EXPERIMENTS ON THE IMPACT OF HIGHER STRENGTH STEEL ON LOCAL BUCKLING AND OVERSTRENGTH OF LINKS IN EBFs

Progress Report to the American Institute of Steel Construction

Prepared by:
Gabriela Arce
Taichiro Okazaki
Michael D. Engelhardt
University of Texas at Austin

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SCOPE

This report summarizes progress on the experimental portion of the AISC sponsored research project on the behavior of link beams in seismic-resistant steel eccentrically braced frames (EBFs). Experiments for this project are being conducted at the University of Texas at Austin, with concurrent analytical studies being conducted at the University of California at San Diego. This progress report pertains only to the experimental portion of the project.

This progress report discusses the design and construction details of the test setup and the test specimens, and concludes with a report on the project status and anticipated schedule for testing.

BACKGROUND STUDIES OF TEST SETUP

A schematic representation of the test setup is shown in Fig. 1. The setup consists of a link, the beam segment outside of the link, and a column segment. Displacements will be applied at the bottom end of the column segment, while holding the beam segment outside of the link in place. The rigid-plastic kinematics of the test setup are shown in Fig. 2. Figures 3 and 4 qualitatively illustrate the shear forces and moments developed within the link, beam and column of the setup. Note that the kinematics and internal forces of the test setup closely replicate those of an actual EBF where the links are attached to the columns.

The design of this test setup combines features of previous EBF test setups by Kasai and Popov (1986) and Engelhardt and Popov (1989). The primary design objective for this test setup is to impose large shear forces on the link, combined with unequal end moments in the link, but with no axial force in the link. In the design of the setup, consideration was given so that the initial elastic link end moments would be unequal (greater at the column end of the link), similar to the condition of an actual EBF with links attached to the columns. With respect to evaluating flange local buckling in the link (a primary objective of this research project), the case of unequal end moment was considered an important case to investigate, due to the larger flexural demands at the column end of the link. Kasai and Popov (1986) demonstrated that unequal end moments result in a significantly different behavior for links as compared to the equal end moment case.

To evaluate typical elastic values of link end moments in an actual EBF, a six story – three bay prototype EBF designed by Ricles and Popov (1987) was analyzed. Details of the frame are shown in Fig. 5. A series of elastic analyses were conducted on this frame using the loading shown in Fig. 6. This figure also shows numbers identifying each link on the frame. This frame was analyzed for six different link lengths. Further, for each link length, the frame was analyzed for two conditions. For the first condition, all beam-to-column connections were modeled as moment-resisting connections (dual system). For the second condition, all beam-to-column connections were modeled as pins (non-dual system). For each case that was analyzed, the ratio of elastic link end moments was computed. This ratio was computed as M_A/M_B, where M_A is the elastic moment at the column end of the link, and M_B is the elastic moment at the beam end of the link.

Results of these frame analyses are plotted in Fig. 7 for the dual system, and in Fig. 8 in the non-dual system. In each plot, the elastic link end moment ratio M_A/M_B is plotted on the vertical axis, and the link length in plotted on the horizontal axis. Note that the link length is normalized by $e_o = M_p/V_p$. The link location identifiers shown in the plots in Figs. 7 and 8 correspond to the links in Fig. 6. Results are only plotted for the links in the outer bays (links attached to columns). For links in the central bay, the end moment ratios were very close to unity regardless of link length. The plots in Figs. 7 and 8 show that a wide range of elastic link end moment ratios are possible, depending on the location of the link within the frame and depending on the link length. Very large link end moment ratios, on the order of 4 or more, are possible for very short links. However, this ratio reduces quickly as link length increases.

To assist in evaluating the elastic link end moment ratio for the test setup, an analysis was conducted on an isolated link under bending and shear, with rotational springs at each end, as illustrated in Fig. 9. In this figure, the "A" end refers to the column end of the link, and the "B" end refers to the beam end of the link. If the spring constants at each end of the link are taken as k_A and k_B, then

$$M_A = - k_A \, \theta_A \tag{1a}$$
$$M_B = - k_B \, \theta_B \tag{1b}$$

A first order elastic analysis results in the following expression for link end moment ratio:

$$\frac{M_A}{M_B} = \frac{\alpha(2+\beta)}{\beta(2+\alpha)} \tag{2}$$

where α and β are measures of the relative flexural stiffness of the link and the end springs. These parameters are defined as follows:

$$\alpha = k_A \frac{e}{EI_{link}} \tag{3a}$$

$$\beta = k_B \frac{e}{EI_{link}} \tag{3b}$$

where e is the link length and EI_{link} is the elastic flexural stiffness of the link section. Equations 2 and 3 show the link end moment ratio is a function of the end restraints, as well as being a function of the ratio of the end spring stiffness to the bending stiffness of the link itself. If this ratio is large, then the link end moment ratio will be close to zero. If this ratio is small, then the end restraint will have a large influence on the link end moment ratio.

Applying the above analysis to the test setup (Fig. 1), the values of k_A and k_B can be computed as follows:

$$k_A = \frac{12EI_{col}}{L_{col}} \tag{4a}$$

$$k_B = \frac{3EI_{beam}}{L_{beam}} \tag{4b}$$

The lines plotted in Fig. 10 show the link end moment ratio computed for four different link sections: W10x19, W10x33, W16x36 and W18x40. As in the test setup, the column and beam sections are taken as W12x120 and W18x76, respectively, with dimensions L_{col} = 96 inches and L_{beam} = 200 inches. In these plots, link length is normalized by $e_o = M_p/V_p$. The plots in Fig. 10 show that the test setup will result in a unequal link end moments, as desired, with a wide range of end moment ratios possible, depending on the link section and link length. The smaller link sections (such as the W10x19) will experience link end moment ratios close to unity, whereas the larger link sections (such as the W16x36 and W18x40) will experience significantly higher end moment ratios. Note that for this research program, W10x19, W10x33 and W16x36 link sections will be tested. However, W18x40 link sections will be tested in this setup as part of a companion study, and so data on W18x40 links (with $e = e_o$, $2e_o$ and $3e_o$) will also be available.

In addition to the analytical study conducted for the test setup, as described above, the setup was also analyzed with the SAP2000 frame analysis program. In this frame analysis, rigid zones were added to each end of the link to more realistically represent the joint conditions. The results of these analyses are plotted as discrete points (marked with an "X") in Fig. 10. Note that the results of the SAP2000 analysis agree quite well with the analytically generated curves. The values of elastic link end moment ratio computed for each specimen from SAP2000 are also listed in Table 2 under the column marked M_A/M_B.

DETAILS OF TEST SETUP

An overall view of the test setup is shown in Fig. 11. Figures 12 to 15 provide additional views of the setup. The same beam and column sections will be reused for all specimens. Only the link section will be changed from one test to another. Note that the link specimen is provided with thick end plates at each end, and is bolted to the column flanges and to a mating end plate on the beam. The entire beam assembly outside of the link (along with the vertical support rods at each end of the beam) can be moved to accommodate different link lengths.

An overall view of the lateral support system is illustrated in Fig. 16, with further details shown in Figs. 17 to 20. Lateral supports are provided near the top and bottom ends of the column, at the beam end of the link, and at the far end of the beam away from the link. Each lateral support is designed to permit free movement in the plane of the specimen, but to prevent out-of-plane movement. All sliding contact surfaces at the column ends and link end are provided with a Teflon coating to minimize friction, so as to permit accurate measurement of link forces.

All components of the test setup were fabricated at the University of Texas Ferguson Laboratory for this project. Details of the test setup components are shown in Appendix B.

TEST SPECIMENS

A total of nine link specimens will be tested to evaluate flange slenderness limits for A992 steel. Three different sections will be used for the specimens: W10x19, W10x33 and W16x36. Key attributes of these sections are listed in Table 1. All specimens of a given section will be taken from the same heat of steel. Certified Mill Test Reports (CMTRs) for the test sections are reproduced in Appendix C. Tensile properties from the CMTRs are also listed in Table 1. Tensile coupon tests will be conducted on flange and web samples from each section later in the project.

Table 2 provides details of each of the nine test specimens. Listed in this table for each specimen is the link section (W10x19, W10x33 or W16x36), the expected elastic link end moment ratio M_A/M_B, the link length, stiffener locations, and target link plastic rotation angle according to the AISC Seismic Provisions for Structural Steel Buildings. Note that the test specimens represent a wide range of link lengths, including shear yielding links, flexural yielding links, and combined shear and flexural yielding links. Drawings of the test specimens are shown in Appendix A.

INSTRUCTION

Load cells are provided in the supporting rods at the top and bottom of the column, at the end of the link, and at the end of the beam. These load cells will permit determination of all internal forces in the link, beam and column. All load cells have been calibrated for this project against a calibration load cell traceable to NIST. In addition to the load cells, displacements and rotations will be measured at each end of the link, and at other locations in the test setup to permit complete determination of specimen deformations.

PROJECT STATUS AND SCHEDULE

The project is currently behind the originally anticipated schedule. The complexity of the test setup and time required to fabricate the setup were considerably greater than originally anticipated. However, the complexity of the test setup was ultimately deemed necessary to adequately control the specimen boundary conditions so as to generate high quality experimental data suitable for meeting the project objectives.

As of early December 2001, all components of the test setup have been fabricated, and the setup is being assembled. Figures 21 and 22 are photos of the setup during assembly. The first specimen has been fabricated (Fig. 23), and fabrication of five additional specimens is proceeding.

It is currently anticipated that the test setup will be completely assembled by early to mid January 2002, and that the first test will be completed by the end of January. It is also anticipated that all testing for this project will be completed by the end of May, 2002.

REFERENCES

Engelhardt, M. D. and Popov, E. P. (1989). "Behavior of long links in eccentrically braced frames." Report No. *UCB/EERC-89/01*, Earthquake Engrg. Res. Ctr., Univ. of California, Berkeley, Calif.

Kasai, K. and Popov, E. P. (1986). "A study of seismically resistant eccentrically braced frames." *Report No. UCB/EERC-86/01*, Earthquake Engrg. Res. Ctr., Univ. of California, Berkeley, Calif.

Ricles, J. M. and Popov, E. P. (1987). "Dynamic analysis of seismically resistant eccentrically braced frames." *Report No. UCB/EERC-87/07*, Earthquake Engrg. Res. Ctr., Univ. of California, Berkeley, Calif.

Table 1 - Test Specimen Sections

Section	$b_f/2t_f$	M_p/V_p	CMTR Tensile Properties	
			F_y (ksi)	F_u (ksi)
W10x19	5.1	15.2”	58.1	74.9
W10x33	9.1	25.2”	58.5	76.9
W16x36	8.1	24.1”	51.9	72.6

Table 2 - Test Specimens

Spec No.	Section	M_A/M_B	Link Length	Intermediate Stiffeners	Target γ_p
1	W10x19	1.23	23″ (=1.5 M_p/V_p)	4 @ 5.75	.08 rad
2	″	1.17	30″ (=2.0 M_p/V_p)	4 @ 6″	.055 rad
3	″	1.11	48″ (=3.2 M_p/V_p)	1 @ 6″ from each end	.02 rad
4	W10x33	1.47	23″ (=0.9 M_p/V_p)	3 @ 5.75″	.08 rad
5	″	1.29	36.6″ (=1.5 M_p/V_p)	5 @ 6.125″	.08 rad
6	″	1.22	48″ (=1.9 M_p/V_p)	4 @ 9.625”	.06 rad
7	″	1.15	73″ (=2.9 M_p/V_p)	1 @12″ from each end	.02 rad
8	W16x36	1.83	36.6″ (=1.5 M_p/V_p)	6 @ 5.25”	.08 rad
9	″	1.63	48″ (=2.0 M_p/V_p)	5 @ 8”	.055 rad

Notes:

1. Target γ_p = maximum link plastic rotation angle specified in Sect. 15.2g – AISC Seismic Spec.

2. Intermediate stiffeners: 3/8” thick; one side only (per Sect. 15.3b – AISC Seismic Spec.); spacing per Sect 15.3b to develop target γ_p

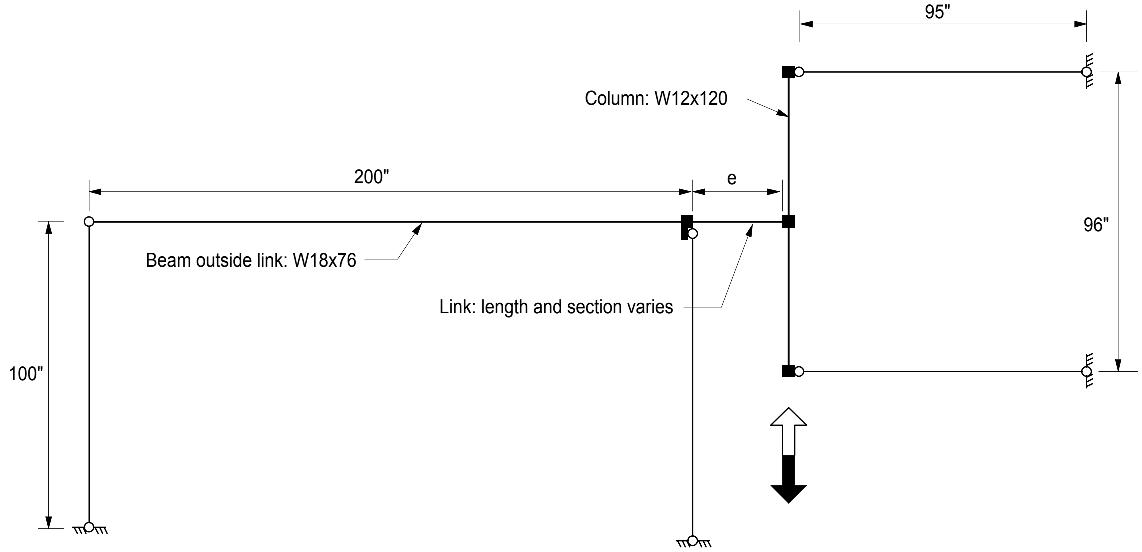


Fig. 1 – Schematic of Test Setup

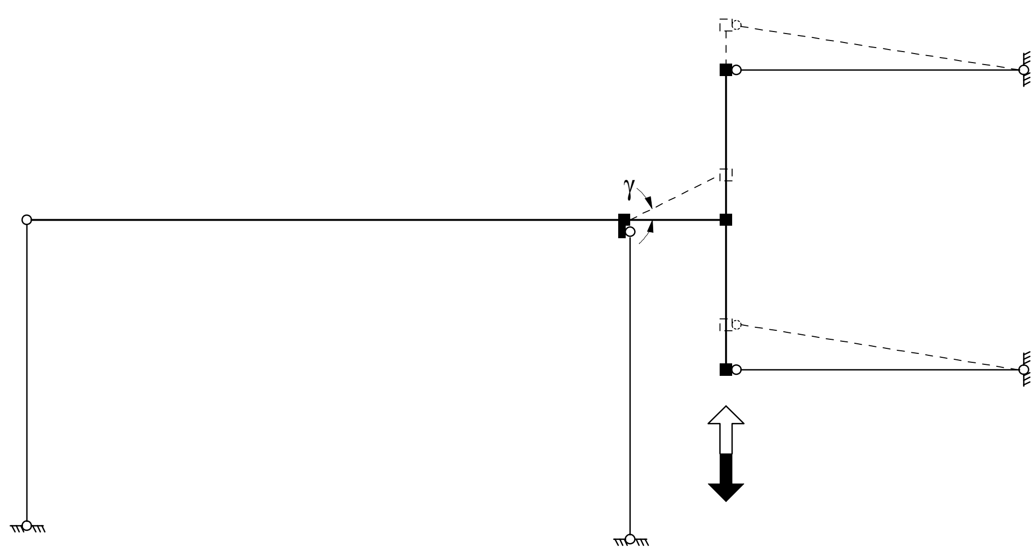


Fig. 2 - Rigid – Plastic Kinematics of Test Setup

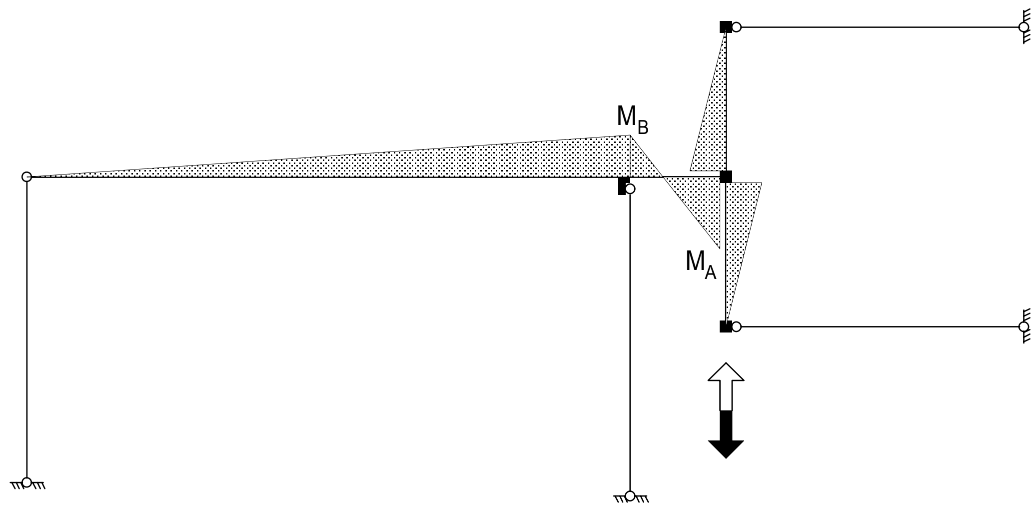


Fig. 3 – Moment Diagram for Test Setup

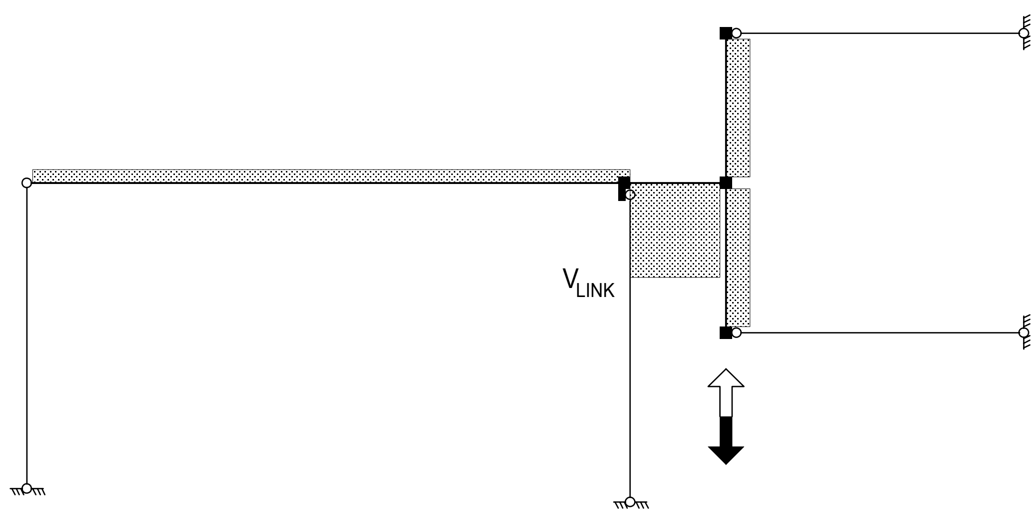


Fig. 4 – Shear Diagram for Test Setup

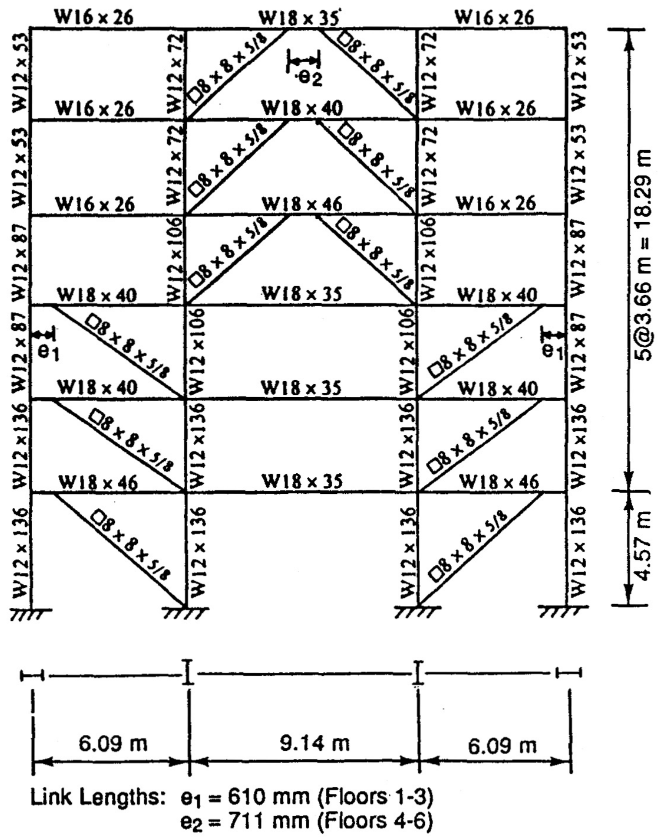


Fig.5 – Prototype Six Story EBF Used for Evaluation of Link End Moment Ratio (Ricles and Popov 1994)

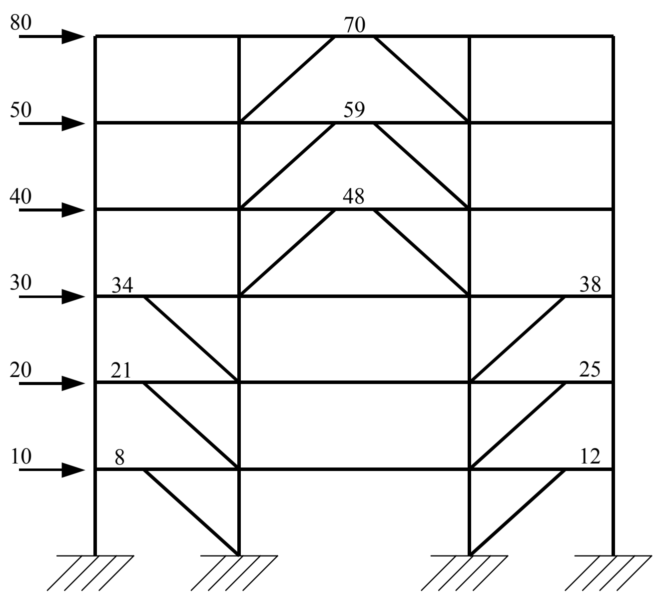


Fig. 6 – Loading and Link Identifiers for Prototype EBF

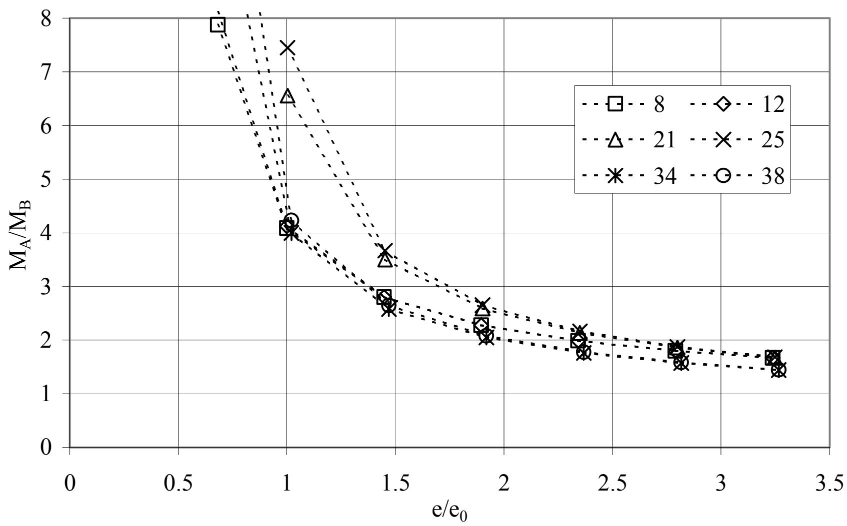


Fig. 7 – Link End Moment Ratios for Prototype EBF: Dual System

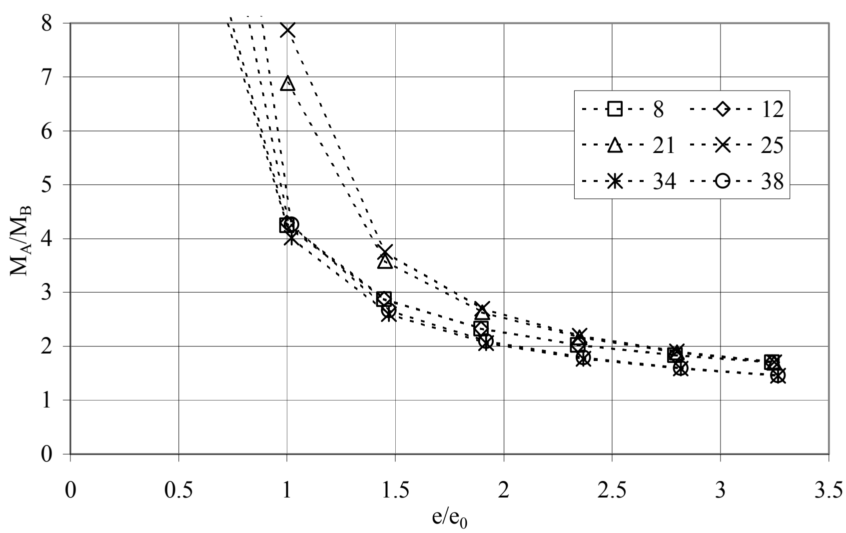


Fig. 8 – Link End Moment Ratios for Prototype EBF: Non-Dual System

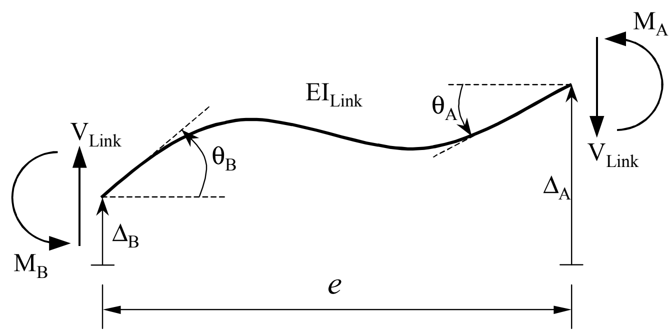


Fig. 9 – Isolated Link for Analysis of End Moment Ratio

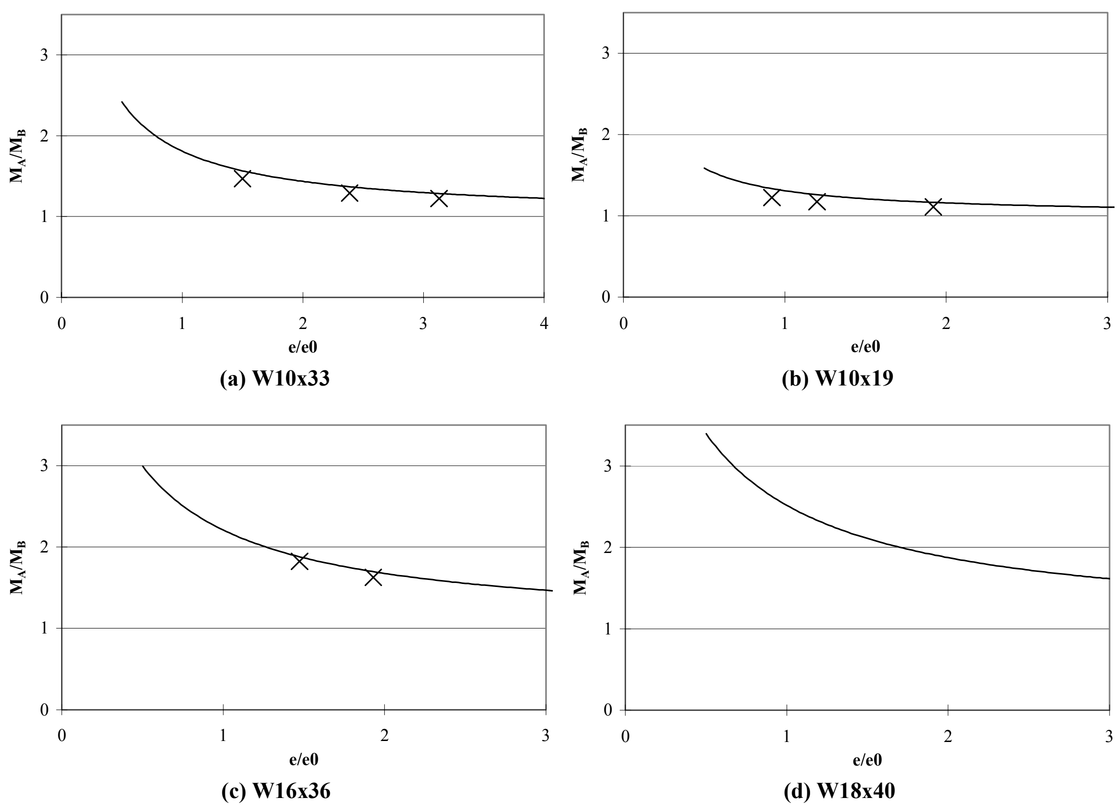


Fig. 10 – Link End Moment Ratios for Test Specimen Sections

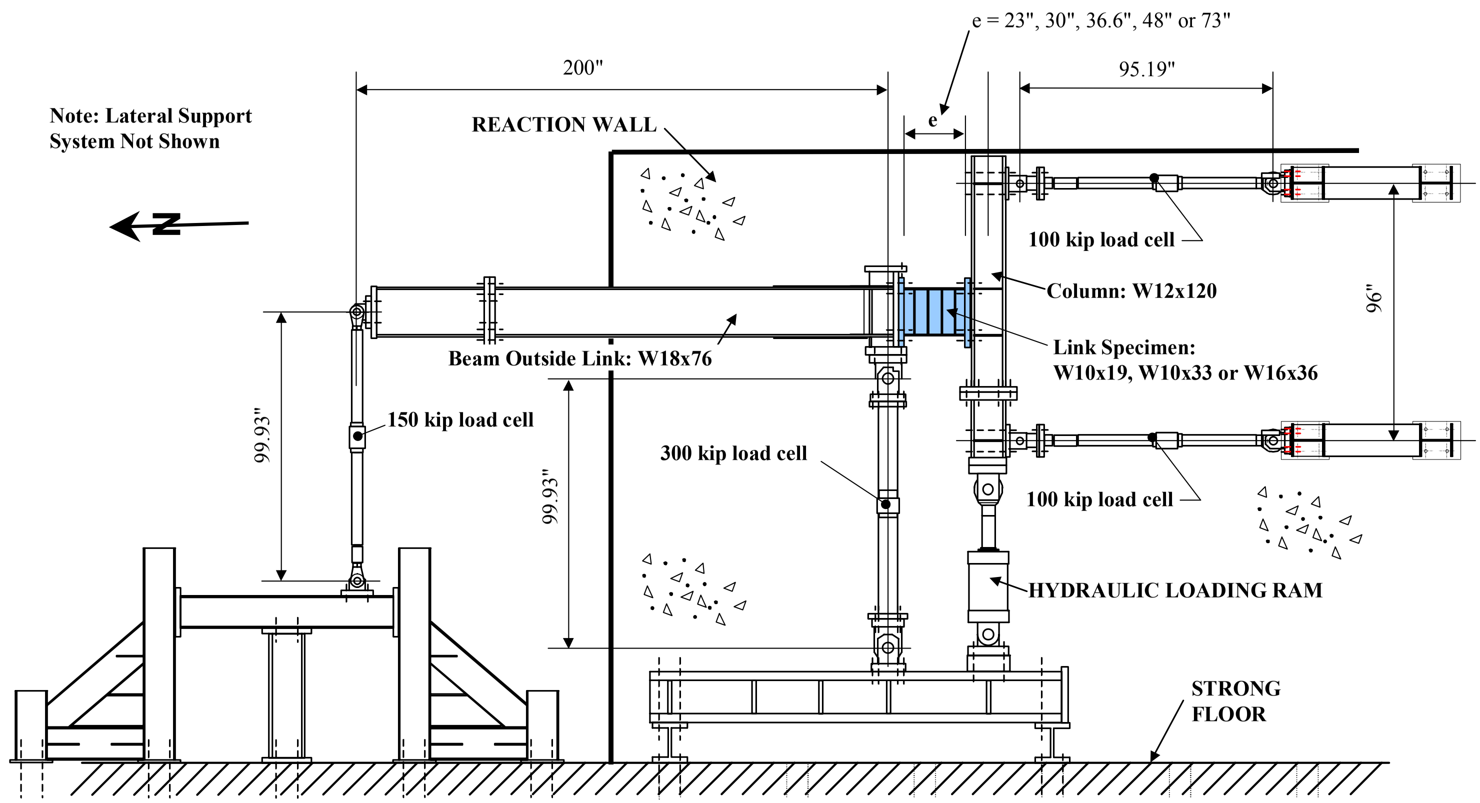


Fig. 11 – Overall View of Test Setup

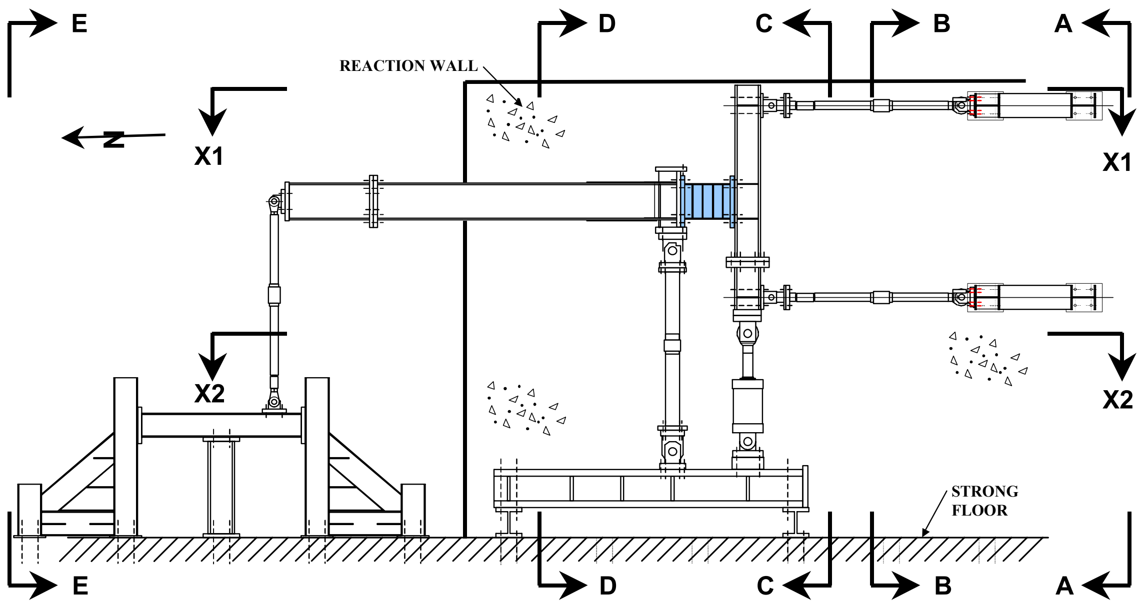


Fig. 12 – Test Setup: Location of Section Views

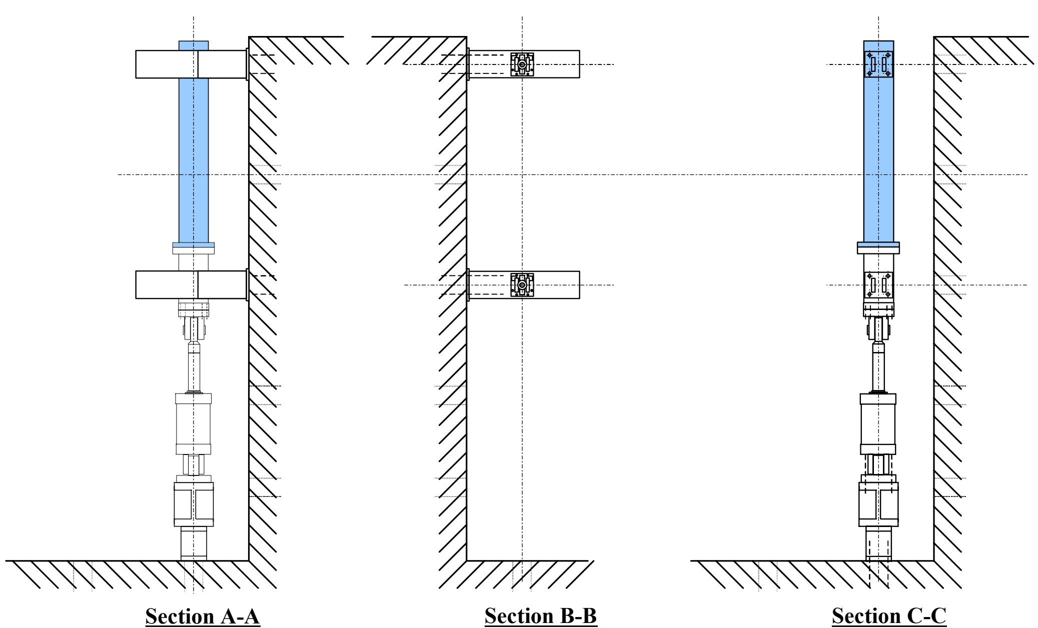


Fig. 13 – Test Setup: Sections A-A, B-B, and C-C

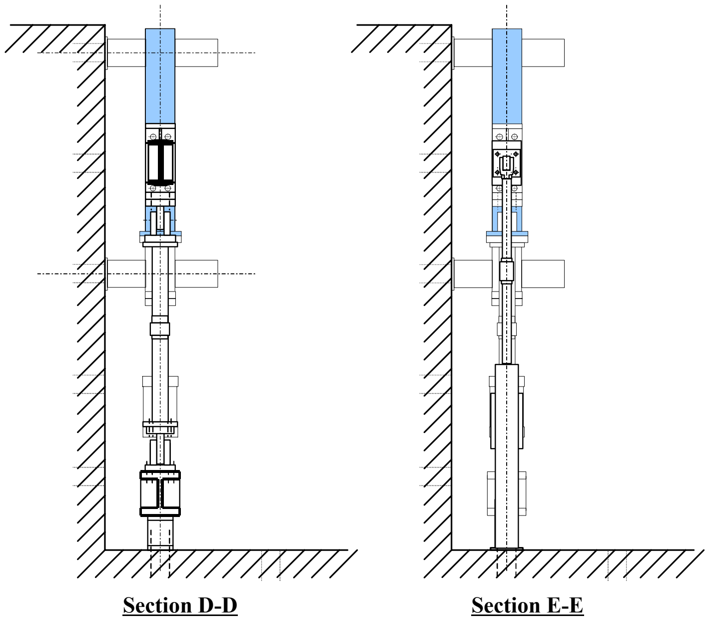


Fig. 14 – Test Setup: Sections D-D and E-E

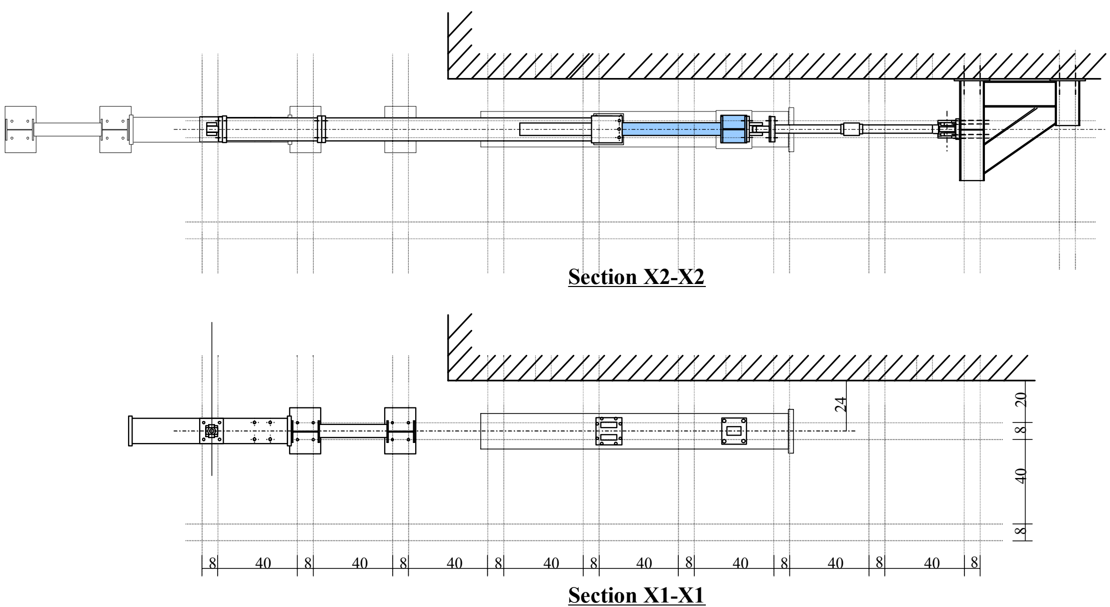


Fig. 15 – Test Setup: Sections X1-X1 and X2-X2

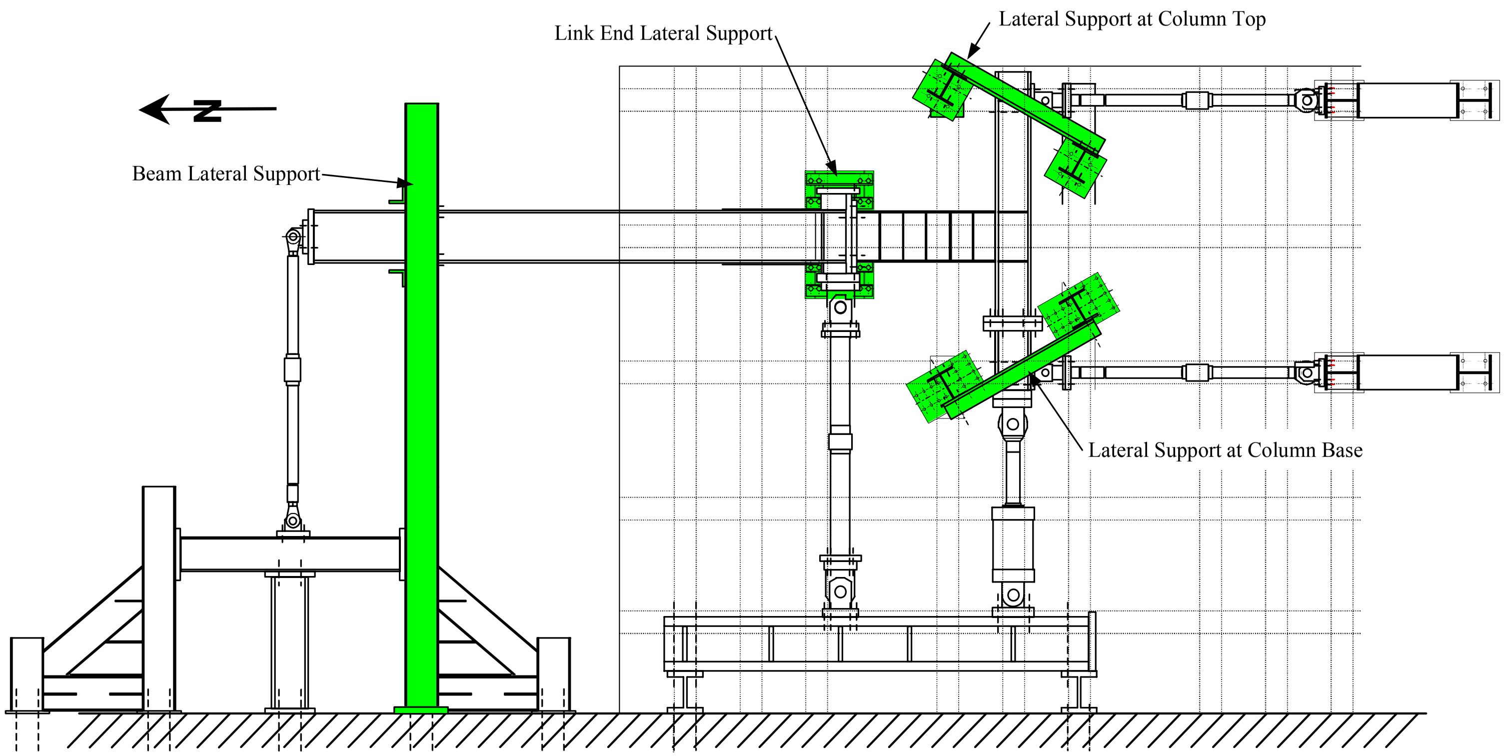


Fig. 16 – Lateral Support System: Overall View

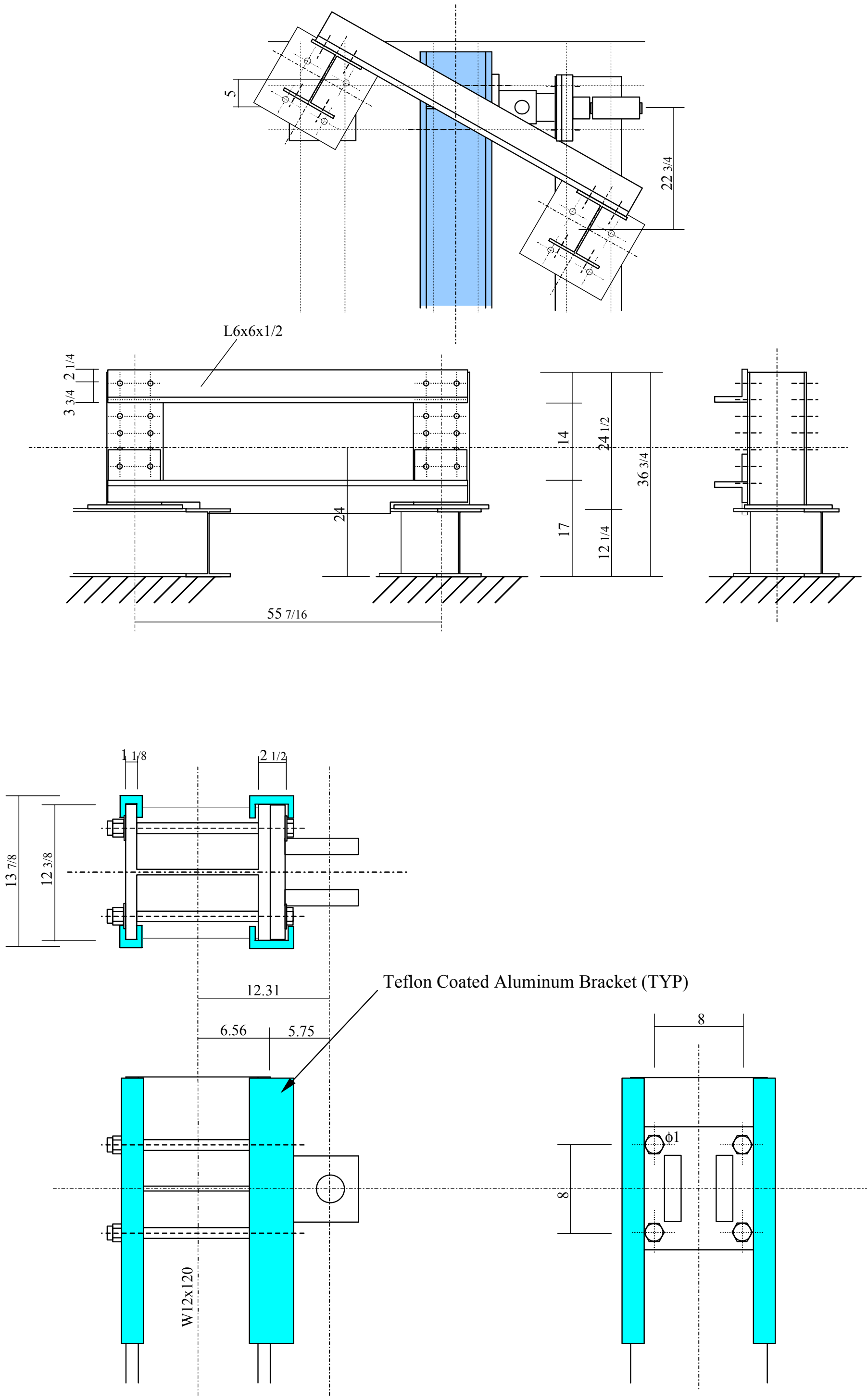


Fig. 17 – Lateral Support System: Details at Top of Column

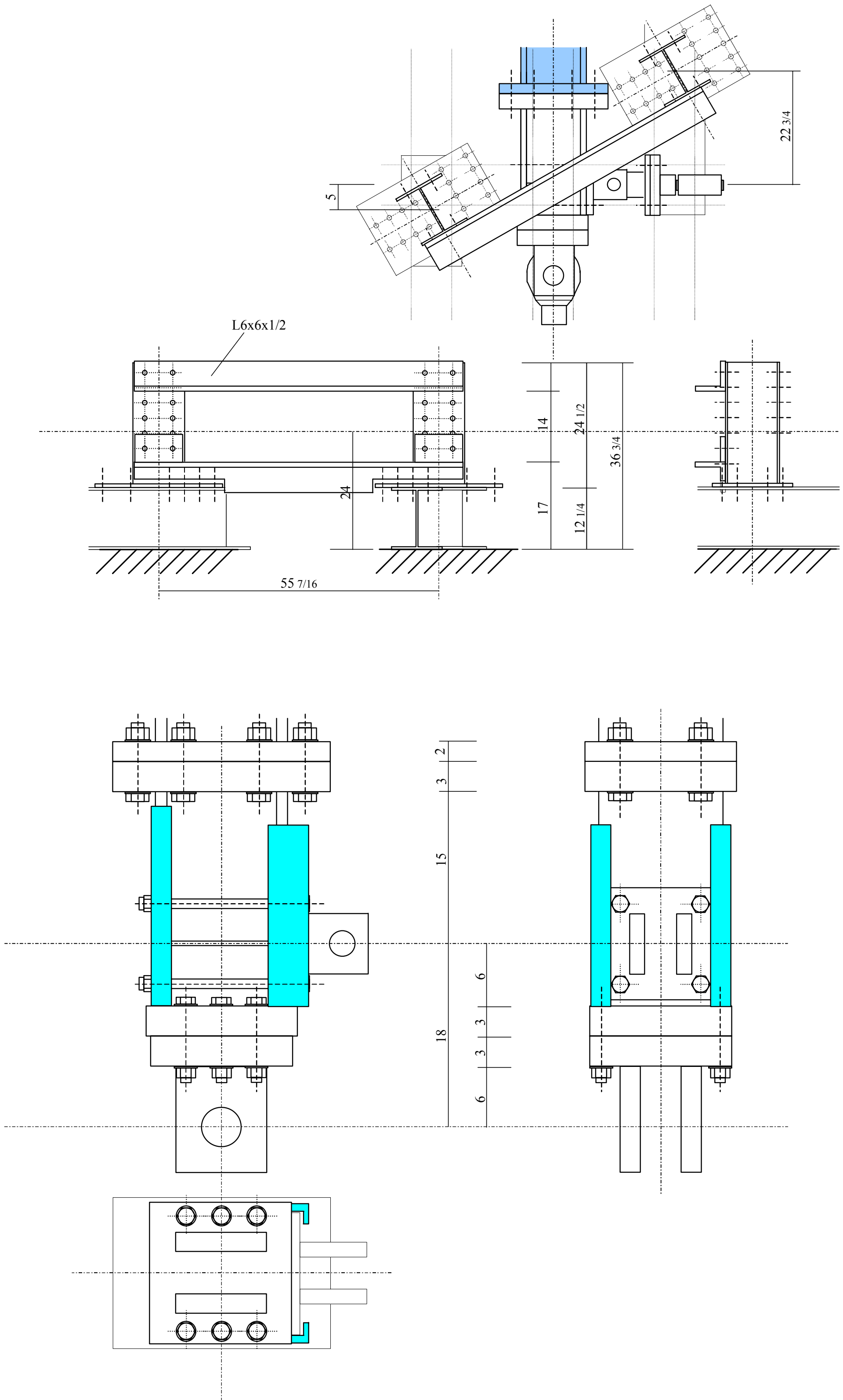


Fig. 18 – Lateral Support System: Details at Base of Column

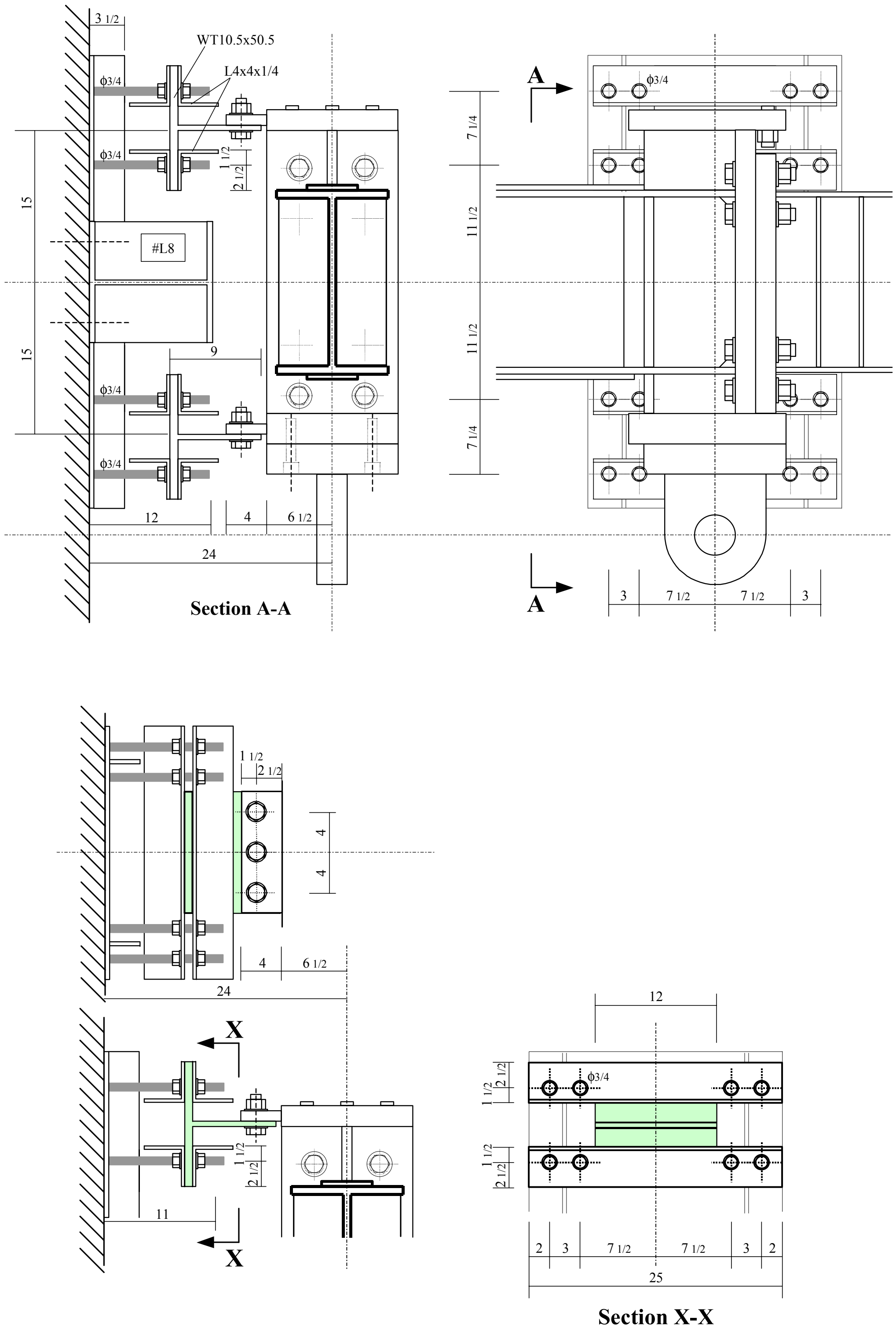
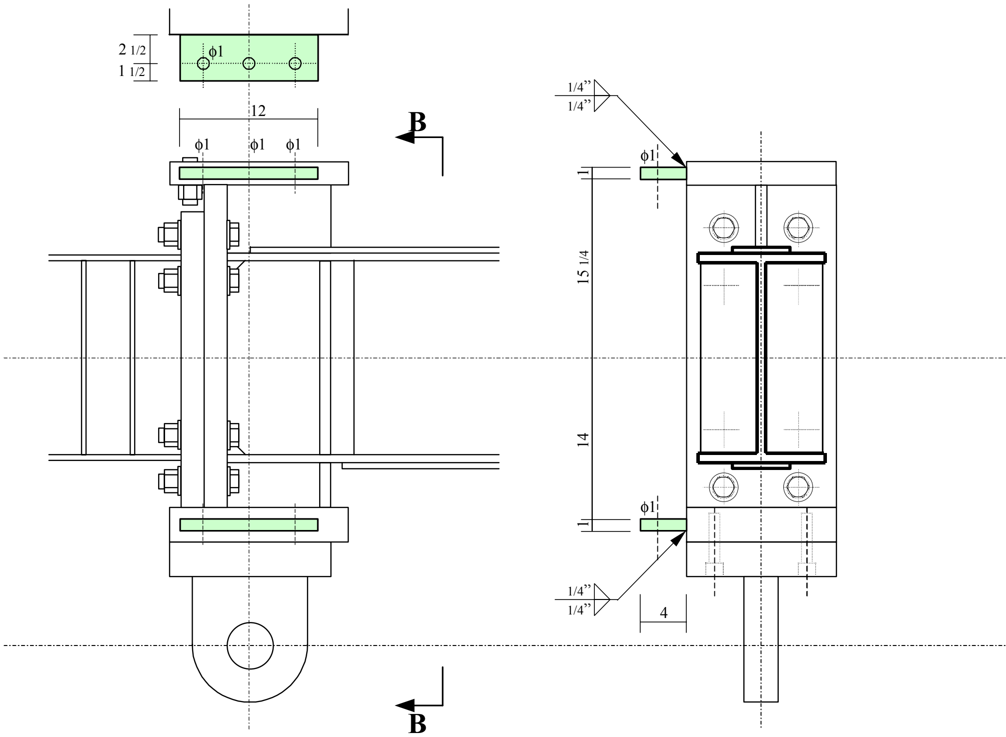
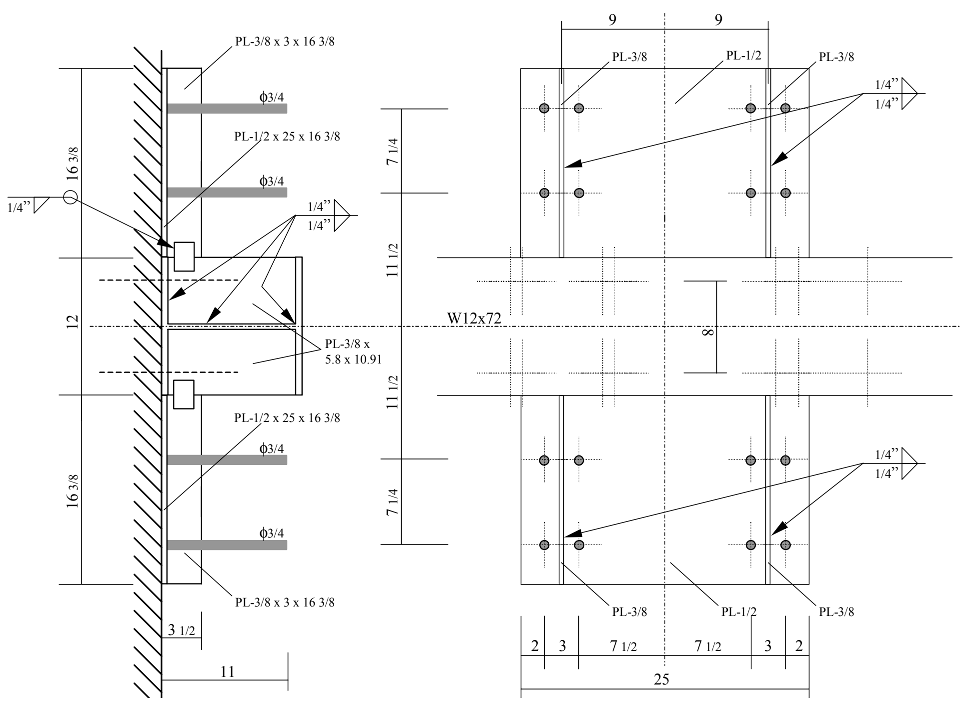


Fig. 19 – Lateral Support System: Details at Link End





Fixture for Attachment of Link End Lateral Support to

Fig. 19 – Lateral Support System: Details at Link End (cont.)

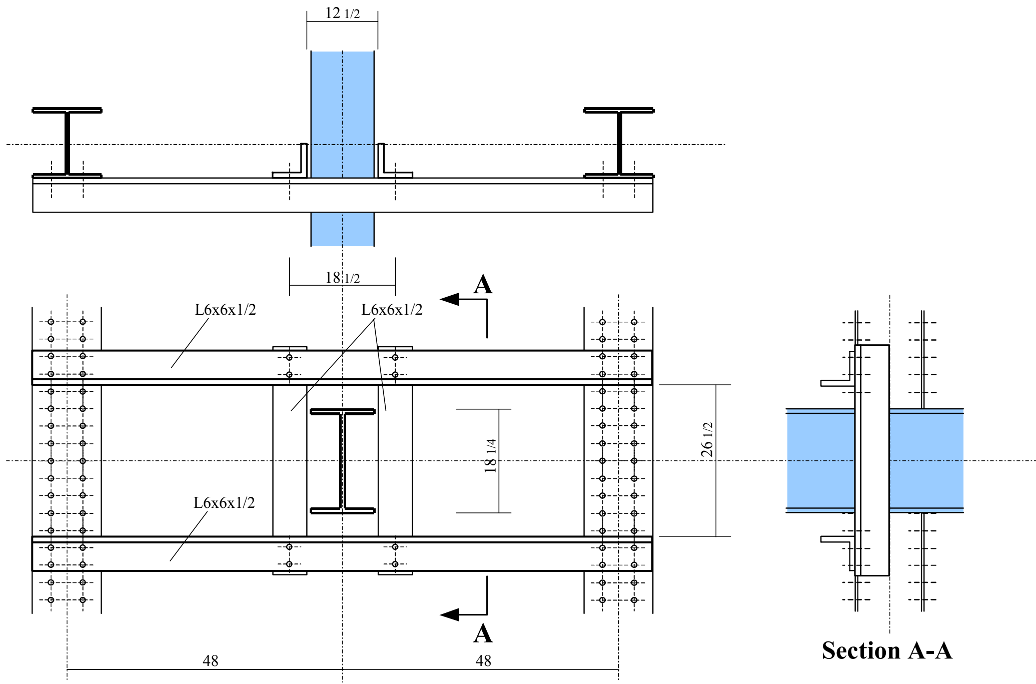


Fig. 20 – Lateral Support System: Details at Beam End

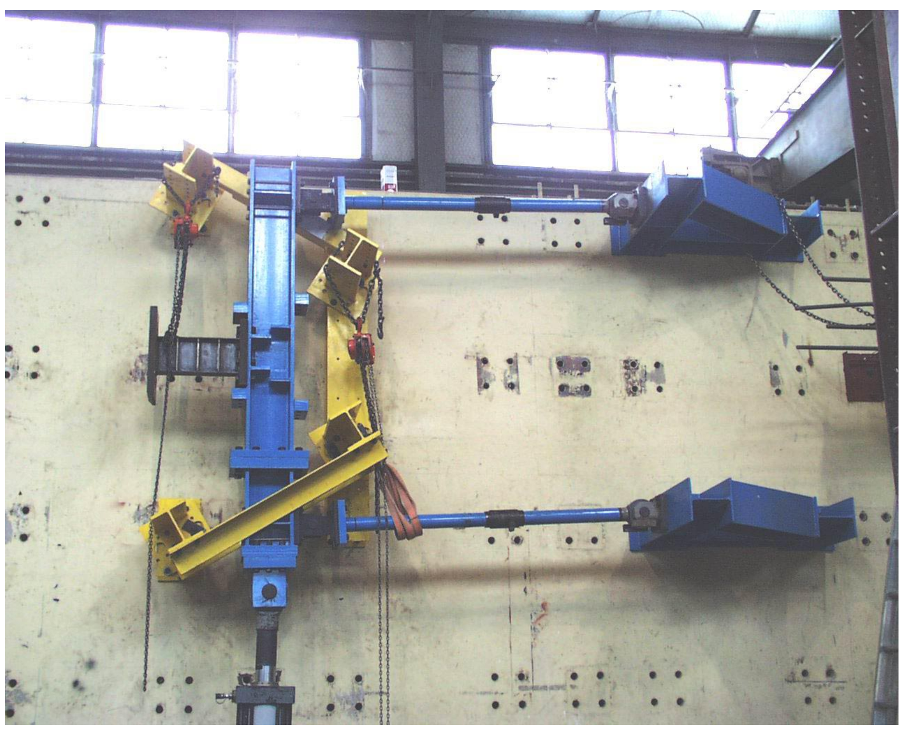


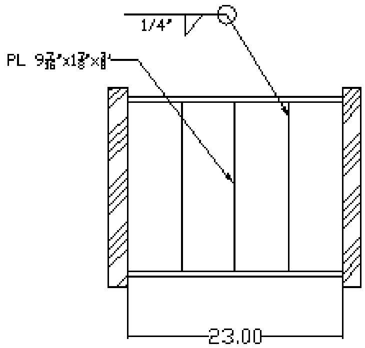
Fig. 21 – Photo of Test Setup During Assembly

Fig. 22 – Photo of Test Setup During Assembly

Fig. 23 – Specimen 1 After Fabrication

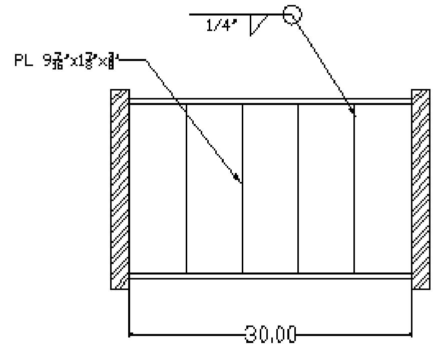
APPENDIX A

TEST SPECIMENS



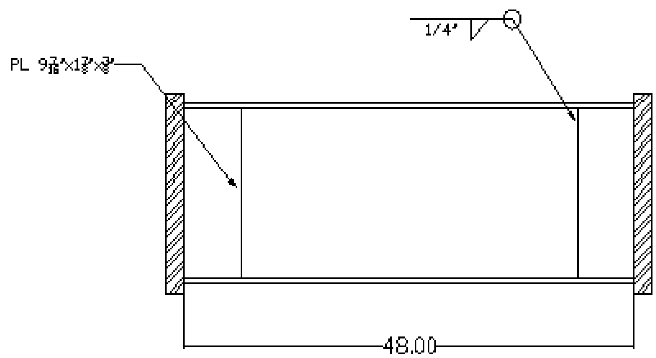
Specimen 1: W10x19

Stiffeners: 3@5 3/4"



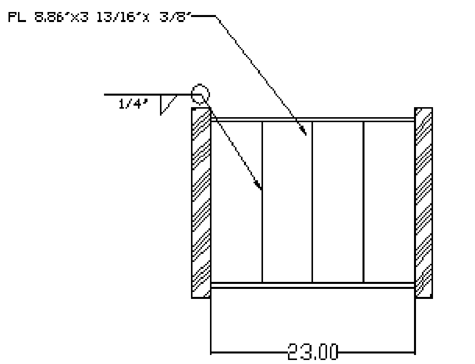
Specimen 2: W10x19

Stiffeners: 4@6.0"



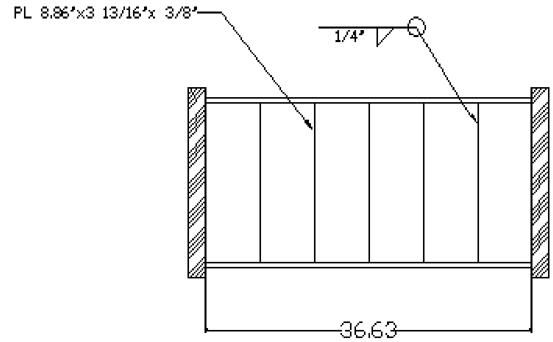
Specimen 3: W10x19

Stiffeners: 1@6.0" from each end



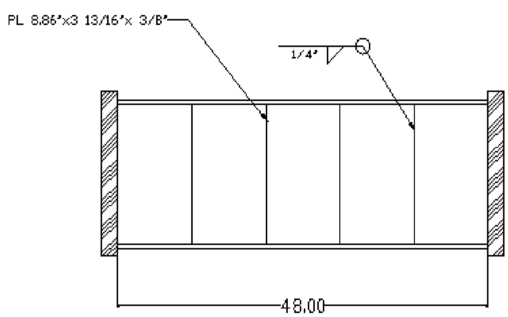
Specimen 4: W10x33

Stiffeners: 3@5$\frac{3}{4}$"



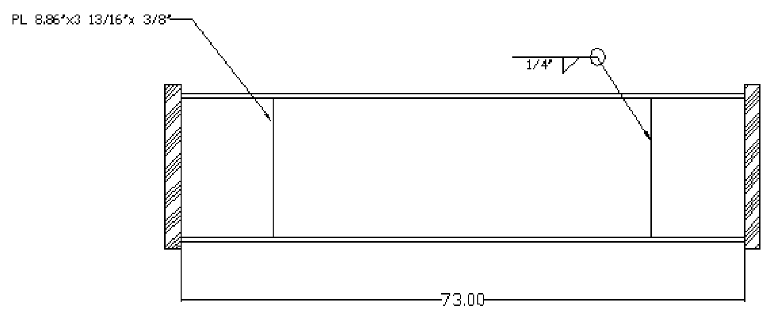
Specimen 5: W10x33

Stiffeners: 5@6$\frac{1}{8}$"



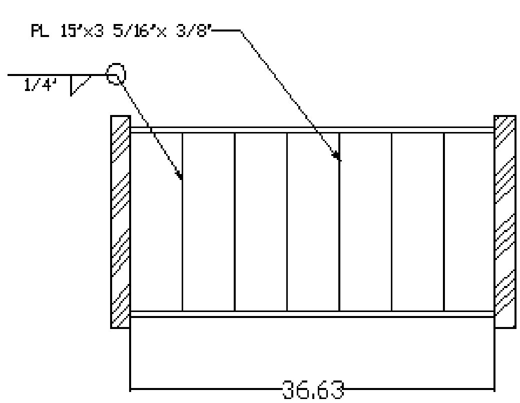
Specimen 6: W10x33

Stiffeners: 4@9 5/8"



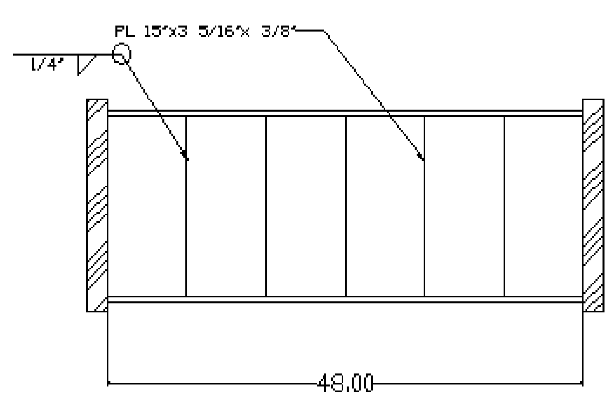
Specimen 7: W10x33

Stiffeners: 1@12.0" from each end



Specimen 8: W16x36

Stiffeners: 6@5$\frac{1}{4}$"

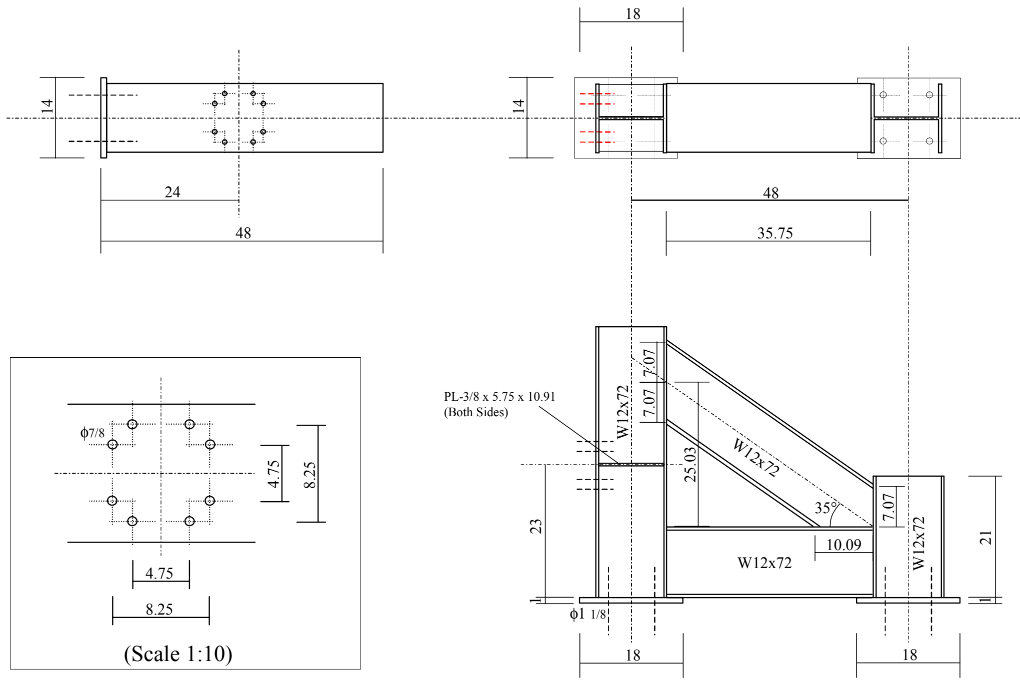


Specimen 9: W16x36

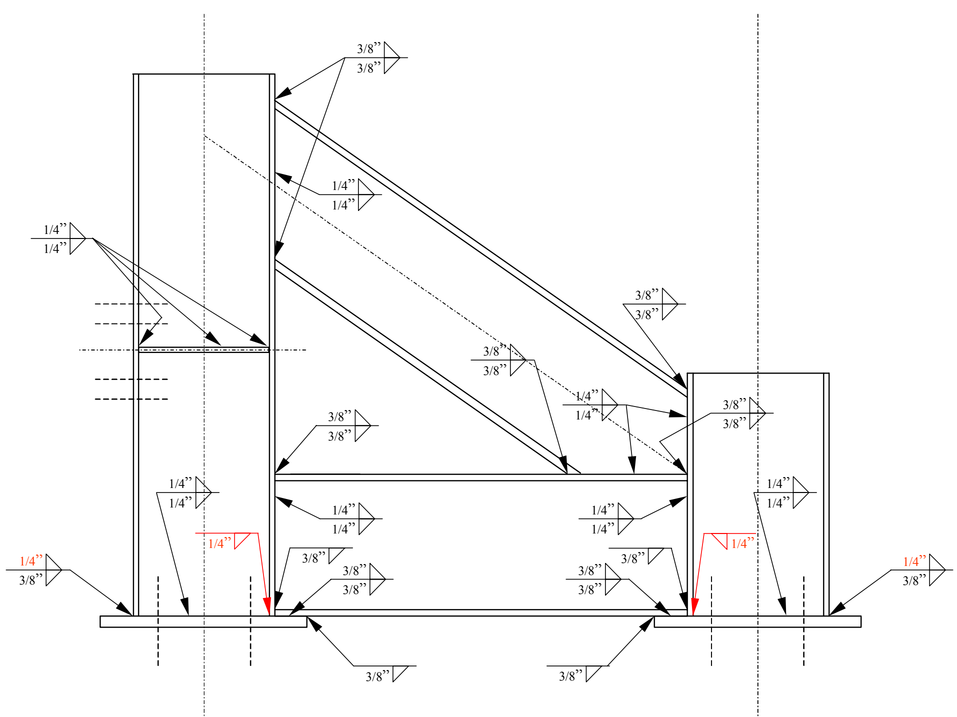
Stiffeners: 5@8.0"

APPENDIX B

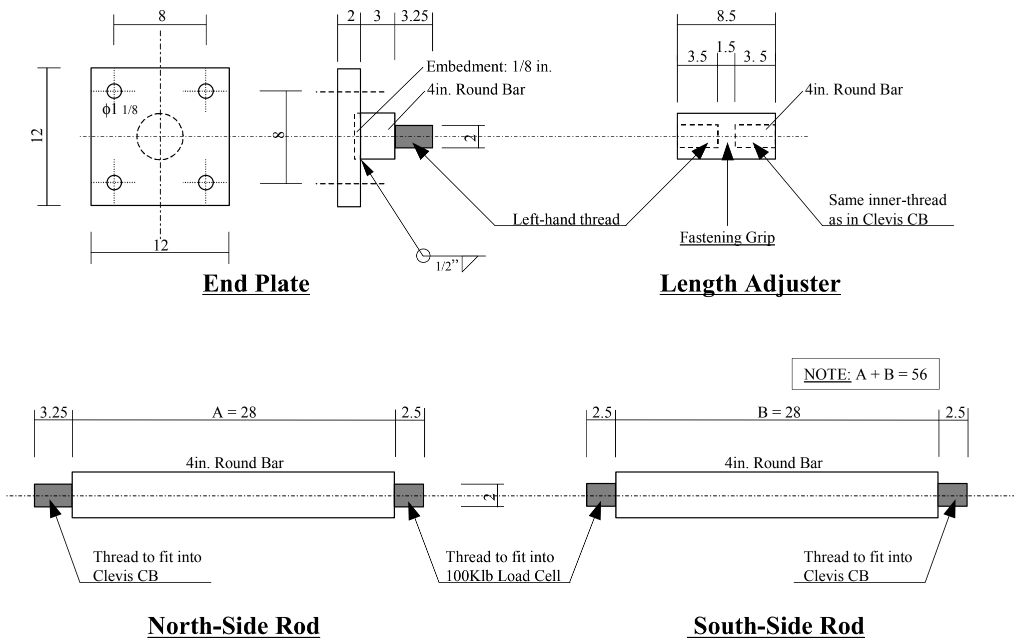
DETAILS OF TEST FRAME COMPONENTS



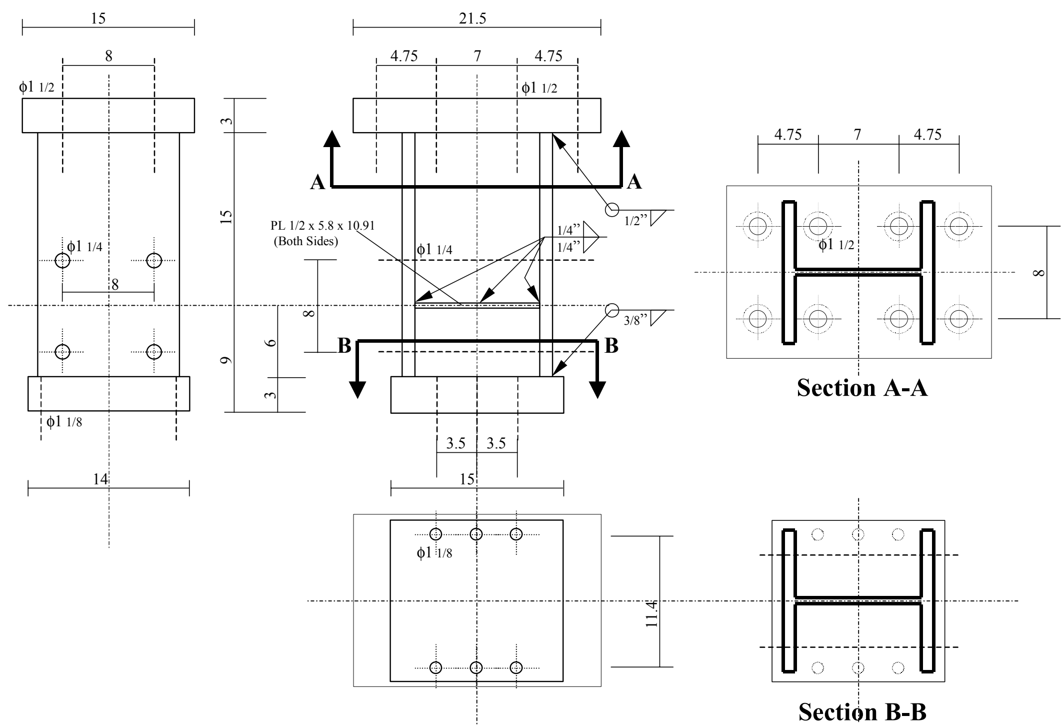
Reaction Frame for Column Lateral Reactions (2 Req'd)



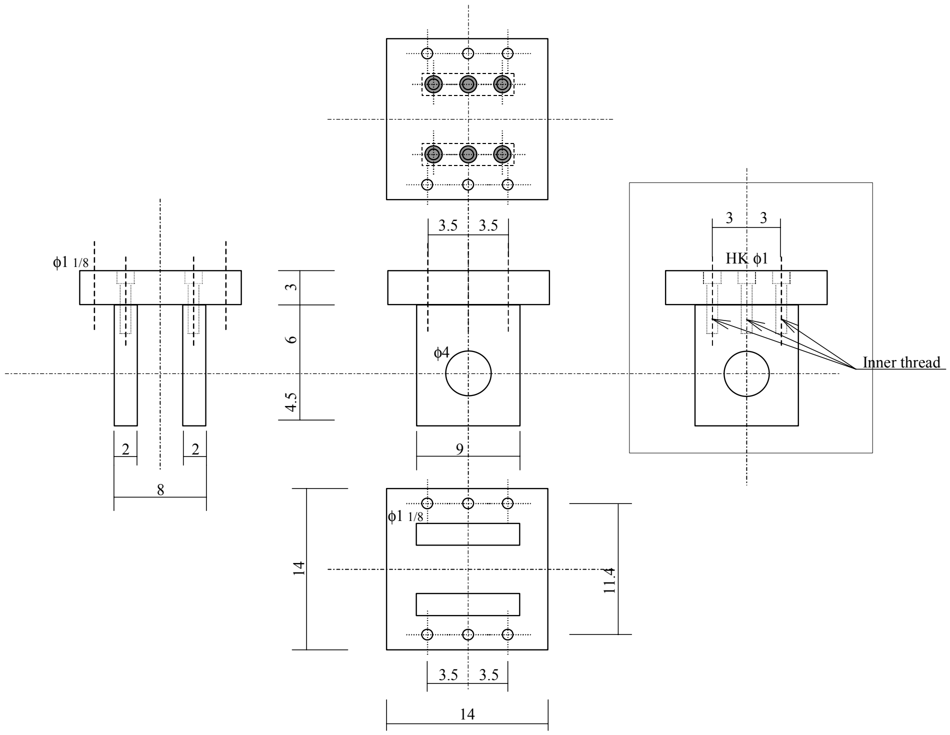
Reaction Frame for Column Lateral Reactions – Add'l Details



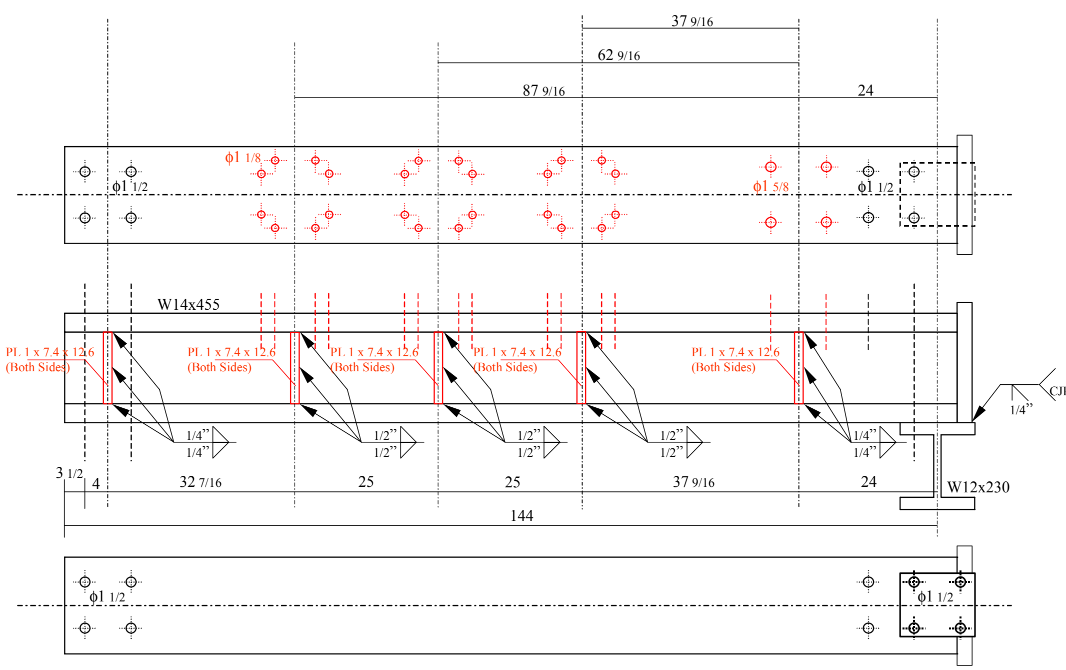
Column Lateral Reaction Rods (2 Complete Assemblies Req'd)



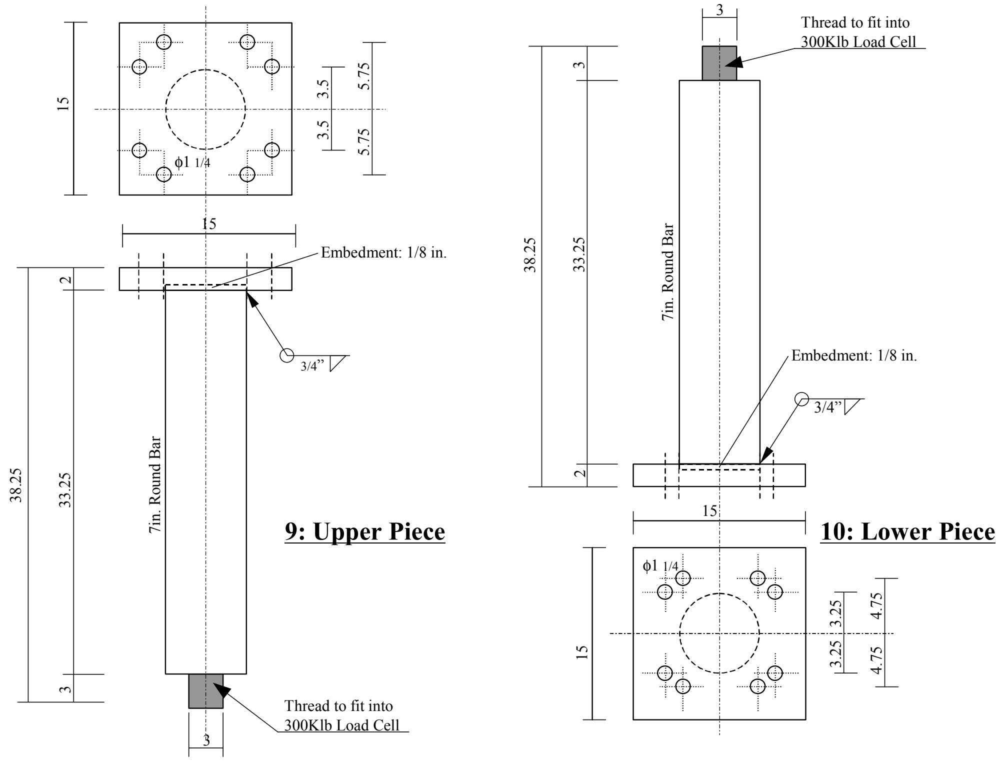
Column Base Stub Assembly



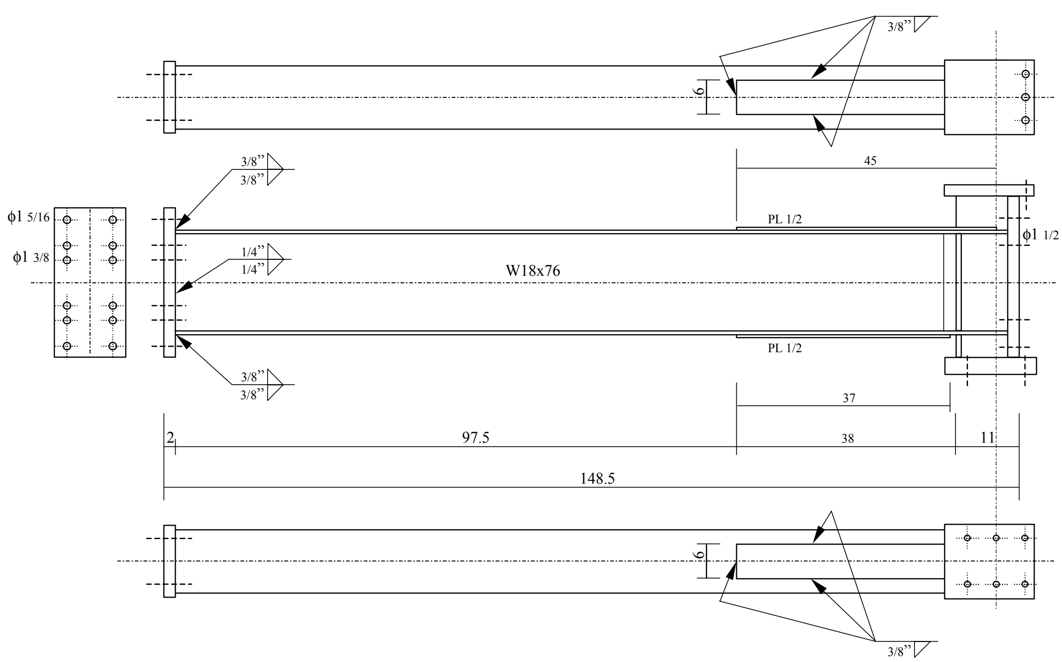
Heavy Two Plate Clevis



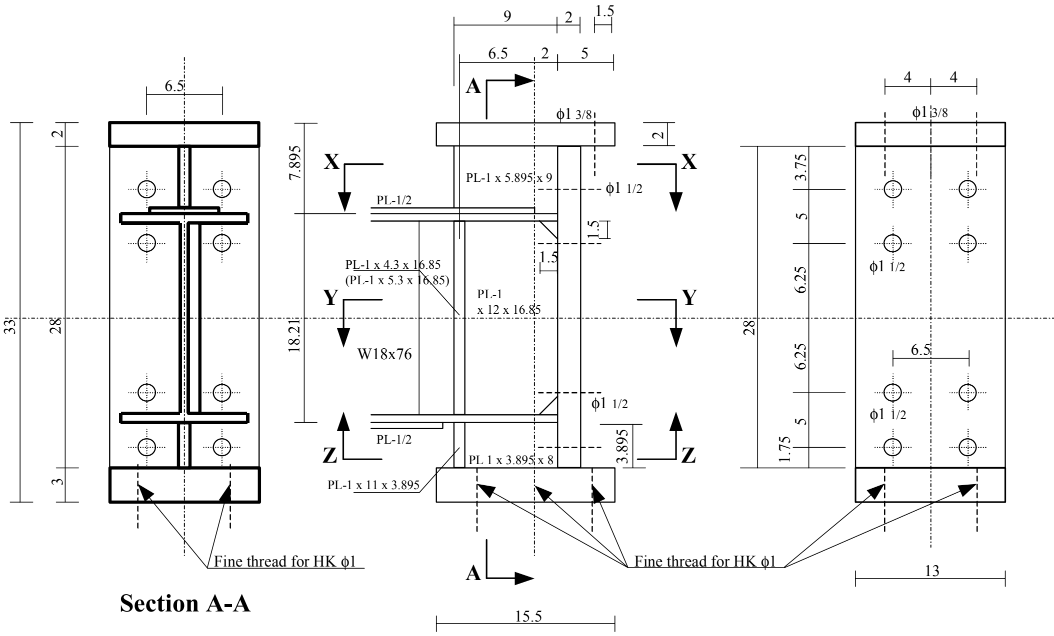
Reaction Beam for Hydraulic Ram Base and Link End Reaction



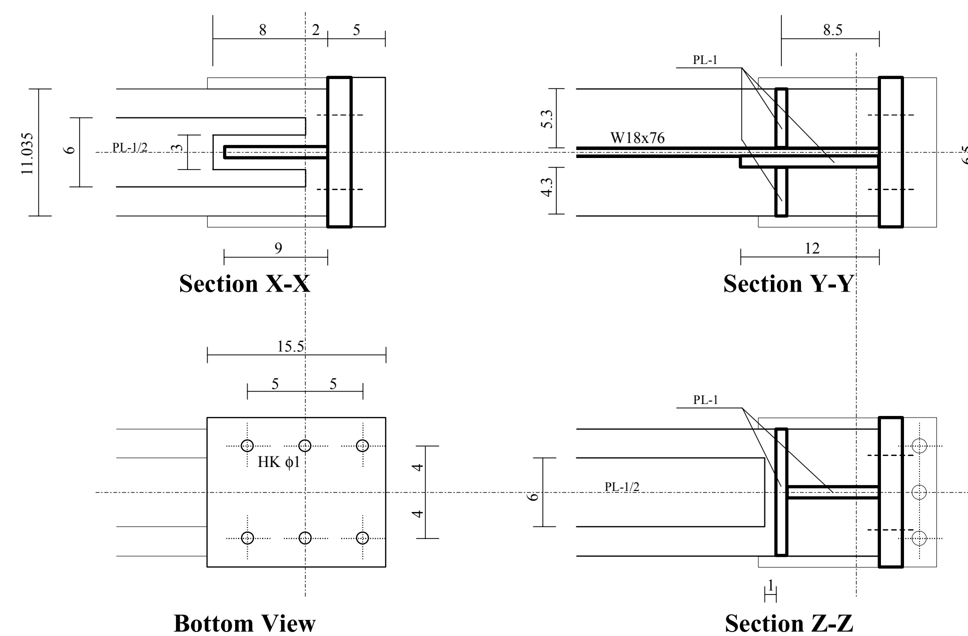
Link End Reaction Rod Assembly



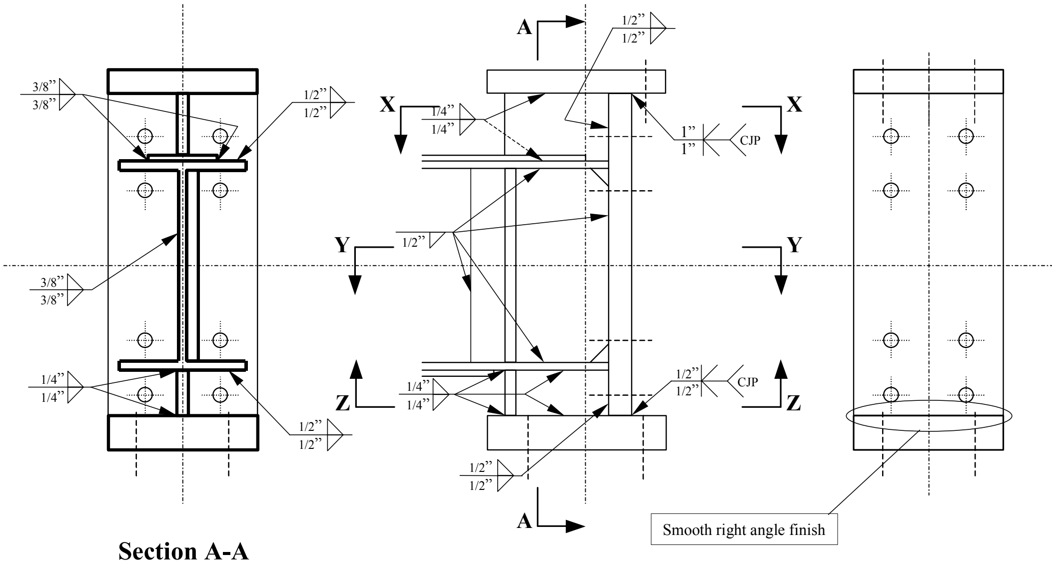
Beam Outside Link – Overall View



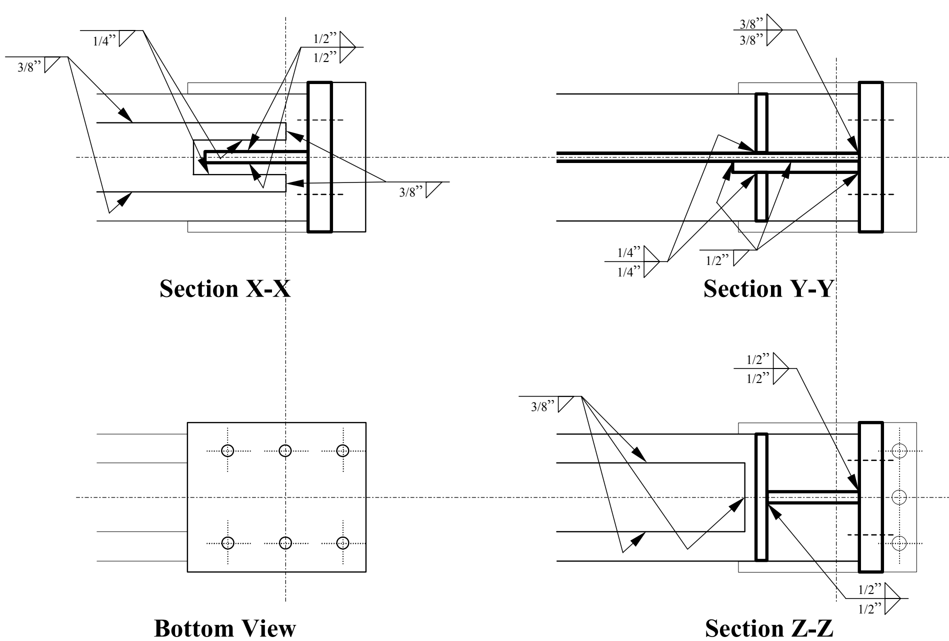
Beam Outside Link – Details at South End



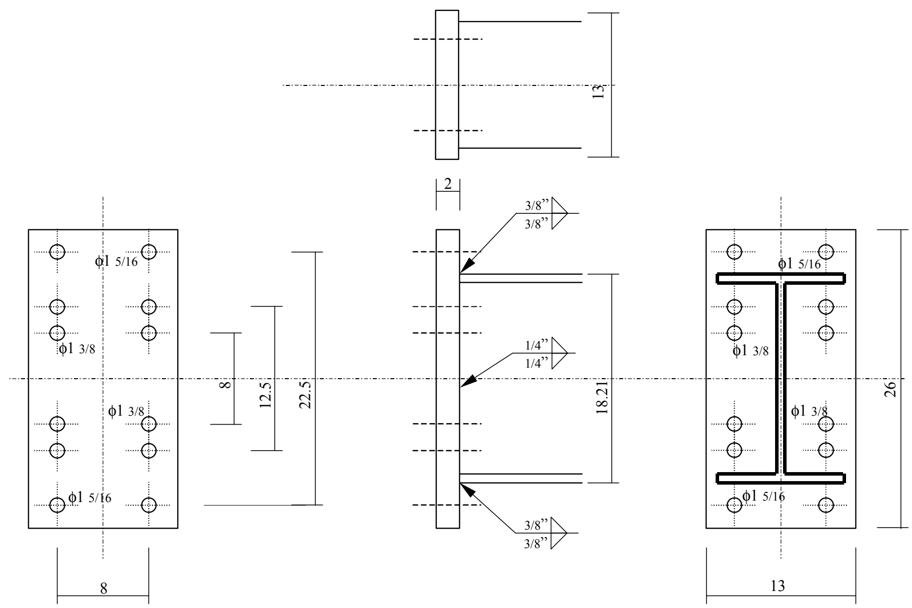
Beam Outside Link – Details at South End



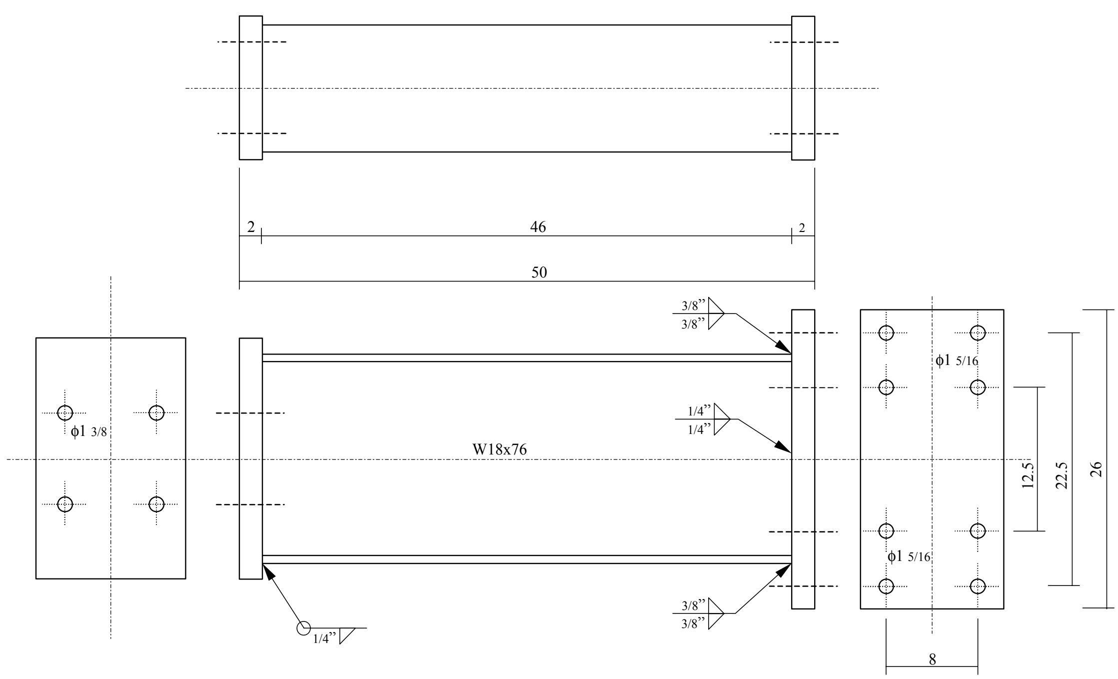
Beam Outside Link – Details at South End



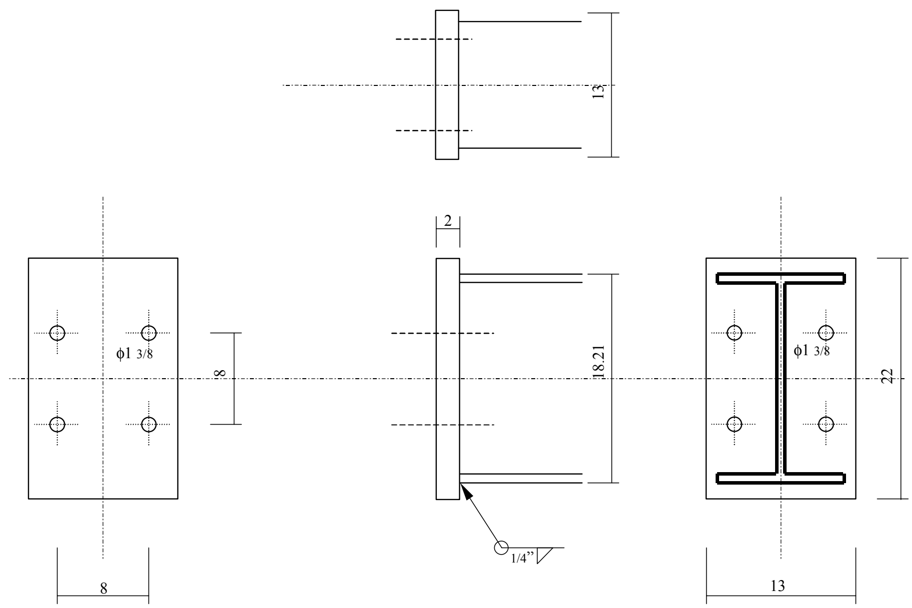
Beam Outside Link – Details at South End



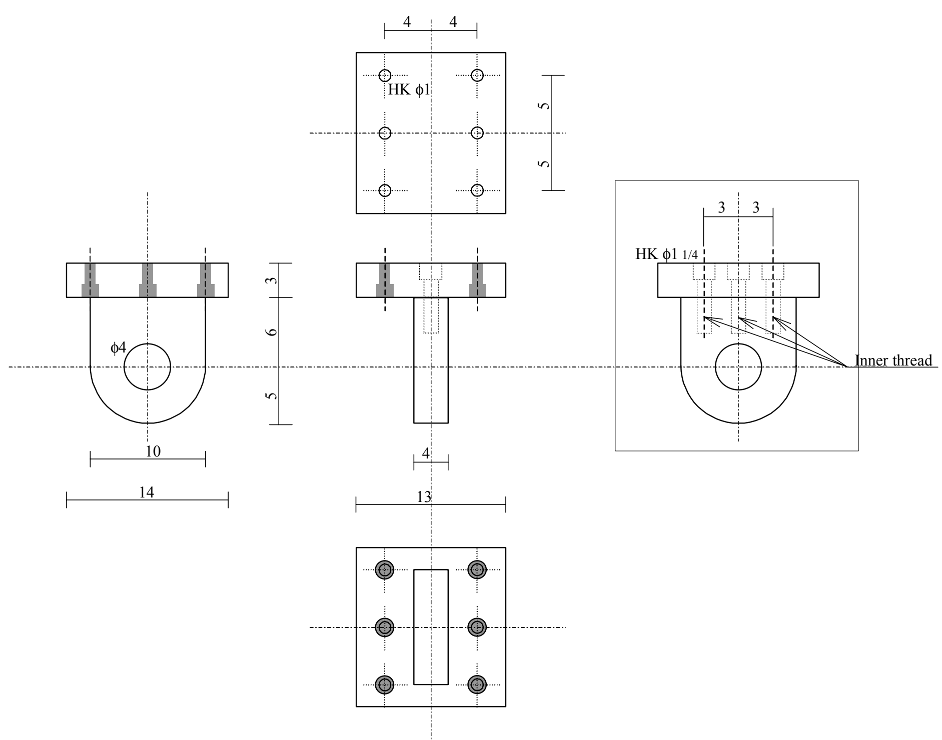
Beam Outside Link – Details at North End



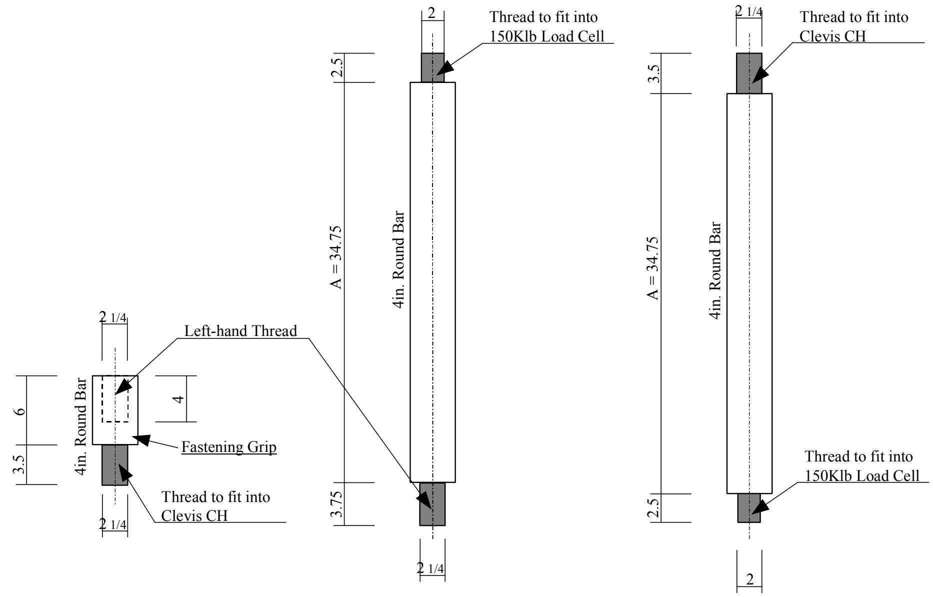
Extension Beam



Extension Beam - Details

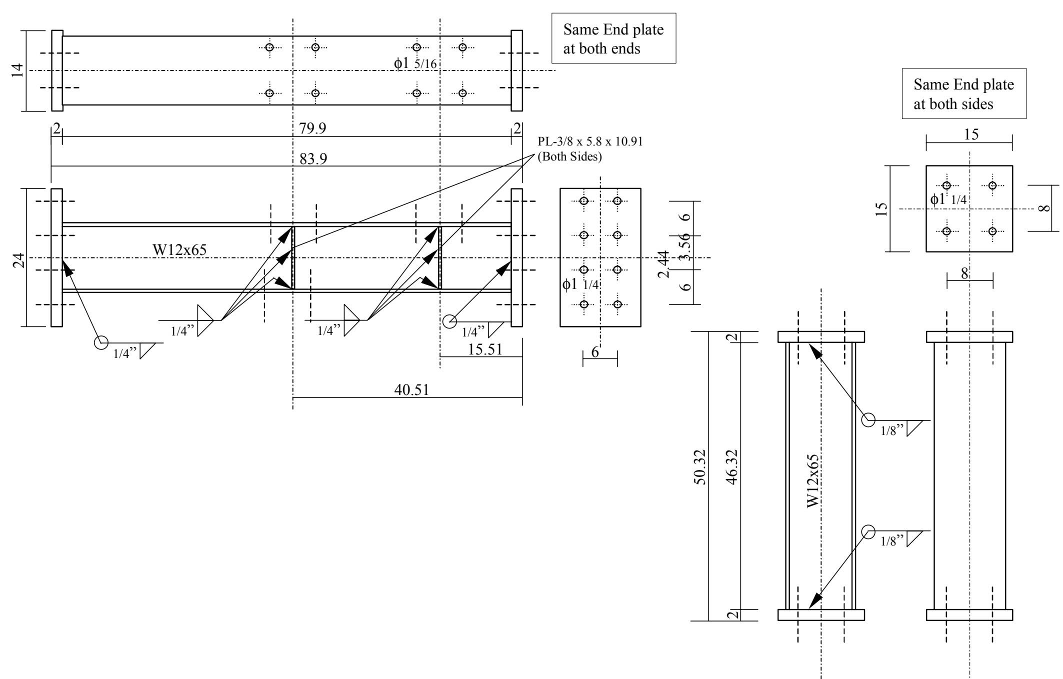


Heavy Single Plate Clevis

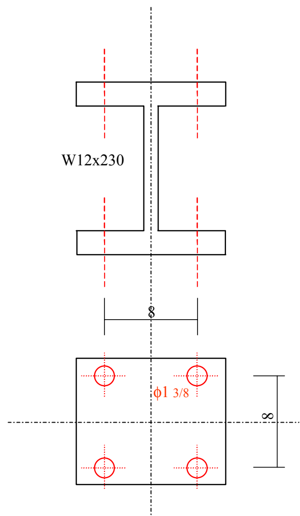


14: Length Adjuster **15: Lower Rod** **16: Upper Rod**

Beam End Reaction Rod Assembly



Beam End Reaction Frame Components



Supports for W14x455 Reaction Beam (2 Req'd)

APPENDIX C

CMTRs FOR TEST SPECIMEN SECTIONS

CHAPARRAL STEEL
CERTIFIED MATERIAL TEST REPORT
300 Ward Rd.
Midlothian, TX
76065-9651
(972) 775-8241

Bill To:	Ship To: 6	Order Date:01/05/2000
DELTA STEEL, INC.	DELTA STEEL, INC.	PO No:DSA-407366
P.O. BOX 2289	1840 ACKERMAN	Mill Order No:2021229
		Load No:567770
HOUSTON TX	SAN ANTONIO TX	Manifest No:1323815
77252 US	78219 US	

SIZE	GRADE	LENGTH	PRODUCT
W 10 X 19#	A36/A57250	40 FT	WF BEAMS

SPECIFICATION
ASTM A6-98a, A36-97a, A572-97, A992-98

HEAT NO:11452310

CHEMICAL ANALYSIS

C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Sn	V	Al	Nb	CE
.11	.98	.012	.038	.26	.29	.09	.10	.021	.010	.002	.002	.010	.32

PHYSICAL PROPERTIES

Yield Strength KSI	Yield Strength MPa	Tensile Strength KSI	Tensile Strength MPa	Specimen Area Sq In	Specimen Area Sq cm	Elongation %	Gage Length	Gage Length	Bend Test Dia.	Bend Test Result	ROA %
58.2	401.3	74.9	516.4	0.360	2.32	22.8	8In	200 mm	0.5	PASS	
58.0	399.9	74.8	515.7	0.364	2.35	23.9	8In	200 mm	0.5	PASS	

Remarks

MEETS REQUIREMENTS OF AISC TECH BULLETIN #3 - MARCH,1997, ASTM A992-98

All manufacturing processes of this product, including melting and casting, occurred in the U.S.A.

"I hereby certify that the contents of this report are correct and accurate. All tests and operations performed by this material manufacturer or its sub-contractors, when applicable, are in compliance with the requirements of the material specifications and applicable purchaser designated requirements."

Signed: Tom L. Harrington
Tom L. Harrington: Quality Assurance Manager
Date:Jan. 13, 2000
Signed: ______________
Notary Public (if applicable)
Date: ______________
Page: 6of 7

W10x19

19-JUL-2000 03:29 Chaparral Steel MILLFAX 1-800-645-4155 Page 2/6

CHAPARRAL STEEL
CERTIFIED MATERIAL TEST REPORT
300 Ward Rd.
Midlothian, TX
76065-9651
(972) 775-8241

Bill To:
DELTA STEEL, INC.
P.O. BOX 2289
HOUSTON TX
77252 US

Ship To: 1
DELTA STEEL, INC.
7355 ROUNDHOUSE LANE
HOUSTON TX
77078 US

Order Date:07/10/2000
PO No:109522
Mill Order No:2102827
Load No:607535
Manifest No:1357979

SIZE	GRADE	LENGTH	PRODUCT
W 10 X 33#	A36/A57250	50 FT	WF BEAMS

SPECIFICATION
ASTM A6-98a, A36-97a, A572-97, A992-98

HEAT NO:21998880

CHEMICAL ANALYSIS

C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Sn	V	Al	Nb	CE
.11	.94	.019	.019	.22	.40	.10	.17	.022	.011	.001	.001	.010	.34

PHYSICAL PROPERTIES

Yield Strength KSI	Yield Strength MPa	Tensile Strength KSI	Tensile Strength MPa	Specimen Area Sq In	Specimen Area Sq cm	Elongation %	Gage Length		Bend Test Dia.	Bend Test Result	ROA %
58.8	405.4	76.8	529.5	0.625	4.03	23.9	8In	200 mm			
58.1	400.6	76.9	530.2	0.621	4.01	24.3	8In	200 mm			

Remarks

MEETS REQUIREMENTS OF AISC TECH BULLETIN #3 - MARCH,1997, ASTM A992-98

All manufacturing processes of this product, including melting and casting, occurred in the U.S.A.

"I hereby certify that the contents of this report are correct and accurate. All tests and operations performed by this material manufacturer or its sub-contractors, when applicable, are in compliance with the requirements of the material specifications and applicable purchaser designated requirements."

Signed: Tom L. Harrington Date:Jul. 19, 2000 Signed: ____________ Date: ____________
Tom L. Harrington: Quality Assurance Manager Notary Public (if applicable)

Page: 2of 6

W10x33

NORTHWESTERN™
STEEL AND WIRE COMPANY
SINCE 1879

CERTIFIED MILL TEST REPORT

100% MELTED AND MANUFACTURED IN U.S.A.

NSW NUMBER	SHIP DATE	B/L NO.	CUSTOMER P.O. NO.	SHIP METHOD	CAR OR TRUCK NUMBERS	SHIP LOC.	NO. OF COPIES	OGL.	NOT.	CORR.	PAGE
13714	05-07-00	91591	DHO109025	RAIL	CNW 128098	24	1		X	64	1

SHIP TO
DELTA STEEL INC
YD #15 TRACK 732 733
7355 ROUNDHOUSE LN
HOUSTON TX 77078

SOLD TO
DELTA STEEL INC
5599 SAN FELIPE
POBOX 2289 SUITE 600
HOUSTON TEXAS 77252

STERLING, IL

LINE NO.	SECTION	SPEC./ LENGTH	PIECES/ WEIGHT	HEAT #	YIELD POINT KSI	TENSILE STRENGTH KSI	ELONG% 8"	C X100	Mn X100	Si X100	P X1000	S X1000	Cu X100	Ni X100	Cr X100	Mo X1000	V X1000	Nb X1000
				Y/T	MPa	MPa		CE										Sn
	*FAX MTR TO CUSTOMER 713-635-7433 &																	
	*MAIL TO:DELTA STEEL MTR DEPT																	
	*7355 ROUND HOUSE HOUSTON TX 77078																	
3	W 16 X 36	A572/44W/A36/A992	6	66409	56.8	74.5	21.9	16	68	21	15	25	27	10	15	33	27	0
		50'	10800		56.6	74.3	22.7	34										15
	W 410 X 53	15.24 METERS		.76	392	514												
				.76	390	512												
			17	66407	51.9	72.6	21.1	18	71	20	24	24	26	10	17	36	24	0
			30600		51.9	72.6	21.1	37										19
				.71	358	501												
				.71	358	501												

I, Notary Public in WHITESIDE County, State of ILLINOIS hereby certify this affidavit was sworn to by THOMAS E. MURPHY an agent of Northwestern Steel and Wire Company under my Notarial Seal this 8 day of MAY 2000

Wayne D. Spencer
Notary Public

My Commission Expires

OFFICIAL SEAL
WAYNE D SPENCER
NOTARY PUBLIC, STATE OF ILLINOIS
MY COMMISSION EXPIRES: 06/03/02

WE HEREBY CERTIFY THAT THE STEELS HAVE BEEN TESTED IN ACCORDANCE WITH SPECIFICATIONS

Thomas E. Murphy
MANAGER, QUALITY

IMPACT TEST	HEAT	VALUE	O	HEAT	VALUE	O	HEAT	VALUE	O

REMARKS:

W16x36