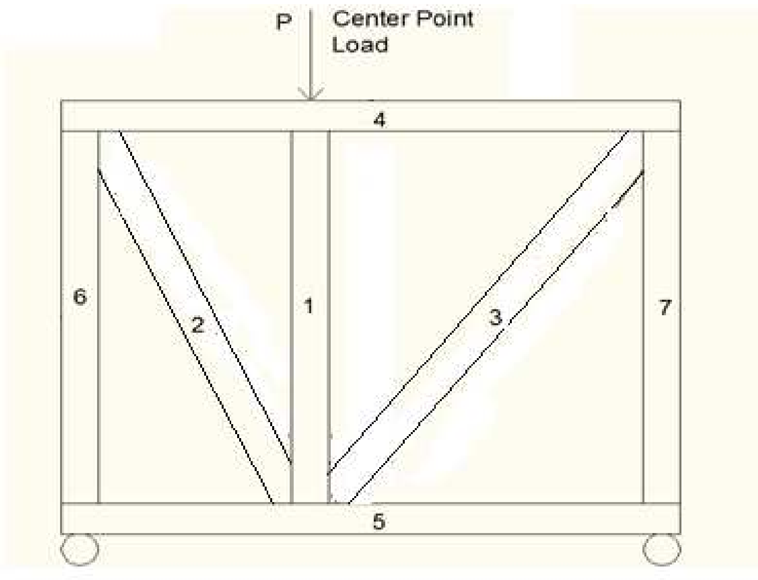


Figure 54: Truss setup for Static Test #7.

Figure 55 shows the placements of the strain gauges for the test.

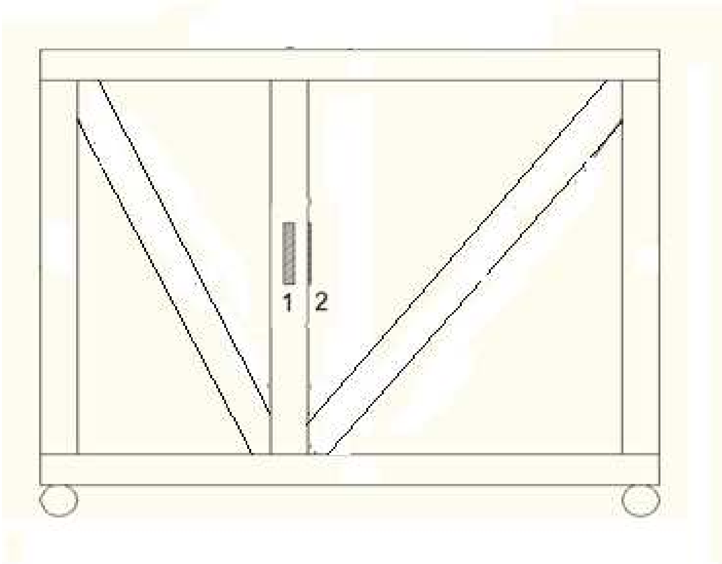


Figure 55: Strain gauge placement on Static Truss #7.

The sample was loaded on the roller at a compression rate of 4,000 lbs per minute until the middle vertical member of the frame plastically deformed. Results of this test based on standard static truss analysis and further study of individual member forces are given in Table 16.

Table 16: Truss #7 static test data.

Member	***Resulting Force (Pounds)***	***Type of Axial Force***
Peak Load (for frame)	60,222	
1	60,222	Compression
2	26,723.33	Tension
3	22,769.82	Tension
4	22,583.25	Compression

5	0	N/A
6	30,111	Compression
7	30,111	Compression

The load versus position diagram for the test is presented in Figure 56.

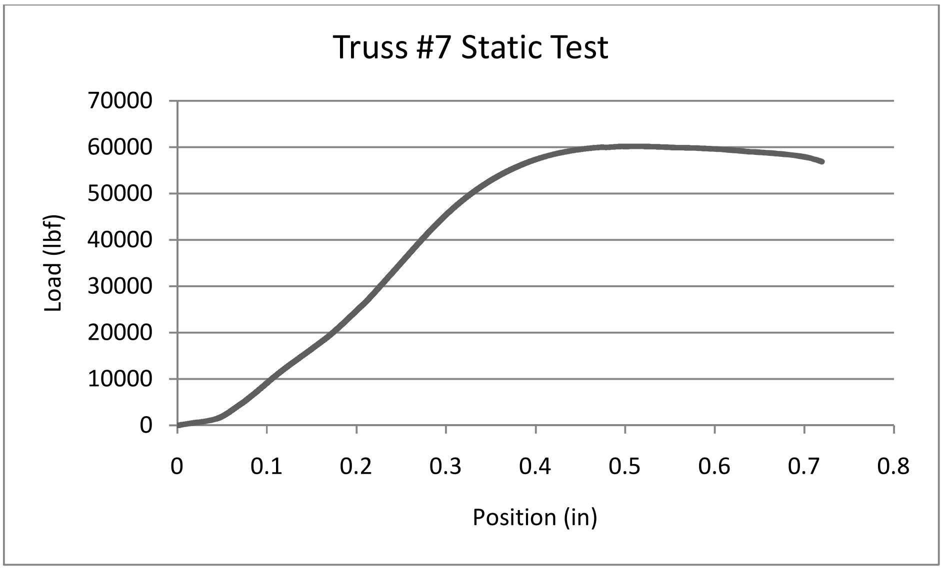


Figure 56: Load versus position graph for static test of Truss #7.

Figure 57 is a picture of the truss frame after the test showing the deformation that occurred.

Figure 57: Result from Truss #7 static test.

Similar to the previous sample, the test was ultimately very successful with deformation occurring in both the top chord and the central column. A graph of the strain gauge readings can be seen in Figure 58.

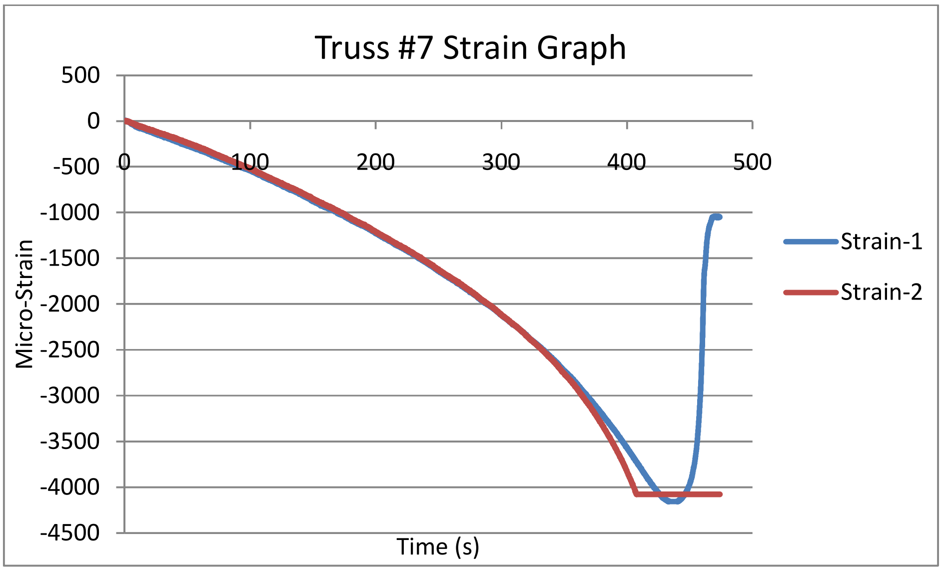


Figure 58: Micro-strain versus time graph for static test of Truss #7.

This strain gauge data is correlated with the stresses of the member by use of Hooke's Law to calculate the peak stress in each member face in Table 17 below.

Table 17: Peak stress on members based on strain gauge measurements

Strain Gauge Position	**Stress (psi)**
1	121,800
2	118,900

The asymmetric truss behaved as expected. Like the previous test, the second strain gauge, which was measuring the side of the middle vertical column, failed near the end of the test due to bending past its capacity. Both strain gauges read compression throughout the duration of the test, implying that the column bent diagonally outward away from the gauges. Strain gauge 1 read considerably more strain then in the previous test which leads to the assumption that the column was able to bend easier due to a smaller weld area from the diagonals where the middle column meets the two diagonal members and the bottom chord. Since the frame was loading directly over the middle column (even though it was asymmetric), the outer vertical members were hardly affected at all. The two diagonal bent slightly during the testing process but appeared to return to their original shape once the load was taken off.

Dynamic Testing

Dynamic testing was performed in order to determine how steel members and trusses would react to impact loads and how they would differ from the information obtained from the quasi-static tests. All tests were performed in a nine-foot drop tower of varying masses. Any changes to the typical test setup were documented in the respective sections below.

The conditions in which the following tests were designed were based on the quasi static calculations and test results from the preceding tests. For columns, the quasi-static result was a failure load of 51,233 lbs with a deflection of approximately 0.2 inches. Using the conservation of energy laws and a drop height of 5 feet after the column and size of the weights were taken into account, a weight of 170.8 pounds was estimated to be required in weight to reach the static failure load. The truss calculations were made using the same process except with a deflection of 0.5 inches and an average failure load of 57,126 lbs based on quasi-static testing, which yielded required weight of 476.6 pounds.

Column #1 Dynamic Test

Dynamic testing began with an axial impact test on a single 24 inch long column in order to determine the dynamic buckling load of a column due to impact forces and to ensure all necessary data could be obtained using the test setup before beginning the tests on the frames. The member was placed vertically with the top end free to rotate and the bottom end held in place by a ½” thick steel plate with a 2” x 2” square cut in the center to keep the column from sliding along the main table or kicking out during testing. These end conditions are consistent with a K value of 0.7 but our calculations for required load are based on a more conservative estimate of 1.0, in order to compare the differences. The sample was then impacted by masses dropped from the tower. Accelerometers were placed on the drop tower head to measure the deceleration at impact. Data was unfiltered and no padding was used. Results of this test are given in Table 18. Calculated values for kinetic energy and force on impact are based on Conservation of Energy, mass being dropped, drop height, and gravity. The force on impact provided is based on the measured acceleration at impact from the 10,000-G accelerometer. Calculations of strain energy are based on the provided force on impact from the accelerometer and a very low deflection on impact (~0.05 inches).

Table 18: Column dynamic test #1 data.

Mechanical Property	***Test Result***
Drop Height (in)	60 +/- 4
Weight Dropped (lbs)	150
Max Impact Acceleration for 10,000G (G’s)	≈675
Max Impact Acceleration for 50,000G (G’s)	≈925
Calculated Kinetic Energy (ft-lbf)	750
Calculated Force on Impact (lbf)	45,000
Force at Impact – 10,000G (lbf)	101,202.96
Strain Energy (based on Impact Force) (ft-lbf)	292.6

The graph displaying measurements taken from the 50,000G and 10,000G accelerometers for the test is presented in Figure 59.

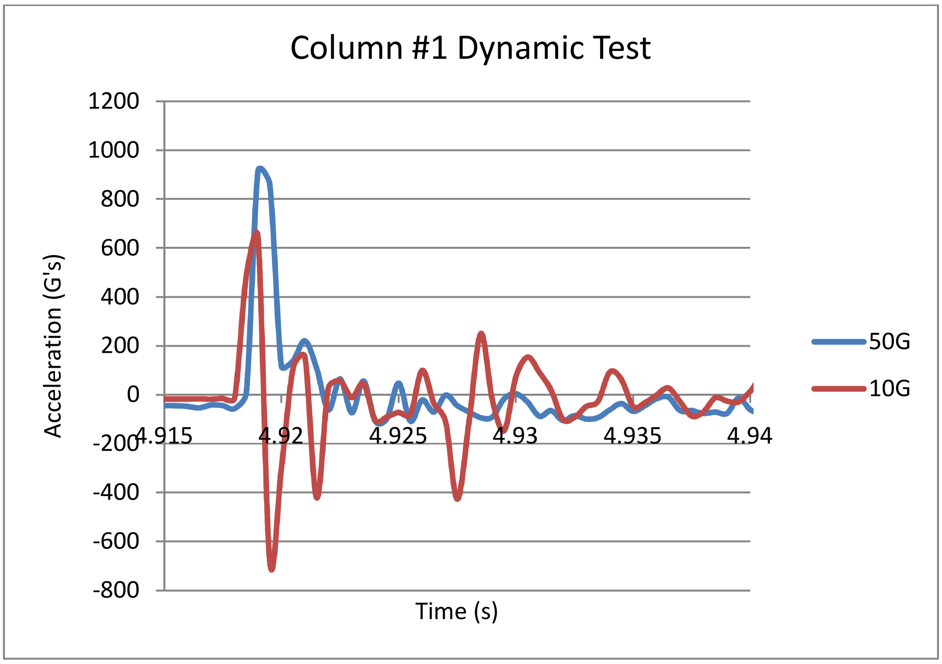


Figure 59: Acceleration versus time graph for column dynamic test #1.

Figure 60 is a picture of column 1 after dynamic testing. The image shows the column with a slight bend due to the impact load and shows a deflection of about ½".

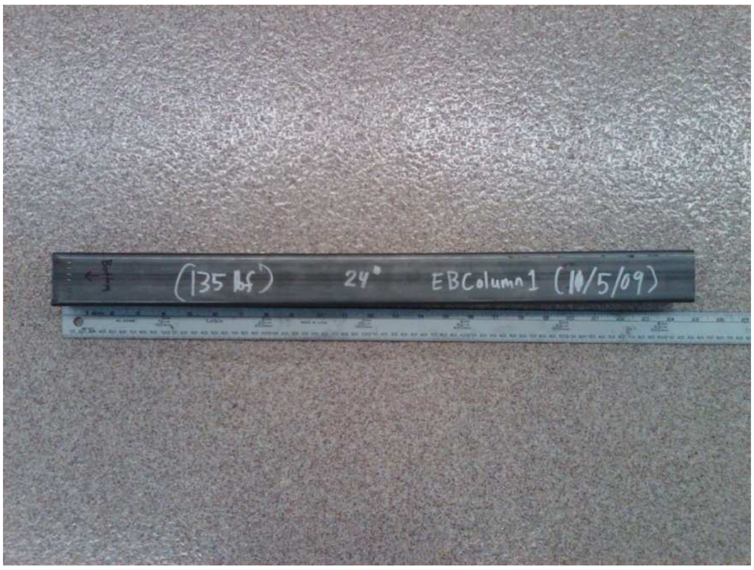


Figure 60: Result from Column #1 dynamic test.

For this test a mass of 150 lbs was dropped in accordance with the assumption that the mass required to buckle the column is only slightly higher at 170 lbs. However, when the column was dropped on it displayed little to no deflection at all, as the measured deflection was less than 1/10 inch. While the instantaneous load on impact was extremely large at 101,202.96 lbs, the small deflection of the column caused the strain energy of the impact to be a mere 292.6 ft-lbs, considerably less than the calculated kinetic energy required of 750 ft-lbs. So while the load is very high on impact, the material properties minimize deflection and the energy provided is insufficient for buckling at this drop weight. In the next test the mass dropped will be brought up above the calculated mass required (170 lbs) to 180 lbs in order to observe any changes in the relationship between energy and dynamic buckling.

Column #2 Dynamic Test

Dynamic Test #2 involved an axial impact test on a single 24 inch long column in order to determine the dynamic buckling load of a column due to impact forces and to ensure all necessary data could be obtained using the test setup before beginning the tests on the frames. The member was placed vertically with the top end free to rotate and the bottom end held in place by a ½" thick steel plate with a 2" x 2" square cut in the center to keep the column from sliding along the main table or kicking out during testing. The sample was then impacted by masses dropped from the tower. The amount of weight used in the tower was increased from the first two dynamic column tests. Accelerometers were placed on the drop tower head to measure the deceleration at impact. Results of this test are given in Table 19.

Table 19: Column dynamic test #2 data

Mechanical Property	***Test Result***
Drop Height (in)	*60 +/- 4*
Weight Dropped (lbs)	*180*
Max Impact Acceleration (G's)	*≈930*
Calculated Kinetic Energy (ft-lbf)	*900*
Calculated Force on Impact (lbf)	*54,000*
Force at Impact – 10G (lbf)	*167,322.23*
Strain Energy (from Impact Force) (ft-lbf)	*697.2*

The graph measuring the 10,000G accelerometer for the test is presented in Figure 61.

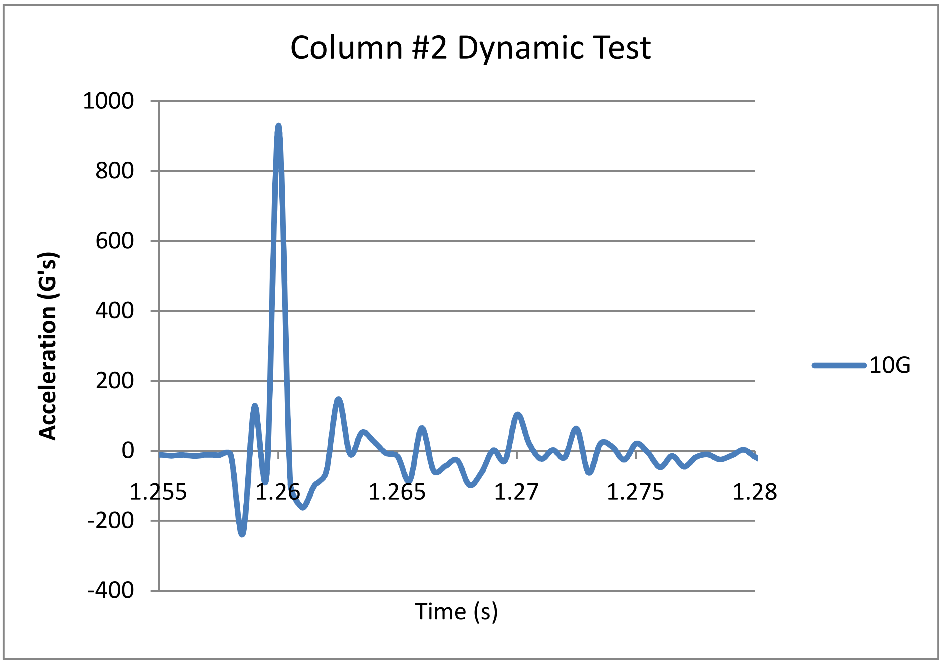


Figure 61: Acceleration versus time graph for column dynamic test #2.

Figure 62 is a picture of the single column after the test showing the deformation in the column. The image shows the column with a slight bend due to the impact load and shows a deflection of about ¼”.

Figure 62: Result from Column #2 dynamic test.

In order to begin to gauge the difference in dynamic and static loadings, the weight dropped was slightly increased to 180 lbs, but the height of the drop remained the same at 60 inches. This increase changed the strain energy of the drop considerably from 292.6 ft-lbs to 697.2 ft-lbs. However, the kinetic energy for the drop is now 900 ft-lbs because of the increase in mass, so the mass is still insufficient for buckling but is closer than before. Further tests will be carried out with this mass in order to observe any variance in the energy and buckling.

Column #3 Dynamic Test

Dynamic Test #3 involved an axial impact test on a single 24 inch long column in order to determine the dynamic buckling load of a column due to impact forces and to ensure all necessary data could be obtained using the test setup before beginning the tests on the frames. The member was placed vertically with the top end free to rotate and the bottom end held in place by a ½" thick steel plate with a 2" x 2" square cut in the center to keep the column from sliding along the main table or kicking out during testing. The sample was then impacted by masses dropped from the tower. Accelerometers were placed on the drop tower head to measure the deceleration at impact. Results of this test are given in Table 20.

Table 20: Column dynamic test #3 data

Mechanical Property	***Test Result***
Drop Height (in)	*60 +/- 4*
Weight Dropped (lbs)	*180*
Max Impact Acceleration (G's)	*≈750*
Calculated Kinetic Energy (ft-lbf)	*900*
Calculated Force on Impact (lbf)	*45,000*
Force at Impact – 10G (lbf)	*134,937.28*
Strain Energy (based on Impact Force) (ft-lbf)	*562.23*

The graph measuring the 10,000G accelerometer for the test is presented in Figure 63.

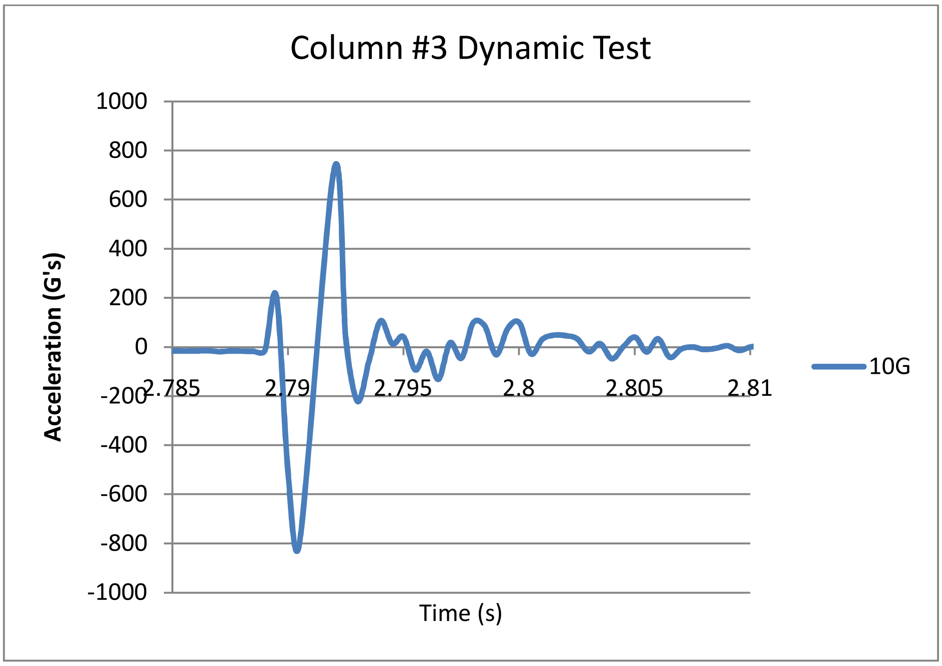


Figure 63: Acceleration versus time graph for column dynamic test #3.

Figure 64 is a picture of the single column after the test showing the deformation in the column. The image shows the column with a slight bend due to the impact load and shows a bow deflection of about ½”.

Figure 64: Result from Column #3 dynamic test.

For this test, identical testing parameters as column #2 were used, displayed a different acceleration. The impact acceleration of this test was 750 G's as opposed to the 930 G's of the previous test. This acceleration yielded an instantaneous load of 134,937 lbs which resulted in strain energy in the column of 562.23 ft-lbs, still much smaller than the required energy for buckling of 900 ft-lbs. However, similar to the second column, this test displayed very little deformation to the sample, although a small bend was noticed in the member, shown in Figure 64 above.

One more test with the same mass was performed to verify the results of dynamic tests #2 and #3 before moving on to a higher mass.

Column #4 Dynamic Test

Dynamic Test #4 involved an axial impact test on a single 24 inch long column in order to determine the dynamic buckling load of a column due to impact forces and to ensure all necessary data could be obtained using the test setup before beginning the tests on the frames. The member was placed vertically with the top end free to rotate and the bottom end held in place by a ½” thick steel plate with a 2” x 2” square cut in the center to keep the column from sliding along the main table or kicking out during testing. The sample was then impacted by masses dropped from the tower. Accelerometers were placed on the drop tower head to measure the deceleration at impact. Results of this test are given in Table 21.

Table 21: Column dynamic test #4 data

Mechanical Property	***Test Result***
Drop Height (in)	*60 +/- 4*
Weight Dropped (lbs)	*180*
Max Impact Acceleration (G’s)	*≈430*
Calculated Kinetic Energy (ft-lbf)	*900*
Calculated Force on Impact (lbf)	*45,000*
Force at Impact – 10G (lbf)	*77,364.04*
Strain Energy (ft-lbf)	*322.4*

The graph measuring the 10,000G accelerometer for the test is presented in Figure 65.

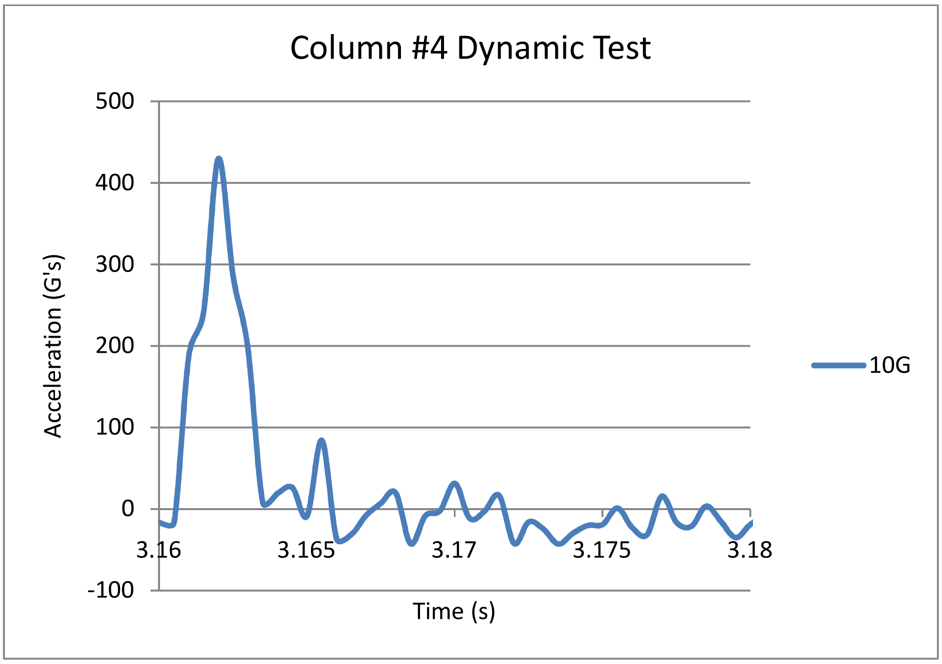


Figure 65: Acceleration versus time graph for column dynamic test #4.

Figure 66 is a picture of the single column after the test showing the deformation in the column. The image shows the column with a slight bend due to the impact load and shows a deflection of about ½”.

Figure 66: Result from Column #4 dynamic test.

Dynamic Column Test #4 yielded similar physical results to Tests #2 and #3 with an even lower force on impact and strain energy. Ultimately, much more mass is needed to obtain useful results for the dynamic column test. The group followed Test #4 with another test with substantially more weight added to the drop tower.

Column #5 Dynamic Test

Dynamic Test #5 involved an axial impact test on a single 24 inch long column in order to determine the dynamic buckling load of a column due to impact forces and to ensure all necessary data could be obtained using the test setup before beginning the tests on the frames. The member was placed vertically with the top end free to rotate and the bottom end held in place by a ½” thick steel plate with a 2” x 2” square cut in the center to keep the column from sliding along the main table or kicking out during testing. The sample was then impacted by masses dropped from the tower. The weight was increased from 180 pounds to 281 pounds in hopes that the large difference would help deform the column on impact. Accelerometers were placed on the drop tower head to measure the deceleration at impact. For the calculation of strain energy the deflection was increased from 1/10 inch to 1/5 inch because of increased deflection noticed in the column after impact. Results of this test are given in Table 22.

Table 22: Column dynamic test #5 data

Mechanical Property	***Test Result***
Drop Height (in)	*60 +/- 4*
Weight Dropped (lbs)	*281*
Max Impact Acceleration (G’s)	*≈250*
Calculated Kinetic Energy (ft-lbf)	*1,405*
Calculated Force on Impact (lbf)	*84,300*
Force at Impact – 10G (lbf)	*70,217.36*
Strain Energy (ft-lbf)	*583*

The graph measuring the 10,000G accelerometer for the test is presented in Figure 67.

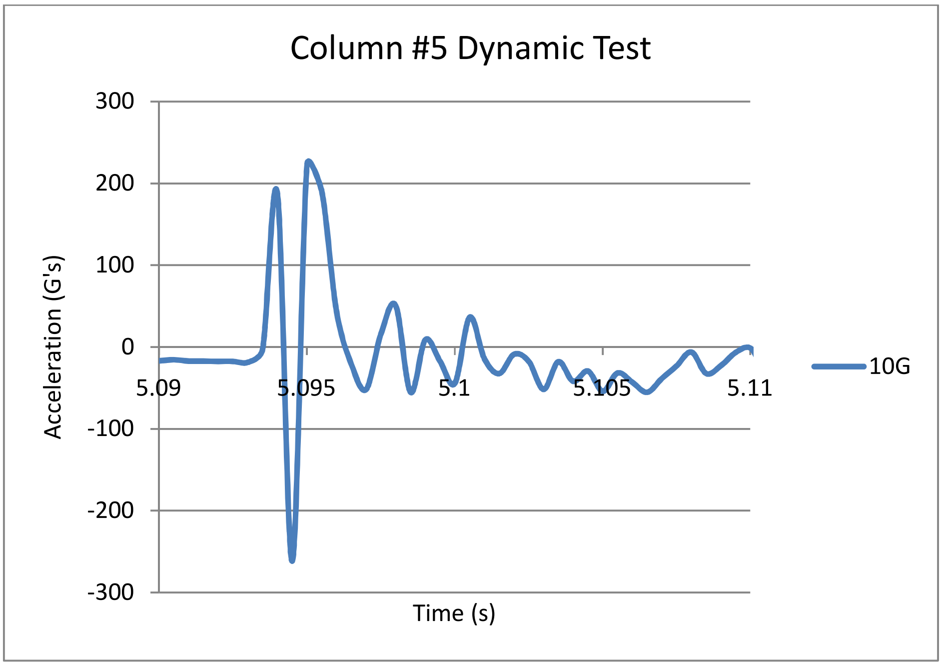


Figure 67: Acceleration versus time graph for column dynamic test #5.

Figure 68 is a picture of the single column after the test showing the deformation in the column. The image shows the column with a slight bend due to the impact load and shows a deflection of about ¼”.

Figure 68: Result from Column #5 dynamic test.

For this test the weight of the drop was increased to 281 lbs in order to observe the difference in acceleration and forces on impact. The acceleration on impact was decreased from previous drops to 250 G’s decreasing the force on impact of 70,217 lbf and corresponding strain energy of 583 ft-lbs. This instantaneous load corresponds with the calculated kinetic energy and force of 1,405 ft-lbs and 84,300 lbs respectively. This loading displayed considerably more deformation on the column then preceding tests, but still more weight is required for failure mode observations.

An important note to make at this point is that the buckling of the column does not seem to depend on the instantaneous load of the weight but on the kinetic energy coinciding with the quasi-static failure load. A further test bringing the strain energy closer to the kinetic energy required for buckling (853.88 ft-lbs) should be sufficient to see significant deformation in a column.

Column Dynamic Test (Longer)

Dynamic Column Test (Long) involved an axial impact test on a single 36 inch long column in order to determine the dynamic buckling load of a column due to impact forces and to ensure all necessary data could be obtained using the test setup before beginning the tests on the frames. The extra length was used to compare the results to those of the 24” columns. The member was placed vertically with the top end free to rotate and the bottom end held in place by a ½” thick steel plate with a 2” x 2” square cut in the center to keep the column from sliding along the main table or kicking out during testing. The sample was then impacted by masses dropped from the tower weighing 747 pounds. Accelerometers were placed on the drop tower head to measure the deceleration at impact. Results of this test are given in Table 23.

Table 23: Column dynamic test (long) data

Mechanical Property	***Test Result***
Drop Height (in)	*48 +/- 4*
Weight Dropped (lbs)	*747*
Max Impact Acceleration (G’s)	*≈90*
Calculated Kinetic Energy (ft-lbf)	*2,988*
Calculated Force on Impact (lbf)	*179,280*
Force at Impact – 10G (lbf)	*67,230*
Strain Energy (ft-lbf)	*1,680*

The graph measuring the 10,000G accelerometer for the test is presented in Figure 69.

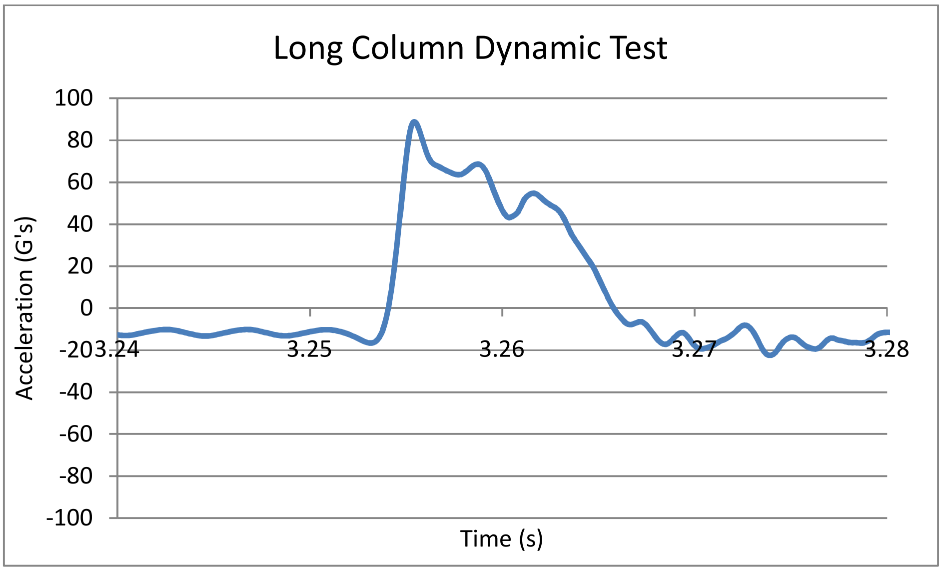


Figure 69: Acceleration versus time graph for column dynamic test (long).

Figure 70 is a picture of the single column after the test showing the deformation in the column. The image shows the column with a slight bend due to the impact load and shows a deflection of about ¾".

Figure 70: Result from Column (long) dynamic test.

This test had a test maximum weight for the impact of 747 lbs in order to observe the difference in acceleration and forces on impact and to obtain a certain bend in the column. The fact that the column was 50% long also helped in achieving this goal. The acceleration on impact was decreased from previous drops to around 90 G's, significantly decreasing the force on impact to around 67,000 lbf. This instantaneous load corresponds with the calculated kinetic energy and force of 2,988 ft-lbf and 179,280 lbf respectively. The loading displayed even more deformation on the column then even the best previous column test, but that is to be expected with the extra length.

Obtaining all previous column impact information became vital before moving into the truss tests, as it helped determine how much load would be needed in order to create a bend in the vertical members. Since the last successful 24" dynamic test took a load of 281 lbf and still bent, the next dynamic truss tests would begin at this point.

Truss #1 Dynamic Test (Symmetric)

The first frame was tested with the sample resting on roller supports on either end of the bottom chord directly underneath the outer vertical members. The truss was oriented such that the two diagonal members ran from the top corners to the bottom center of the frame. The local deformation issue was addressed by inserting a solid piece of steel stock (1.75”x1.75”x4”, henceforth known as a “plug”) into either end of the bottom chord directly over the roller supports. By doing so, local deformation due to the reactive forces of the supports would be eliminated. The truss was held in place by four vertical I-beams to restrict movement forward and back. The drop tower contained 281 lbf of weight and was dropped at a height of 60”. A cylindrical roller was also welded on the bottom of the falling mass to simulate a point load on the top of the frame. This set up and loading is summarized in Figure 71 along with number labels for each member.

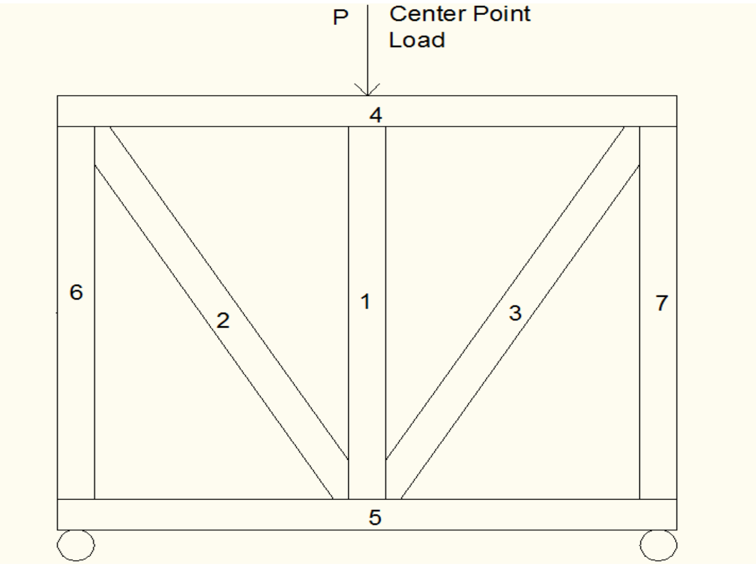


Figure 71: Truss setup for Dynamic Test #1.

Figure 72 is a picture of the top center of the truss frame showing the local deformation that occurred upon impact.

Figure 72: Result from Truss #1 dynamic test.

The weight applied to the truss at impact was by no means enough to even bend a single member. The only result from the test came from a very small amount of local deformation at the area of impact due to the point load simulation created by the roller. While this test was ultimately not successful, it provided insight into what could be done to fix the next dynamic truss test. It was apparent that more weight would be needed and if possible, more height. Also, the truss was not secured down to the ground so at impact, it bounced about a foot off of the rollers as a reaction to being hit by the falling mass. Finally, the readings of acceleration were very noisy and needed to be filtered to obtain pertinent data. All of these issues would be dealt with for "Truss #2 Dynamic Test".

Drop Tower Calibration

After various tests were performed using both columns and trusses, it became apparent that it was required to slow the frequency of the loading to bring the instantaneous loading of impact down to size in order to spread the impact pulsation. This was first done using a symmetric truss with three different tests: the first with no padding at all, the second with half an inch of padding, and the third with one inch of padding. The drop head was lifted to less than twelve inches above the center of the truss and dropped in order to get an idea of how successful this would be.

The accelerometer was the primary tool for measurement of this test. The results of the test with no pads measured from the accelerometer are displayed in Figure 73 below.

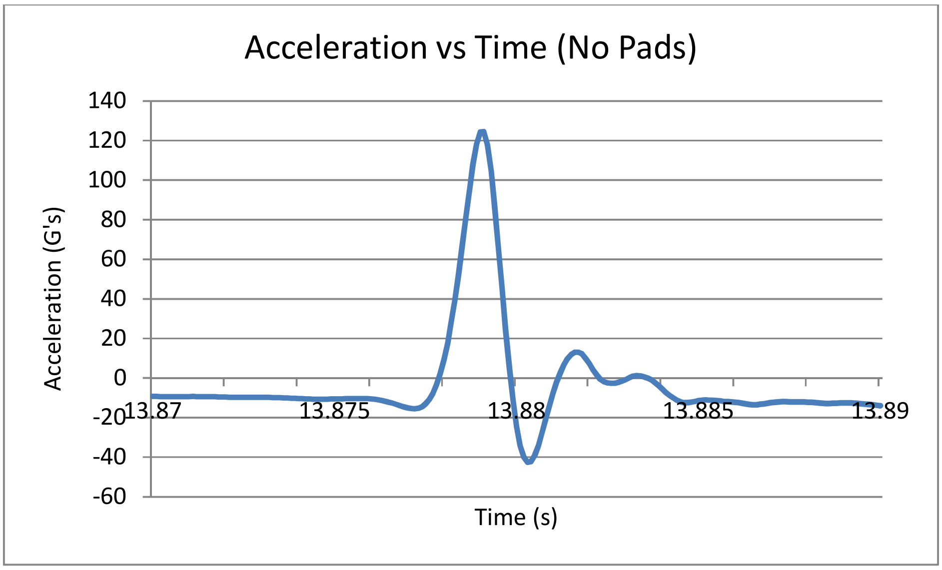


Figure 73: Acceleration vs Time graph for low height drop using no pads.

The next test utilized a half inch of padding and the results are displayed in Figure 74 below.

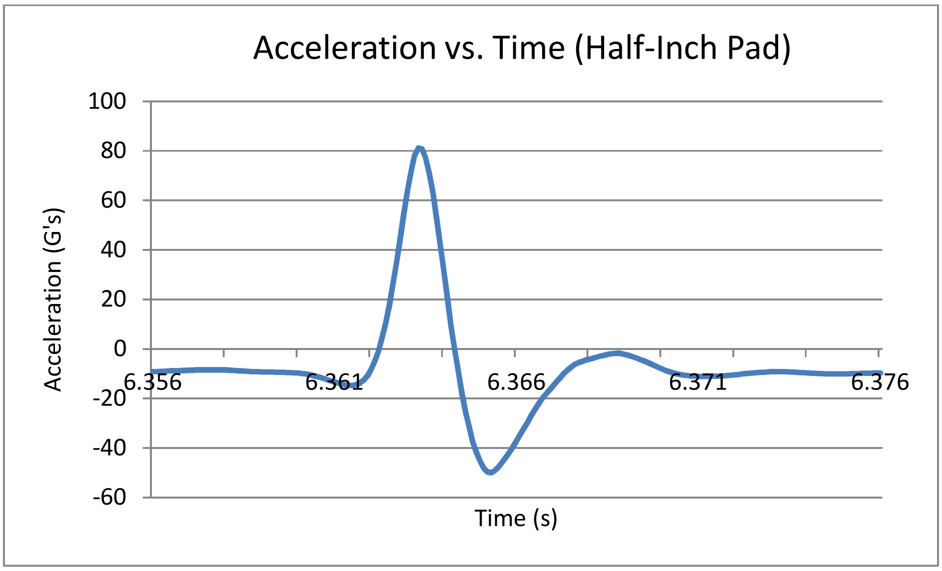


Figure 74: Acceleration vs Time graph for low height drop with half inch of padding.

The last part of this testing utilized an inch of neoprene padding and the results are displayed in Figure 75 below:

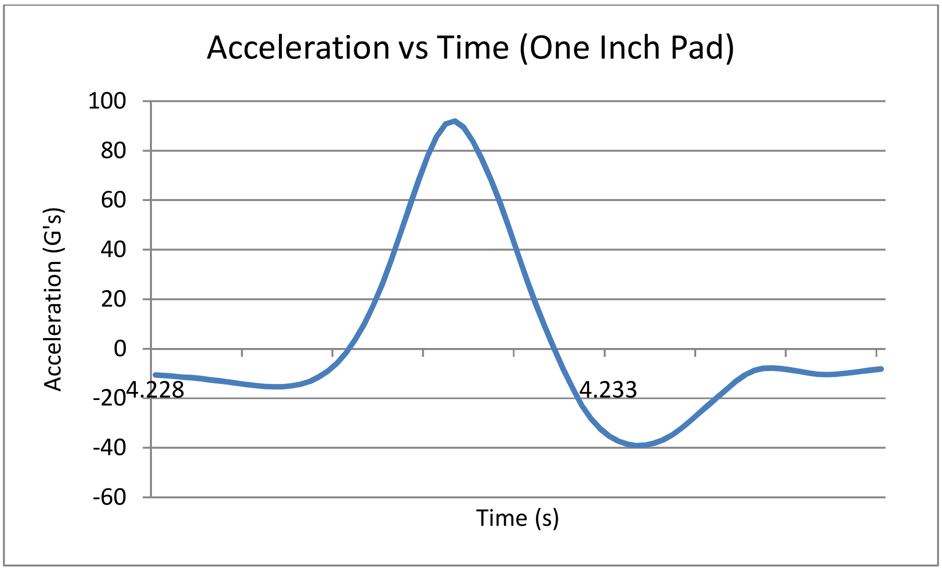


Figure 75: Acceleration vs Time graph for low height drop with one inch of padding.

The major result of this test is that using the padding the time of impact increases from 0.01 seconds with no padding up to almost 0.1 seconds using an inch of padding. This change in load frequency corresponds to a decrease in acceleration on impact from 120 G's down to 85-90 G's. This decrease (based on the mass dropped, 695 lbs) yields an impact force of 62,550 lbf with an inch of padding as opposed to a force of 83,400 lbf when impacting the truss with no padding.

Truss #2 Dynamic Test (Symmetric)

The second frame was tested with the sample resting on roller supports on either end of the bottom chord directly underneath the outer vertical members. The truss was oriented such that the two diagonal members ran from the top corners to the bottom center of the frame. The local deformation issue was addressed by inserting a solid piece of steel stock (1.75”x1.75”x4”, henceforth known as a “plug”) into either end of the bottom chord directly over the roller supports. By doing so, local deformation due to the reactive forces of the supports would be eliminated. The truss was held in place by four vertical I-beams to restrict movement forward and back. The drop tower contained 695 pounds of mass, which was a substantial increase from the previous test, and was dropped at a height of 60”. A cylindrical roller was also welded on the bottom of the falling mass to simulate a point load on the top of the frame.

Rubber matting was placed in all areas consisting of metal-to-metal contact including between the rollers and the frame as well as between the frame and the I-beams. This would help reduce the noise that appeared in the acceleration plots. The test was performed three times with three different types of padding between the frame and the drop tower head: ½” rubber, 1” rubber, and thin rubber matting. The bottom chord of the frame was clamped down to the base of the drop tower in order to prevent any movement upon impact. All acceleration plots were also filtered by importing the Microsoft Excel data collected during the tests into MATLAB and applying an SAE filter to smooth out the data and eliminate most of the noise. This set up and loading is summarized in Figure 76 along with number labels for each member.

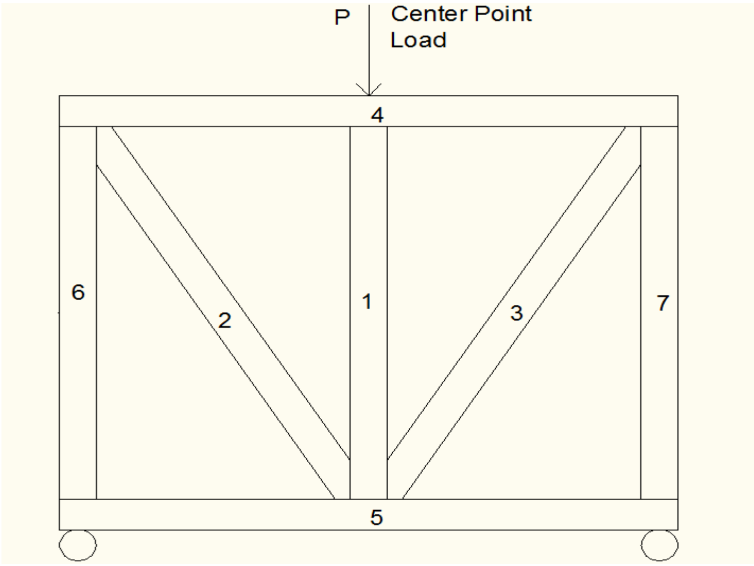


Figure 76: Truss setup for dynamic test #2.

The strain gauges used to gather material data of each of the key members (mainly the top chord and the center column) were arranged as follows in

Figure 77.

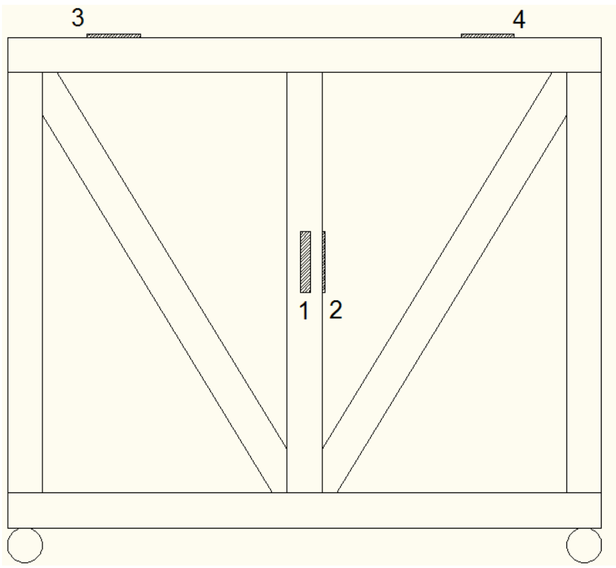


Figure 77: Strain gauge assembly for dynamic frame test.

The parameters for this test are summarized below in Table 24.

Table 24: Testing Parameters for Dynamic Truss #2.

Property	**Value**
Drop Height (inches)	*50*
Mass Dropped (lbs)	*695*
Kinetic Energy Calculated (ft-lbf)	*2,895.9*
Force Calculated (lbf)	*69,500*

The first part (A) of the test used 1/2” rubber as a medium between the drop head and the frame. The acceleration data was collected in Excel through LabView and transferred to MATLAB for filtering. The filtered data can be seen in Figure 78.

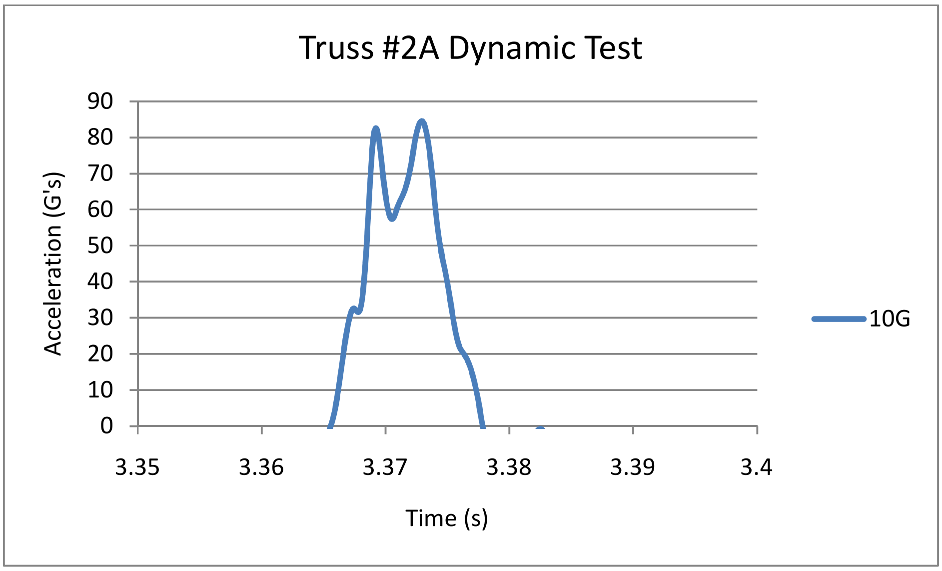


Figure 78: Truss #2A Accelerometer Results.

This graph displays a maximum acceleration of 85 G's which corresponds to a force on impact of 59,075 lbf.

The strain gauge readings for this test are presented in Figure 79 below:

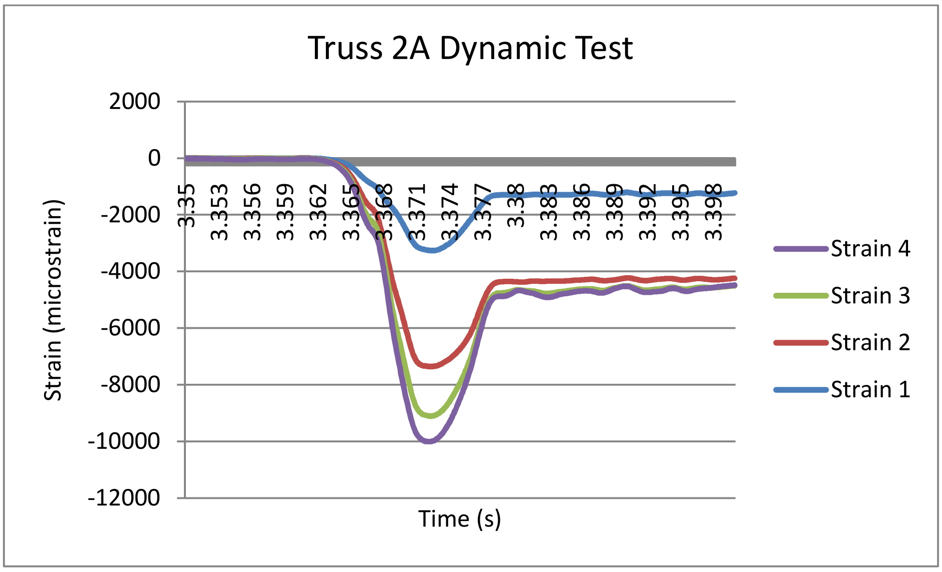


Figure 79: Truss #2A strain gauge results.

These strain readings are summarized below in Table 25

Table 25: Summary of Strain Readings for Truss #2A

Strain Gauge	Maximum Strain Reading
1	0.003
2	0.007
3	0.009
4	0.01

The second part (B) of the test used 1” rubber as a medium between the drop head and the frame. The acceleration data was collected in Excel through LabView and transferred to MATLAB for filtering. The filtered data can be seen in Figure 80.

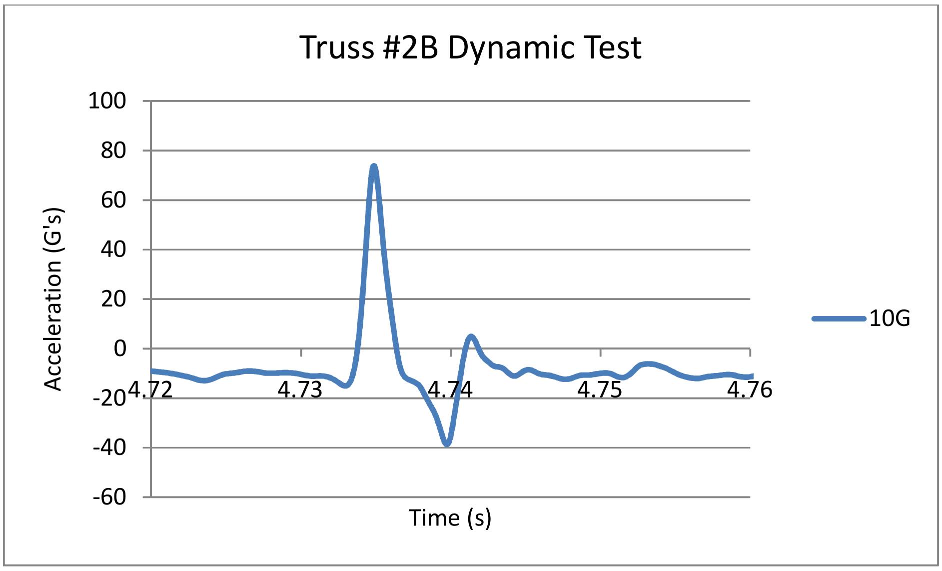


Figure 80: Truss #2B Accelerometer Readings.

This graph displays a maximum acceleration of 72 G's which corresponds to a force on impact of 50,040 lbf.

Corresponding strain gauge readings are displayed below in Figure 81.

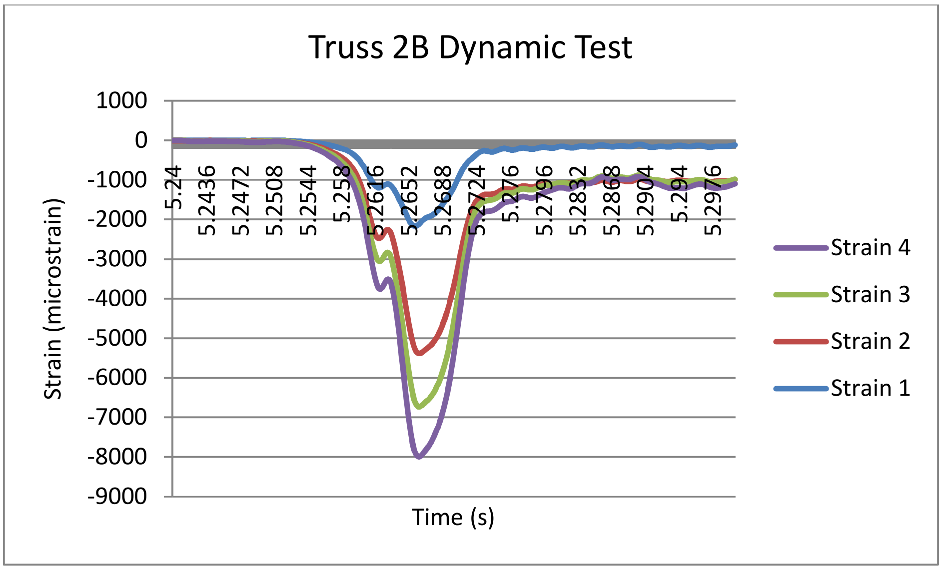


Figure 81: Truss #2B strain gauge results.

These strain readings are summarized below in Table 26.

Table 26: Summary of Strain Readings for Truss #2B

Strain Gauge	**Maximum Strain Reading**
1	0.002
2	0.0052
3	0.0065
4	0.008

The third part (C) of the test used thin rubber matting (approximately 1/16”) as a medium between the drop head and the frame. The acceleration data was collected in Excel through LabView and transferred to MATLAB for filtering. The filtered data can be seen in Figure 82.

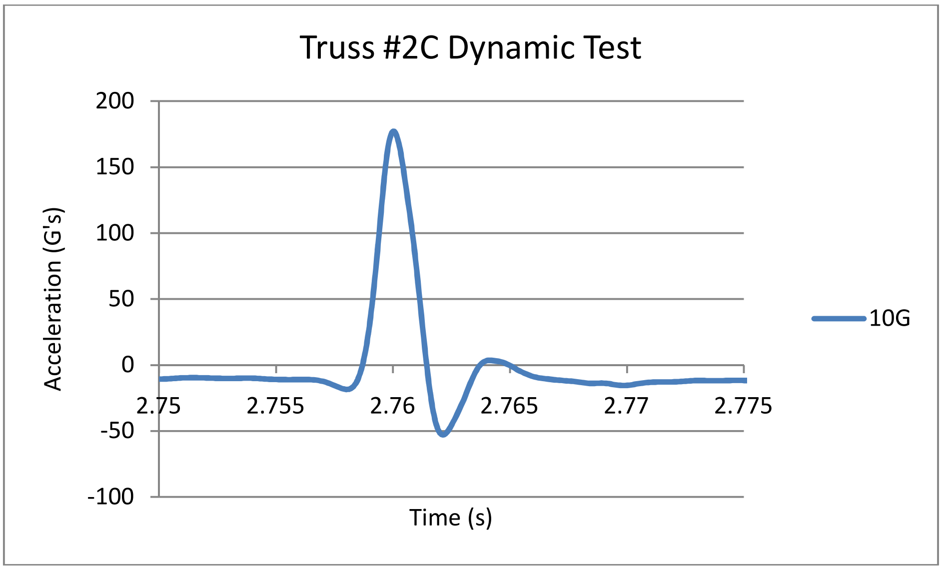


Figure 82: Truss #2C Accelerometer Readings.

This graph displays a maximum acceleration of 175 G’s which corresponds to a force on impact of 121,625 lbf.

At this point the strain gauge readings displayed a lot of noise rather than clear curves, which could be either because of the very small amount of padding used or because the strain gauges failed during the test.

The drops described above finally bent the truss and manages to buckle the top of the center column. This failure can be observed in Figure 83 below.

Figure 83: Result of dynamic test #2.

With this test the weight was dropped three times on the same truss with different programming materials (the neoprene pads). The first two used thick and stiff pads of different thickness and yielded impact forces of 59,075 lbf and 50,040 lbf respectively, while the last test was with a thin rubber mat and yielded 121,625 lbf. The thicker pads managed to bring the force observed closer to the static load and managed to weaken the truss for failure in the third drop.

Discussion

The purpose of this report was to observe through experimentation the internal structures of a submarine module and investigate any conservatism inherent in their design. At first a series of background investigations had been performed to gain familiarity with a submarine's structure and the process by which it is designed and built. This background included documentation provided by Electric Boat and tours of Electric Boat's facilities in Connecticut. After the background research was carried out, sample designs were provided by EB representing scaled Pratt-like trusses representative of a truss configuration in the modular structures on board the submarines along with material specifications for procurement and use of steel in the testing samples.

After the steel was procured and the trusses fabricated, the testing portion began. The first part of testing was to establish the properties of the materials used utilizing the ASTM E8 standard for testing of metallic materials. The material used was ASTM A500 Grade B steel in the form of HSS shapes and the ASTM standard specifies that the yielding strength of the material must be at least 46,000 psi. The tensile tests we did and in accordance with ASTM E8 provided an average yielding strength over three tensile samples of 56,367 psi—nearly 20% higher than the specified minimum. So automatically some question was placed on the conservatism of theoretical design in those members if the yield strength used in the design is already by itself 20% lower than is experimentally observed in the material.

The next step in the testing process was to get an idea as to the failure modes of the columns in the truss by themselves. At first a 24 inch column representative of the center column in the truss design was tested under quasi-static loading to observe its buckling behavior in comparison to what the AISC empirical data suggests. Based on a conservative effective length factor (K) estimate of K=1.0 (pin-ended columns free to rotate at both ends) and a yield strength of 46,000 psi (ASTM minimum), the calculated buckling load of a column like this would be 36,138 lbs based on AISC equations. However, the static testing of a single column did not actually buckle until 51,233 lbs—a 29.5% increase! This loading is more consistent with an end condition of fixity, suggesting the real end conditions of the columns to be closer to 0.65-0.7 rather than the 1.0 used.

After quasi-static loading, new columns were then placed under dynamic load using the drop tower, a specified drop mass, and accelerometers in order to measure acceleration on impact which was used to calculate the instantaneous load on impact. Using the quasi-static column buckling load of 51,233 lbs and the conservation of energy, it was calculated that a mass of 170.8 lbs would be required at a height of 5 feet in order to achieve a kinetic energy of 853.88 ft-lbs required to buckle the column. However, with a mass of 180 lbs dropped from 5 feet, very little deformation was observed in the columns despite imposing an extremely large instantaneous load of between 70,000-167,000 lbs of force on the column at impact. After 3 columns at this drop height it was apparent that we lacked sufficient weight and height to attain the energy required to buckle these compact columns (as the strain energy observed with this weight was only seen to be between 322-562 ft-lbs using the 180 lb drop mass). Thus the mass dropped was increased to 281 lbs dropped from 5 feet, which still only yielded 583 ft-lbs—still not close enough to the required energy of 853.88 ft-lbs.

Through this it was observed that we did not have enough capacity of either height or weight to buckle these columns with their material properties and compactness. So the length of the column was increased to 36 inches in order to make the column more slender to observe the column's buckling pattern in dynamic loading. This column in quasi-static loading was observed to handle a peak load of 52,282 lbs before buckling, which is extremely high once again considering it has an AISC calculated buckling load of 33,239 lbs. For the dynamic test the springs to the drop tower were removed to increase the allowable height and the mass was increased enormously to 747 lbs as opposed to the 281 lbs of the preceding drop. This test finally buckled the column considerably, deflecting it at impact almost ½ inch and producing a strain energy of 1,680 ft-lbs, considerably larger than the kinetic energy required of 837 ft-lbs. The strange part however was the fact that the kinetic energy for the drop was enormous at nearly 3,000 ft-lbs, nearly twice the strain energy!

For the truss configuration a series of setups were utilized, but at the request of Electric Boat emphasis was placed on the centered point loading of the quasi-static truss tests 5 and 6. The buckling load from these tests however, did not differ that much from that of the column, with only a slight increase from a quasi-static column loading of 51,233 lbs to an average peak load in the trusses of 55,202 lbs. A major difference though was the fact that due to the welds in the truss and surrounding structure the ends stayed more in place with more fixity in the center column (the column that buckled in this loading) bringing the realistic end condition closer to that of fixity with 0.6 as the effective length factor (K) rather than the conservative estimate of 1.0 (pin ended and free to rotate) used in the design which through the AISC buckling equations would yield a buckling force in the center column of 43,620 lbs for a 24 inch column.

When this truss setup was placed under dynamic loading it was initially under the final weight dropped on the 24 inch columns of 281 lbs dropped from 5 feet, which after the drop did absolutely nothing but dent the top chord of the truss slightly. It was at this point that it was decided to increase the mass to 695 lbs and remove the springs. The tests that followed also employed various thickness neoprene pads as programming mediums in order to lengthen the pulse of impact (stretch out the time of loading at impact).

The first of these tests was the test used to calibrate what the difference would be when using the pads as opposed to dropping directly onto steel. This test dropped the 695 lbs from less than a foot above a truss with no padding, half an inch of padding, and then an inch of padding. Accelerometer results displayed that with no padding at all (just steel on steel impact) the time of impact was only $1/100^{th}$ of a second, but this loading frequency changed to nearly $1/10^{th}$ of a second with an inch of padding, and this addition also lowered the peak acceleration from over 120-G's with no padding down to 90-G's with an inch of padding. This would allow us to get a lower force on impact as the load is no longer instantaneous as before, bringing down the gap between quasi-static and dynamic buckling loads.

The last test using the symmetric trusses was similar to the calibration test just explained, but the height of the drop was increased to the maximum allowable height of 50 inches and the programming mediums were different thickness neoprene pads of one half an inch, one inch, and then a mere $1/16^{th}$ inch rubber mat. The first two tests used the thick and stiff neoprene pads used before, and resulted in loadings at impact of 59,075 lbs and 50,075 lbs respectively; very close to the quasi-static loadings, but very little deformation was observed. However, the third test with the rubber matting (simply to dampen the noise from the steel on steel contact) produced an instantaneous loading of 121,625 lbs and a deformation of approximately $1/4^{th}$ of an inch, resulting in a strain energy of 2,500 ft-lbs which finally buckled the weakened truss.

The final test to discuss is that of the asymmetric quasi-static truss test. Concern had risen regarding increased moments in the center column if the column was off center and the truss not symmetric. However, the asymmetric truss actually managed to take on a higher loading than that of the symmetric trusses and still managed to buckle almost entirely in the center column at 60,222 lbs. Based on analysis of the truss using both classical methods as well as data from the strain gauges, the only difference made by this change was that the diagonals managed to take a heavier burden in the loading with one taking more than the other. The failure mode however, displayed an interesting level of torsion rather than simple deflection in a single plane, both buckling out towards the shorter diagonal as well as buckling out of the plane of the truss at the same time.

Recommendations

In the beginning, the stated goal of this project was to identify and possibly quantify conservatism in the design of submarine modules (where trusses are employed) through analytical prediction and experimentation. Current modules are designed to withstand dynamic loadings with elastic deformation analysis methods for computing stress and buckling. The potential conservatisms inherent in these methods begins with the fact that elastic methods assume the deformation remains elastic, not plastic, and continue with the fact that traditional analysis methods are based on empirical data from static loadings. Based on the AISC limiting values for slenderness from Chapter E of the Steel Construction Manual (Chapter E, Steel Construction Manual, 2005), the members used in the testing (both 24 inch and 36 inch) are not considered slender and should be analyzed using inelastic buckling methods. Also, the modules in these submarine structures are not subjected to significant static loading, as the design methods are based on, but are intended to survive certain severe dynamic loading conditions.

In order to quantify the conservatism, various tests were run to observe the behavior of the truss members when subjected to static and dynamic loads until failure, with an emphasis placed on the dynamic loading. In the dynamic tests in particular, it was observed that, despite an increasingly high load on the instant of impact, the columns and frames were not buckling. When the height of the drop was increased and the mass dropped was nearly doubled, the columns finally buckled and after repeated drops while dampening the pulse of the loading, so did the frames. However, these drops that buckled the members dynamically also reflected lower instantaneous loads on impact than the samples that did not buckle. This brings us to the ultimate conclusion drawn from the dynamic testing, that the buckling in a dynamic situation is almost entirely dependent on the strain energy.

This conclusion can be drawn based on the idea that as the column leaves its elastic region and begins to fail, there are no longer forces springing the column back to its original shape; the member deforms and permanently stays that way. This means that the deformation increases and the load on impact decreases, resulting in the strain energy required for dynamic buckling being approximately the same as the kinetic energy required for static buckling. As the theoretical energy required for buckling is calculated based on static loading, design of members for buckling would be the same for both static and dynamic.

While this is a valuable conclusion, it still does not pinpoint conservatism in the design; however, an examination of the static design and results can shed further light on this conservatism. Observing the 36 inch column (as we received failure results both statically and dynamically to compare), the peak static load was 52,282 lbs which yields a critical buckling stress of 62,240.48 psi. Using the AISC equation E3-2 for inelastic buckling design and back calculating using this load and the theoretical yielding stress of 46,000 psi, one gets an Euler buckling load of 172,910.72 psi, which (based on material and geometric properties of the column) would yield a K value of 0.86 rather than the K value of 1.0 used in the standard design for these stresses.

This leads to the main recommendation of all of these tests, which is to continue with static design of the members since in the worst case scenario, these members will buckle relying on strain energy and at failure, strain energy acts almost identically between static and dynamic loadings. However, the static design of the members has conservative assumptions; the major one being that the K value of the truss is 1.0 rather than a value of between 0.8 and 0.9 realized in the tests carried out. If one replaced the K value in the analysis of the single column buckling with 0.8 and the yielding stress in the design being the experimental yield stress of 56,367 psi, the estimated peak load for the 24 inch column would increase from 43,620.76 lbs to approximately 45,000 lbs.

Thus, it is recommended that consideration be given to changing the assumption to K=0.8 and keeping the yielding stress at the ASTM specified 46,000 psi. It is proposed that this would still result in a conservative design, but would save materials and money because this would allow for the radius of gyration in the slenderness ratio to decrease, which would in turn keep the truss members at a compact and conservative design while making the cross sectional area less and ultimately using less material. Less material means less structural weight, easier manufacturability, and higher cost efficiency. However, further changes beyond this point should be verified through additional research using more emphasis on dynamic situations and the impact of these changes for Electric Boat's specific members.

References

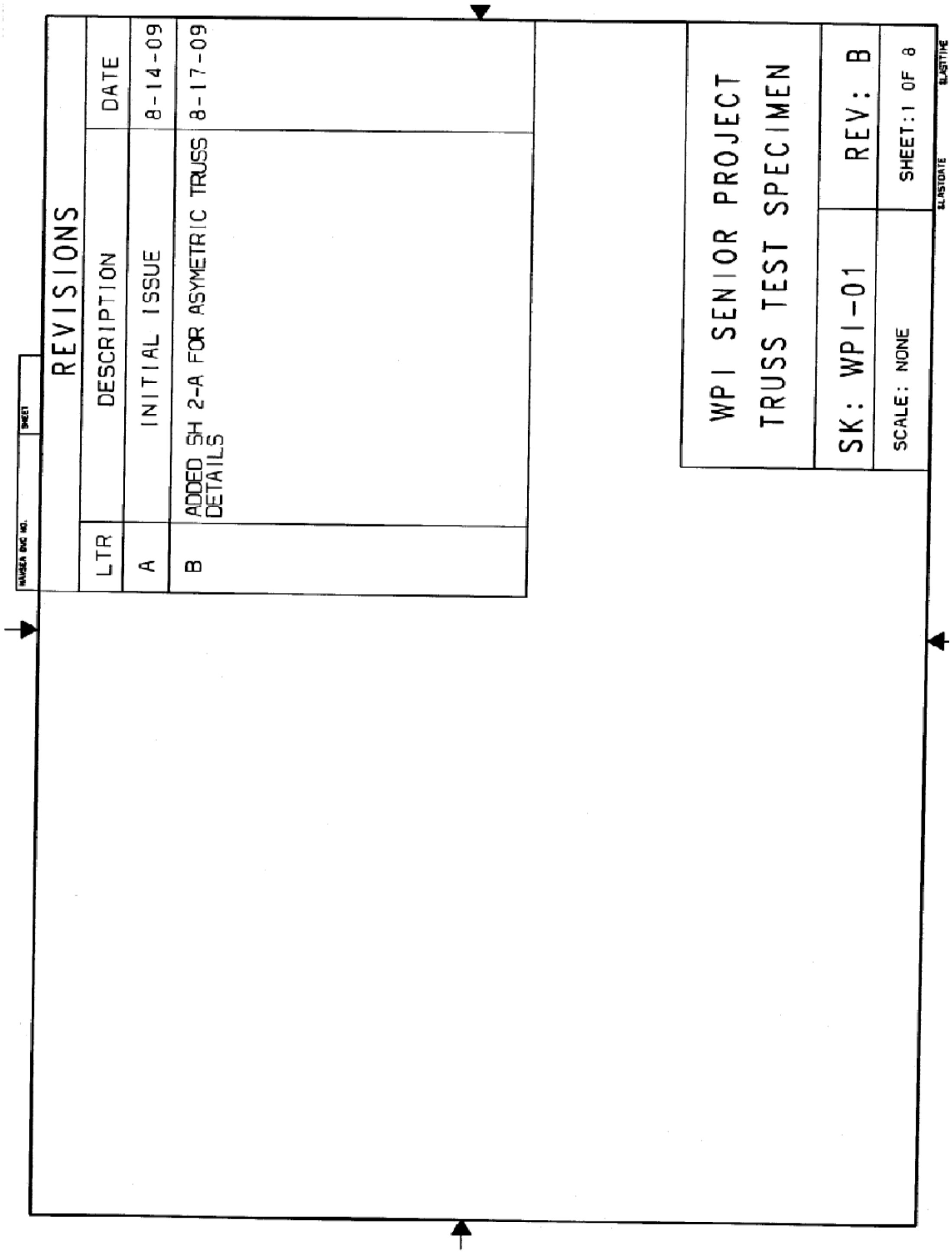
American Institute of Steel Construction. "Chapter E: Design of Members for Compression." In *Steel Construction Manual*. 2005.

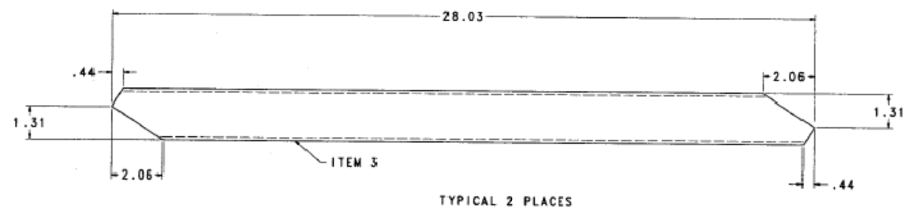
American Institute of Steel Construction. "Chapter H: Design of Members for Combines Forces and Torsion." In *Steel Construction Manual*. AISC, 2005.

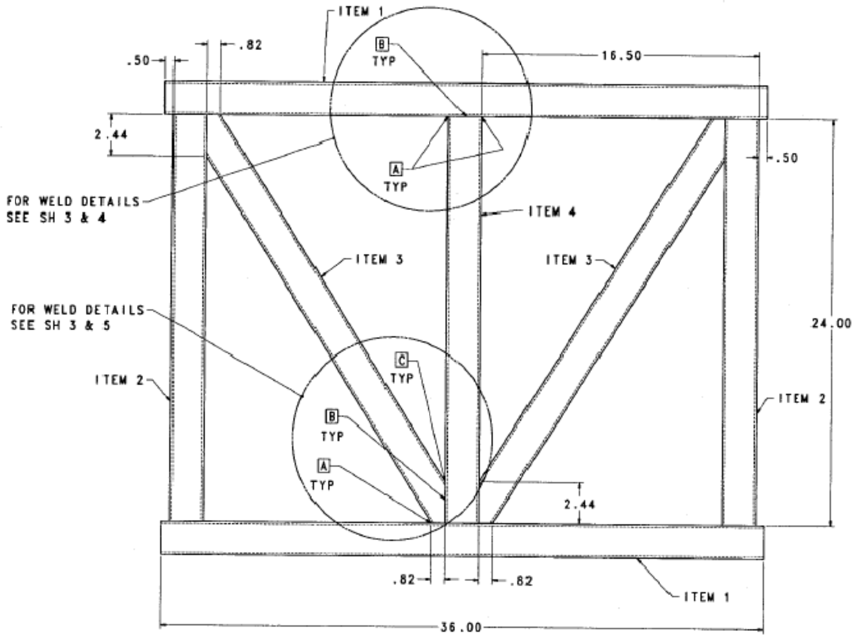
ASTM Standard E8, 2008, "Standard Test Methods for Tension Testing of Metallic Materials", ASTM International, West Conshohocken, PA, 2008, DOI: 10.1520/E0008_E0008M-09, www.astm.org.

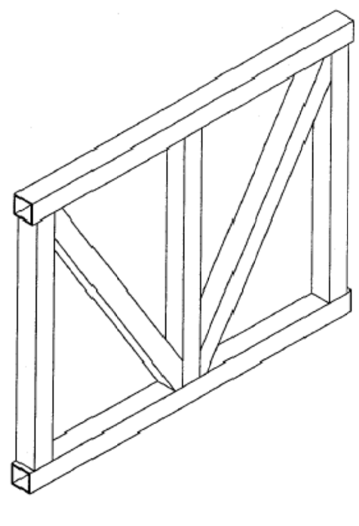
Appendices

Appendix A: Sketch: WPI-01 Rev B – WPI Senior Project Truss Test Specimen







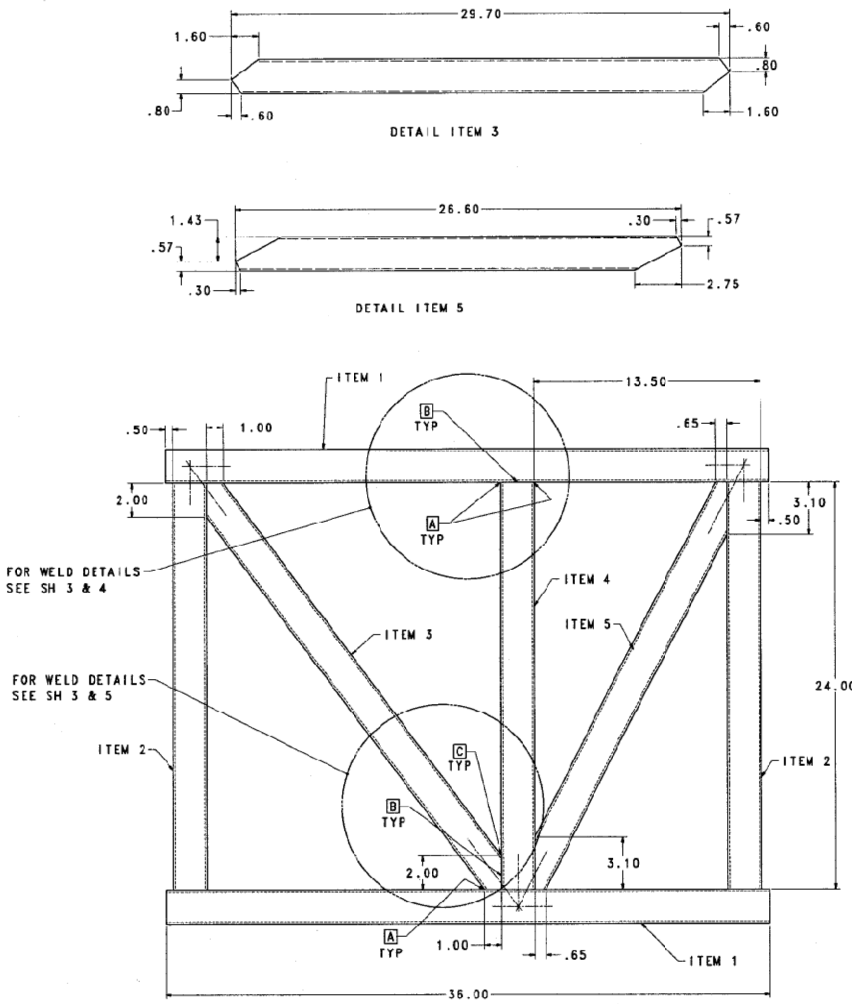


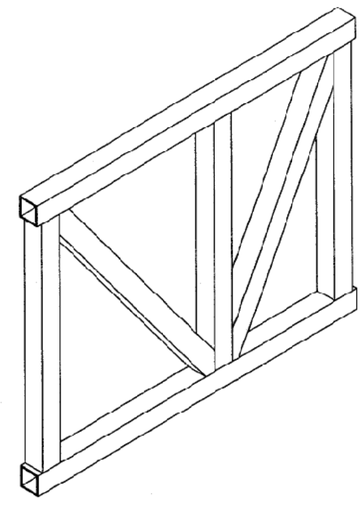
NOTES

1. ALL MATERIAL TO BE 2X2X1/8 TUBE, ASTMA500,GR B
   (MADE FROM SEMI-KILLED OR KILLED STEEL)
2. FOR WELD DETAILS SEE SH 2-8

TRUSS TEST SPECIMEN
WPI SENIOR PROJECT
SKETCH WPI-01 REV B

SHEET 2





NOTES

1. ALL MATERIAL TO BE 2X2X1/8 TUBE, ASTMA500,GR B (MADE FROM SEMI-KILLED OR KILLED STEEL)
2. FOR WELD DETAILS SEE SH 2-8

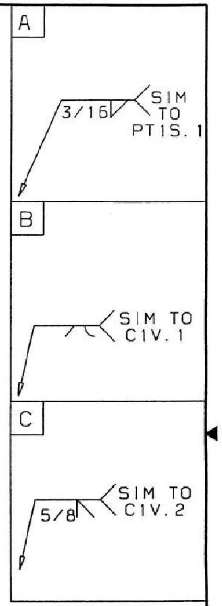
TRUSS TEST SPECIMEN
WPI SENIOR PROJECT
SKETCH WPI-01 REV B

SHEET 2-A

NAVSEA DWG NO. | SHEET

WELDING NOTES

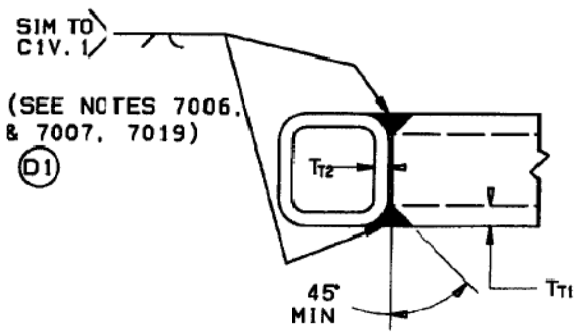
1. ALL WELDS SHALL BE FABRICATED USING THE GMAW (MIG) PROCESS.
2. ALL WELDS SHALL BE FABRICATED USING 70XX ELECTRODE (MIN ULT TENSILE STRENGTH OF 70KSI), OR EQUAL.
3. ALL FIT-UP SHALL MEET THE REQUIREMENTS OF SH 6, 7 & 8, EXCEPT BEVEL ANGLES SHALL BE IN ACCORDANCE WITH SH 4 & 5.
4. ALL WELDS SHALL BE VISUALLY INSPECTED.
5. WELD UNDERCUT SHALL BE LIMITED TO 1/64" MAX.
6. ALL WELDING REQUIREMENTS NOT DEFINED ON THIS SKETCH SHALL BE IN ACCORDANCE WITH AWS D1.1.



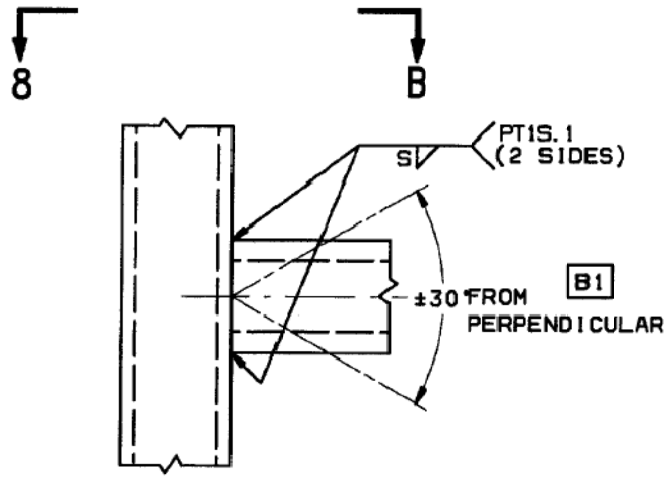
TRUSS TEST SPECIMEN
WPI SENIOR PROJECT
SKETCH WPI-01 REV A

SHEET 3

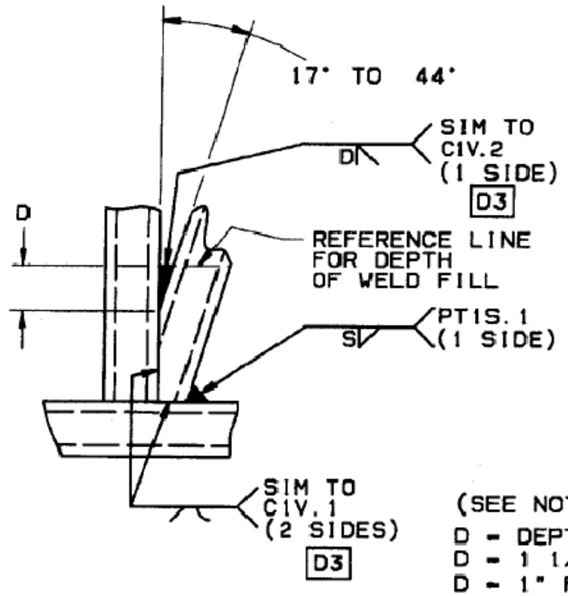
&LASTDATE &LASTTIME



C1 DETAIL 8-B

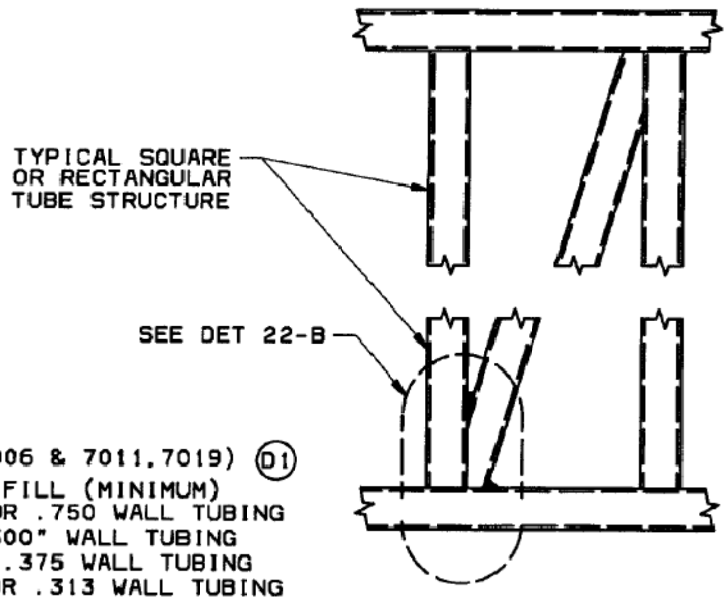


TRUSS TEST SPECIMEN
WPI SENIOR PROJECT
SKETCH WPI-01, REV A
SHEET 4



DETAIL 22-B

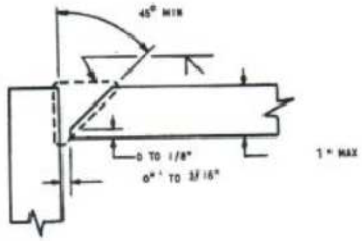
(SEE NOTES 7006 & 7011,7019) D1
D - DEPTH OF FILL (MINIMUM)
D - 1 1/4" FOR .750 WALL TUBING
D - 1" FOR .500" WALL TUBING
D - 7/8" FOR .375 WALL TUBING
D - 13/16" FOR .313 WALL TUBING
D - 3/4" FOR .250 WALL TUBING
D - 11/16" FOR .188 WALL TUBING
S - DETERMINED BY DESIGN



DETAIL 22-A

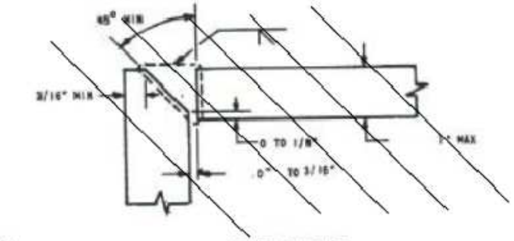
TRUSS TEST SPECIMEN
WPI SENIOR PROJECT
SKETCH WPI-01, REV A
SHEET 5

CIV.2 (SEE NOTE)



WELDED ONE SIDE

~~CIV.3 (SEE NOTE)~~



SH9026A

WELDED ONE SIDE

NOTE: Joints welded from one side shall not be used when root of weld is subject to bending tension stress equivalent to one-half the yield strength of the base metal or greater.

FIGURE 10. Corner joints, outside single-bevel, CIV.2, CIV.3.

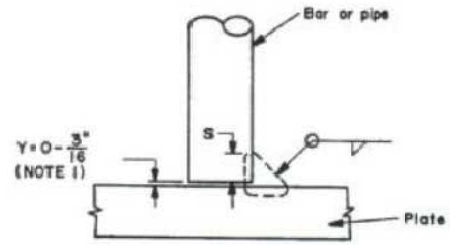
TRUSS TEST SPECIMEN

WPI SENIOR PROJECT

SKETCH WPI-01, REV A

SHEET 7

PT 1S-1



SH 11390

NOTE: Where Y is greater than 1/16 inch as a nominal condition, S shall be increased by an amount equal to the excess of the opening above 1/16 inch.

FIGURE 21. Tee joint, partial penetration, PT1S.1.

TRUSS TEST SPECIMEN

WPI SENIOR PROJECT

SKETCH WPI-01, REV A

SHEET 8

Appendix B: Weld Inspection Data Sheet

WORCESTER COUNTY WELDING INC.

Weld Inspection Data Sheet

WORCESTER COUNTY WELDING INC.

CUSTOMER W.P.I.

JOB NO. 3951 **DATE** 8/18/09

WELDING PERFORMED BY JASON FERREIRA

WELDING AUTHORIZED BY MICHAEL FLETCHER

WELDING PROCESS MIG (GMAW)

MANUAL ✓ MACHINE ✓ SEMI-AUTOMATIC ✓ AUTOMATIC____

JOINT DESIGN USED:

TYPE____________ **SINGLE WELD** ✓ **DOUBLE WELD** ✓

BACKING YES___ NO ✓ BACKING MATERIAL NONE

ROOT OPENING — **ROOT FACE DIMENSION** —

GROOVE ANGLE YES **RADIUS (J-U)** YES

BACK GOUGING YES___ NO ✓

METHOD____________________________

BASE METALS:

MATERIAL SPEC.________________ **TYPE OF GRADE**__________

THICKNESS GROOVE________ FILLET________

DIAMETER (PIPE)____________

FILLER METALS:

AWS SPECIFICATION. A5.18 **AWS CLASSIFICATION** 70S-6

SHIELDING:

FLUX. — **GAS** — **COMPOSITION (% GAS)** 75/25

ELECTRODE-FLUX (CLASS) —

FLOW RATE 20GFH **GAS CUP SIZE** —

PREHEAT:

PREHEAT TEMP., MIN. —

INTERPASS TEMP., MIN. — MAX. —

POSITION:

POSITION OF GROOVE FLAT **FILLET** ✓

VERTICAL PROGRESSION UP — DOWN —

ELECTRICAL CHARACTERISTICS:

MACHINE PARAMETERS

TRANSFER MODE SHORT-CIRCUITING

SHORT-CIRCUITING ✓ GLOBULAR___ SPRAY___

CURRENT VOLTS 21 AMPS 120

AC___ DCEP ✓ DCEN___ PULSED___

OTHER___

TUNGSTEN ELECTRODE (GTAW) SIZE — TYPE —

TECHNIQUE:

FEED RATE 16IPM **STRINGER OR WEAVE BEAD** —

MULTI-PASS OR SINGLE PASS ✓ **NO OF ELECTRODES**___

ELECTRODE SPACING: LONGITUDINAL___ LATERAL___ ANGLE___

CONTACT TUBE TO WORK DISTANCE 3/4

PEENING — **INTERPASS CLEANING** HAND BRUSH FOR GLASSING

POSTWELD HEAT TREATMENT:

TEMPERATURE — **TIME.** —

Appendix C: Material Specifications

1. SCOPE

1.1 This specification covers carbon steel bar size shapes, bars and square and rectangular structural tubing.

1.2 The following classes of bars, bar size shapes and structural tubing are included:

Class A - Hot rolled carbon steel bars and bar size shapes with defined chemical and mechanical properties.

Class B - Cold formed square and rectangular structural steel tubing with defined chemical and mechanical properties.

2. APPLICABLE DOCUMENTS

2.1 Unless otherwise specified, the date of issue of all specifications, standards, publications, etc., invoked by this procurement Specification shall be those in effect on the date specified in the Purchase Order or request for proposal.

SPECIFICATIONS

INDUSTRIAL

ASTM-A6 - General Requirements for Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use

ASTM-A36 - Structural Steel

ASTM-A500 - Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

3. REQUIREMENTS

3.1 Material furnished under this specification shall conform to the applicable requirements of ASTM A6, as defined herein and in referenced specifications.

3.2 The steel shall be made by one or more of the following processes: open-hearth, basic-oxygen, or electric furnace.

3.2.1 No rimmed or capped steel shall be used for bars over 1/2 inch. No rimmed or capped steel shall be used for bar size shapes.

3.2.2 Class B material shall be seamless or ERW (Electric Resistance welded).

Steel used to produce this tube shall be strand (continuous) cast from semi-killed or killed steel.

3.3 Chemical Requirements

3.3.1 Class A bars and bar size shapes. The heat analysis of Class A material shall conform to the requirements described in Table 1. Material shall be supplied as special quality.

3.3.2 Class B tubing. The product analysis for Class B structural steel tubing in square and rectangular sizes shall conform to the requirements described in Table 2. Heat analysis is not required for Class B material.

3.3.3 Product analysis. The steel shall conform, on product analysis, to the requirements prescribed in Table 1 or 2, as applicable.

3.3.3.1 Product analysis is not applicable to bar size shapes or flat bars 1/2 in. and under in thickness.

TABLE I
CHEMICAL REQUIREMENTS FOR CLASS A MATERIAL
CHEMICAL RANGES AND LIMIT, %
HEAT AND PRODUCT ANALYSES

Thickness, Inches	Carbon		Manganese		Phosphorus		Sulfur	
	Heat Max.	Product Max.	Heat	Product	Heat Max.	Product Max.	Heat Max.	Product Max.
BARS								
To 3/4, incl.	0.26	0.30	-	-	0.04	0.05	0.05	0.06
Over 3/4 to 1-1/2 incl.	0.27	0.31	0.60 0.90	0.54 0.98	0.04	0.05	0.05	0.06
Over 1-1/2 to 4 incl.	0.28	0.32	0.60 0.90	0.54 0.98	0.04	0.05	0.05	0.06
Over 4	0.29	0.33	0.60 0.90	0.54 0.98	0.04	0.05	0.05	0.06
BAR SIZE SHAPES								
All sizes	0.26	0.30	-	-	0.04	0.05	0.05	0.06

TABLE 2
CHEMICAL REQUIREMENTS FOR CLASS B MATERIAL
CHEMICAL RANGES AND LIMITS, %
PRODUCT ANALYSES

Size	Carbon, Max. Product	Phosphorus, Max. Product	Sulfur, Max. Product
All	0.30	0.05	0.063

3.4 Tensile Requirements

3.4.1 Class A material, as represented by the test specimen provided in ASTM A6, and except as specified in 3.4.1.1, shall conform to the tensile property requirements specified in Table 3.

3.4.1.1 Shapes less than 1 sq. inch in cross section and bars other than flats, less than 1/2 inch in thickness or diameter need not be subjected to tension tests by the producer.

3.4.1.2 For material under 5/16 inch in diameter or thickness, a deduction from the percentage of elongation in 8 inch, as specified in Table 3, of 1.25% shall be made for each decrease of 1/32 inch of the specified thickness or diameter below 5/16 inch.

TABLE 3
MECHANICAL REQUIREMENTS
FOR CLASS A MATERIAL

	Bars	Bar Size Shapes
Ult. Tensile Strength, psi	58,000 - 80,000	58,000 - 80,000
Yield Point, min., psi	36,000	36,000
Elongation in 8 in. min. %	20 *	20 *
Elongation in 2 in., min %	23	23

* - See 3.4.1.2.

3.4.2 Class B material, as represented by the test specimen, as provided in ASTM A500 and ASTM A370 Supplement II, shall conform to the tensile property requirements specified in Table 4.

TABLE 4
MECHANICAL REQUIREMENTS
FOR CLASS B MATERIALS
(STRUCTURAL TUBING)

Tensile Strength, min., psi	58,000
Yield Strength, min., psi	46,000
Elongation in 2 in., min.	23 *

* - Applies to specified wall thicknesses 0.180 in. and over. For wall thickness under 0.180 in., the minimum elongation shall be calculated by the formula: percent elongation in 2 in. = 61t + 12.

The following table gives computed minimum values for longitudinal strip tests:

Wall Thickness	Elongation in 2 in., min. %
0.180	23
0.165	22
0.148	21
0.134	20
0.120	19.5
0.109	19
0.095	18
0.083	17
0.065	16
0.049	15
0.035	14

3.5 When galvanizing is required, procurement documents should specify the appropriate coating specification.

3.6 When painting is required, procurement documents should indicate the type and color of paint required. NOTE: Material to be painted must be descaled prior to painting.

3.7 Identification Requirements.

3.7.1 Class A material shall be marked in accordance with the Ordering Data specified elsewhere in this order.

3.7.2 Class B material shall be marked in accordance with the requirements of 5.1.

4. QUALITY ASSURANCE PROVISIONS

4.1 Responsibility for inspection. Unless otherwise specified in the contract, the contractor is responsible for the performance of all inspection requirements specified herein. Except as otherwise specified in the contract, the contractor may use his own or any other facilities suitable for the performance of the inspection requirements specified herein, unless disapproved by the Electric Boat Division or the Government. The Electric Boat Division reserves the right to perform any of the inspections set forth in the specification where such inspections are deemed necessary to assure supplies and services conform to prescribed requirements.

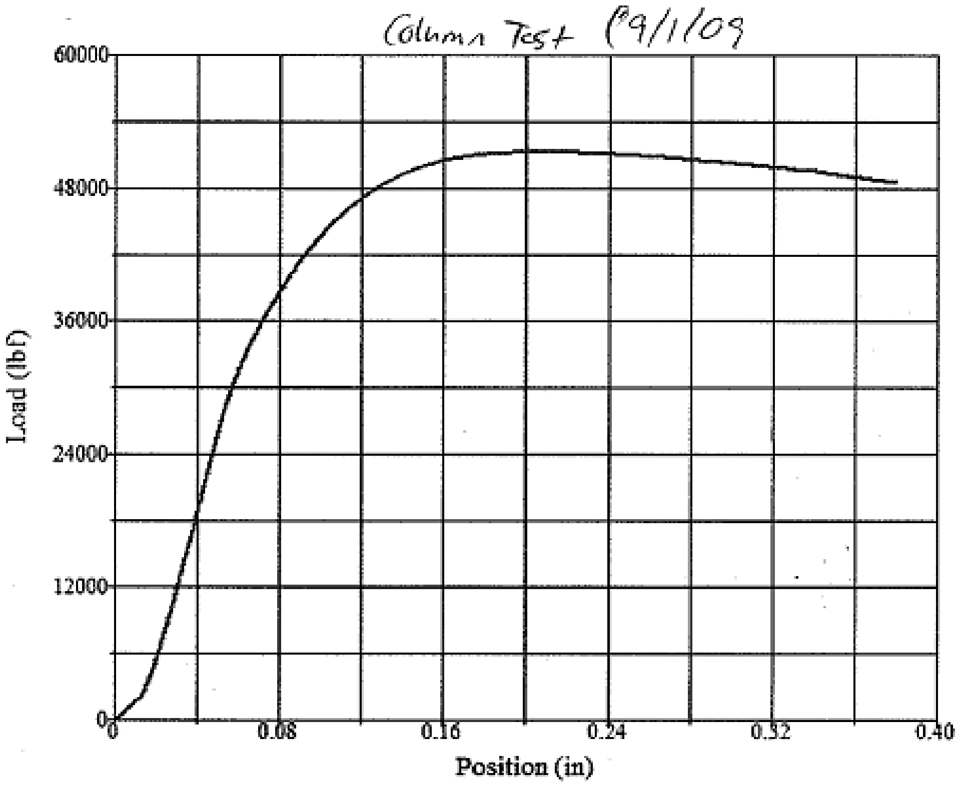
4.2 Quality Conformance Inspection.

4.2.1 Lot Size.

4.2.1.1 A lot for Class A material shall consist of material from the same heat or blow submitted for inspection at one time unless otherwise specified.

Appendix D: Static Test Results

CEINSTRON1307 9:44:43 AM 9/1/2009



Test Summary

Counter:	307
Elapsed Time:	00:12:58
Procedure Name:	6X12 Cylinder
Start Date:	9/1/2009
Start Time:	9:30:31 AM
End Date:	9/1/2009
End Time:	9:43:29 AM
Workstation:	CEINSTRON1
Tested By:	default
Material:	A500
Comments:	EB-2

Test Results

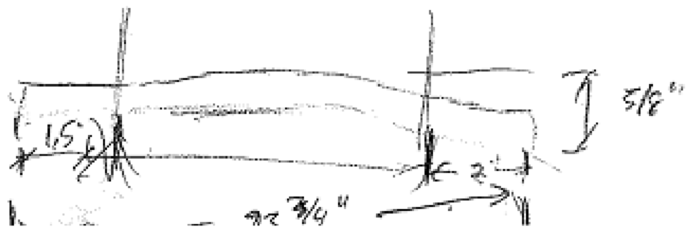
Area:	0.8973 in²
Compressive Strength:	57097 psi
Peak Load:	51233 lbf
Width:	2.0000 in
Corner Radius:	0.2500 in
Wall Thickness:	0.1250 in

$23\frac{3}{4} - 3\frac{1}{2}$

$20 - \frac{1}{4}$

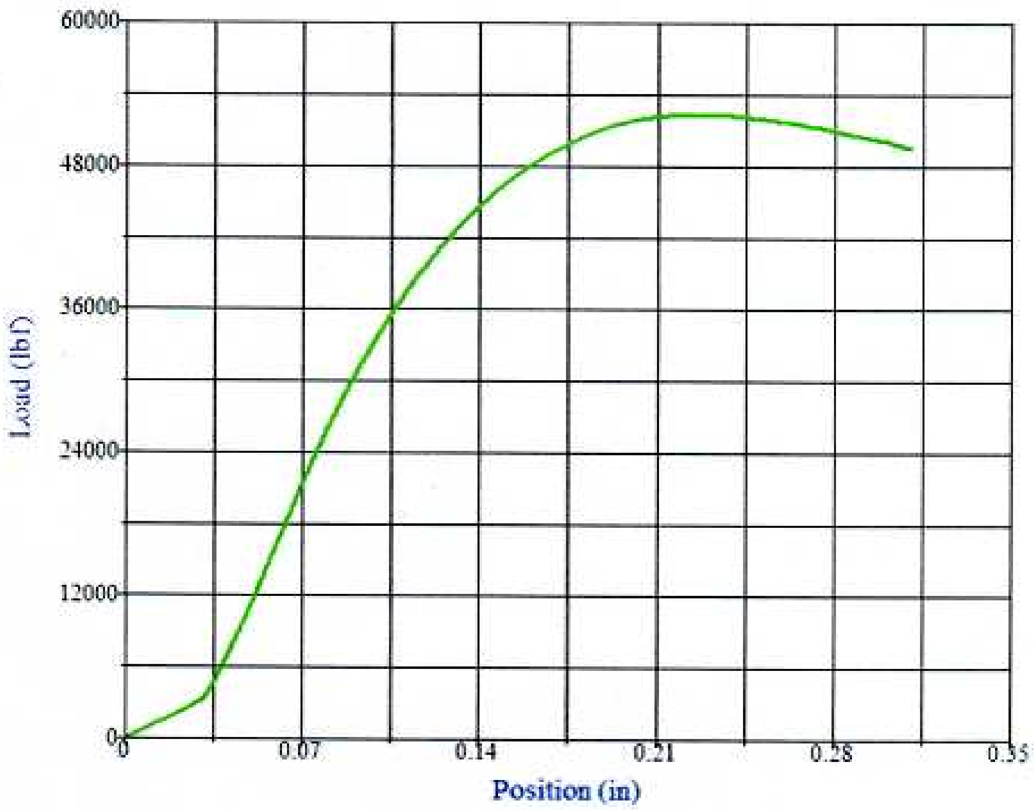
$\frac{20.25}{23.75} =$ K 0.86

1.0



CEINSTRON1547 12:34:19 PM 3/3/201(

Long column static



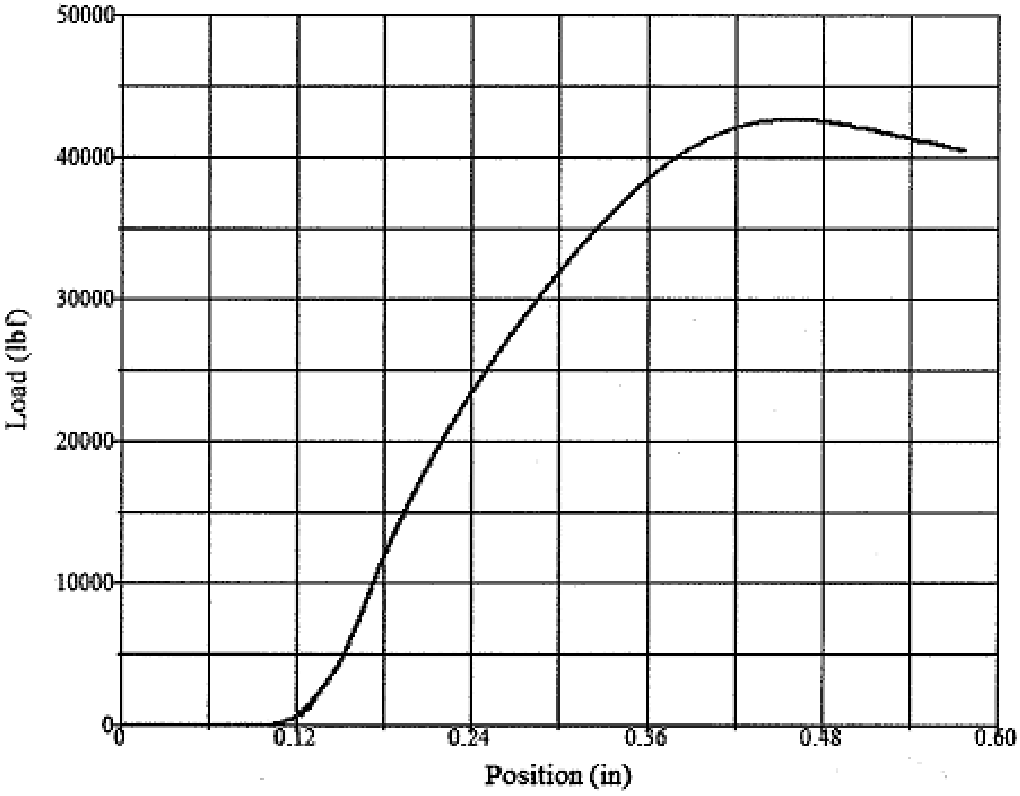
Test Summary

Counter:	547
Elapsed Time:	00:25:59
Specimen Identification:	EB Long
Material:	A500
Procedure Name:	Tube Comp. -2 Ext
Start Date:	3/3/2010
Start Time:	10:04:07 AM
End Date:	3/3/2010
End Time:	10:30:06 AM
Workstation:	CEINSTRON1
Tested By:	default

Test Results

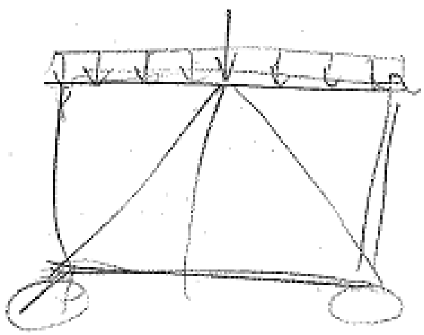
Area:	0.8973 in^2
Peak Load:	52282 lbf
Width:	2.0000 in
Corner Radius:	0.2500 in
Wall Thickness:	0.1250 in
Compressive Strength:	58266 psi

#1 (9/8/09)



Test Summary	
Counter:	309
Elapsed Time:	00:10:47
Procedure Name:	Tube
Start Date:	9/8/2009
Start Time:	8:27:36 AM
End Date:	9/8/2009
End Time:	8:38:23 AM
Workstation:	CEINSTRON1
Tested By:	default
Material:	A500
Comments:	EB Full Frame

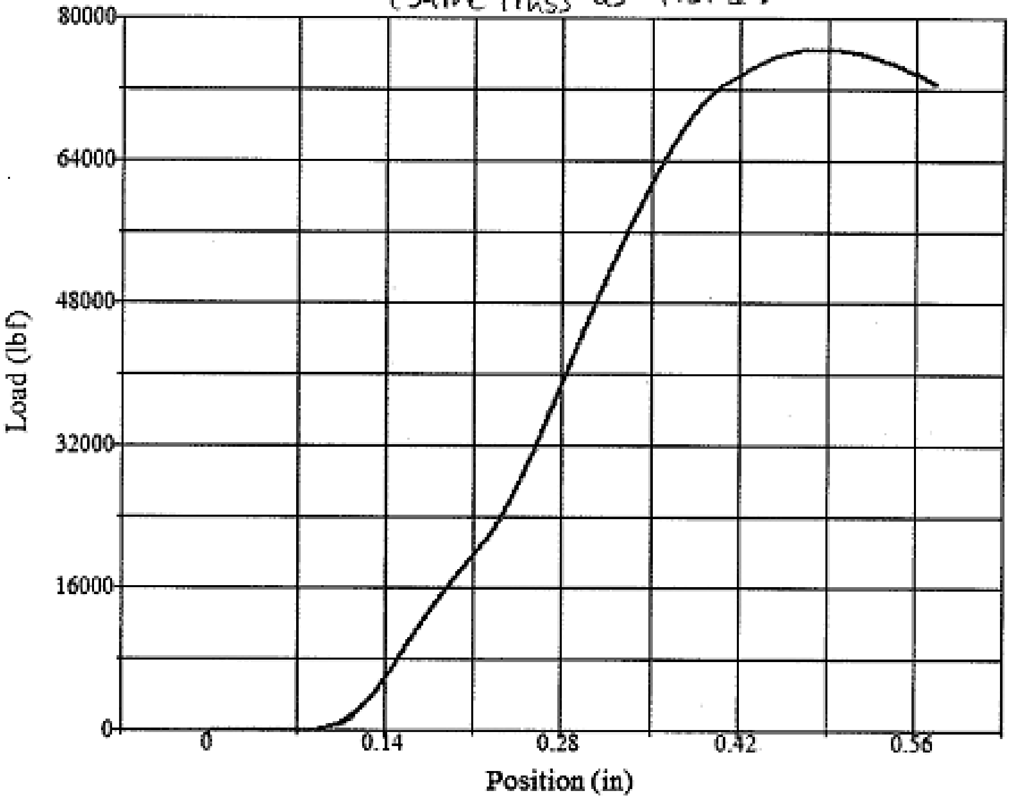
Test Results	
Area:	14.6741 in^2
Peak Load:	42611 lbf
Corner Radius:	0.1250 in
Wall Thickness:	0.1250 in
Width1:	34.0000 in
Width2:	25.0000 in



CEINSTRON1310 8:40:19 AM 9/10/2009

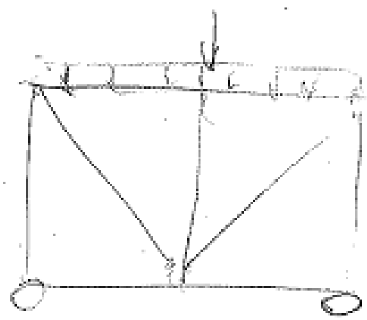
Test 2 9/10/09

(Same Truss as Test 1)



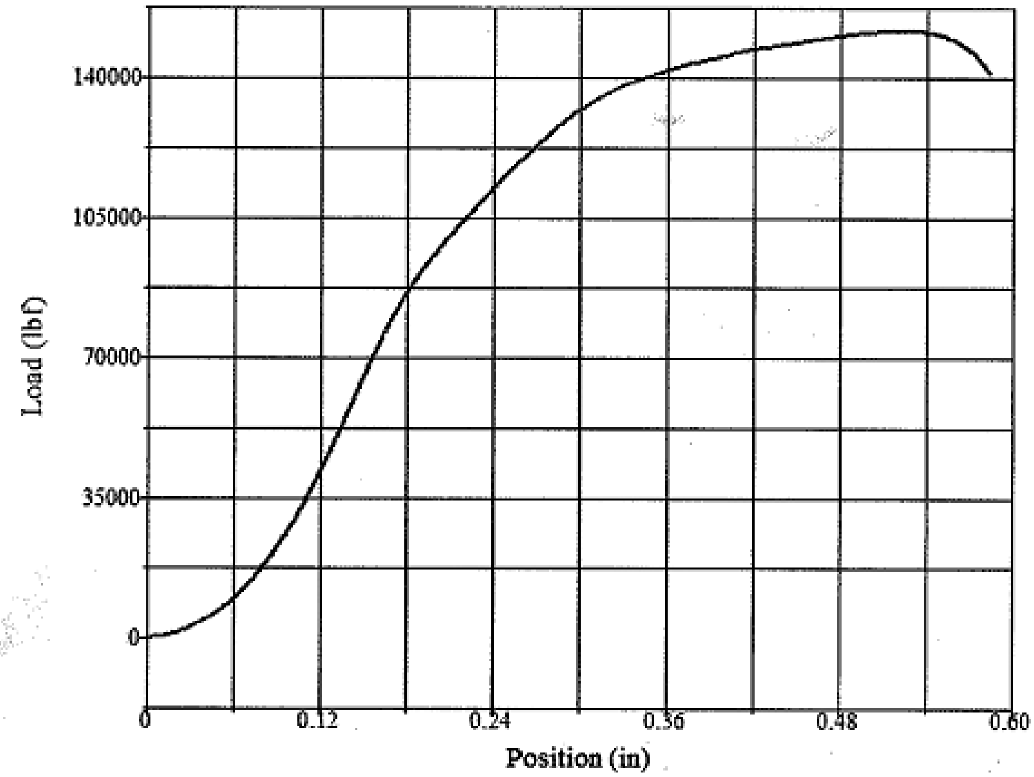
Test Summary	
Counter:	310
Elapsed Time:	00:19:14
Procedure Name:	Tube
Start Date:	9/10/2009
Start Time:	8:18:56 AM
End Date:	9/10/2009
End Time:	8:38:10 AM
Workstation:	CEINSTRON1
Tested By:	default
Material:	A500
Comments:	9-10-09 EB

Test Results	
Area:	14.6741 in²
Peak Load:	76382 lbf
Corner Radius:	0.1250 in
Wall Thickness:	0.1250 in
Width1:	34.0000 in
Width2:	25.0000 in



CEINSTRON1313 9:30:07 AM 9/16/2009

#3 (9/16/09)



Test Summary

Counter:	313
Elapsed Time:	00:19:03
Procedure Name:	Tube
Start Date:	9/16/2009
Start Time:	9:06:26 AM
End Date:	9/16/2009
End Time:	9:25:29 AM
Workstation:	CEINSTRON1
Tested By:	default
Material:	A500
Comments:	9-16-09 EB Plugs-2

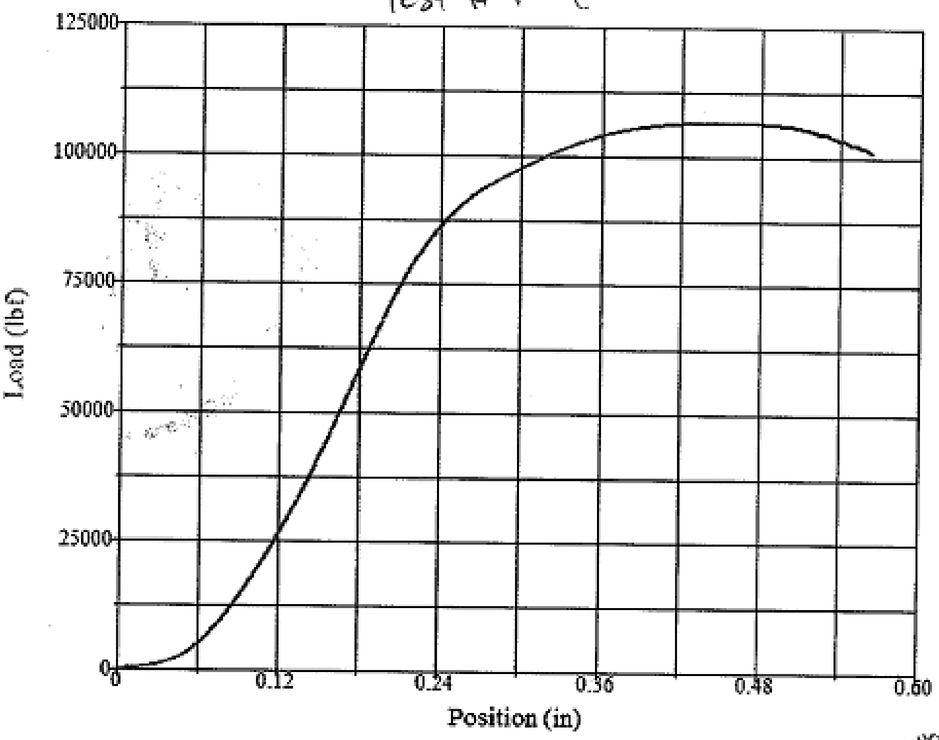
Test Results

Area:	29.6964 in²
Peak Load:	151617 lbf
Corner Radius:	0.2500 in
Wall Thickness:	0.2500 in
Width1:	24.0000 in
Width2:	36.0000 in

CEINSTRON1316

3:33:01 PM 9/25/2009

Test #4 (First Successful Test)



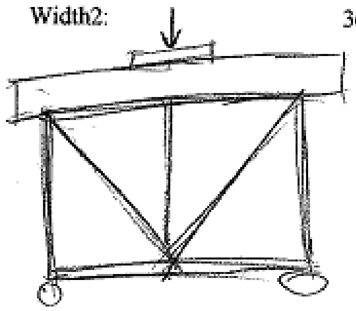
previously loaded truss ~ 80,000 lbs.

Test Summary

Counter:	316
Elapsed Time:	00:13:28
Procedure Name:	Tube
Start Date:	9/25/2009
Start Time:	3:17:41 PM
End Date:	9/25/2009
End Time:	3:31:09 PM
Workstation:	CEINSTRON1
Tested By:	default
Material:	A500
Comments:	9-25-09-EB

Test Results

Area:	14.9241 in²
Peak Load:	106549 lbf
Corner Radius:	0.1250 in
Wall Thickness:	0.1250 in
Width1:	24.0000 in
Width2:	36.0000 in

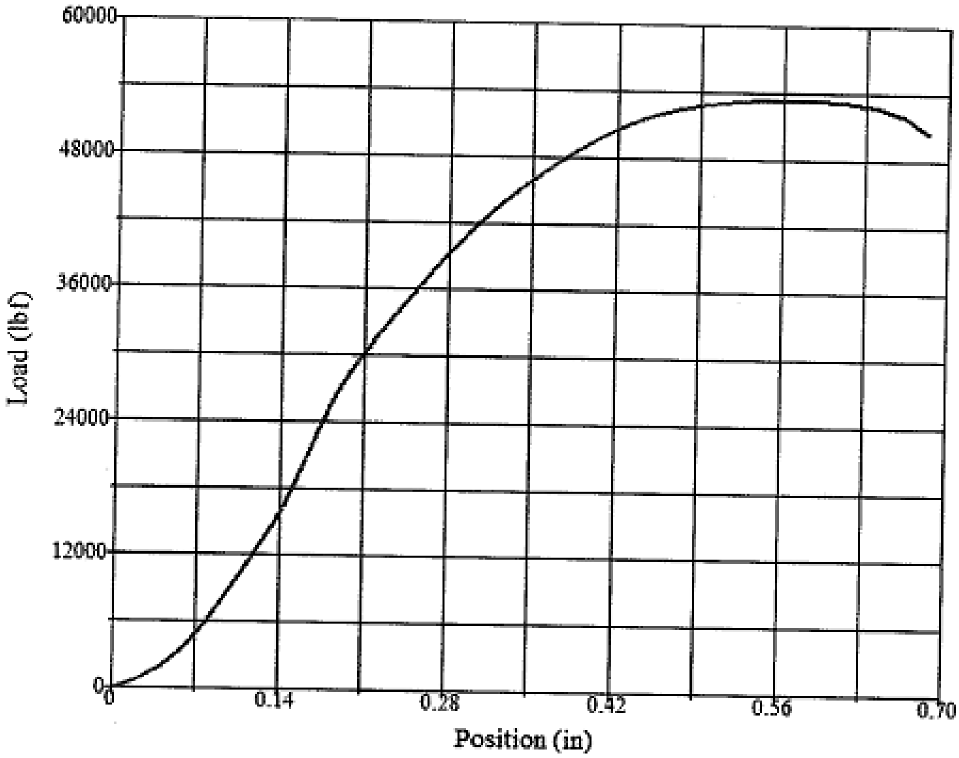


4x4 Solid Stock ASTM 1018 Steel Bar

CEINSTRON1320

12:37:03 PM 10/21/2009

Centric Point Load (Static)



Test Summary

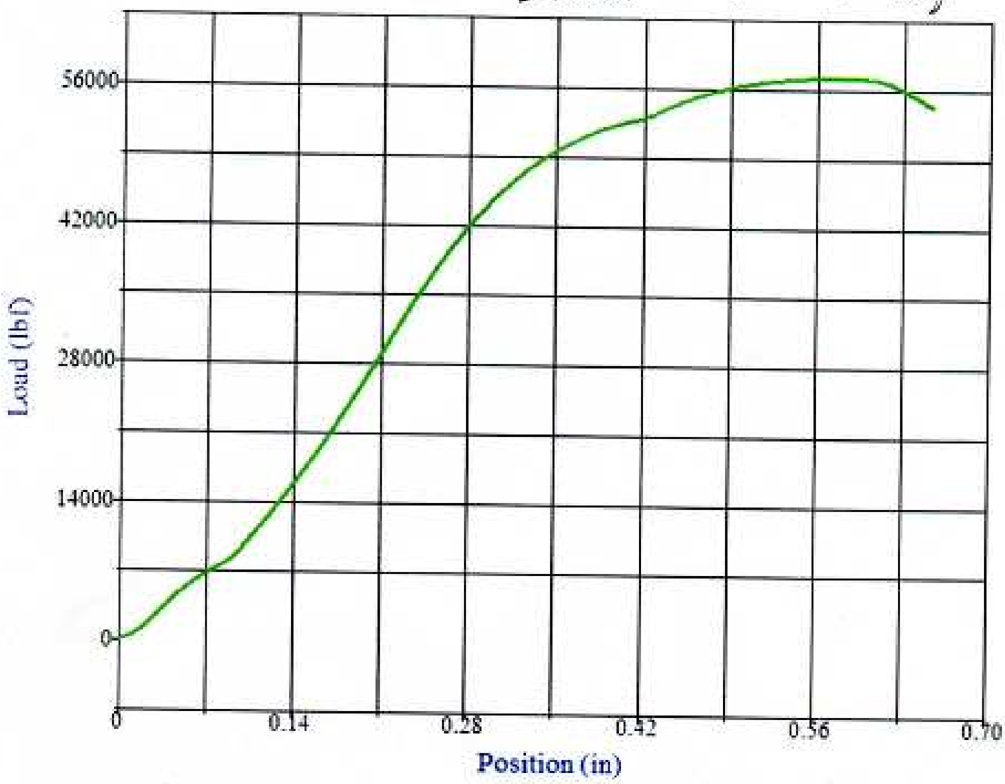
Counter:	320
Elapsed Time:	00:06:47
Procedure Name:	Tube
Start Date:	10/21/2009
Start Time:	12:29:15 PM
End Date:	10/21/2009
End Time:	12:36:02 PM
Workstation:	CEINSTRON1
Tested By:	default
Material:	A500
Comments:	EB 10-21-09-3

Test Results

Area:	30.1964 in²
Peak Load:	53278 lbf
Corner Radius:	0.2500 in
Wall Thickness:	0.2500 in
Width1:	25.0000 in
Width2:	36.0000 in

CEINSTRON1350 8:58:52 AM 12/16/2009

Symmetric test strain (Truss 6)



Test Summary

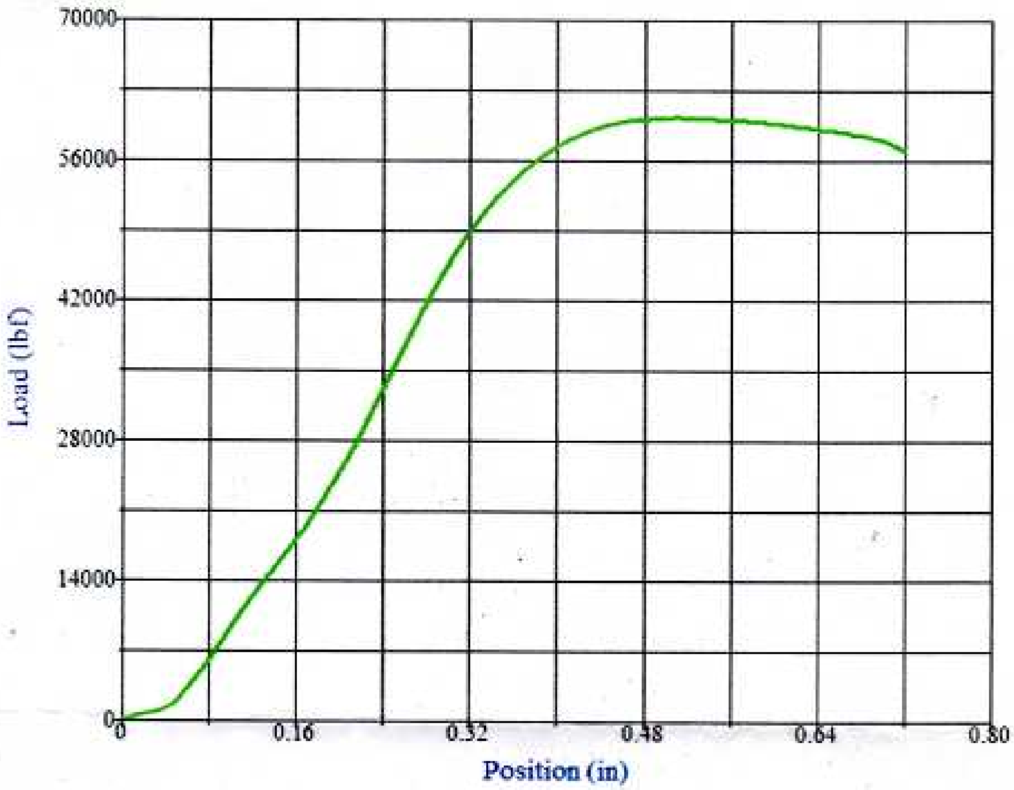
Counter:	350
Elapsed Time:	00:14:24
Specimen Identification:	EB-12-16-09
Material:	A500
Comments:	
Procedure Name:	dpel-block comp
Start Date:	12/16/2009
Start Time:	8:37:57 AM
End Date:	12/16/2009
End Time:	8:52:21 AM
Workstation:	CEINSTRON1
Tested By:	default

Test Results

Width:	28.0000 in
Length:	34.0000 in
Area:	952.0000 in²
Peak Load:	57126 lbf

CEINSTRON1544 8:44:53 AM 3/1/2010

Asymmetric (Truss 7)



Test Summary	
Counter:	544
Elapsed Time:	00:07:41
Procedure Name:	Tube
Start Date:	3/1/2010
Start Time:	8:32:34 AM
End Date:	3/1/2010
End Time:	8:40:15 AM
Workstation:	CEINSTRON1
Tested By:	default
Material:	A500
Comments:	EB Asem. 3/1/10

Test Results	
Area:	14.8973 in²
Peak Load:	60222 lbf
Corner Radius:	0.2500 in
Wall Thickness:	0.1250 in
Width1:	24.0000 in
Width2:	36.0000 in

Appendix E: Calculations

Conservation of Energy Calculations for Dynamic Columns and Trusses

In order to discover the drop weight required to buckle the columns and frames, the average buckling stress from the static test we taken (for a centered point load) and put through the conservation of energy laws and the mass is solved for:

COLUMN (Drop Height = 5 ft)

Deflection at Buckling ≅ 0.2 inches = d

Buckling load from Static Column test = F = 51,233 lbf

Energy to reach this load:

$$U = Fd = 51{,}233\ \text{lbf}\,(0.2'')\left(\frac{1\text{ft}}{12\text{in}}\right) = \underline{853.88\ \text{ft-lbf}}$$

$$853.88\ \text{ft-lbf} = \frac{1}{2} m v^2$$

$$v = \sqrt{2gh} = \sqrt{2(32.2)(5)} = 17.9\ \text{ft/s}$$

$$m = \frac{853.88\ \text{ft-lbf}\ (2)}{v^2} = \frac{853.88(2)}{(17.9)^2} = 5.3\ \text{slugs}$$

mass = 5.3 slugs (32.2 ft/s²) = 170.8 mass-pounds

$$\boxed{\text{mass required} = 170.8\ \text{lbm}}$$

FRAME (Drop Height = 5 ft)

Deflection at Buckling ≅ 0.5 inches = d

Average Buckling Load from Static Truss Tests = F = 57,125 lbf

Energy to reach this load:

$$U = Fd = 57{,}125\ \text{lbf}\,(0.5'')\left(\frac{1\text{ft}}{12''}\right) = \underline{2{,}380.25\ \text{ft-lbf}}$$

$$2{,}380.25\ \text{ft-lbf} = \frac{1}{2} m v^2 \quad (= 17.9\ \text{ft/s})$$

$$\text{mass} = \frac{2{,}380\,(2)}{(17.9)^2} = 14.8\ \text{slugs} \Rightarrow 14.8\ \text{slugs}\ (32.2\ \text{ft/s}^2) = 478\ \text{lbm}$$

$$\boxed{\text{mass required} = 478\ \text{lbm}}$$

Single Column Buckling Strength (AISC Formulas)

Using the AISC Specifications, Part E, the critical buckling load of a structural steel member in axial compression, is determined using the following formulas:

$$P_n = F_{cr} A_g$$

A_g for HSS 2" x 2" x 1/8" members

$$A_g = 0.84 \text{ in}^2 \quad \text{(AISC Table 1-12)}$$

F_{cr} (column flexural buckling stress) is determined using AISC Equations E3-2 and E3-3, as follows

a.) AISC Eqn E3-2

If $\frac{KL}{r} \leq 4.71\sqrt{E/F_y}$ (or $F_e \geq 0.44 F_y$)

$$F_{cr} = \left[0.658^{F_y/F_e}\right] F_y$$

b.) AISC Eqn E3-3

If $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ (or $F_e < 0.44 F_y$)

$$F_{cr} = 0.877 F_e$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

Using a conservative estimate for $\frac{KL}{r} \cong \frac{L}{r}$, for HSS 2x2x1/8 $r = 0.761$ in (AISC Table 1-12) and the length is 24", thus

$$\frac{KL}{r} = \frac{24}{0.761} = 31.54$$

Thus, using $F_{y,avg}$ from tests ($F_y = 56{,}367$ psi) and $E = 29{,}000{,}000$ psi

F_e = Euler Buckling Load

$$= \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29{,}000{,}000 \text{ psi})}{(31.54)^2}$$

$$F_e = 287{,}722 \text{ psi}$$

$$0.44 F_y = 0.44(56{,}367) = 24801.5 \text{ psi}$$

$$F_e > 0.44 F_y$$

To check:

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000000}{56367}} = 106.83$$

$$\frac{KL}{r} = 31.54 < 4.71\sqrt{\frac{E}{F_y}} = 106.83$$

Use Equation E3-2

$$F_{cr} = \left[0.658^{F_y/F_e}\right] F_y = \left[0.658^{0.1959}\right](56{,}367)$$

$$\boxed{F_{cr} = 51{,}929.48 \text{ psi}}$$

P_n = Nominal Strength = $F_{cr} A_g$

$$P_n = \left(51{,}929.48 \frac{\text{lb}}{\text{in}^2}\right)\left(0.84 \text{ in}^2\right)$$

$$\boxed{P_n = 43{,}620.76 \text{ lbs}}$$

ASD Allowable Compression Strength

$$\frac{P_n}{\Omega_c} = \frac{43{,}620.76 \text{ lb}}{1.67} = \boxed{26{,}120.22 \text{ lbs.}}$$

$$\text{LRFD} \Rightarrow \phi_c P_n = 0.90(43{,}620.76) = \boxed{39{,}258.68 \text{ lbs}}$$

AISC Column Buckling | 24" Column | $F_y = 46{,}000$ psi | 1/1

$\frac{KL}{r} = 31.54$ (Assuming $K = 1.0$)

As $\frac{KL}{r} < 4.71\sqrt{E/F_y}$ use AISC Eqn. E3-2

Critical Buckling Stress $= \left[0.658^{(F_y/F_e)}\right] F_y = F_{cr}$

F_e = Euler Buckling Load = 287,722 psi

$F_{cr} = \left[0.658^{(46{,}000/287{,}722)}\right](46{,}000) = \underline{43022.58 \text{ psi}}$

P_n = Nominal Strength = $F_{cr} A_g$ = 43,022.58 (0.84)

$\boxed{P_n = 36{,}138.96 \text{ lbs}}$

LRFD Factored Load = $\phi_c P_n = 0.90(36{,}138.96) = \underline{32{,}525.1 \text{ lbs}}$

AISC Buckling Stress | 36" Column

$\frac{KL}{r} = \frac{36}{0.761} = 47.31$, assuming a K value of $K=1.0$ (pin-ended column) to coincide with the estimates given by Electric Boat. (using ASTM $F_y = 46,000$ psi)

$4.71\sqrt{E/F_y} = 4.71\sqrt{\frac{29000000}{46,000}} = 118.26$

$\frac{KL}{r} < 4.71\sqrt{E/F_y}$, use AISC Eqn E3-2

$F_{cr} = \left[0.658^{(F_y/F_e)}\right] F_y$ (AISC Eqn E3-2)

F_e = Euler Buckling Load = $\frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29,000,000)}{(47.31)^2}$

$F_e = 127,876.83$ psi

$F_{cr} = \left[0.658^{\left(\frac{46,000}{127,876.83}\right)}\right](46,000) = 39,570$ psi

$\boxed{\text{Critical Buckling Stress} = F_{cr} = 39,570 \text{ psi}}$

P_n = Nominal Strength = $F_{cr} A_g = (39,570 \frac{lb}{in^2})(0.84 in^2)$

$\boxed{P_n = 33,239.1 \text{ lbs}}$

($A_g = 0.84$ from AISC Table 1-12)

Using $F_y = 56,367$ psi (from ASTM E8 Tensile Tests)

$F_{cr} = \left[0.658^{(56,367/127,876.83)}\right](56,367) = \underline{46,870.6 \text{ psi}}$

Nominal Strength = $P_n = 46,870.6 \text{ psi} (0.84 in^2) = \underline{39,371.3 \text{ lbs}}$

LRFD Factored Allowable Load = $\phi_c P_n = 0.90(39,371.3)$

$\boxed{\phi_c P_n = 35,434.2 \text{ lbs}}$

HSS 2" × 2" × 1/8" and the Compactness Criteria of the AISC

Based on the AISC Specifications, Part B, the compactness of a square HSS member in axial compression depends on whether or not its width-thickness ratio is equal to or less than the limiting value for the ratio, λ_p, given in AISC Table: B4.1.

Width-Thickness Ratio of HSS 2×2×1/8 (AISC Table 1-12)

$$b/t = 14.2$$

Limiting b/t value from Table B4.1

$$b/t \leq \lambda_p = 1.12\sqrt{E/F_y}$$

Using specified $F_y = 46$ ksi and standard $E = 29000$ ksi for ASTM A500 Gr. B steel:

$$\lambda_p = 1.12\sqrt{\frac{29000}{46}} = 28.12$$

Using the measured F_y from tensile tests in accordance to the ASTM E8 Material Testing Specification:

$F_{y,avg} = 56,367$ psi $\quad E = 29,000,000$ psi

$$\lambda_p = 1.12\sqrt{\frac{29000000}{56,367}} = 25.4$$

No matter what the value used for F_y is, the b/t ratio for HSS 2×2×1/8 is equal to 14.2, thus $b/t < \lambda_p$, and is a stock, compact member.

Single Column Failure Load (Static)

$\boxed{\begin{array}{l}\text{Column Peak Load} = 51{,}233 \text{ lbs} \\ \text{Compressive Strength} = 57{,}097 \text{ psi}\end{array}}$

Single Column Failure Load (Dynamic)

$\boxed{\begin{array}{l}\text{Peak Load} = 152{,}468.64 \text{ lbs} \\ \text{Strength} = 181{,}510.29 \text{ psi}\end{array}}$ (average between two columns that failed)

Compared to $\Rightarrow$ AISC calculated

Critical Stress = $F_{cr} = 51{,}929.48$ psi

$\frac{F_{cr}}{\Omega_c} = 31{,}095.~~50~~$ psi (ASD)

$\phi_c F_{cr} = 46{,}736.53$ psi (LRFD)

Critical Load = $P_n = 43{,}620.76$ lbs

$\frac{P_n}{\Omega_c} = 26{,}120.22$ lb (ASD)

$\phi_c P_n = 39{,}258.68$ lbs (LRFD)

Static Failure of Truss

Avrege Peak Load on two separate trusses

= $\boxed{55{,}202 \text{ lb w/ buckling center column}}$

Max Load applied Dynamically (w/ no failure)

$\boxed{= 167{,}322.32 \text{ lbs}}$

column (24"): $d = 0.2$ $F = 51{,}233$ lbF

$$U = (0.2 \text{ in})(51{,}233 \text{ lbF})(1 \text{ Ft}/12 \text{ in}) = 853.88 \text{ Ft lbF}$$

$$\text{mass} = \frac{2(853.88 \text{ Ft·lbF})}{(17.9)^2} = 5.3 \text{ slugs}$$

$$(5.3 \text{ slugs})(32.2 \text{ Ft}/s^2) = 170.8 \text{ lbm}$$

column (36"): $d = 0.2$ $F = 52{,}282$ lbF

$$U = (0.2 \text{ in})(52{,}282 \text{ lbF})(1 \text{ Ft}/12 \text{ in}) = 871.37 \text{ Ft·lbF}$$

$$m = \frac{2(871.37 \text{ Ft·lbF})}{(17.9)^2} = 5.44 \text{ slugs}$$

$$(5.44 \text{ slugs})(32.2 \text{ Ft}/s^2) = 175.14 \text{ lbM}$$

truss: $d = 0.5$ $F = 57{,}126$ lbF

$$U = (0.5 \text{ in})(57{,}126 \text{ lbF})(1 \text{ Ft}/12 \text{ in}) = 2{,}380.25 \text{ Ft·lbF}$$

$$\text{mass} = \frac{2(2{,}380.25 \text{ Ft·lbF})}{(17.9)^2} = 14.8 \text{ slugs}$$

$$(14.8 \text{ slugs})(32.2 \text{ Ft}/s^2) = 476.6 \text{ lbm}$$