EXPERIMENTAL INVESTIGATION OF TRUSS TYPE RIGID FRAMES INCLUDING CONNECTION STUDIES
-MOMENT SPLICE CONNECTIONS-
VOLUME I
TH, CN, KN and CB TEST RESULTS

by

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CHAPTER I

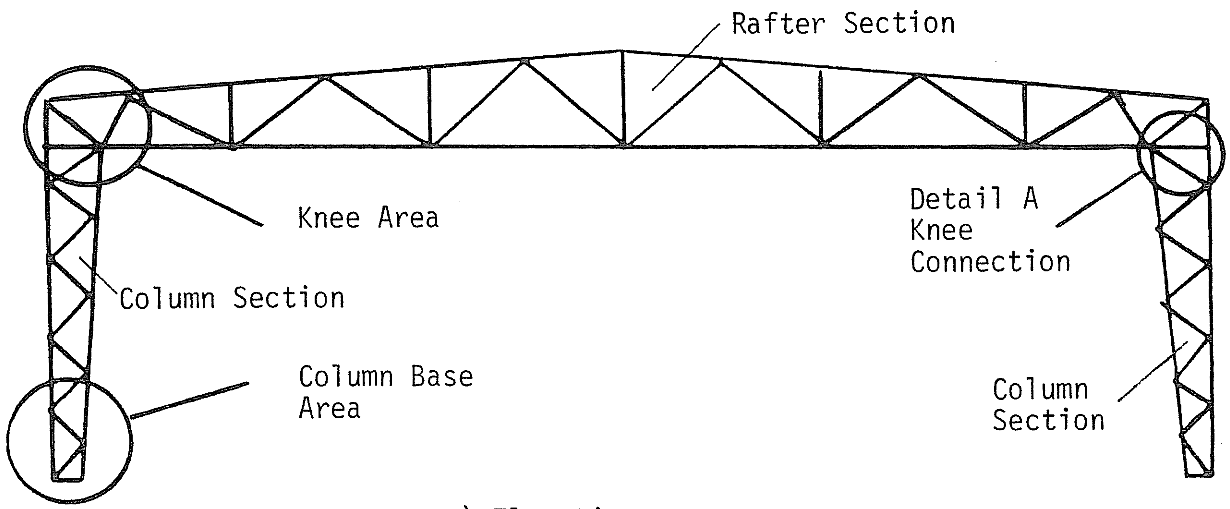
EXPERIMENTAL INVESTIGATION OF TRUSS TYPE RIGID FRAMES INCLUDING CONNECTION STUDIES - MOMENT SPLICE CONNECTIONS

INTRODUCTION

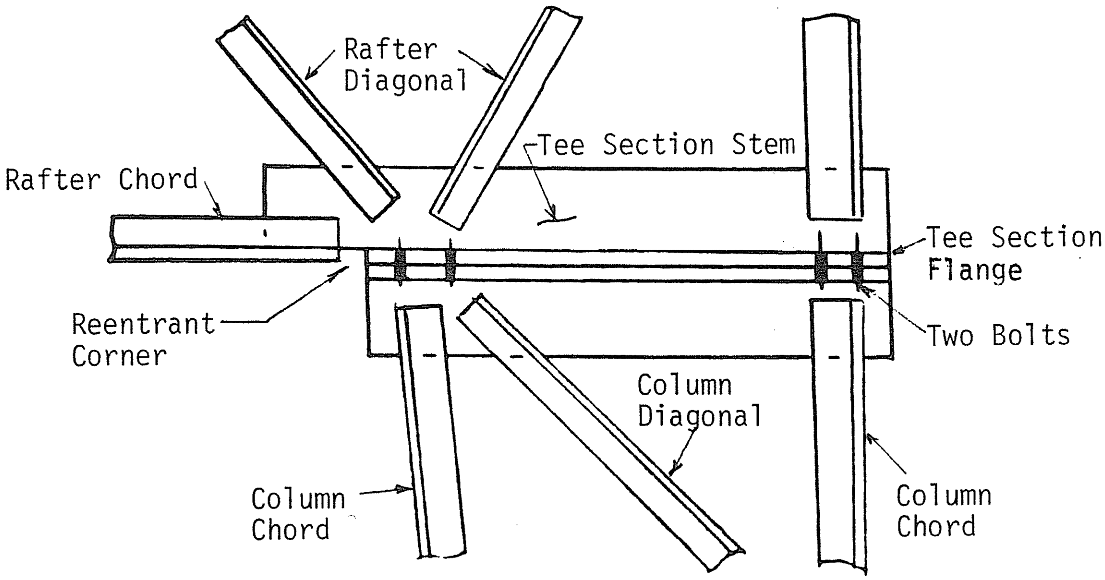
1.1 Background

An extensive research program to study the behavior of truss type, single story, single bay, rigid frames is being conducted at the Fears Structural Engineering Laboratory, University of Oklahoma under the sponsorship of VULCRAFT, a division of NUCOR Corporation, hereafter referred to as VULCRAFT. The frames are fabricated from angles and plates and consist of three major components: two columns and a rafter. Figure 1.1 is an elevation view of a typical frame. Both columns and the rafter are tapered members. Connection between the components is made using tees fabricated from 1 in. thick plates and high-strength bolts as shown in Detail A of Figure 1.1.

VULCRAFT fabricates the frame components using angle and plate material having a nominal yield strength of 50 ksi. Plate material is generally 1 in. thick. Angle sizes are chosen from the list in Table 1.1. All welding is done using the manual stick electrode (shielded metal



a) Elevation



b) Detail A

Figure 1.1 Elevation View of a Typical Truss-Type Rigid Frame

Table 1.1
Angle Sizes Used to Fabricate Frames

SEC	ANGLE	AREA	RX	RZ	RY-1	SL	SS	Y	Q	IXX	WELD
3	1.00X.109	0.206	0.3066	0.1962	0.8464	0.067	0.027	0.290	1.0000	0.019	1619
8	1.25X.111	0.265	0.3830	0.2500	0.9373	0.113	0.044	0.353	0.9841	0.040	1648
12	1.50X.109	0.315	0.4675	0.2972	1.0278	0.166	0.063	0.415	0.9050	0.069	1619
14	1.50X.115	0.332	0.4670	0.2990	1.0294	0.174	0.067	0.418	0.9277	0.072	1708
16	1.50X.130	0.373	0.4660	0.2980	1.0336	0.191	0.075	0.424	0.9753	0.080	1856
17	1.50X.138	0.395	0.4640	0.2970	1.0357	0.200	0.079	0.426	0.9964	0.085	1856
18	1.50X.145	0.414	0.4640	0.2970	1.0383	0.207	0.083	0.429	1.0000	0.089	1856
19	1.50X.156	0.444	0.4610	0.2950	1.0403	0.217	0.088	0.433	1.0000	0.096	1856
20	1.50X.170	0.481	0.4590	0.2940	1.0430	0.232	0.095	0.437	1.0000	0.101	2060
22	2.00X.137	0.529	0.6245	0.3967	1.2223	0.375	0.142	0.551	0.8796	0.206	1856
23	2.00X.148	0.570	0.6229	0.3960	1.2250	0.399	0.153	0.555	0.9129	0.221	1856
25	2.00X.163	0.625	0.6208	0.3951	1.2285	0.430	0.167	0.560	0.9522	0.241	1956
26	2.00X.175	0.669	0.6191	0.3944	1.2322	0.454	0.179	0.565	0.9788	0.257	2136
27	2.00X.186	0.717	0.6170	0.3940	1.2337	0.478	0.190	0.569	1.0000	0.272	2320
28	2.00X.200	0.760	0.6156	0.3932	1.2379	0.502	0.202	0.574	1.0000	0.288	2320
[illegible]	2.00X.213	0.807	0.6140	0.3930	1.2405	0.526	0.214	0.578	1.0000	0.304	2320
[illegible]	2.00X.232	0.874	0.6113	0.3918	1.2455	0.558	0.231	0.585	1.0000	0.327	2517
33	2.50X.197	0.946	0.7769	0.4943	1.4279	0.819	0.317	0.698	0.9389	0.571	2320
36	2.50X.212	1.015	0.7740	0.4930	1.4308	0.866	0.339	0.703	0.9673	0.609	2320
38	2.50X.230	1.097	0.7720	0.4920	1.4354	0.921	0.365	0.710	0.9964	0.654	2487
39	2.50X.250	1.187	0.7690	0.4910	1.4399	0.980	0.394	0.717	1.0000	0.703	2784
42	3.00X.227	1.310	0.9330	0.5930	1.6283	1.369	0.527	0.834	0.9223	1.142	2442
43	3.00X.250	1.437	0.9300	0.5920	1.6321	1.473	0.577	0.842	0.9607	1.240	2734
46	3.00X.281	1.607	0.9260	0.5907	1.6404	1.614	0.642	0.854	1.0000	1.378	3244
48	3.00X.313	1.780	0.9220	0.5890	1.6500	1.738	0.707	0.869	1.0000	1.510	3712
51	3.50X.287	1.927	1.0860	0.6910	1.3365	2.315	0.902	0.981	0.9545	2.273	3333
52	3.50X.313	2.093	1.0800	0.6900	1.8414	2.475	0.976	0.990	0.9566	2.450	3712
54	3.50X.344	2.290	1.0781	0.6833	1.8488	2.657	1.065	1.002	1.0000	2.661	4179
57	3.50X.375	2.484	1.0700	0.6870	1.8535	2.842	1.150	1.010	1.0000	2.870	4640
59	4.00X.375	2.859	1.2300	0.7880	2.0530	3.825	1.520	1.140	1.0000	4.360	4640
61	4.00X.438	3.312	1.2300	0.7850	2.0631	4.234	1.750	1.160	1.0000	4.970	5568
63	4.00X.500	3.750	1.2200	0.7820	2.0749	4.712	1.970	1.180	1.0000	5.560	5568
64	5.00X.438	4.188	1.5500	0.9860	2.4563	7.092	2.790	1.410	0.9792	10.000	5568
66	5.00X.500	4.750	1.5400	0.9830	2.4706	7.902	3.160	1.430	1.0000	11.300	5568
67	5.00X.563	5.313	1.5310	0.9801	2.4844	8.542	3.512	1.457	1.0000	12.445	5568
	6.00X.500	5.750	1.8600	1.1300	2.8659	11.345	4.610	1.680	0.9607	19.900	5568
69	6.00X.563	6.439	1.8500	1.1900	2.8833	12.923	5.140	1.710	1.0000	22.100	5568
70	6.00X.625	7.109	1.8400	1.1800	2.8943	13.938	5.660	1.730	1.0000	24.200	5568
71	6.00X.688	7.772	1.8400	1.1800	2.9041	14.930	6.160	1.750	1.0000	26.200	5568
72	6.00X.750	8.438	1.8300	1.1700	2.9224	19.843	6.660	1.780	1.0000	28.200	5568

arc) process. A325 bolts are used in the rafter-to-column connections. The frames are designed using the Steel Joist Institute (SJI) "Standard Specifications for Longspan Steel Joists LH Series and Deep Longspan Steel Joists DLH Series" as adopted by SJI on February 15, 1978.

Specimens selected for testing in this project were chosen based on the following ranges of frame parameters:

Clear Span	30 ft. - 150 ft.
Eave Height	10 ft. - 30 ft.
Frame Spacing	20 ft. - 40 ft.
Roof Slope	1/2:12 and 1:12
Design Live Load	12 psf - 40 psf
Design Wind Load	10 psf - 20 psf

Bolt diameters of 5/8 in., 3/4 in., 7/8 in., 1 in. or 1-1/8 in. are used depending on the frame span and loading. Tee flange thicknesses vary from 1/2 in. to 1 in. and the nominal knee area dimensions vary from 10 in. by 60 in. by 60 in.

Details and results of connection, knee area and column base area studies are included in this volume. Supporting test data for these studies are found in Volume II. Full scale frame tests are described in Volume III.

1.2 Scope of Research

To thoroughly study the structural characteristics of frames configured as shown in Figure 1.1, five series of tests were conducted.

1. Tee-hanger tests (TH-series),
2. Knee-connection tests (CN-series),
3. Knee-area tests (KN-series),
4. Column base area tests (CB-series),
5. Full frame tests (FR-series).

The purposes of each test series are as follows.

Tee-Hanger Tests. The bolted column-to-rafter splice connection shown in Detail A of Figure 1.1 may be described as an eccentric tee-hanger connection. The tension portion of the connection can conceivably be analyzed as a tee-hanger by considering a portion of the web and flange plates symmetric about the tension bolts. Example calculations using typical dimensions for the splice connections in the frames under investigation showed that for a given loading and configuration large differences in required bolt diameter and flange plate thickness resulted. Thus, a limited experimental program of nine tee-hanger tests (TH-1 thru TH-9) was undertaken. The test specimens consisted of two tee-hangers connected together with four A325 bolts. The specimens were loaded using a universal type testing machine.

Details of the TH- testing programs, comparison of results with two design procedures, and recommendations are found in Chapter III of this volume and Appendix B in Volume II. Descriptions of the design procedures are found in Appendix A of this volume.

Knee Connection Tests. To verify the validity of using tee-hanger specimens to study the behavior of the rafter-to-column splice connection, four tests (CN-1 thru CN-4) of complete connections were conducted. The specimens consisted of two complete tee-sections with angles welded to one end to simulate the tension chord of the column. Tension load was applied thru these angles using a universal type testing machine.

Details of the CN- testing program, comparison of results with tee-hanger test results and with predictions from the two tee-hanger design procedures, and recommendations are found in Chapter III of this volume and Appendix C in Volume II.

Knee Area Tests. The objectives of the knee area tests was to study the behavior of the knee portion of typical frames. Six specimens (KN-1 thru KN-5 and KN-7) were used to conduct seven tests. Each specimen consisted of a rafter section and a column section connected as shown in Detail A of Figure 1.1. The specimens were subjected to a single applied force, located so that the moment, shear and thrust in the knee area approximated actual design values for combined dead plus live loads.

Special test setups were constructed to provide lateral support equivalent to that provided in an actual building and to facilitate loading.

Details of the KN- testing program, comparison of deflections predicted using standard stiffness analyses and with member capacities calculated using the SJI specification provisions, and recommendations are found in Chapter IV of this volume and Appendix D of Volume II.

Column Base Tests. The objectives of these tests were to study the behavior of a typical column base area when subjected to both wind and gravity loadings. Four specimens (CB-1 thru CB-4) were used to conduct five tests, three for simulated wind loading and two for simulated gravity loading. Both two and four bolt base plate configurations were used. A universal type testing machine was used to apply the simulated loading.

Chapter V of this volume and Appendix E in Volume II give test details and results, including comparisons with stiffness analysis data and member strength predictions.

Frame Tests. To verify analytical procedures used by VULCRAFT to predict frame strength and stiffness, a complete bay of a typical building was constructed over the laboratory reaction floor. The setup consisted of two frames spaced 24 ft. on center, roof and wall joists, joist bridging, chord brace angles, and roof and end wall rod braces. Roof deck and end and side wall panels were not installed. Simulated live and wind loads, alone and in combination, were applied to the homes using hydraulic cylinders. A total of six tests were conducted: (1) working live load on one frame (both slopes loaded), (2) unbalanced live load on both frames simultaneously (one slope on each frame loaded), (3) wind load on both frames simultaneously, (4) combined wind and unbalanced live load on both frames simultaneously, and (5) full live load load on each frame to failure (2 tests).

Complete test results and comparisons with analytical predictions are found in Volume III of this report.

CHAPTER II

TEE-HANGER TESTS AND ANALYSES

2.1 General

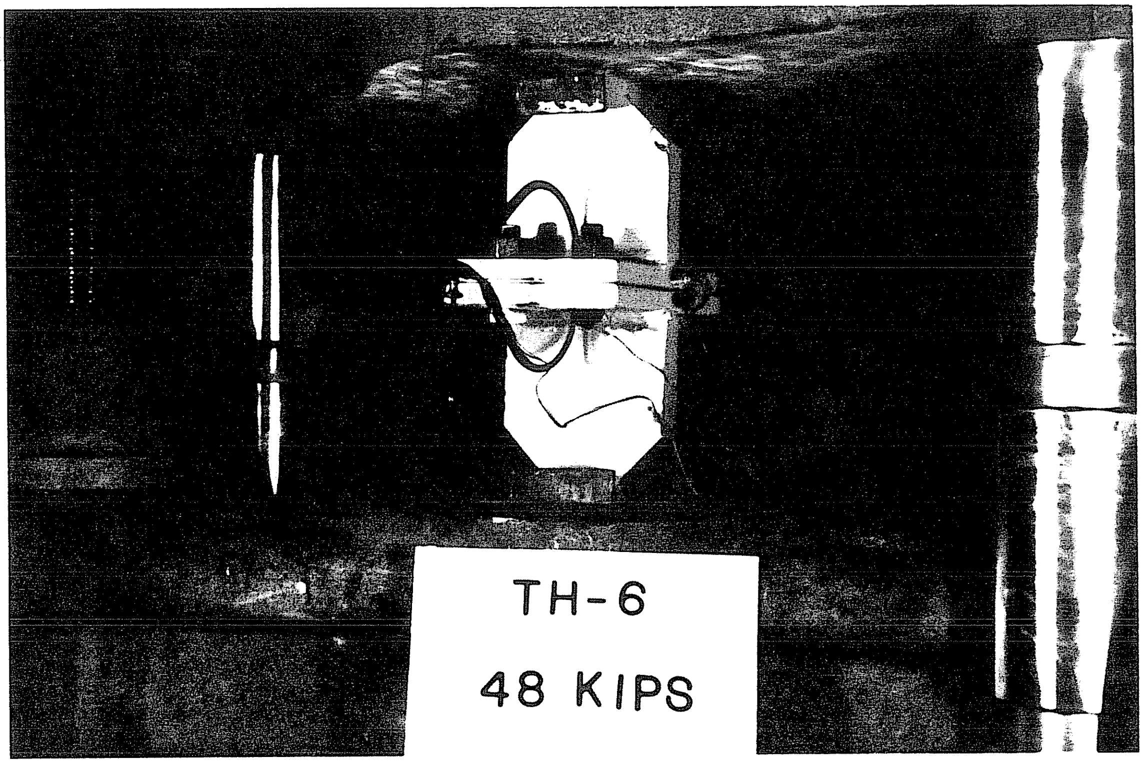
The bolted column-to-rafter splice connection used in the truss-type rigid frames currently under investigation may be described as an eccentric tee-hanger connection. The tension portion of the connection can conceivably be analyzed as a tee-hanger by considering a portion of the web and flange plates symmetric about the tension bolts. A review of the literature concerning tee-hanger design revealed a number of procedures. Example calculations showed that for a given loading and configuration, differences in flange plate thickness of up to 200% result from the various procedures.

To ascertain the validity of specific procedures, a limited experimental program consisting of nine tee-hanger tests, was undertaken. Based on initial results, the procedures suggested by Fisher and Struik[1] and Kennedy, Vinnakota and Sherbourne[2] were selected for further evaluation. The former method is the basis for the tee-hanger design procedure found in the 8th edition AISC Manual of Steel Construction[3]. Explanations of the two methods are found in Appendix A of this volume. In the comparisons that follow, experimental results will be compared to predictions from the two methods which will be referred to as the "AISC" and "Kennedy" procedures. Coupon tests for the material used to fabricate the specimen were not made, therefore, all calculations are based on the nominal yield stress of the plate material, 36 ksi. As a result, predicted results are, in general, low since the actual yield stress is probably in the 39-42 ksi range, based on previous experience with plate material coupon tests.

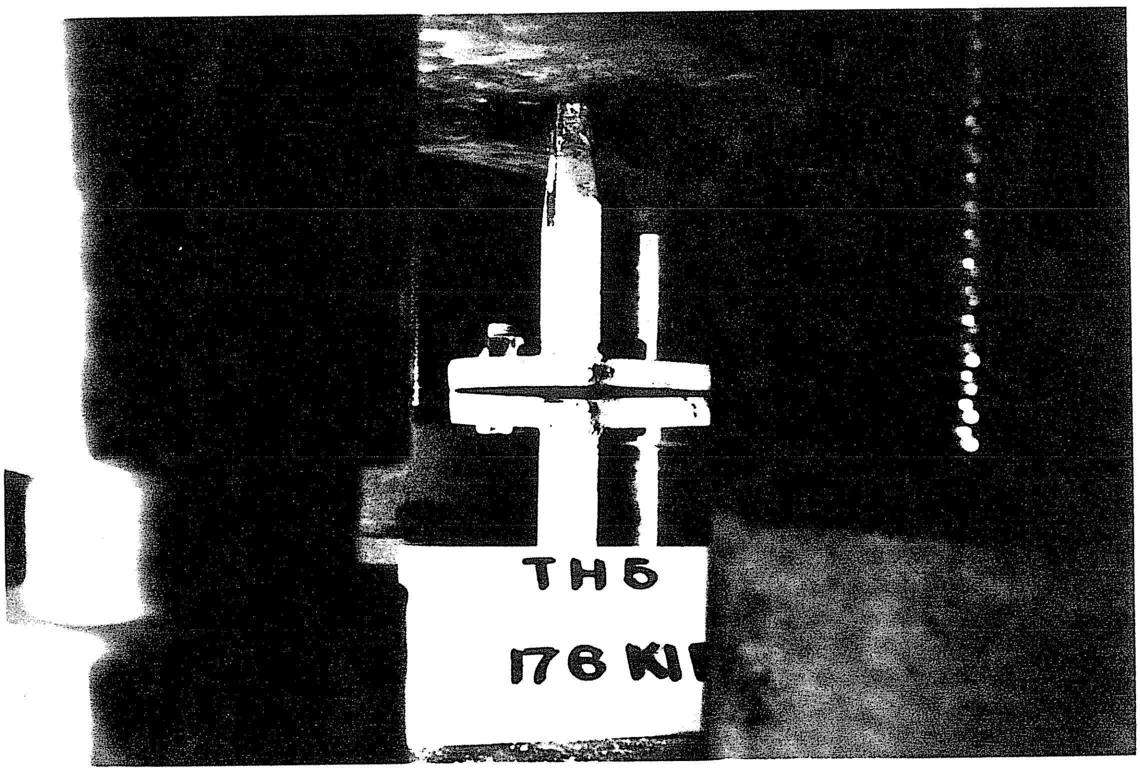
In the following section, details of the testing program and results are discussed, and comparison with the two design procedures are made.

2.2 Testing Details

The photographs in Figure 2.1 show the details of the test setup. Nine specimens were tested. Each specimen consisted of two "hanger" sections fabricated from plates as shown in Figure 2.2. The hanger sections were connected using four A325 bolts, pretensioned to the level specified in Table 1.23.5 of the 1978 AISC "Specifications for the Design, Fabrication and Erection of Structured Steel for Buildings." The tests were conducted using a universal testing machine with a tension capacity of 200 kips and a micro-computer based data ac-



a) Overview of Test Setup



b) After Bolt Rupture

Figure 2.1 Photographs of Tee-Hanger Testing

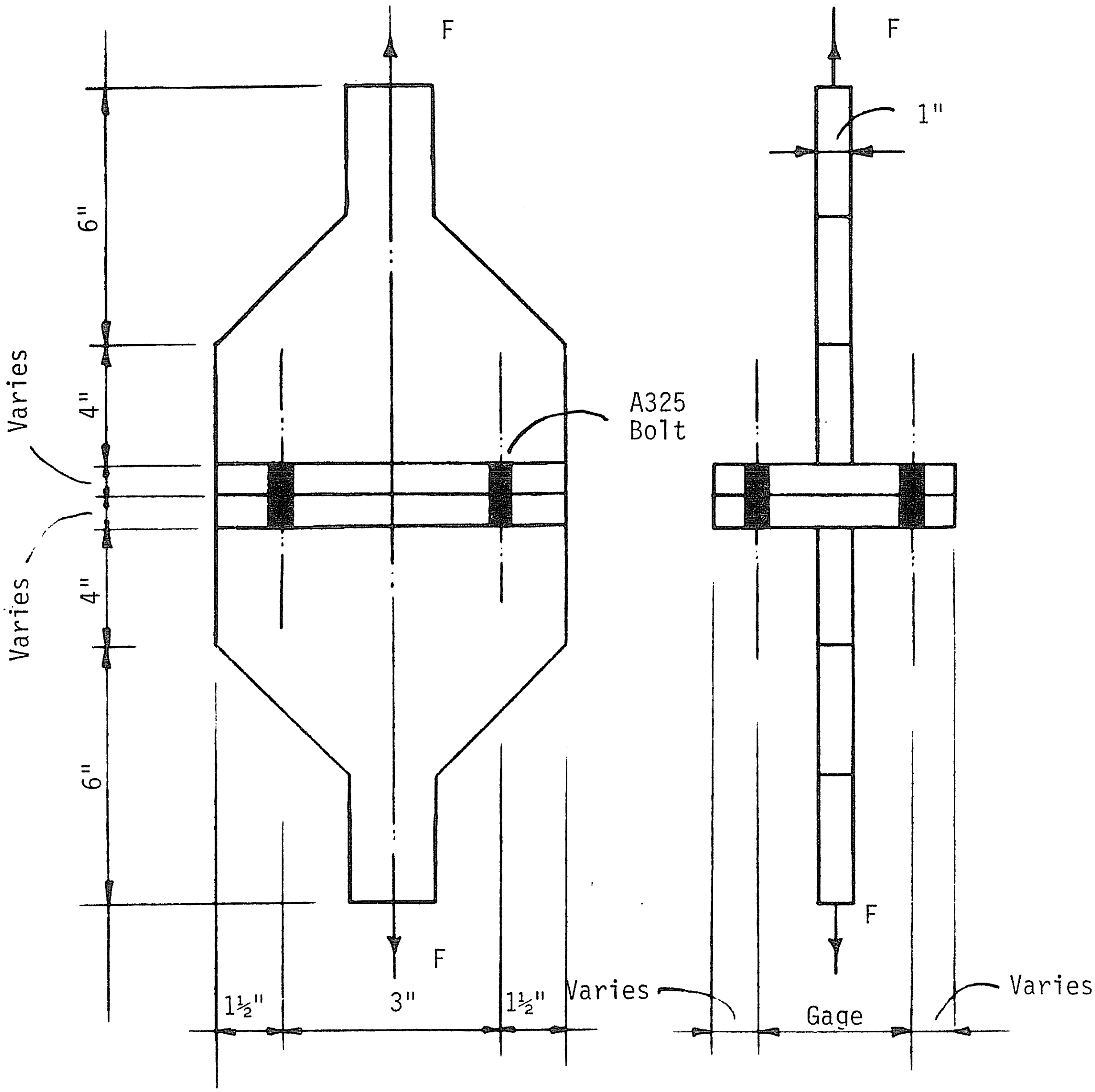


Figure 2.2 Typical Tee-Hanger Test Specimen

quisition system.

Test specimens. General details of the test specimens are shown in Figure 2.2. For all tests, the stem (or web) plates were 1 in. thick and 6 in. wide. The flange plate length was also 6 in., but the width varied from 6¼ in. to 9 in. to accommodate bolt gages of 3¼, 3½, 4½, 5½, and 6 in. Flange plate thicknesses were 3/8, 1/2, 3/4 and 1 in. and bolt diameters were 5/8, 3/4, and 1 in. The bolt pitch was 3 in. for all tests. Table 2.1 lists the nominal dimensions and weld sizes for each of the nine tests.

All plate material was obtained from W&W Steel Company, Oklahoma City, Oklahoma and had a nominal yield stress of 36 ksi. All bolts were ASTM A325 and the weld electrode was E70XX. The welds were designed to carry the anticipated ultimate capacity of the tee-hangers without exceeding their tensile strength. All welding was done by Fears Structural Engineering Laboratory personnel using the manual stick electrode (shielded metal arc) process.

Instrumentation. Instrumentation consisted of strain gaged, calibrated bolts; plate separation calipers; and the load indicator of the universal testing machine. Data was collected, processed, printed and plotted as the tests progressed using a microcomputer based data acquisition system.

Two instrumented bolts, located diagonally on the flange plates, were used in each test. Four calipers were used to measure plate separations: two in the plane of the tee-hanger stems at opposite ends of the flange plates and two adjacent to the sides of the stem plates at the longitudinal centerline of the flange plates. In the subsequent discussion, the former locations will be referred to as the "edge" locations and the latter as the "centerline" locations. Figure 2.3 shows the location of all instrumentation.

Testing Procedure. All specimens were first "white washed" so that flaking of mill scale due to high local strains could be detected. The specimens were then placed in the testing machine and the bolts pretensioned. The calibrated bolts were connected to the data acquisition prior to tightening so that the force could be monitored during tightening. At the beginning of each test, approximately 15% of the calculated failure load (AISC design procedure working load times 2.0) was applied without recording data and then removed. Following this initial step, "zero" readings were recorded. The specimen was then loaded in increments of approximately 10% of the calculated failure load. After each increment, reading from all instrumentation were recorded, the specimen was carefully inspected for any yielding and a photographic record was made. Load versus plate separations and

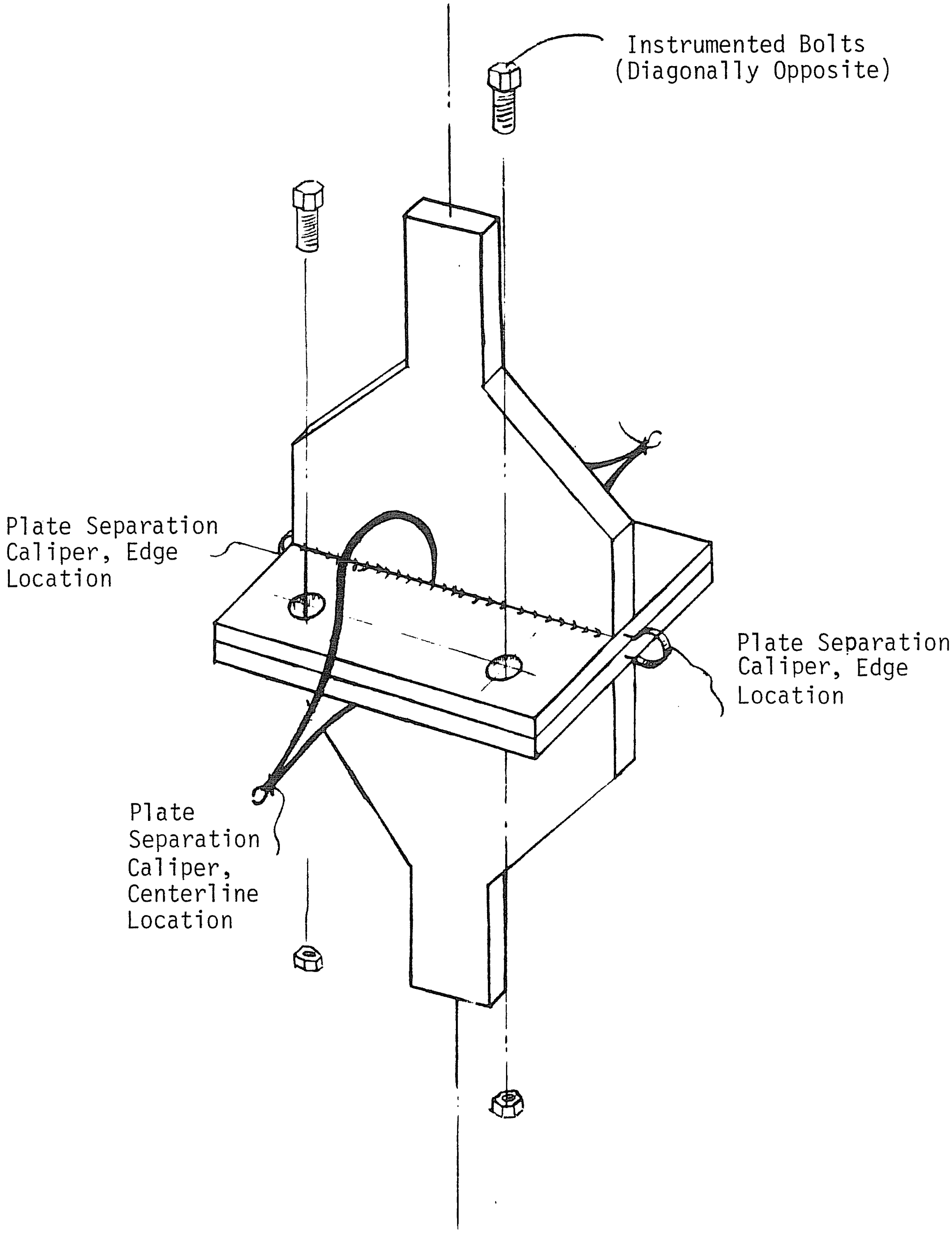


Figure 2.3 Tee-Hanger Instrumentation

bolt force plots were made as the test progressed. If the load-deflection plot softened or the bolt forces began to increase at an increasing rate, the loading increments were decreased.

The loading was continued until failure occurred. Failure was defined as (1) bolt strains greatly exceeding the yield strain of the bolt material, (2) excessive deformation of the flange plate measured as plate separation and (3) weld rupture. Bolt forces were calculated from strain data assuming elastic material properties (modulus of elasticity of 29,000 ksi). Yielding of the bolt material was not considered, hence, the reported measured bolt forces can exceed the rupture strength of the bolt.

2.3 Test Results and Comparisons

Test results consist of load versus bolt force data, load versus plate separation data, photographic record and description of test procedure and failure mode. Load versus bolt force data includes applied tensile load versus bolt forces at the diagonal locations on the flange plates. Load versus plate separation data includes applied tensile load versus average plate separation at the centerline and edge locations of the flange plates.

Detailed results for each test are found in Appendix B, Volume II. The test data "packages" contain a test summary sheet, specimen details, results of theoretical analyses, instrumentation drawing, an applied load versus bolt force plot and an applied load versus plate separation plot. The specimen details are summarized in Table 2.1 and the results are summarized in Table 2.2. A discussion of each test follows.

Test TH-1. The primary parameters for this test were 5/8 in. bolt diameter, 3/8 in. flange plate, and $5\frac{1}{2}$ in. gage. The bolts were tightened to the AISC pretension force, 19 kips, prior to the start of testing. The test summary sheet is on page B.2 of Volume II.

The maximum load applied in the test was 34 kips. The test was stopped after a yield plateau developed in the load versus plate, Figure B.5. Maximum plate separation was 0.150 in. The measured bolt forces remained at the pretension level until near the end of the test. The maximum measured bolt force was 23.0 kips. The AISC allowable tension load for 5/8 in. diameter A325 bolts is 13.5 kips.

The maximum applied load was 4.9 times the AISC working load. The AISC factored load, 2.0 times the working load was 13.88 kips. The Kennedy method predicted an ultimate load of 15.49 kips. The governing failure mode in both analyses was plate yielding as opposed to bolt rupture. It is noted that the load-plate separation curve, Figure B.5, began to soften at approximately 17

Table 2.1

Nominal Tee-Hanger Specimen Details

Test No.	Bolt Diameter (in.)	Flange Plate (in.)	Pitch (in.)	Gage (in.)	Length (in.)	Stem Thickness (in.)	Stem to Flange Weld (in.)
TH-1	5/8	3/8 x 8	3	5 1/2	6	1	5/16
TH-2	5/8	1/2 x 6 1/4	3	3 1/4	6	1	5/16
TH-3	5/8	1/2 x 8	3	5 1/2	6	1	5/16
TH-4	3/4	1/2 x 8	3	5 1/2	6	1	5/16
TH-5	3/4	3/4 x 6 1/2	3	3 1/2	6	1	5/16
TH-6	3/4	3/4 x 8	3	5 1/2	6	1	5/16
TH-7	1	3/4 x 9	3	6	6	1	5/16
TH-8	1	1 x 7 1/2	3	4 1/2	6	1	3/16*
TH-9	1	1 x 9	3	6	6	1	5/16

Full penetration with 3/16 in. fillet.

Table 2.2

Tee Hanger Test Results--Plate Behavior

Test No.	Tee Hanger Loads						Separation	Failure Mode
	Experimental		Predicted		Ratios		Max. Measured Plate Separation (in.)	
	Max. (kips)	Yield[1] (kips)	AISC x 2.0 (kips)	Kennedy (kips)	AISC/Yield	Kennedy/Yield		
TH-1	34.0	18.0	13.8	15.9	0.77	0.88	0.150	Plate yielding
TH-2	100.4	60.0	58.8	46.1	0.98	0.77	0.367	Large bolt forces[2]
TH-3	48.0	23.0	24.6	24.5	1.07	1.07	0.087	Plate yielding
TH-4	66.2	38.0	25.0	26.6	0.66	0.70	0.155	Plate yielding
TH-5	176.0	76.0	120.0	83.2	1.58	1.10	0.029	Rupture of bolts[3]
TH-6	90.0	51.0	56.0	50.8	1.10	0.99	0.508	Plate yielding
TH-7	105.8	52.0	50.0	51.4	0.96	0.99	0.086	Plate yielding
TH-8	170.0	118.0	142.2	105.8	1.21	0.90	0.040	Large bolt forces
TH-9	150.0	76.0	88.8	79.1	1.17	1.04	0.094	Large bolt forces

[1]Load at which the experimental load versus plate separation curve deviates 10% from the initial linear portion.

[2]Large bolt forces as a result of excessive plate separation

[3]After removal of instrumentation

kips, indicating that the Kennedy procedure is possible a good indicator of the initial yield load.

Test TH-2. For this test, 5/8 in. diameter bolts at 3¼ in. gage and a 1/2 in. thick flange plate were used. The bolts were tightened to 19 kips pretension force prior to starting the test. The test summary sheet is page B.8 of Volume II.

The maximum load applied in the test was 100.4 kips. The test was stopped because of rapidly increasing bolt forces (strains), Figure B.9. The maximum observed bolt force, assuming elastic material properties, was 33.6 kips. For comparison purposes, the AISC allowable tension load for a 5/8 in. diameter, A325 bolt is 13.5 kips.

From Figure B.10, a plateau had not formed in the applied load versus plate separation curves upon completion of the test. Maximum plate separation was 0.37 in., measured at the centerline location.

The maximum applied load was 3.41 times the AISC working load. The AISC factored load was 58.85 kips. The Kennedy method predicted an ultimate capacity of 46.06 kips. See Figure B.7. Both methods predicted plate yielding as the failure mode. Figure B.10 shows that the AISC factored load is close to the load level at which inelastic behavior becomes evident in the applied load versus plate separation curve.

Test TH-3. Primary parameters for this test were

5/8 in. diameter bolts, 1/2 in. thick flange plate and 5½ gage. Bolt pretension was 19 kips. The test summary sheet is page B.14 of Volume II.

High bolt forces, Figure B.14, and plate yielding, Figure B.15, caused the test to be terminated at an applied load level of 48.0 kips. The maximum measured bolt force was 26.6 kips (versus an ultimate of 27 kips). A well-defined yield plateau had developed in the load versus plate separation curve as shown in Figure B.15. The maximum plate separation, again at the centerline location, was 0.087 in.

The ratio of the maximum applied load-to-AISC service or working load is 3.87 for this test. The AISC factored load is 24.68 kips and the Kennedy ultimate load is 24.46 kips. Both methods predict plate yielding. Again, the onset of inelastic behavior in the load versus plate separation curves, Figure B.15, corresponds to the AISC and Kennedy ultimate load levels.

Test TH-4. For this test, the bolts were 3/4 in. in diameter, the flange plate 1/2 in. thick and the gage was 5½ in. Bolt pretension was 28 kips. The test summary sheet is page B.20 of Volume II.

The test was stopped at 66.2 kips after a yield plateau had developed in the load versus plate separation curve, Figure B.20. The maximum plate separation occurred at the edge location and was 0.143 in. Bolt forces initially dropped below the pretension level, but increased rapidly near the end of the test, Figure B.21. The maximum observed bolt force was 36.1 kips. The AISC allowable tension force in a 3/4 in. diameter A325 bolt is 19.4 kips.

The AISC and Kennedy predictions are found on page B.22. Both methods predicted plate yielding as the failure mode. The maximum applied load was 5.32 times the AISC working load. The AISC factored load is 24.90 kips and the Kennedy ultimate load is 26.57 kips. For this test, the onset of inelastic behavior occurred at about 35 kips of applied load, Figure B.20.

Test TH-5. The primary parameters in this test were 3/4 in. bolt diameter, 3/4 in. flange plate and 3½ in. gage. Bolt pretension was 28 kips. Page B.26 of Volume II is the test summary sheet.

Failure in this test was at 176 kips due to bolt rupture. As shown in Figure B.24, the bolt forces remained at the pretension level until the applied load reached 90 kips, at which time the forces began to increase at a rapid rate. The test was stopped at 110 kips and the specimen was unloaded. All instrumentation were removed including the instrumental bolts. Standard bolts were used to replace the instrumented bolts and the specimen was reloaded to failure.

The centerline load versus plate separation curve,

Figure B.25, is erratic at low loads. Both the centerline and edge location curves show that the limit of elastic behavior is about 95 kips of applied load.

The failure load was 2.93 times the AISC working load. The predicted ultimate loads from the AISC and Kennedy methods were 120.1 kips and 83.2 kips, respectively. Both methods predicted plate yielding as the failure mode.

Test TH-6. In the sixth tee-hanger test, the bolts were 3/4 in. diameter, the flange plate was 3/4 in. thick and the bolt gage was 5½ in. The bolt pretension force was 28.0 kips. The test summary sheet is found on page B.32, Volume II.

The test was stopped at 90.5 kips due to large bolt forces, Figure B.29, and plate separation, Figure B.30. The maximum observed bolt force was 44.6 kips (19.4 kips is the allowable). The maximum measured plate separation, before the instrumentation was removed at 85 kips of applied load, was 0.508 in. at the centerline location.

The predicted failure modes by both methods was plate yielding. The AISC method factored load is 56.0 kips, whereas the Kennedy method ultimate load is 50.8 kips. The maximum applied load was 3.23 times the AISC service load.

Test TH-7. Bolts, 1 in. diameter, at 6 in. gage and

3/4 in. thick flange plates were used in this test. The bolt pretensioning force was 51 kips. The test summary sheet is found on page B.38 of Volume II.

The test was stopped after the bolt forces exceeded 60 kips, Figure B.34, and plate separation exceeded 0.08 in., Figure B.35. The maximum applied load was 105 kips.

Bolt forces remained at the pretension level until the applied load reached approximately 65 kips. The proportional limit of the plate separation curves is approximately 50 kips. Maximum measured bolt force was 63.2 kips (the AISC allowable is 34.6 kips) and the maximum plate separation was 0.085 in. at the centerline location.

The maximum applied load was 4.20 times the AISC allowable load. The AISC and Kennedy ultimate loads were 50.0 kips and 51.4 kips, respectively, and both predict the onset of inelastic behavior in the plate separation curves, Figure B.35.

Test TH-8. The summary sheet for this test is found on page B.44 of Volume II. The bolts were 1 in. diameter and the pretension force was 51 kips. The flange plates were 1 in. thick and the bolt gage was $4\frac{1}{2}$ in. The maximum applied force was 170 kips. The test was stopped due to high bolt forces, Figure B.39, before the development of a flat yield plateau, Figure B.40.

The maximum observed bolt force was 76.5 kips (34.6

kips allowable), based on assumed elastic behavior. The maximum measured plate separation was at the edge location of magnitude 0.038 in.

The failure loads for both analysis procedures were predicted on plate yielding. The AISC procedure factored load was 142.2 kips and the Kennedy procedure ultimate load was 105.7 kips. The maximum applied load was 2.39 times the AISC working load.

Test TH-9. This test differed from TH-8 in that the gage was increased to 6 in. Four 1 in. diameter bolts, pretensioned to 51 kips, and 1 in. thick flange plates were used. The test summary sheet is page B.50 of Volume II.

The test was stopped at 151.2 kips due to high bolt forces, Figure B.44, and large plate separations, Figure B.45. The maximum bolt force at the final load level as 82 kips (AISC allowable is 34.6 kips) and the maximum plate separation was 0.086 in., measured at the edge location.

The maximum applied load was 3.40 times the AISC service load. The predicted ultimate loads due to plate yielding are 88.9 kips and 79.1 kips by the AISC and Kennedy methods, respectively. These loads correspond to the proportioned limit, approximately 75 kips, of the plate separation curves, Figure B.45.

2.4 CONCLUSIONS

Test results and comparisons with the AISC and Kennedy procedures are summarized in Tables 2.2 and 2.3. From the previous discussion concerning individual test results, it is evident that the ultimate strength of tee-hanger connections greatly exceeds the factored AISC capacity (2.0 times the allowable load) and the predicted ultimate load by the Kennedy method. Infact, the ratio of maximum applied experimental load to the factored AISC load ranges from 1.20 to 2.46 and the ratio of maximum applied load to the ultimate Kennedy load ranges from 1.61 to 2.53. However, extensive yielding in the flange plates occurred in all tests before the ultimate load is reached which in time caused a significant and rapid increase in bolt forces. Consequently, an attempt was made to define a load level so that bolt forces remain at or near the pretension level.

The third column of Table 2.2 lists the "yield" load for each of the nine tests. This load is determined from the applied load versus plate separation curves and corresponds to the load level at which the deviation from the initial straight portion of the curve equals 10%. The ratios of the AISC factored loads and the Kennedy ultimate loads to the yield loads are shown in Table 2.2. The range for the AISC loads is 0.66 to 1.58 with a mean of 1.056 and a standard deviation of 0.27. The range for the Kennedy loads is 0.77 to 1.10 with a mean of 0.938 and a standard deviation of 0.12. It is noted that a ratio less than one is conservative.

From the above results, it is concluded that (1) design of tee-hangers should be based on a "yield" load to present the possibility of premature bolt rupture and, (2) that the Kennedy method is a better predictor of the yield load as defined herein.

Table 2.3 summarizes bolt force data and comparisons. The bolt forces shown in the table were calculated from strain measurement data assuming elastic material properties with the modulus of elasticity taken as 29,000 ksi. Bolt forces calculated in this manner can exceed the rupture strength of the bolt if yielding occurs, thus, the forces should be used for comparison purposes only.

The table lists the maximum measured bolt force, the AISC assumed tensile strength of the bolt (2.0 times the allowable tension load) and the ratio of maximum applied load to the AISC factored load. This ratio varies from 0.85 to 1.24 for the nine tests indicating that the bolts were at or near their yield load in all tests. It is emphasized that a ratio greater than 1.0 indicates that the yield strain had been exceeded.

The table also shows the bolt force (maximum between the two instrumented bolts) at the previously defined

Table 2.3

Tee Hanger Test Results--Bolt Behavior

Test No.	Experimental		Rupture		AISC Method		Kennedy Method	
	Force at Yield Load[1] (kips)	Max. Bolt Force[1] (Kips)	AISC Allowable x 2.0 (kips)	Max. / AISC	Predicted Force at Fac. Ld. (kips)	Force at Factored Load[2] (kips)	Predicted Force at Fac. Ld. (kips)	Force at Factored Load[2] (kips)
TH-1	19.1	23.0	27.0	0.85	19.0	19.0	19.0[3]	19.0
TH-2	19.8	33.6	27.0	1.24	19.0	19.7	19.0[3]	19.5
TH-3	19.1	26.6	27.0	0.99	19.1	19.1	19.0	19.0
TH-4	26.4	36.1	38.8	0.93	28.0	26.7	28.0	26.7
TH-5	29.5	35.5	38.8	0.91	35.9	N.A.	28.0	29.3
TH-6	28.9	44.6	38.8	1.15	28.0	29.8	28.0	28.6
TH-7	50.9	63.2	69.2	0.91	51.0	50.9	51.0	50.9
TH-8	56.6	76.5	69.2	1.11	51.0	62.0	51.0	55.0
TH-9	50.5	82.0	69.2	1.18	51.0	51.7	51.0	50.6

[1] Calculated from strain measurements assuming elastic material properties ($E = 29{,}000$ ksi).

[2] Measured

Note: Bolt pretension for Tests TH-1, TH-2 and TH-3 was 19.0 kips, for Tests TH-4, TH-5 and TH-6 it was 28.0 kips and for Tests TH-7, TH-8 and TH-9 it was 51.0 kips.

yield load. In all nine tests, the bolt force was very near the pretension force indicating that prying forces are negligible at this load level.

Measured bolt forces at the AISC factored and Kennedy ultimate load levels are also listed. Except for Test TH-5, the predicted bolt forces from both methods at these load levels was the pretension force. In Test TH-5, the AISC method predicted a bolt force 8.7 kips or 32% greater than the pretension level. Unfortunately, the instrumented bolts had been removed prior to reaching the AISC factored load (see discussion of Test TH-5). In Test TH-8, the measured bolt forces exceeded the pretension level at both the AISC factored and Kennedy ultimate loads. For all other tests, the measured bolt forces were at or near the pretension level at the reference loads.

From the above bolt data analysis it is concluded that at the "yield" load and at the Kennedy ultimate load levels, bolt forces do not exceed the pretension level. Thus, the controlling failure mode for a tee-hanger designed using the Kennedy procedure in plate yielding and that premature bolt rupture will not occur.

Finally, it is concluded that the most consistent and accurate analysis procedure for tee-hangers is the Kennedy method as outlined in Section A.3. This procedure can be rearranged to a design procedure to find the required plate thickness, t_f for given applied load per bolt, T, and plate geometry. The steps are:

1. Select a bolt diameter such the B > 1.2 T.

2. Calculate the required thickness t_f from

$$t_f = \sqrt{\frac{4\ (1.67Tb - M_B)}{(C_1\ p + C_2\ p')\ F_y}} \tag{A.14}$$

3. Calculate the required ultimate bolt force including prying force from

$$B_u = 2.0T + (p'\ t_f^{\ 2}/4a) \sqrt{F_y^{\ 2} - 3\ [(2.0T)/p'f)]^2} \tag{A.15}$$

4. If B_u is less than twice the allowable bolt tension (2.0B), the design is satisfactory. If not, increase the bolt diameter and repeat steps 2, 3 and 4.

In the above procedure, the factor of safety against plate failure is 1.67 and against bolt rupture 2.0. The lower factor of safety against plate failure is justified by the large reserve strength available in the connections studied.

CHAPTER III

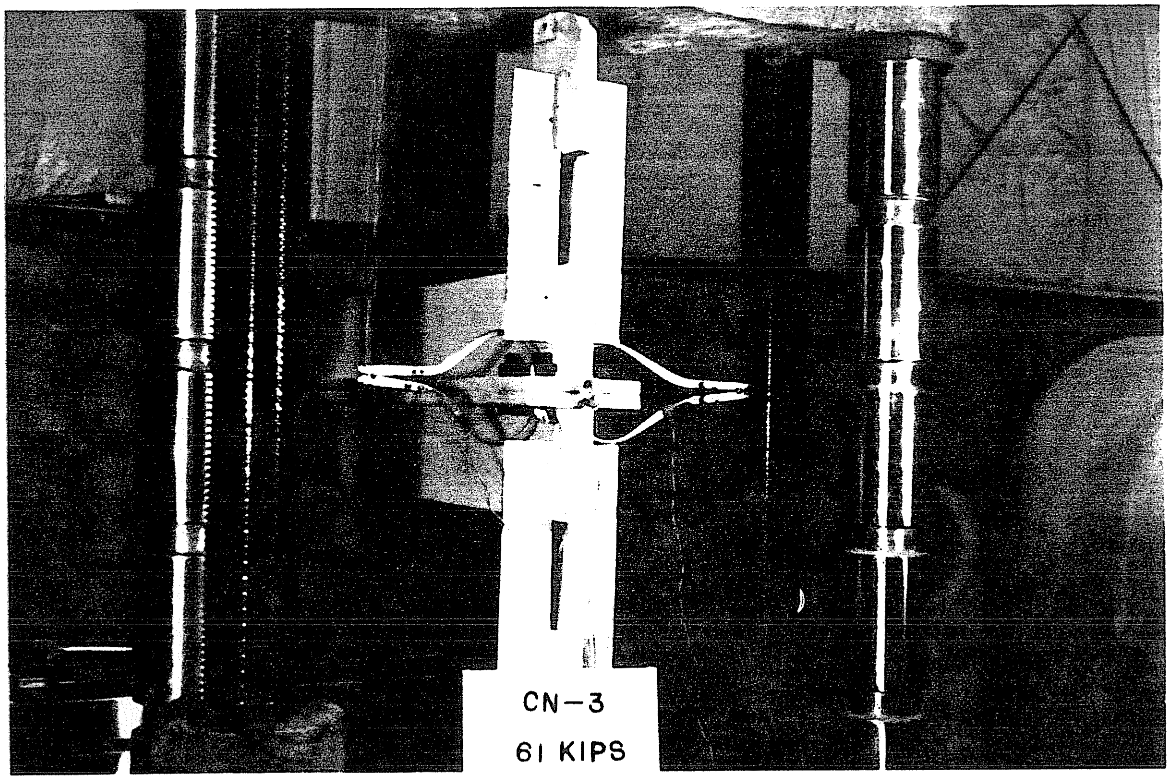
KNEE CONNECTION TESTS AND ANALYSES

3.1 General

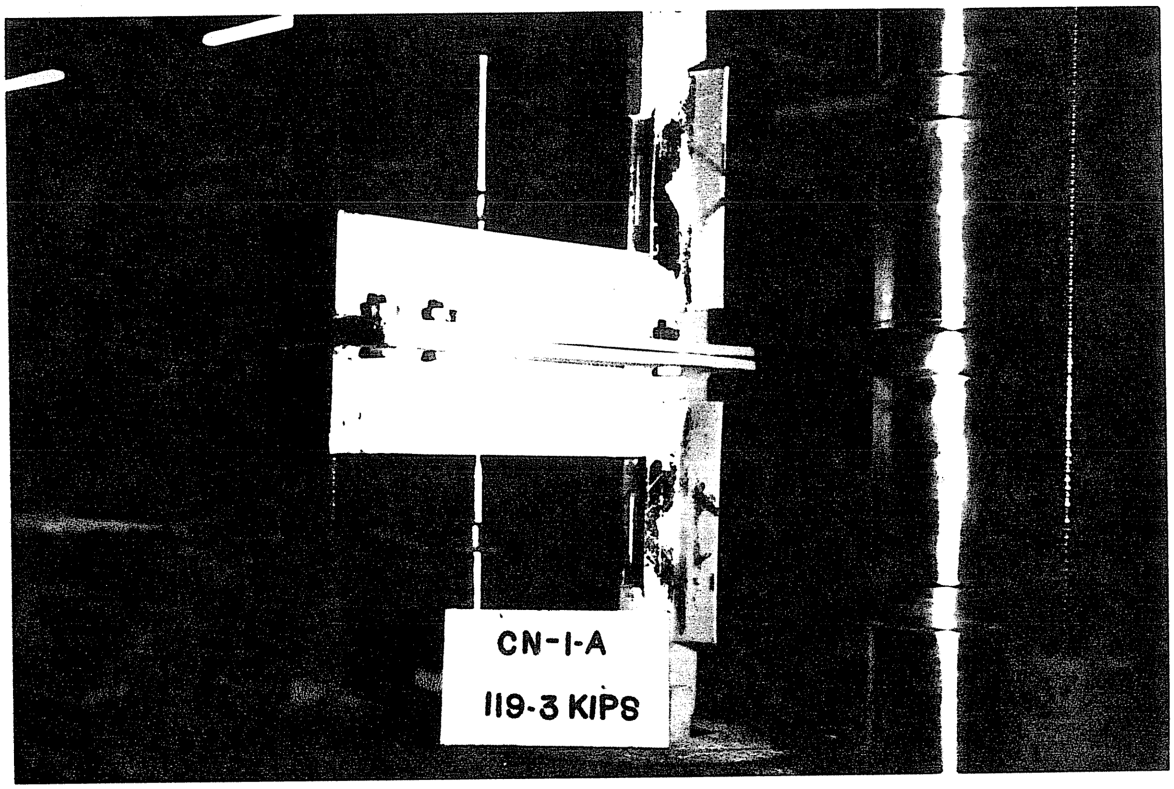
To verify the validity of using tee-hanger specimens to study the behavior of the rafter-to-column moment connections used in the frames under investigation, four tests of complete connections were conducted. Figure 3.1 is photographs of the test setup.

The specimens consisted of the complete tee sections at the rafter-to-column connection. Angles to simulate the tension chord of the column were welded to one end of the tee section stems. Tab plates were then welded to these angles so that standard testing machine friction jaws could be used to apply load to the specimens. Eight bolts were used to connect the two sections of each specimen. Four bolts were located at the angle end and the other four at the far end of the tee flange. The length of the tee sections in the four tests corresponded to the knee area dimension in representative frames.

Test procedures and measurements were similar to those used for the tee-hanger tests. Results from the knee connection tests are compared to results from the



a) Overview of Test Setup



b) Failure of Specimen

Figure 3.1 Photographs of Knee Connection Testing tee-hanger tests and to predictions from the AISC and Kennedy analytical models.

3.2 Testing Details

A total of six tests of four knee connection specimens were conducted. General details of a typical test specimen are shown in Figure 3.2 and detail dimensions are found in Table 3.1. Each specimen consisted of two sections fabricated from plates and angles. The sections were connected using eight A325 bolts, pretensioned to the level specified in Table 1.23.5 of the 1978 AISC "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings." The tests were conducted using a universal testing machine with a tension capacity of 200 kips and a micro-computer based data acquisition system.

Test Specimens. General details of the test specimens are shown in Figure 3.2. For all tests, the stem (or web) plates were 1 in. thick and 6 in. wide. The flange plates varied in width from 6¼ in. to 7½ in. to accommodate bolt gages from 3¼ to 4½ in. and varied in length from 23 in. to 64. Flange plate thicknesses were 1/2, 5/8, 3/4 and 1 in. and bolt diameters were 5/8, 3/4 and 1 in. The bolt edge distances and pitch were 3 in. for all tests.

All plate and angle material had a nominal yield

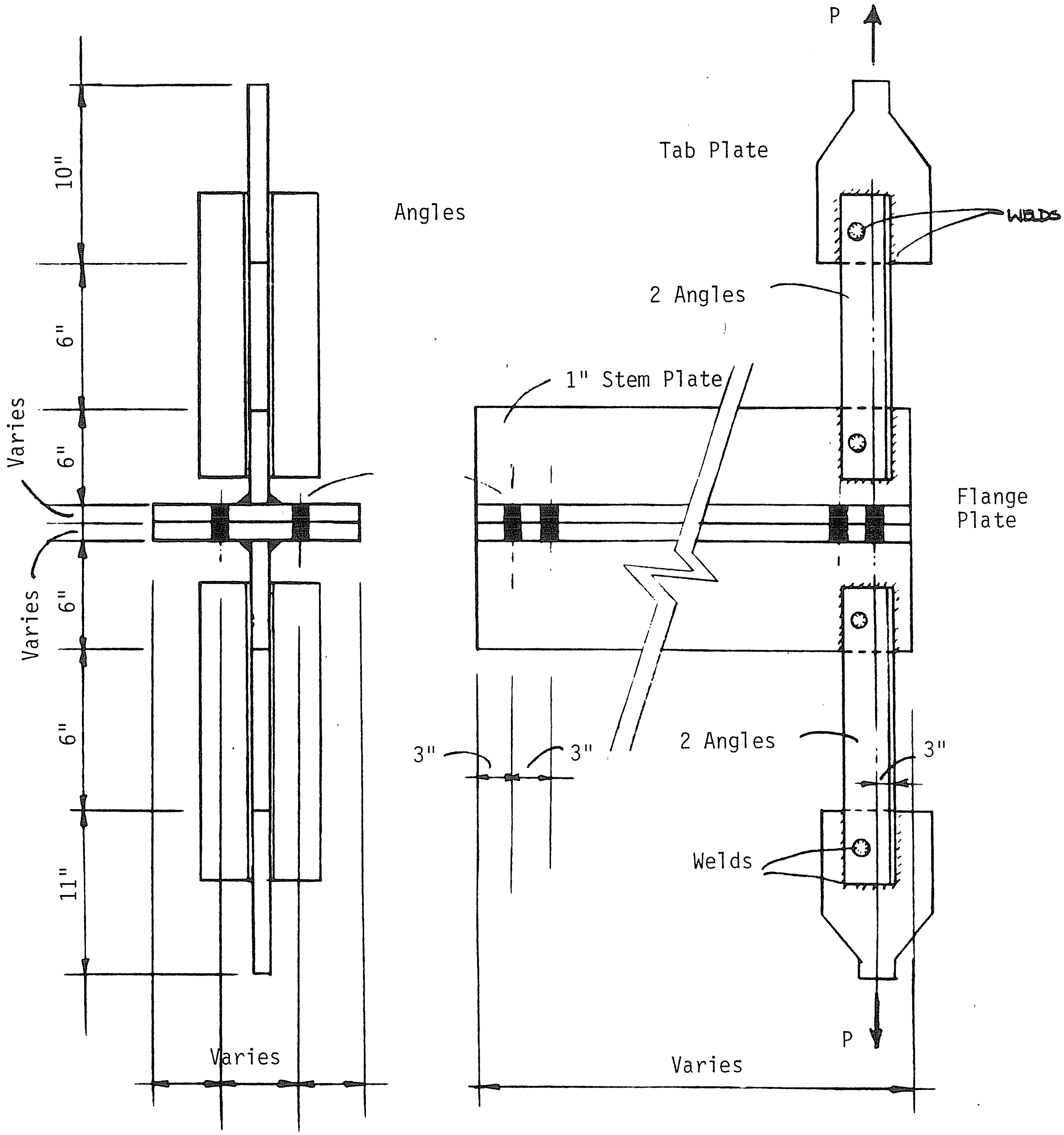


Figure 3.2 Typical Knee Connection Test Specimen

Table 3.1

Nominal Knee Connection Specimen Details

Test No.	Bolt Diameter (in.)	Flange Plate (in.)	Pitch (in.)	Gage (in.)	Length (in.)	Stem Thickness (in.)	Stem to Flange Weld (in.)
CN-1	5/8	1/2 x 6 1/4	3	3 1/4	23	1	3/16 x 6
CN-1A*	5/8	1/2 x 6 1/4	3	3 1/4	23	1	5/16 x 6
CN-2	3/4	5/8 x 7	3	4	35	1	1/4 x 6 1/4
CN-3	3/4	3/4 x 6 1/2	3	3 1/2	47	1	5/16 x 5 3/4
CN-4	1	1 x 7 1/2	3	4 1/2	64	1	5/16 x 6 3/4

*Reweld of web-to-flange weld.

strength of 50 ksi; all bolts were A325. The specimens were fabricated by VULCRAFT at their Brigham City, Utah plant. The welds were designed to carry the estimated ultimate load capacity of the connection without exceeding their factored allowable tensile strength. Welds were placed at the ends of the tee sections, starting from the transverse edge and extending inward for the length shown in Table 3.1 (see Figure 3.2). Weld size is also shown in Table 3.1.

Instrumentation. Instrumentation was the same as used for the tee-hanger tests -- calibrated bolts, plate separation calipers and the load indicator of the universal testing machine. The instrumental bolts were location diagonally opposite each other on the load application side of the connection. Three calipers were used to measure plate separations at the same end of the specimens. Two calipers were located between the bolt rows, one on each side of the stem. The third caliper measured the plate separation at the end of the specimen in the plane of the tee stems. For reference with the tee-hanger test data, the former location will be referred to as the "centerline" location and the latter as the "edge" location. The instrumentations are shown in Figure 3.3.

Testing Procedure. All specimens were first "white washed" so that flaking of mill scale due to high local strains could be detected. The specimens were then

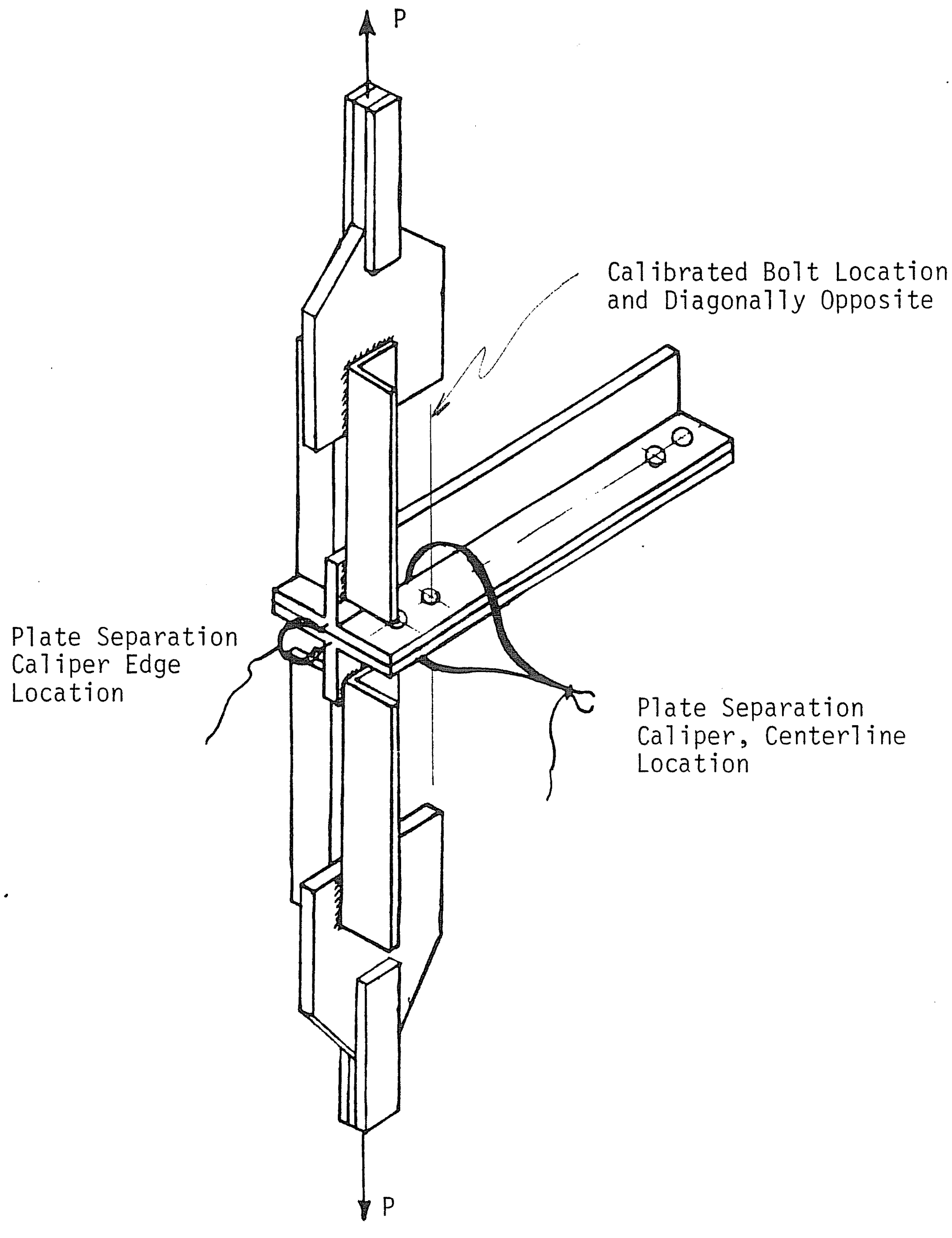


Figure 3.3 Knee Connection Instrumentation

placed in the testing machine and the bolts pretensioned. The calibrated bolts were connected to the data acquisition prior to tightening so that the force could be monitored during tightening. At the beginning of each test, approximately 15% of the calculated failure load (AISC design procedure working load times 2.0) was applied without recording data and then removed. Following this initial step, "zero" readings were recorded. The specimen was then loaded in increments of approximately 10% of the calculated failure load. After each increment, reading from all instrumentation were recorded, the specimen was carefully inspected for any yielding and a photographic record was made. Load versus plate separations and bolt force plots were made as the test progressed. If the load-deflection plot softened or the bolt forces began to increase at an increasing rate, the loading increments were decreased.

The loading was continued until failure occurred. Failure was defined as (1) bolt strains greatly exceeding the yield strain of the bolt material, (2) excessive deformation of the flange plate measured as plate separation and (3) weld rupture. Bolt forces were calculated from strain data assuming elastic material properties (modulus of elasticity of 29,000 ksi). Yielding of the bolt material was not considered, hence, the reported measured bolt forces can exceed the rupture strength of the bolt.

3.3 Test Results and Comparisons

Test results consist of load versus bolt force data, load versus plate separation data, photographic record and description of test procedure and failure mode. Load versus bolt force data includes applied tensile load versus bolt forces at the diagonal locations on the flange plates. Load versus plate separation data includes applied tensile load versus average plate separation at the centerline and edge locations of the flange plates.

Detailed results for each test are found in Appendix C, Volume II. The test data "packages" contain a test summary sheet, specimen details, results of theoretical analyses, instrumentation drawing, shop details, an applied load versus bolt force plot and an applied load versus plate separation plot. The specimen details are summarized in Table 3.1 and the results are summarized in Table 3.2. A discussion of each test follows.

Test CN-1. This test corresponds to test TH-2 in that the basic geometry, flange plate width and flange plate thickness are the same. The primary parameters were 5/8 in. bolt diameter, 1/2 in. flange plate and 3-1/4 in. gage. The bolts were tightened to 19 kips pretension force prior to starting the test. The test summary sheet is page C.2 of Volume II.

Failure occurred at 95.4 kips because of rupture of a flange-to-stem weld. Figure C.5 shows that the measured bolt forces had increased above the pretension level prior to failure. The maximum measured bolt force was 28.1 kips; the AISC allowable tension load for a 5/8 in. diameter, A325 bolt is 13.5 kips. The centerline plate separation curve had begun to soften prior to failure, but a yield plateau had not formed, Figure C.6. The edge plate separation was negligible. Maximum centerline plate separation was 0.019 in.

The AISC factored load for the specimen was 81.7 kips and the Kennedy method predicted an ultimate capacity of 61.5 kips, see Figure C.2. Both loads were exceeded prior to the weld rupture and both loads were based on plate yielding as the failure mode. Figure C.6 shows that the AISC factored load is close to the load level at which inelastic behavior becomes evident in the applied load versus plate separation curve, approximately 55 kips.

Test CN-1A. Since weld rupture was not the desired failure mode and the specimen used in Test CN-1 appeared to be undamaged, the test was repeated after rewelding. Weld size was also increased from 3/16 in. to 5/16. The length was kept at 6 in. See Table 3.1. The test summary sheet is page C.9 of Volume II.

Failure occurred by rupture of the bolts at the load application end of the specimen at a load of 119.3 kips. The maximum measured bolt force was 42.3 kips (AISC allowable is 13.5 kips). The load versus plate separation curve had softened, Figure C.12, but a yield plateau had not developed prior to bolt rupture. The maximum plate separation was 0.026 in. at the edge location.

The ratio of the maximum applied load to AISC working load is 2.92 for this test. The AISC factored load is 81.7 kips and the Kennedy ultimate load is 61.5 kips both based on plate yielding. The onset of substantial yielding in the plate separation curves occurred at approximately 65.0 kips.

Test CN-2. For this test, the bolts were 3/4 in. diameter, the flange plate was 5/8 in. thick and the bolt gage was 4 in. Bolt pretension was 28.0 kips. The test summary sheet is page C.16 of Volume II.

The test was stopped due to yielding of the angles used to apply the tension load, however, the bolt forces were also very high. The maximum measured bolt force was 40.7 kips; the AISC allowable for the 3/4 in. diameter bolts used in the test is 19.4 kips. A yield plateau had not formed in the plate separation curves, Figure C.18. Maximum plate separation was 0.035 in.

The AISC factored and Kennedy ultimate loads for the specimen are 90.1 kips and 71.9 kips, respectively; both ielding became evident in the plate separation curves at approximately 70.0 kips.

Test CN-2A. This test is a repeat of Test CN-2. The only difference being the insertion of 9/6 gage steel shim between the tee section flanges. The shim was approximately 6 in. long and 2½ in. wide. It was placed between and centered on the bolts at the load application end of the specimen. The 6 in. length of shim was in the same direction as the longitudinal direction of the specimen. The test summary sheet is found on page C.23, Volume II.

The maximum applied force was 136 kips. The test was stopped due to high bolt forces; maximum was 45.6 kips. However, the bolt forces stayed at the pretension level until approximately 100 kips had been applied, Figure C.23. The plate separation curves also remained linear to the 100 kips level, Figure C.24. Yield plateaus had developed in both curves prior to the termination of the test. The AISC and Kennedy models do not apply to this test because of the shim.

Figures C.25 and C.26 compare the bolt force and plate separation curves for the two tests. Ideal bolt behavior in a tee-hanger type connection is for the bolt to remain at the pretension level until the connected plates begin to separate, e.g., prying forces are negligible. It is obvious from Figures C.25 and C.26 that this behavior was achieved in Test CN-2A. Furthermore, the stiffness of the connection greatly improved with the use of the shim; the yield load increased about 30%. Finally, the ultimate capacity increased about 10% when the shim was used.

Test CN-3. This test corresponds to TH-5: 3/4 in. diameter bolts, 3/4 in. flange plate, 3½ gage and 28 kips bolt pretension force. The test summary sheet is page C.32 of Volume II.

The test was stopped at 150.0 kips due to high bolt forces. Instrumentation was removed at 135 kips.

Bolt forces exceeded 42 kips prior to failure, Figure C.31. The AISC allowable for the bolt is 19.9 kips. A yield plateau had not developed in either plate separation curve, Figure C.32, before the instrumentation was removed. Maximum separation was 0.026 in., at the edge location.

The factored AISC and the ultimate Kennedy loads for the connection were 155.5 kips and 112.2 kips, respectively. Both loads are determined by plate yielding. The yield load, based on the plate separation curves, is 100 kips.

Test CN-4. This test corresponds to Test TH-8. The flange plates were 1 in. thick, bolts were 1 in. in diameter at a 4½ in. gage and were pretensioned to 51.0 kips. The test summary sheet on page C.39 of Volume II describes the test in detail. A flange-to-stem weld ruptured at 192.3 kips of applied load.

Maximum measured bolt force was 73.0 kips versus the AISC allowable of 34.6 kips. Maximum plate separation occurred at the edge location and was 0.031 in. As shown in Figure C.38 a yield plateau had not developed prior to the termination of the test. The yield load from the plate separation curves is 150 kips.

The factored AISC load is 197.5 kips and the ultimate Kennedy load is 141.0 kips, both based on plate yielding.

3.4 Conclusions

Test results and comparisons with the AISC and Kennedy procedures are summarized in Tables 3.2 and 3.3. Not shown in the tables are the ratios of the maximum applied loads to the factored AISC and ultimate Kennedy loads. For tests where the welds did not fail, the range of this ratio for the AISC loads is 0.97 to 1.51 and that for the Kennedy loads is 1.34 to 1.94. The corresponding ranges for the tee-hanger (TH) tests were 1.20 to 2.46 and 1.61 to 2.53. Comparison of the two sets of ranges indicates that the knee connection configuration has an adverse effect on strength, however, the measured strength nearly equals or exceeds the predicted failure loads based on nominal yield strength of the flange plate

Table 3.2

Knee Connection Test Results--Plate Behavior

Test No.	Tee Hanger Loads						Separation	Failure Mode
	Experimental		Predicted		Ratios			
	Max. (kips)	Yield* (kips)	AISC x 2.0 (kips)	Kennedy (kips)	AISC/Yield	Kennedy/Yield	Max. Measured Plate Separation (in.)	
CN-1	95.4	55.0	81.7	61.5	1.49	1.12	0.019	Weld rupture
CN-1A	119.3	65.0	81.7	61.5	1.26	0.95	0.026	Rupture of bolts
CN-2	124.4	70.0	90.1	71.9	1.29	1.03	0.035	High bolt forces
CN-2A	136.0	90.0	**	**	**	**	0.054	High bolt forces
CN-3	150.0	100.0	155.5	112.2	1.56	1.12	0.026	High bolt forces
CN-4	192.3	150.0	197.5	141.0	1.32	0.94	0.031	Weld rupture

*Load at which the experimental load versus plate separation curve deviates 10% from the initial linear portion.

**A shim was used in Test CN-2A. Theoretical results do not apply.

Table 3.3

Knee Connection Test Results--Bolt Forces

Test No.	Experimental		Rupture		AISC Method		Kennedy Method	
	Force at Yield Load[1] (kips)	Max. Bolt Force[1] (kips)	AISC Allowable x 2.0 (kips)	Max./AISC	Predicted Force at Fac. Ld. (kips)	Force at Factored Load[2] (kips)	Predicted Force at Fac. Ld. (kips)	Force at Factored Load[2] (kips)
CN-1	20.0	28.1	27.0	1.04	24.4	19.0	19.7	19.0
CN-1A	22.0	43.3	27.0	1.60	24.4	19.2	19.7	19.2
CN-2	33.1	40.7	38.8	1.05	28.2	30.0	27.2	30.0
CN-2A	28.8	45.6	38.8	1.18	**	**	**	**
CN-3	34.3	42.1	38.8	1.09	38.9	29.6	36.3	29.4
CN-4	61.4	73.0	69.2	1.05	61.5	54.0	48.4	53.6

[1]Calculated from strain measurements assuming elastic material properties (E = 29.000 ksi).

[2]Measured

Note: Bolt pretension for Tests CN-1 and CN-1A was 19.0 kips, for Tests CN-2, CN-2A and CN-3 it was 29.0 kips and for Test CN-4 it was 51.0 kips.

**A shim was used in Test CN-2A. Theoretical results do not apply.

material.

The third column of Table 3.2 lists the "yield" load for each of the nine tests. This load is determined from the applied load versus plate separation curves and corresponds to the load level at which the deviation from the initial straight portion of the curve equals 10%. The ratios of the AISC factored loads and the Kennedy ultimate loads to the yield loads are shown in Table 3.2. The range for the AISC loads is 1.00 to 1.56 with a mean at 1.32 and a standard deviation of 0.20. The range for the Kennedy loads is 0.80 to 1.12 with a mean of 0.99 and standard deviation of 0.12. A ratio less than one is conservative.

Even though three of the CN specimens were geometrically identical to three of the TH specimens (CN-1 and TH-2, CN-3 and TH-5, CN-4 and TH-8), direct comparisons cannot be made since the nominal yield strength of the plate material in the CN tests was 50 ksi and that used in the TH tests was 36 ksi. Table 3.4 lists the ratios of the factored AISC and the ultimate Kennedy loads to the experimental maximum and yield loads for the four sets of corresponding tests. Examination of this data shows fair agreement between the corresponding yield load ratios and poor agreement for the corresponding maximum load ratios. Note that all calculations are based on nominal yield stresses.

Table 3.4

Comparison of Knee Connection and Tee-Hanger Results

Ratios	CN-1A	TH-2	CN-3	TH-5	CN-4*	TH-8
$\frac{\text{AISC}}{\text{Max.}}$	1.46	1.71	0.96	1.47	0.97	1.20
$\frac{\text{Kennedy}}{\text{Max.}}$	1.94	2.18	1.34	2.53	1.36	1.61
$\frac{\text{AISC}}{\text{Yield}}$	1.26	0.98	1.56	1.58	1.32	1.21
$\frac{\text{Kennedy}}{\text{Yield}}$	0.95	0.77	1.12	0.92	0.94	0.90

*Failure mode was weld rupture.

AISC = factored load from AISC procedure.

Kennedy = ultimate load from kennedy procedure.

Max. = maximum experimental load.

Yield = experimental load at 10% deviation from initial portion of plate separation curves.

From the tee-hanger and knee connection tests results, it is concluded that; (1) design of the type of knee connections tested here should be based on a "yield" load to prevent the possibility of premature bolt rupture and, (2) that the Kennedy method is a better predictor of the yield load as defined herein.

CHAPTER IV

KNEE AREA TESTS AND ANALYSES

4.1 General

The objective of the knee area tests was to study the behavior of the knee portion of typical truss-type rigid frames when subjected to moment, shear and thrust caused by gravity loading. Each specimen consisted of a rafter section and a column section connected using the bolted tee-sections described in Chapter II. The assemblies were subjected to a single force as shown in Figure 4.1. The configuration of the test specimen and location of the line of action of the applied force was determined for each specimen so that the moment, shear and thrust at the connection were identical to actual frame design values for combined dead plus full live loads. The splice connections for all specimens were designed to have a strength greater than required so that failure would occur in the members and not the connection.

To facilitate loading and support, the specimens were tested in a rotated position relative to use in a building. Figure 4.2 shows schematic drawings of the test setups. As shown in Figure 4.2(a), hot-rolled steel

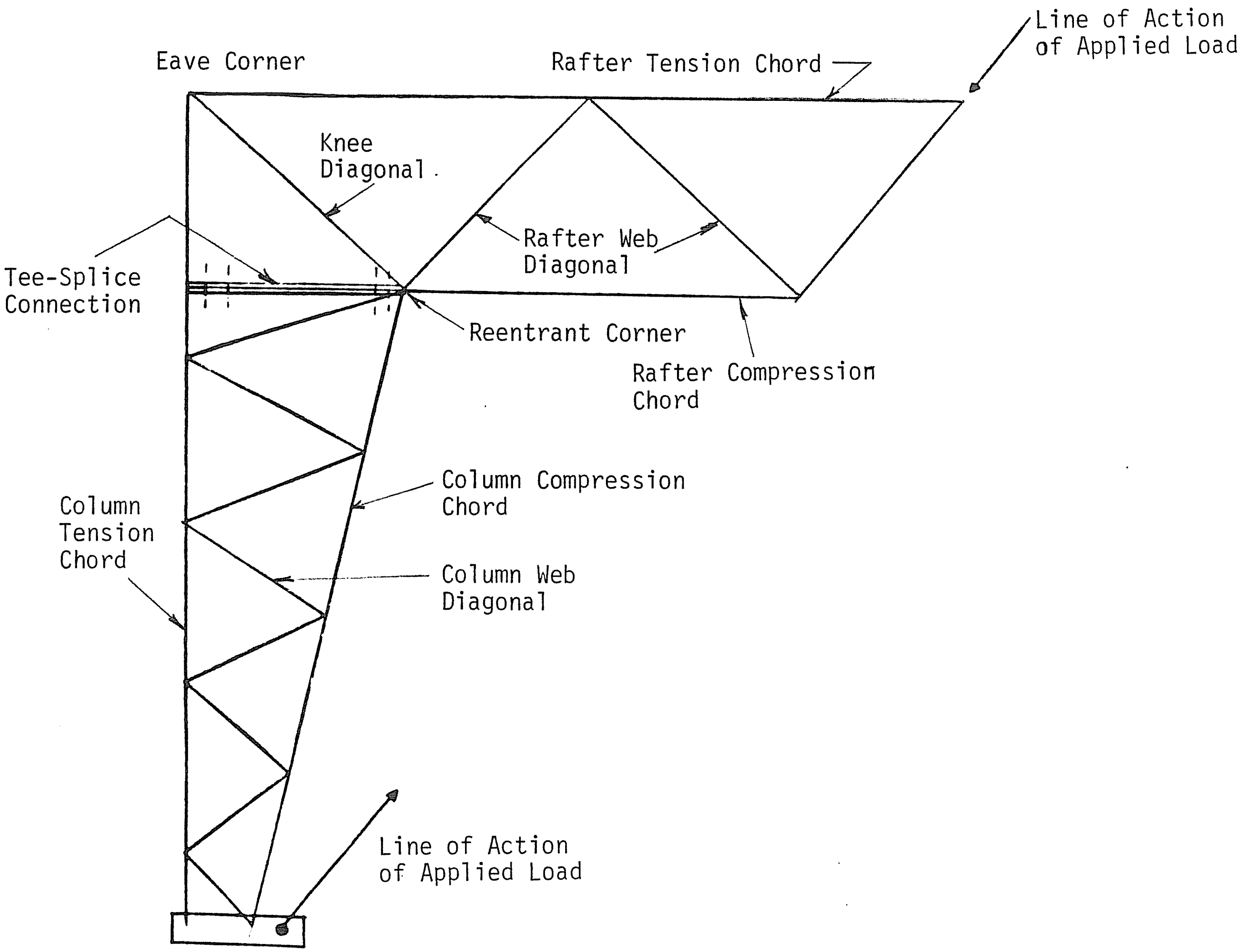
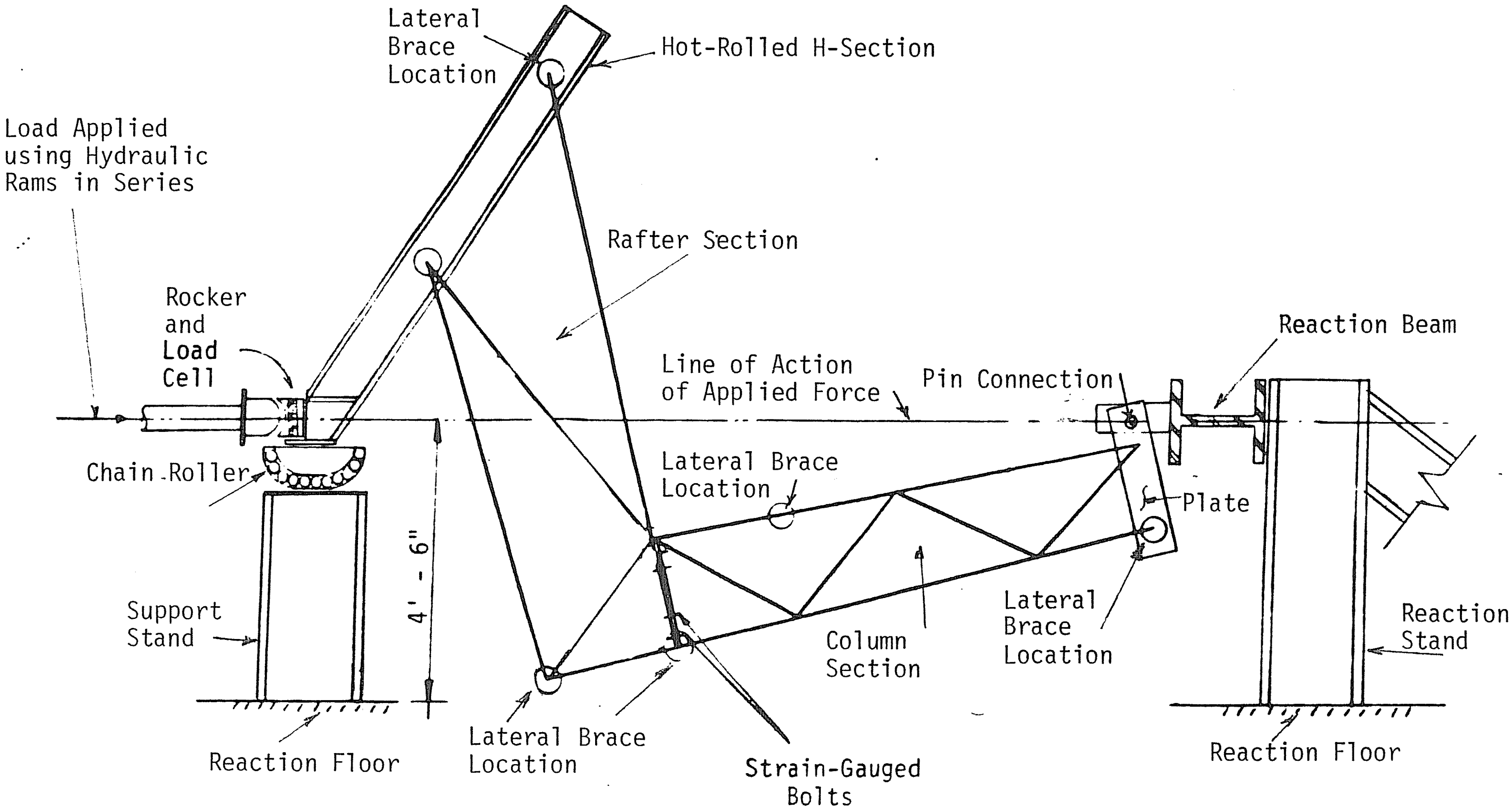
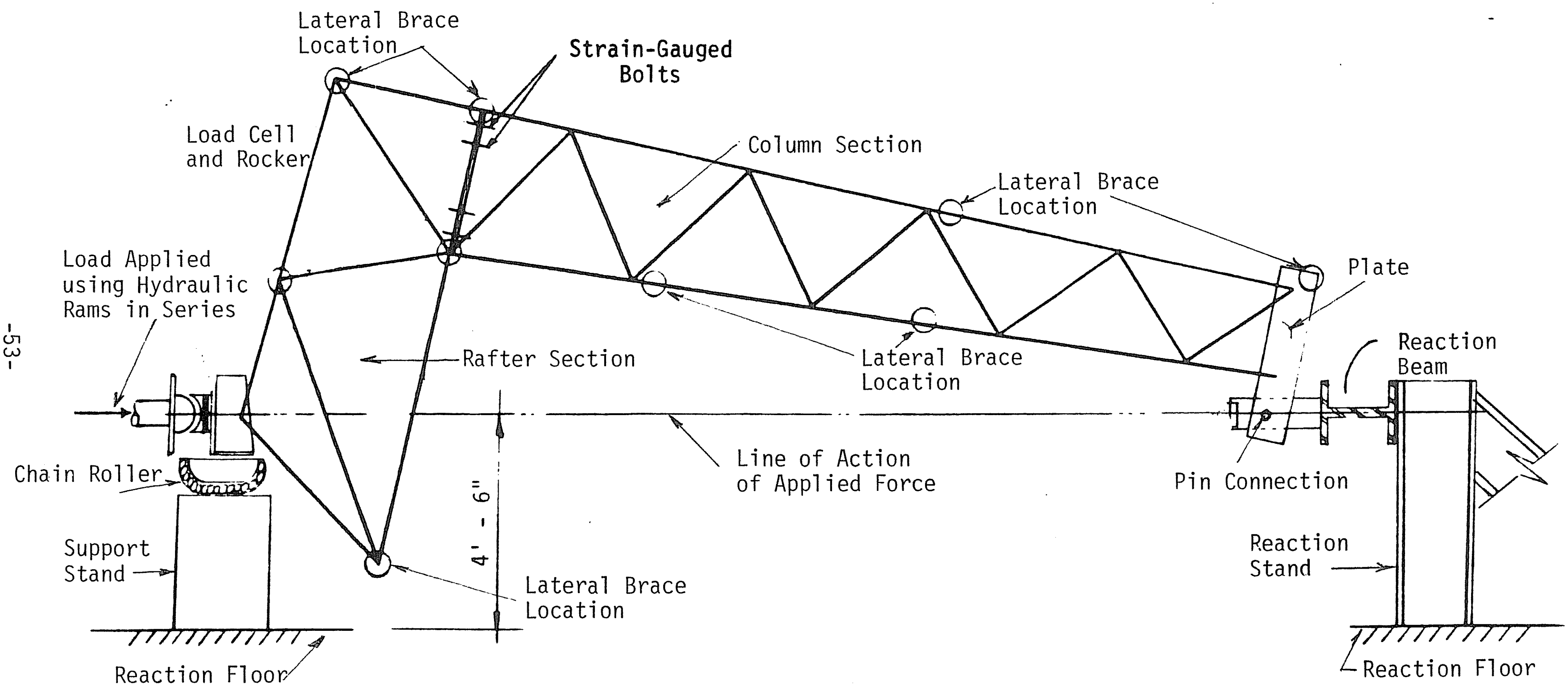


Figure 4.1 Knee Area Loading



a) Specimen with Hot-Rolled Section

Figure 4.2 Knee Area Test Setup Schmatics



b) Specimen Loaded Directly

Figure 4.2 Knee Area Test Setup Schmatics, Continued beams were used in some test setups so that the line of action of the applied force could be correctly positioned without affecting the relative magnitude of the chord and diagonal member forces.

Seven tests were conducted using six specimens; one specimen was tested twice. Table 4.1 lists the test parameters and Table 4.2 summarizes the results. Specimen details and test data are found in Appendix D of Volume II.

4.2 Testing Details

Test Setup. The test setups, Figure 4.2, consisted of the knee area specimen, two reaction stands, two 120 kips capacity hydraulic rams in series, a 300 kips capacity load cell, a chain roller, lateral base mechanisms and lateral brace support frames. The lateral brace mechanisms are three link devices which permit unrestricted longitudinal and vertical movement of the braced point but prevent transverse movement. See Reference 5 for details.

The reaction stands and the lateral brace support frames were bolted to the laboratory reaction floor. The concrete/steel reaction floor is 30 ft. by 60 ft., weighs one million pounds and is capable of reacting 320,000 pounds of uplift at any location. Four W36 x 150 x 58 feet steel beams are buried in concrete with top flanges

Table 4.1

Nominal Knee Area Specimen Details

Test No.	Bolt Diameter/Length (in.)	Flange Plate (in.)	Pitch (in.)	Gage (in.)	Length (in.)	Stem Thickness (in.)	Stem to Flange Weld (in.)
KN-1	3/4 - 2 1/4	1/2 x 7	3	3 1/2	17 1/2	1	5/16 x 6
KN-2	3/4 - 2 1/4	1/2 x 7	3	3 1/2	29 1/2	1	5/16 x 6
KN-3	3/4 - 2 1/4	3/4 x 7	3	3 1/2	34 1/8	1	3/8 x 6
KN-4	1 - 3 1/4	1 x 8	3	4 1/2	40 1/8	1	3/8 x 6.4
KN-5	1 - 3 1/4	1 x 8	3	4	36 1/4	1	3/8 x 7.7
KN-6	1 1/8 - 3 1/2	1 x 10	3	4	67 1/4	1	3/8 x 9
KN-7	3/4 - 2 1/4	3/4 x 7	4	4	36	1	3/8 x 7

Table 4.2

Knee Area Test Results

Test No.	Predicted Loads				Experimental Loads		Applied Load / Working Load	Failure Mode
	Working* Load (kips)	Ultimate** Load (kips)	Bolt Allowable (kips)	Bolt Ultimate (kips)	Max. Applied Load (kips)	Max. Bolt Force (kips)		
KN-1	20.64	34.26	19.4	38.8	33.3	28.9	1.61	Local buckling of member
KN-2	17.36	28.82	19.4	38.8	36.0	28.6	2.07	Weld rupture, local buckling of member
KN-3A	38.86	64.50	19.4	38.8	58.6	29.6	1.51	Buckling of member
KN-3B***	38.86	64.50	19.4	38.8	60.1	31.3	1.55	Considerable lateral displacement
KN-4	73.68	122.31	34.6	69.2	109.6	72.8	1.49	Lateral buckling of stem plate
KN-5	58.16	96.54	34.6	69.2	80.0	60.7	1.38	Out-of-plane buckling
KN-6	69.33	115.08	43.7	87.4	—	—	—	Not tested
KN-7	38.86	64.50	19.4	38.8	71.8	36.1	1.85	Buckling of member

* given by Vulcraft.

** working load times 1.66.

*** same as KN-3A except failed member was replaced.

exposed for attaching test set-ups.

To assemble a test setup, the rafter and column sections were first bolted together. The rafter end of the specimen was then placed on the roller stand (Figure 4.2) and the column end pin connected to the section frame. Lateral brace mechanisms were then attached to all points that would be braced in an actual frame. The specimen was then "white washed" and instrumented. Finally, the connection bolts were tensioned to the required level.

Test Specimens. All specimens were fabricated by VULCRAFT at their Brigham City plant from angles and plates having a nominal yield stress of 50 ksi. Shop details of each of the specimens are found in Appendix D, Volume II. A325 bolts were used in all connections.

Instrumentation. Instrumentation consisted of strain gaged and calibrated bolts, displacement transducers, and plate separation calipers. Wire type displacement transducers were used to measure deformation of the specimen parallel to the line of action of the applied force and out-of-plane movement at selected locations. Two probe type (DVDT) transducers were used to measure deformation on each side of the knee area diagonal from which member force was calculated. Calipers were used to measure plate separations at the tension end of the connection plates. The locations were the same as for the knee connection tests as shown in Figure 3.3. The terms "centerline" and "edge" locations will again be used in the following test descriptions. Figure 4.3 shows the location of the instrumentation.

Testing Procedure. The testing procedure was similar to that used for the previously described tests. After the initial loading, load was applied in equal increments, approximately 10% of the estimated failure, until a softening of the load-deflection curve, which was being plotted in real time, was observed. The load increments were then decreased and loading continued until failure occurred. Failure in all tests was either member or specimen lateral buckling, or angle or plate local buckling.

4.3 Test Results and Comparisons

Test results consist of load versus a) displacement along the line of action of the applied load, b) lateral displacements, c) bolt forces, d) plate separations and, e) member forces. Comparisons are made to predicted displacements and member forces from results of a standard stiffness analysis computer program. AISC and Kennedy analyses were made for the connection flange plates and bolts. Individual member strength was determined from tables supplied by the research sponsor. The working load shown in Table 4.1 was calculated from the control-

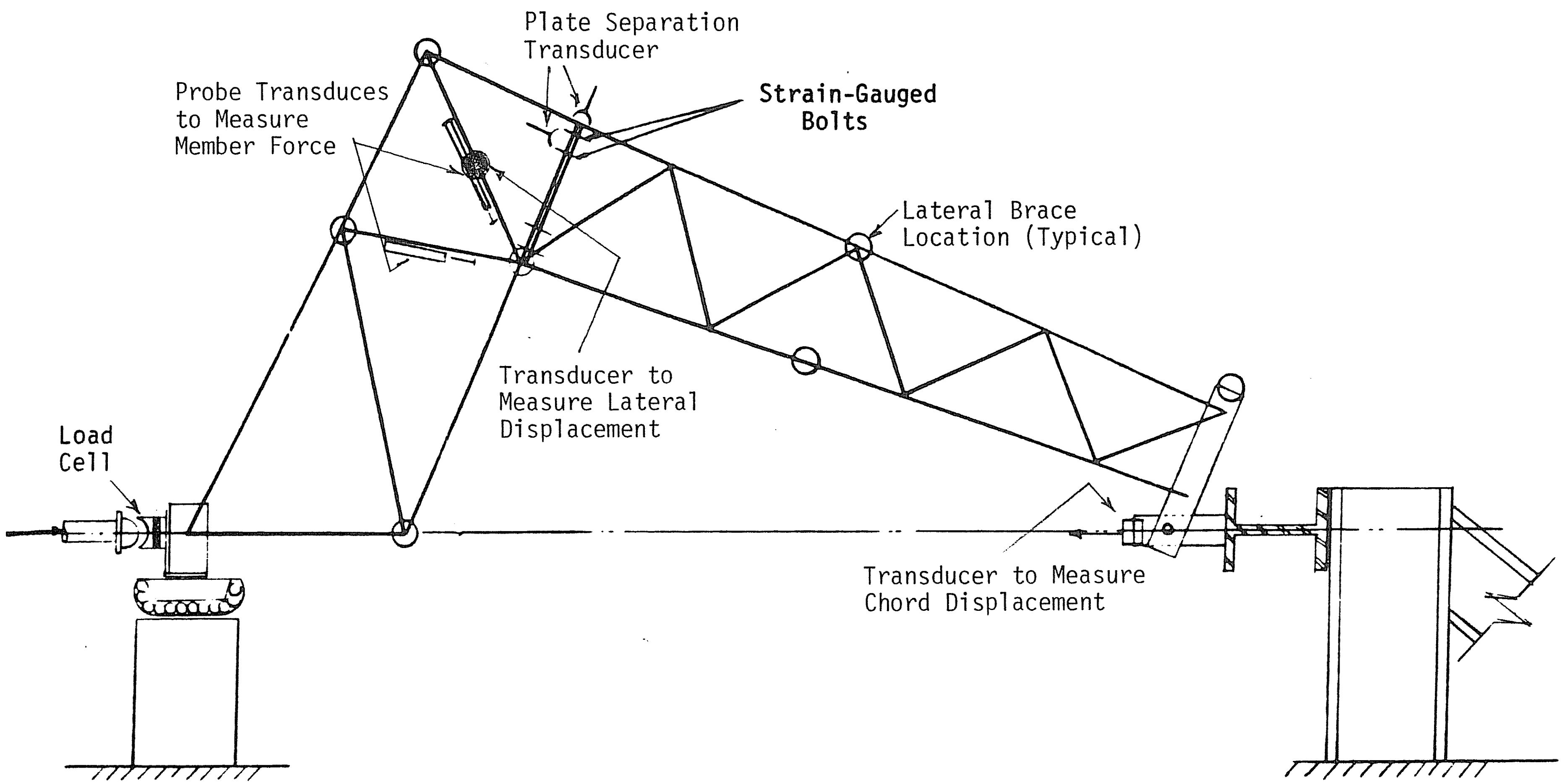


Figure 4.3 Typical Knee Area Test Instrumentation

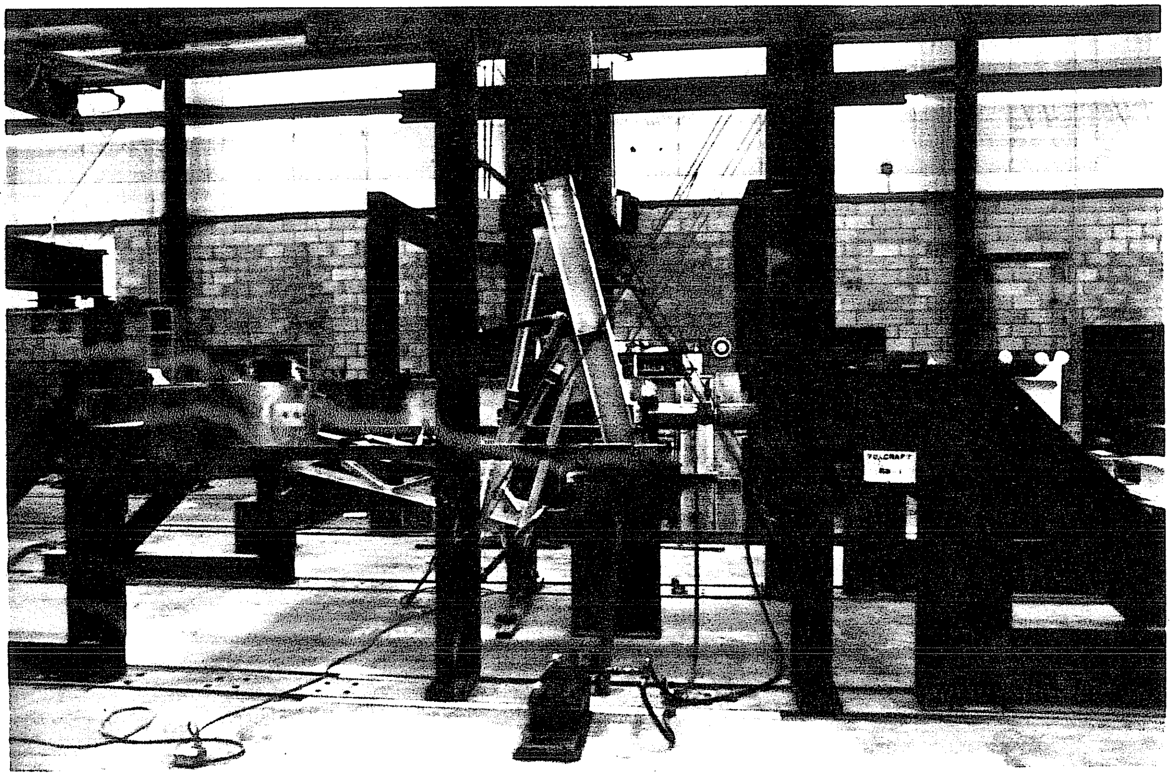
ling member capacity assuming a linear relationship between applied load and member force. The predicted ultimate load is taken as the working load times 1.66.

Detailed results for each test are found in Appendix D, Volume II. The test data "packages" contain a test summary sheet, specimen details, results of theoretical analyses, instrumentation drawing, and the various plots. The specimen details are summarized in Table 4.1 and the results in Table 4.2. A discussion of each test follows.

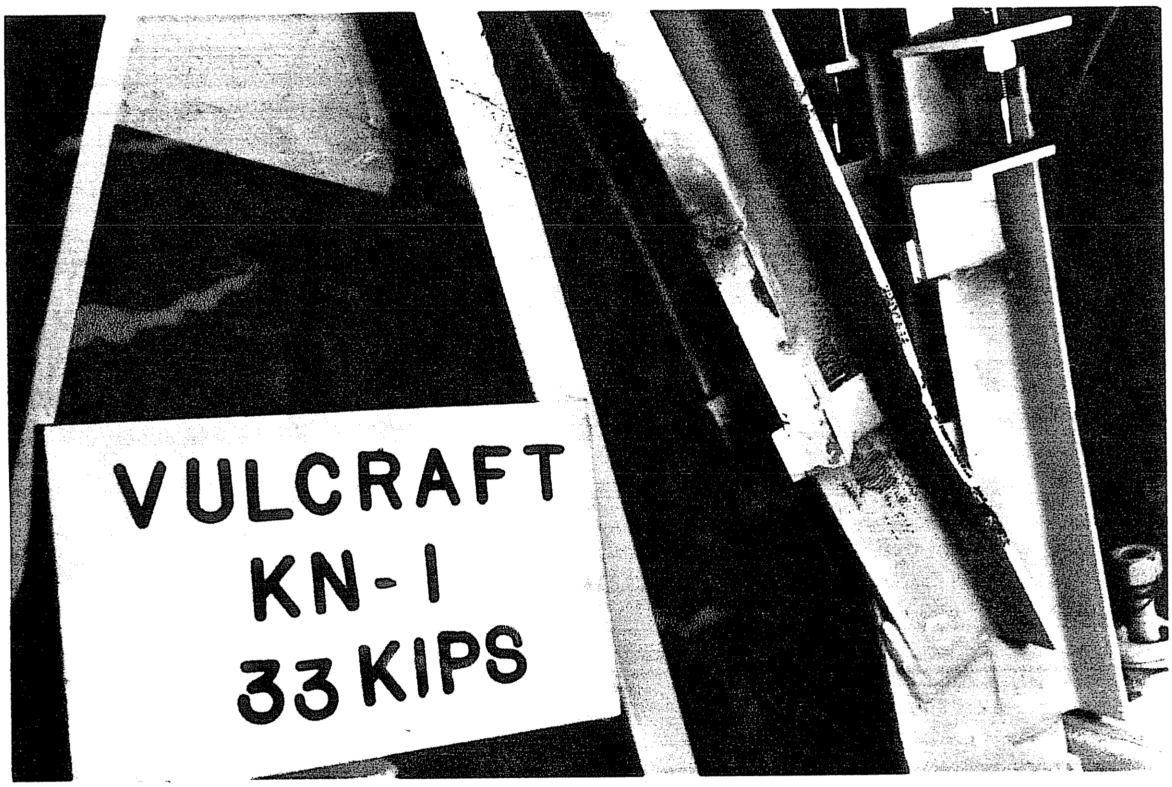
<u>Test KN-1</u>. The test summary sheet on page D.2 of Volume II describes this test in detail. Photographs of the test are found in Figure 4.4. Primary parameters of the specimens were 3/4 in. diameter bolts, 3½ in. bolt gage, 1/2 in. thick tee flange plates and a nominal 12 in. square knee area.

Failure occurred at 33.3 kips by local buckling at both ends of the first diagonal member, from the knee area, on the rafter section, member #33 in Figure D.1. See photograph in Figure 4.4(b). The predicted failure load was 34.26 kips (1.66 times the working load given by Vulcraft).

Figure D.2 shows theoretical (from the stiffness analysis, Figure D.3) and experimental displacements in the loading direction, e.g., chord deflections. The specimen was slightly stiffer than predicted until the applied load reached approximately 22 kips. Above this



a) Overview of Test Setup



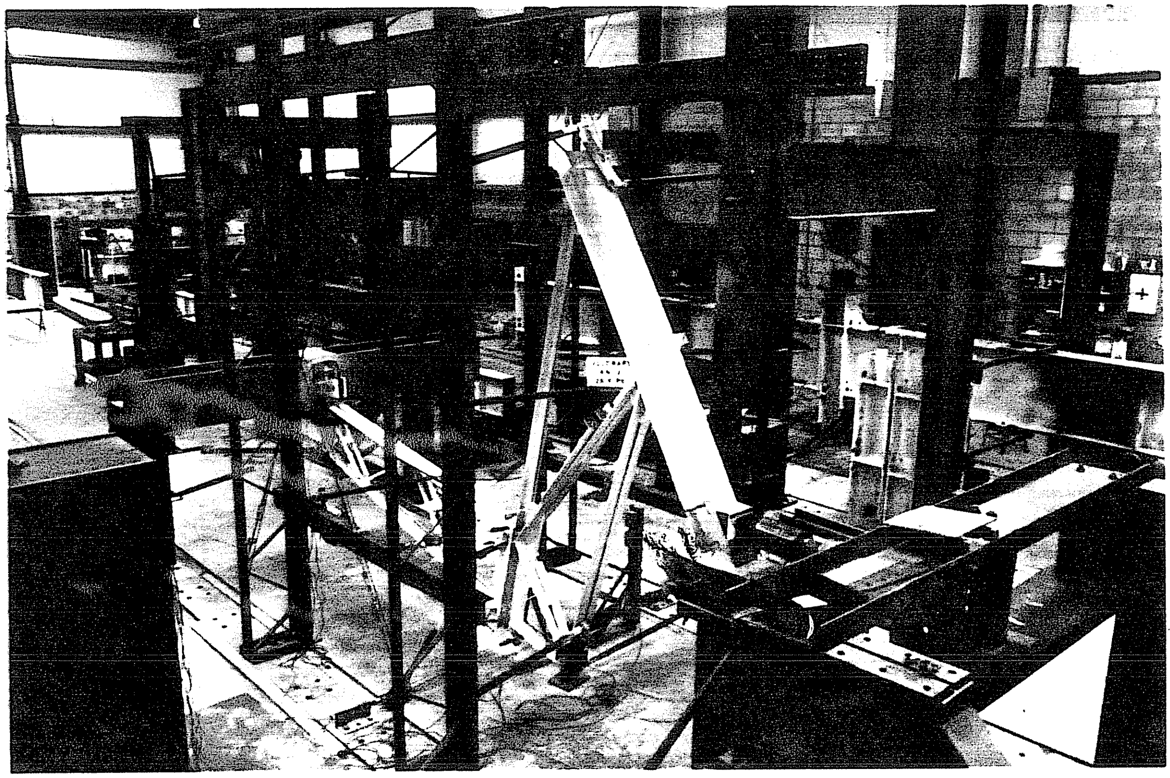
b) Local Buckle of Member #33

Figure 4.4 Photographs of Test KN-1

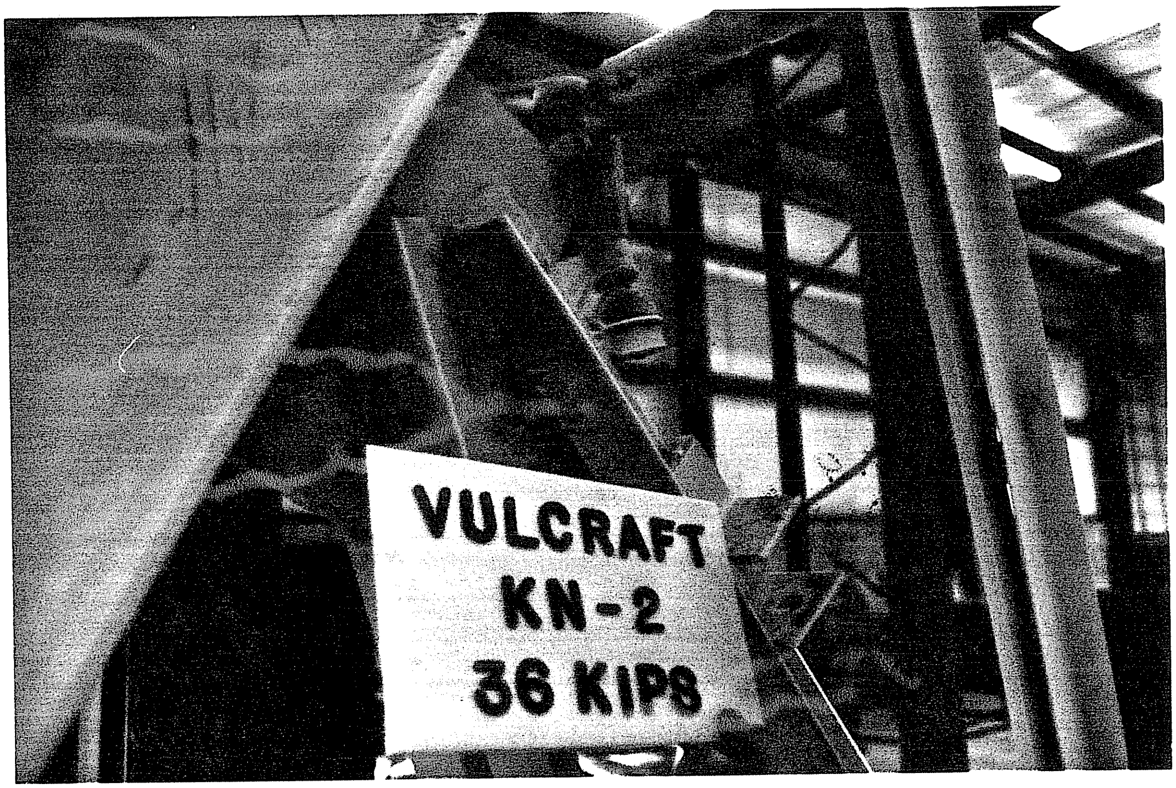
level, the load-deflection curve softened considerably. Figure D.7 shows the measured lateral displacement of the center of the knee diagonal. Movement was not detected until the 20 kips load level. From the shape of the curve it is possible the member buckled laterally which in turn caused the local buckling of the first rafter diagonal because of force redistribution. Figure D.8 shows that the bolt forces remained at the pretension level throughout the test. The plate separation curves in Figure D.9 are erratic, indicating possible instrumentation problems. Figure D.10 and D.11 are predicted and experimental plots of applied load versus member forces for members #33 and #18 (the rafter compression chord), respectively. The measured force in member #33 was greater, approximately 30%, than predicted. The measured force in member #18 was approximately 30% less than predicted.

Test KN-2. The data package for this test starts on page D.19, Volume II. The knee area of the specimen was nominally 24 in. square. The connection was made using 3/4 in. diameter bolts at 3½ gage and a 7 in. by 1/2 in. tee section flange plate.

Failure occurred at 36.0 kips due to buckling of the first rafter diagonal member, member #43 in Figure D.12, and rupture of the welds connecting this member to a gusset plate. Figure 4.5 shows the test set-up and the



a) Overview of Test Setup



b) Weld Failure at End of Member #43

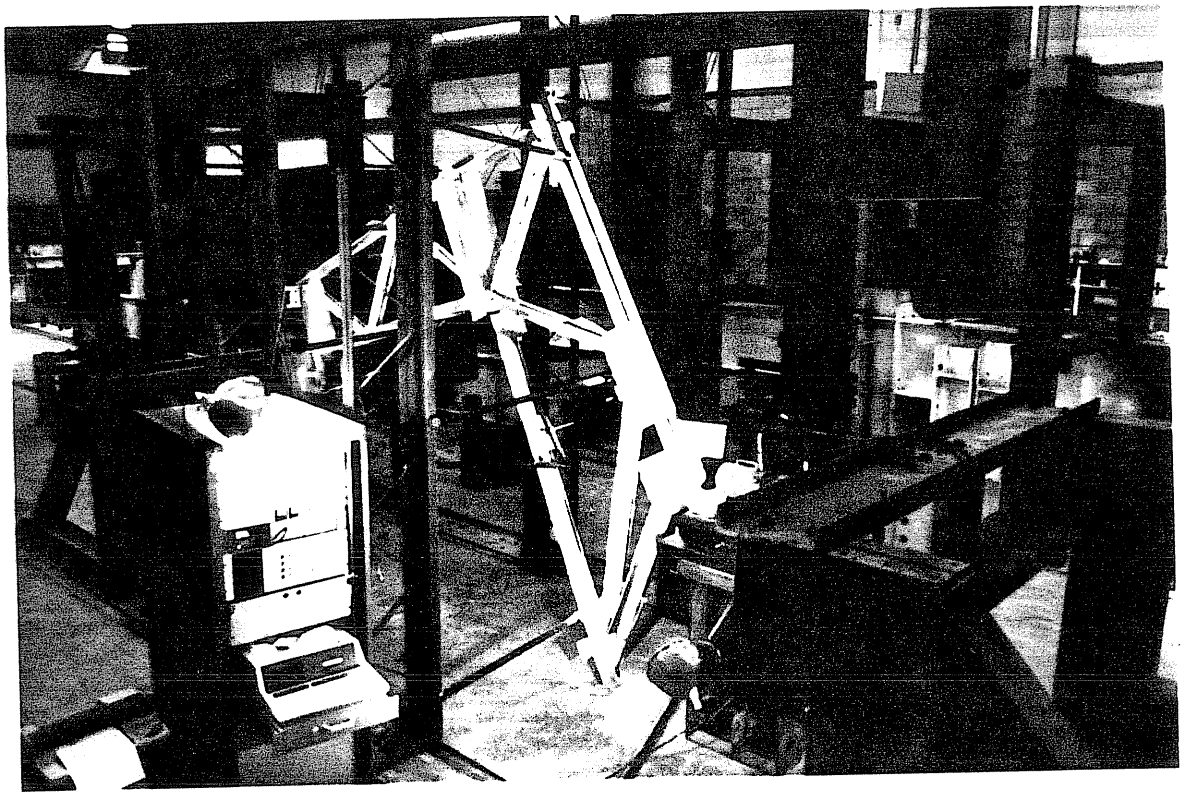
Fegure 4.5 Photographs of Test KN-2

location of the weld failure. The predicted failure load was 28.82 kips (1.66 times the working load given by Vulcraft).

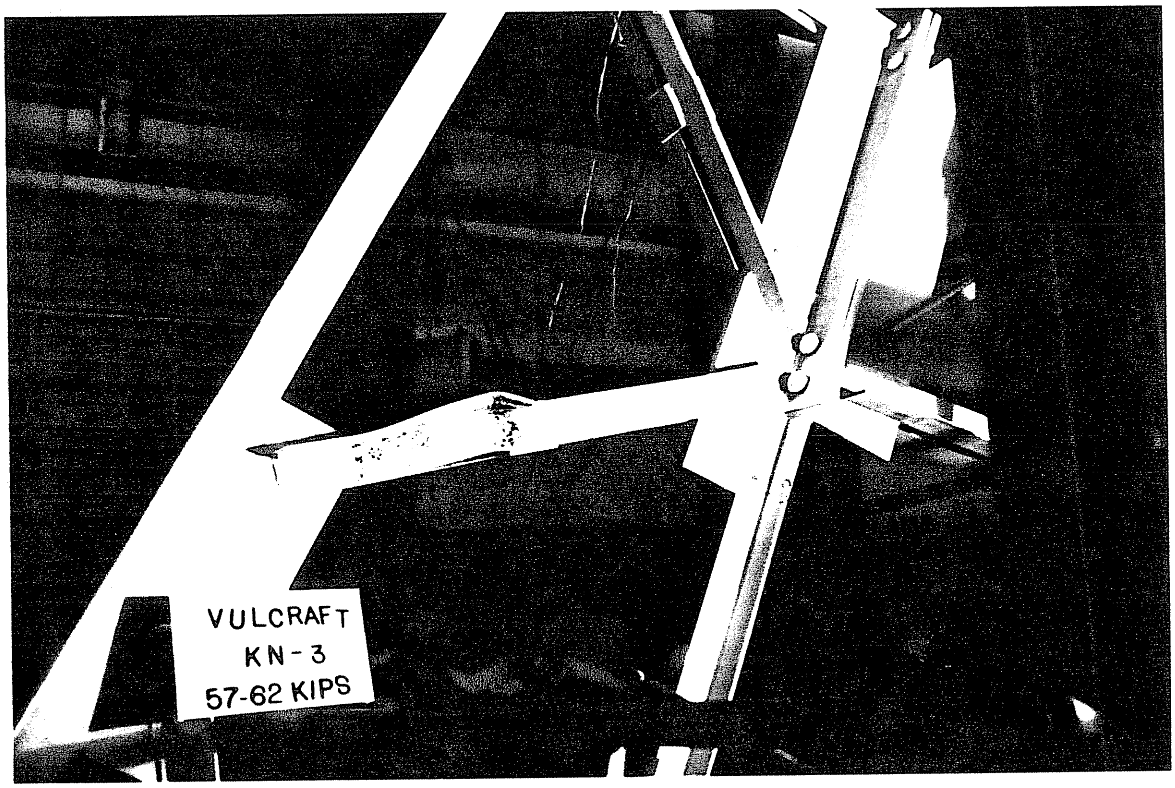
Figure D.17 shows excellent agreement between the predicted and measured chord displacements. Figure D.18 is a plot of load versus the measured lateral displacement of the web diagonal. Movement began to occur at 20 kips and increased rapidly until failure occurred. Bolt forces did not increase above the pretension level prior to failure, Figure D.19. The plate separation curves, Figure D.20, are erratic, again indicating instrumentation error. Excellent agreement was attained between predicted and measured forces in member #42, the knee diagonal, and member #43, the member which failed, as shown in Figures D.21 and D.22.

Test KN-3A. This test was the first of two test conducted using the KN-3 specimen. The nominal knee area for the specimen was 30 in. by 30 in., bolts were 3/4 in. diameter, bolt gage was 3½ in. and tee flange thickness was 3/4 in. The test summary sheet is page D.36 of Volume II. Failure occurred at 58.6 kips when the first diagonal member on the rafter buckled, member #11 in Figure D.23. The predicted failure load was 64.5 kips (1.66 times the working load given by Vulcraft). Figure 4.6 is photographs taken during the testing.

Excellent agreement was found between predicted and



a) Overview of Test Setup



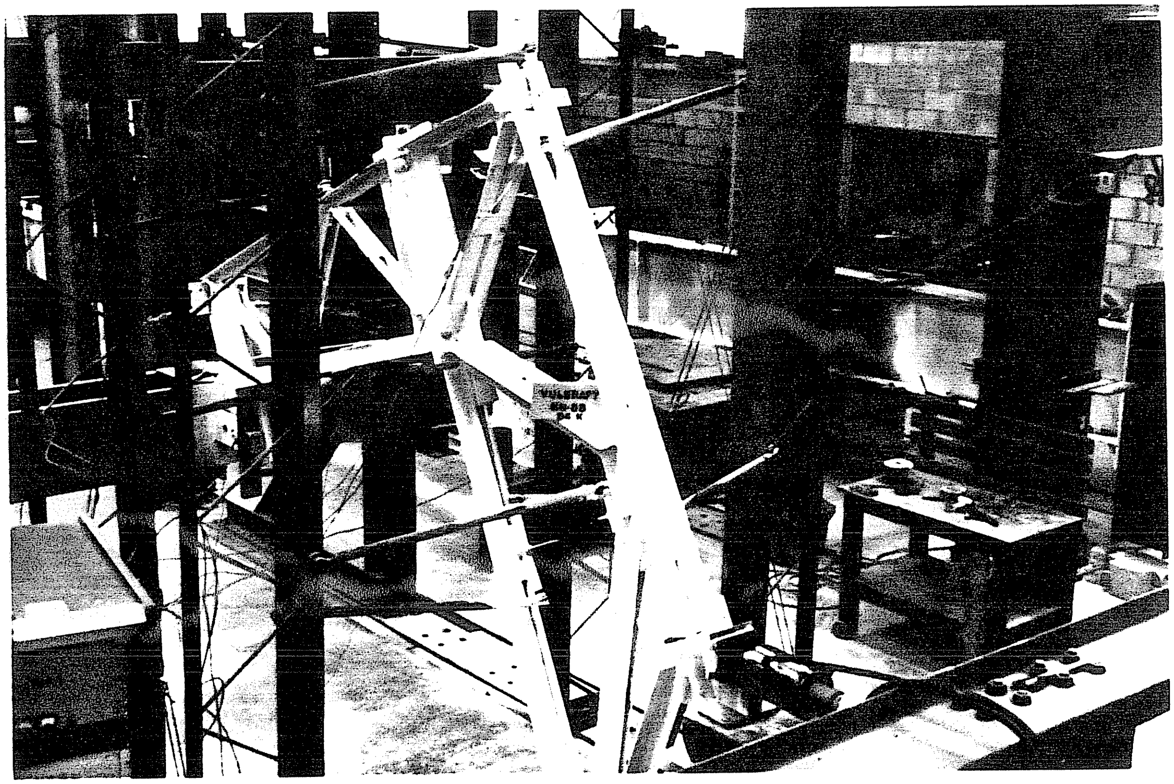
b) Failed Member #11

Figure 4.6 Photographs of Test KN-3A

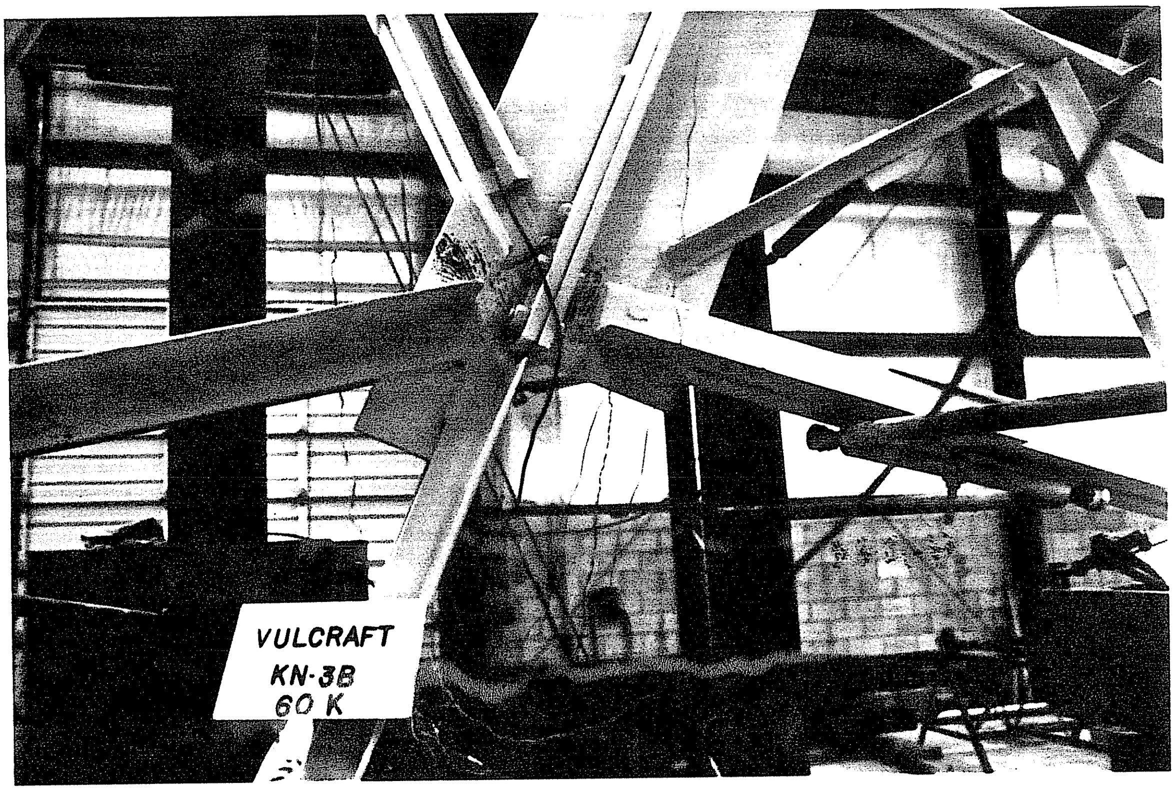
measured chord displacements, Figure D.28. As with the previous tests, bolt forces remained at the pretension level prior to failure, Figure D.29. The plate separation instrumentation failed during the test and plots were not made. Figure D.30 shows measured stresses, computed from strains measured with strain gages, for the knee diagonal.

Test KN-3B. This test is a repeat of Test KN-3A with the buckled rafter diagonal replaced with 2 L $3\frac{1}{2}$ x $3\frac{1}{2}$ x 1/4, A36 steel. Failure occurred at 60.1 kips when a plastic hinge formed in the compression region of the stem web plate. Photographs in Figure 4.7 show the test set-up and the failure. The predicted failure mode and load, 64.50 kips, were the same as for Test KN-3A. The test data packet is pages D.50 to D.66 of Volume II.

The measured load versus chord displacement curve for this test was considerably stiffer than the predicted curve, Figure D.36, possibly indicating strain hardening effects due to the first test. The lateral displacement plot in Figure D.37 clearly shows the load level at which buckling occurred. There was a slight increase above the pretension level in the force in one bolt, Figure D.38. The plate separation curves, Figure D. 39, were again erratic. Excellent agreement exist between the measured and predicted forces for member #9, the knee diagonal, as shown in Figure D.40.



a) Overview of Test Setup



b) Local Failure of Tee Section Stem

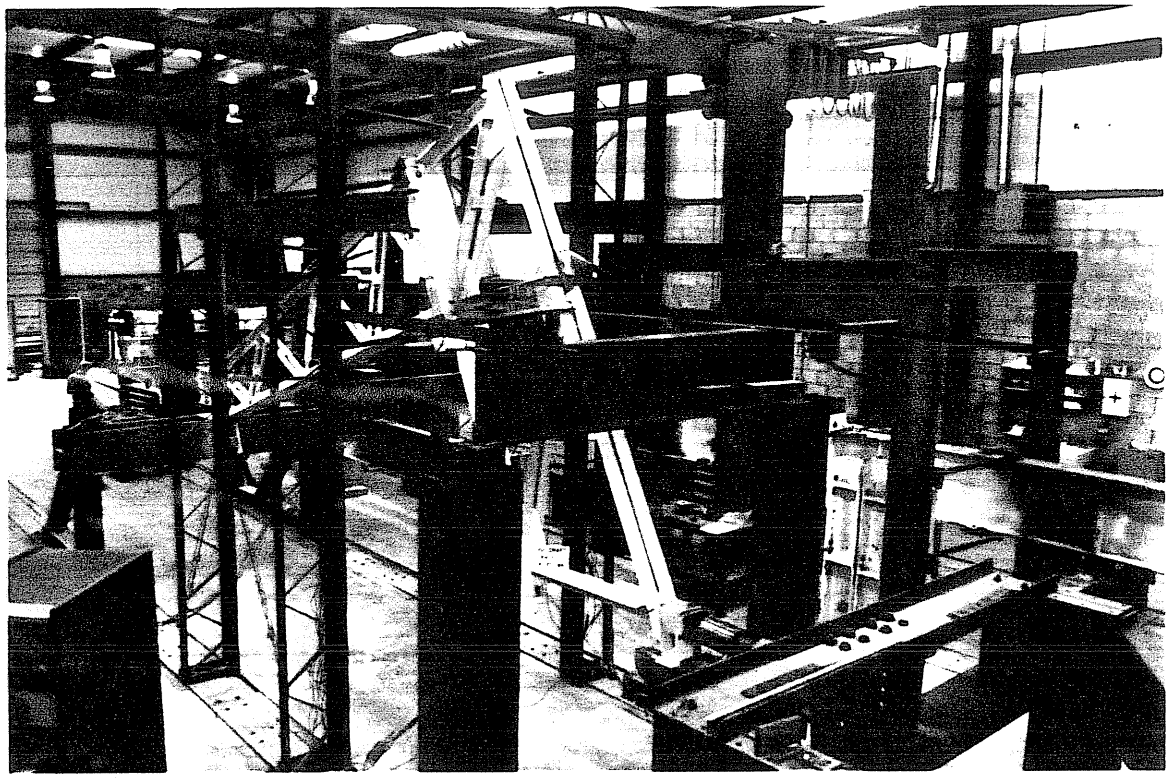
Figure 4.7 Photographs of Test KN-3B

Test KN-4. The nominal knee area for this test was 36 in. by 36 in., connection bolts were 1 in. diameter, tee flange thickness was 3/4 in. and the bolt gage was 4½ in. Details concerning the test are found on page D.67, Volume II. Photographs of the test are found in Figure 4.8.

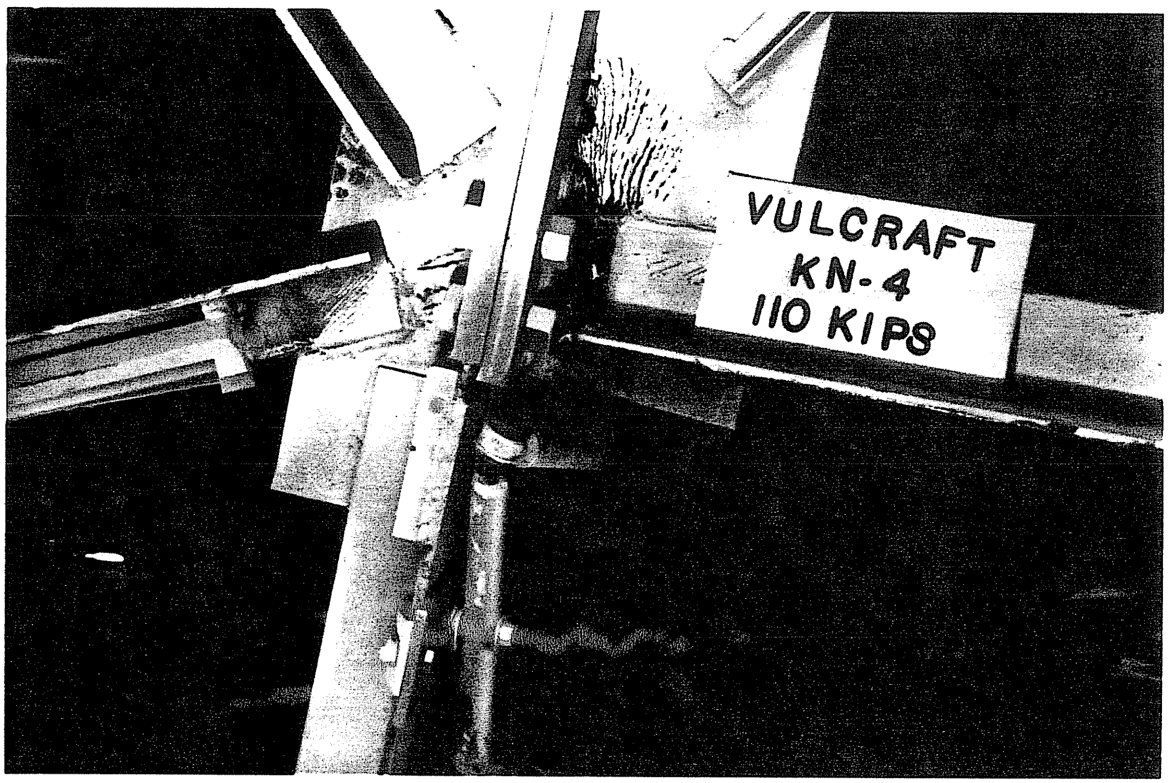
Failure occurred at 109.6 kips due to buckling of the stem plate on the rafter side of connection, Figure 4.8(b). The predicted failure load was 122.3 kips (1.66 times working load given by Vulcraft).

Figure D.46 shows fair agreement between the measured and predicted chord displacement curves. Figure D.47 shows that lateral movement started at the 50 kips level and approached 1 in. near the end of the test. Bolt forces increased above the pretension level from the start of the test. Maximum measured bolt force was 72.8 kips. As in the previous tests, the plate separation curves are erratic, Figure D.49. Excellent agreement exists between the measured and predicted forces in member #69, the knee area diagonal, Figure D.50. The measured force in the first rafter diagonal, member #70, was up to 60% greater than the predicted force, Figure D.51.

Test KN-5. The data pack for this test starts on page D.88 of Volume II. The nominal knee area was 36 in. by 44 in., connection bolts were 1 in. diameter at 4 in. gage. The tee flange was 1 in. thick.



a) Overview of Test Setup



b) Failure of Tee Stem Plate

Figure 4.8 Photographs of Test KN-4

Failure occurred at 80 kips by out-of-plane buckling of the first rafter diagonal. The predicted failure was 96.5 kips (1.66 times working load given by Vulcraft).

As shown in Figure D.57, the predicted chord displacements were approximately 15% more than the measured displacements. From Figure D.58, it is seen that lateral displacement at the center of the knee diagonal occurred from the beginning of the test. Bolt forces essentially remained at the pretension level, Figure D.59. Plate separation measurements were erratic, Figure D.60. Forces were measured in member 55, the knee area diagonal, and member #56, the rafter web member. The measured force in member #55 was less than the predicted force by 20-30%, Figure D.61. The opposite was true for member #56, Figure D.62.

Test KN-6. A specimen was fabricated for this test, but the test was not conducted.

Test KN-7. The specimen for this test was similar to that used for KN-3 except that the first rafter web member was placed in line with the column compression chord and the remaining web diagonals adjusted for this change. The knee area was a nominal 30 in. square, bolts were 3/4 in. diameter at 4 in. gage, and the tee flange was 3/4 in. thick. The test summary sheet is page D.108 of Volume II.

Failure occurred at 71.8 kips when the rafter web member in line with the column compression flange (member #12 in., Figure D.63) buckled. The predicted failure load was 64.5 kips (1.66 times working load given by Vulcraft). For comparison purposes, the actual and predicted failure loads for test KN-3B were 64.5 kips and 60.1 kips, respectively.

Figure D.68 shows fair agreement between predicted and measured chord displacements and clearly shows the effect of member buckling. From Figure D.69, it is found that lateral displacements were near zero until buckling occurred. Bolt forces increased only slightly prior to failure, Figure D.70. Again the measured plate separations were erratic, Figure D.71. Figure D.72 shows good agreement between the measured and predicted forces in member #11, the knee area diagonal. Figure D.73 shows that the measured force was less than the predicted force by 10-15% for member #12. The failure of this member is clearly shown in Figure D.73.

4.4 Conclusions

Test results and comparisons with predicted values are summarized in Table 4.2. The working load for the knee area specimen, calculated from the most highly stressed member, as well as the factored (1.66) or ultimate capacity of the knee area are shown. Also shown are the allowable AISC tension load for the bolts and the ultimate bolt capacity, two times the allowable load. Two experimental loads are listed: the maximum applied load and the maximum measured bolt force. Also shown for each test are the ratio of maximum applied load-to-working load and the failure mode.

The range of the load ratio was 1.38 for Test KN-5 to 2.07 for Test KN-2; the average value was 1.64. However, in only two tests, KN-2 and KN-7 was an acceptable factor of safety, 1.66, obtained.

Except for Test KN-4, the calculated bolt force from measured strains, assuming elastic material properties, did not exceed the ultimate bolt force based on factored AISC tension allowable forces.

In all tests, except KN-3A, the failure mode was associated with instability in the knee area. Comparison of the results from Tests KN-3B and KN-7, which were similar except for the configuration of the knee area and rafter web members, shows that the configuration used in Test KN-7 results in a stronger knee area. The reason for the improved results is that less eccentricity exists at the reentrant corner in the arrangement used for Test KN-7.

CHAPTER V

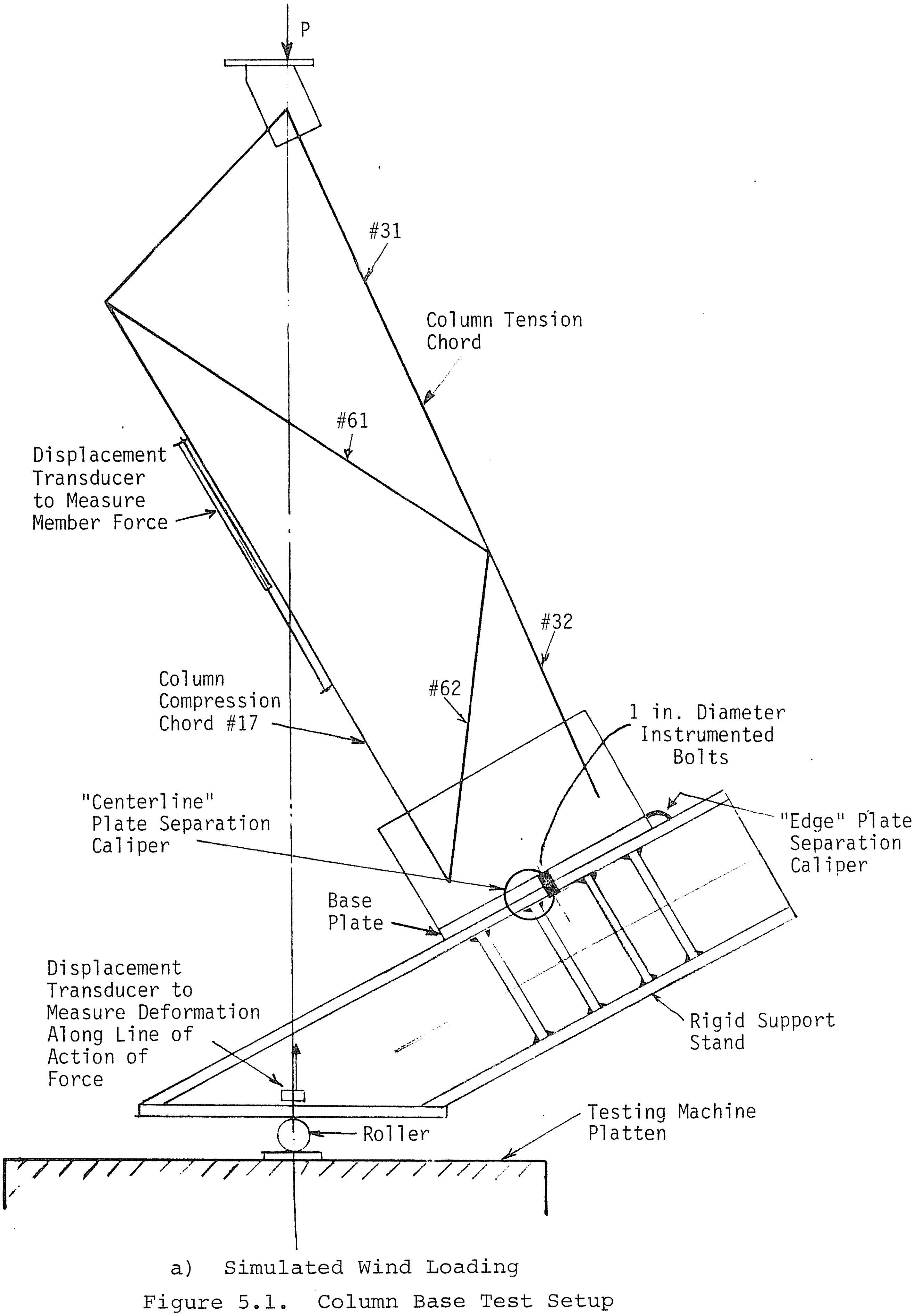
COLUMN BASE TESTS

5.1 General

The objective of the column base test was to study the behavior of a typical column base area when subjected to both wind and gravity loadings. Four tests were conducted: two were for simulated wind loading and two for simulated gravity loading. Column base connection configurations using two and four bolts were used for each loading. One test was repeated with a change in bolt pretensioning.

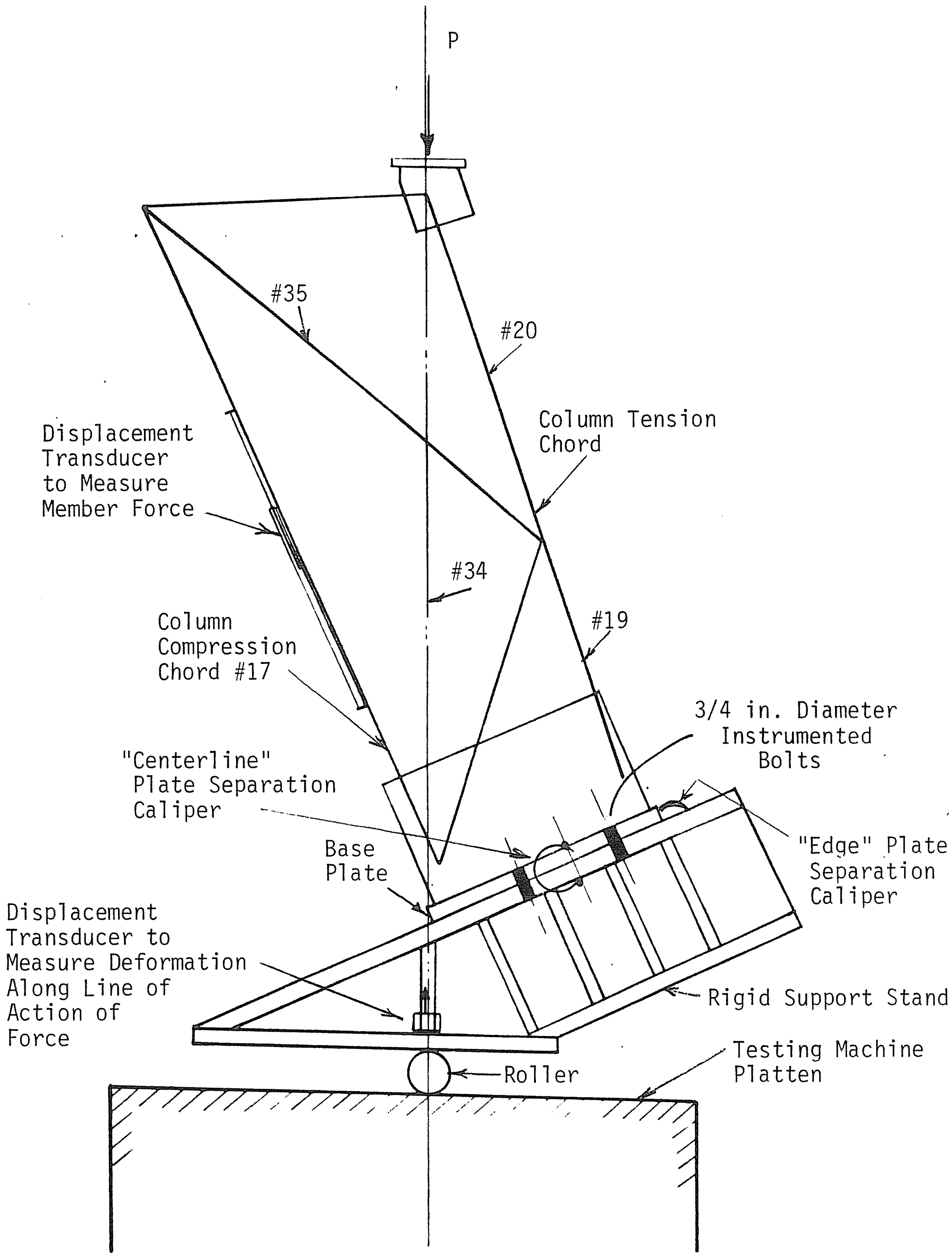
The tests were conducted using a universal testing machine. Figures 5.1 and 5.2 show overviews of the test setup and instrumentation used. The line of action of the force P was moved to simulate gravity and wind loadings. The resulting forces in the column members were nearly identical in magnitude and direction to those found in a typical 60 ft. span rigid frame.

Table 5.1 lists the test parameters and summarizes the results. Specimen details and test data are found in Appendix E of Volume II.



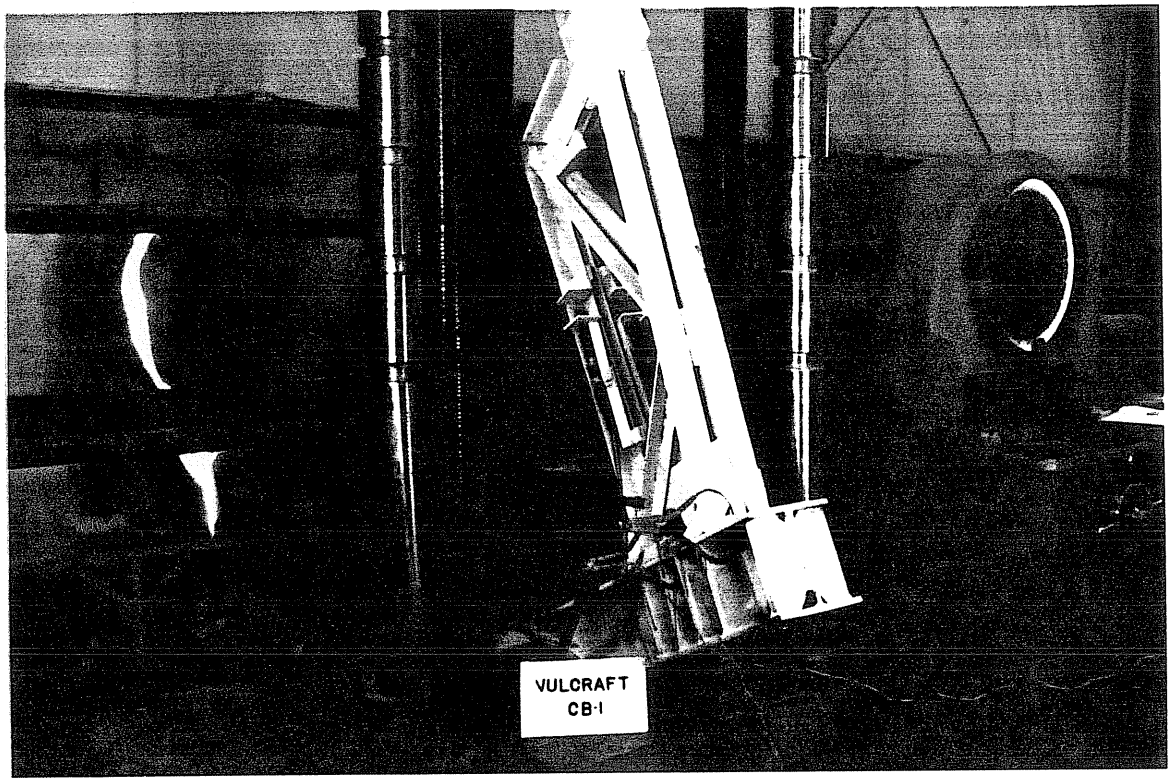
a) Simulated Wind Loading

Figure 5.1. Column Base Test Setup

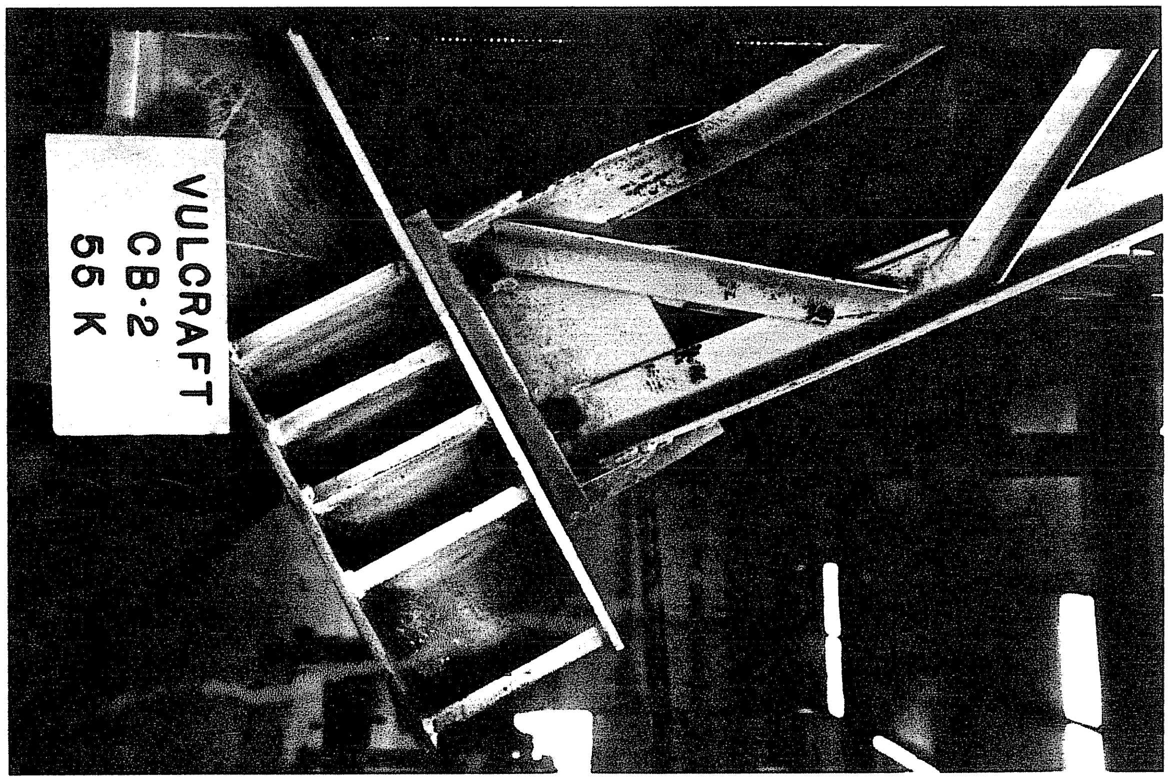


b) Simulated Gravity Loading

Figure 5.1. Column Base Test Setup, Continued



a) Four Bolt Wind Loading Test



b) Two Bolt Gravity Loading Test

Figure 5.2 Photographs of Column Base Tests

Table 5.1

Column Base Specimen Details and Test Results

Test No.	No. of Bolts	Bolt Diameter (in.)	Load Eccent. (in.)	Working Load (k)	Ultimate Load (k)**	Experimental Loads		Max. Plate Sep. (in.)	Applied Load / Working Load	Failure Mode
						Max. applied Load (k)	Max. Bolt Force (k)			
CB-1A	2	1	10.59	17.66	29.31	35	52.90	0.250	1.98	Excessive plate Separation
CB-1B*	2	1	10.59	17.66	29.31	37.5	37.80	0.063	2.12	Bolt Yielding
CB-2	4	3/4	10.59	17.66	29.31	55	28.90	0.123	3.11	Member Buckling
CB-3	2	1	1.44	27.86	46.24	89	13.64	0.070	3.19	Member Buckling
CB-4	4	3/4	1.44	27.86	46.24	83	7.50	0.020	2.98	Member Buckling

*The same as CB-1A, except anchor bolts were pretensioned to 90% capacity.

**Ultimate Load = Working Load x 1.66

5.2 Testing Details

Test Setup. The test setup consisted of a short section of typical column base and supported by a rigid stand to simulate a concrete footing. Two stands were used so that the line of action of the applied force could be varied to simulate the two loading conditions. The column section and stand were bolted together and placed in a 200 kips capacity universal testing machine. A rocker was placed between the platten of the testing machine and the stand so that the line of action of the applied force would be known.

Test Specimens. All specimens were fabricated from angles and plates having a nominal yield stress of 50 ksi. The base plate for all test specimens was 8 in. by 12 in. by 1 in. thick. One inch diameter bolts were used for the two bolt tests and 3/4 in. diameter bolts were used for the four bolt tests. All anchor bolt material was A36 steel.

Instrumentation. Instrumentation consisted of strain gaged and calibrated bolts, displacement transducers, and plate separation calipers. A wire displacement transducer was used to measure deformation of the specimen parallel to the line of action of the applied force. Two probe type (DVDT) transducers were used to measure deformation on each side of one member from which member force was calculated. Calipers were used to measure plate separation at the centerline and edge of the base plates. The "centerline" location was as near to the intersection of the two base plate centerlines as possible. The "edge" location was on the longitudinal centerline of the base plate at the tension side edge. Figure 5.1 shows the location of the instrumentation.

Testing Procedure. The testing procedure was similar to that used for the previously described tests. The load was applied in equal increments until a softening of the load-deflection curve, which was being plotted in real time, was observed, whereupon, the load increments were decreased until failure occurred. Failure was defined as either member buckling and a corresponding decrease in load or excessive separation of the base plate from the support stand.

5.3 Test Results and Comparisons

Test results consist of load versus displacement along the line of action of the applied load, load versus anchor bolt forces, load versus base plate/stand separation and load versus member force. Comparisons are made to predicted displacements, member forces, and bolt forces from results of a standard stiffness analysis computer program. Prying action considerations are neglected for the predicted bolt forces. Member strength was determined from tables supplied by the research sponsor.

The working load shown in Table 5.1 was calculated from the controlling member capacity assuming a linear relationship between applied load and member force. The predicted ultimate load is taken as the working load times 1.66.

Test CB-1A. The purpose of this test was to study the behavior of the two anchor bolt column base configuration when subjected to wind loading. The simulated anchor bolts were 1 in. diameter by 4 in. long, A36 steel studs. Bolt gage was 4 in. The bolts were snug tightened using a standard "spud" wrench prior to the beginning of the test.

The maximum load applied in the test was 35 kips or 1.98 times the predicted working load. The test was stopped because of excessive plate separation and resulting end rotation. The test summary sheet on page E.2 of Volume II describes the test in detail.

Figure E.5 shows the predicted and experimental load versus displacement (along the line of action of the applied load) curves. As can be seen, no agreement exists between the theoretical and experimental curves because of the relatively large plate separations as shown in Figure E.7. The maximum plate separation was 0.25 in.

Bolt force versus applied load relationships are shown in Figure E.6. At the maximum applied load, 35 kips, the maximum observed bolt force was 52.9 kips.

Figure E.8 shows the measured and predicted forces for member #17, the inside (compression) column chord. Again poor agreement exists between the experimental and predicted curves.

Test CB-1B. This test was identical to test CB-1A (including the column base and anchor bolt studs) except that the anchor bolts were pretensioned to 90% of the nominal yield strength of the unthreaded shank, approximately 25 kips.

The maximum load applied in the test was 37.5 kips or 2.12 times the predicted working load. The test was stopped due to yielding of the anchor bolt studs. The test summary sheet is found on page E.14 of Volume II.

Figure E.13 shows predicted and experimental displacements in the load direction. Better agreement exists between the two curves. The maximum measured displacement was 0.221 in.

The anchor bolt forces versus applied load relationships are shown in Figure E.14. An extremely sharp increase in bolt forces occurred when the applied load reached approximately 35 kips.

Maximum plate separation at the edge location was 0.063 in. as shown in Figure E.15. The separation at the centerline location was significantly less.

The relationship between predicted and measured member #17 force is shown in Figure E.16. Correlation is poor.

Test CB-2. The purpose of this test was to study the behavior of a four bolt column base subjected to wind load. Anchor bolts were 3/4 in. diameter by 3 in. long, A36 steel, threaded studs. The bolt gage was 4 in. and the studs were snug tightened prior to testing. Failure occurred at an applied load of 55 kips by buckling of the compression column chord, member #17. The resulting failure load-to-working load ratio is 3.11. The test summary sheet is found on page E.26 of Volume II.

Figure E.21 shows predicted and experimental displacement in the load direction. Again, agreement is not good. The maximum measured displacement was 0.533 in. under maximum applied load.

Maximum measured bolt force was 28.9 kips. As shown in Figure E. 22, bolt force increased almost immediately with application of simulated wind loading.

Maximum plate separation occurred at the edge location and was 0.123 in. As shown in Figure E.23, plate separations at the edge location were approximately 60% greater than at the centerline location.

Figure E.24 shows predicted and measured force in member #17. Correlation is better than in the previous tests.

Test CB-3. The purpose of this test was to study the behavior of a column base under gravity load and with two anchor bolts. Anchor bolts were studs, 1 in. in diameter by 4 in. in length, A36 steel. Bolt gage was 4 in. The studs were hand tightened prior to the test. Failure occurred at 89 kips by buckling of a diagonal member (#34) or 3.19 times the predicted working load. The test summary sheet on page E.38 of Volume II describes the test details.

Figure E.29 shows predicted and experimental displacements in the load direction. Maximum displacement was 0.345 kips at 89 kips of applied load. Correlation between the curves is excellent (indicating that plate separation was affecting these relationships in the mind loading tests).

As shown in Figure E.30, the anchor bolt studs were subjected to tension load during the test. The maximum observed bolt force was 13.64 kips.

As shown in Figure E.31, plate separations at the edge locations were approximately 55% greater than at the mid depth location. Maximum separation was 0.07 in.

Predicted and measured forces in member #17 are shown in Figure E.32. Correlation is poor.

Test CB-4. The purpose of this test was to study the behavior of a column base with four anchor bolts when subjected to gravity loading. The anchor bolts were 3/4 in. diameter threaded studs, 3 in. long, A36 steel. The anchor bolt gage was 4 in. The anchor bolts were snug tightened prior to the test. Failure occurred at an applied load of 83 kips by buckling of a diagonal member. The ratio of failure load to working load is 2.98. The test summary sheet is found on page E.50 of Volume II.

Predicted and experimental displacements along the line of action of the applied force are shown in Figure E.37. Correlation is very good. The maximum measured displacement was 0.215 in.

Figure E.38 shows load versus tension bolt force relationships. The maximum measured bolt force was 7.5 kips.

Maximum plate separation was at the edge location, 0.02 in. As shown in Figure E.39, plate separation at the centerline location was close to zero.

Figure E.40 shows predicted and experimental member force is the column compression chord, member #17. The measured forces were greater than the predicted forces until buckling occurred.

5.4 Conclusions

Wind Loading. From the experimental load versus displacement along the line of action of the applied load and the load versus plate separation curves, it is obvious that the column base configurations tested do not provide sufficient stiffness to satisfy the end conditions assumed in the stiffness model unless the anchor bolts are pretensioned. The most flexible connection was Test CB-1A where the measured deformation exceeded the predicted deformation by 2-3 times (Figure E.5) and the plate separation reached 0.25 in. (Figure E.7). Pretensioning of the anchor bolt studs to 90% of their yield strength resulted in a significantly stiffer connection and the line of action of load deformation is very close to the predicted deformation (Figure E.13). Use of four anchor bolts stiffened the connection relative to the two bolt connection, but the correlation with the predicted stiffness (Figure E.21) was not as good as for the two bolt with pretensioning test. Plate separation exceeded 0.12 in.

In all three tests, the bolt forces increased with increasing load and the maximum bolt force exceeded the yield strength of the bolt based on the unthreaded area, 15.9 kips for 3/4 in. diameter and 28.3 kips for 1 in. diameter bolts (see Table 5.1). From Figure E.14, it is seen that the bolt forces did not increase above the pretension level, Test CB-1B, until the applied load exceeded 20 kips. Correlation between measured and predicted column compression chord forces (member #17) was not good in any test. It is possible that the instrumentation technique was the major cause of the discrepancy.

Failure for the two bolt test without pretension (CB-1A) was defined as excessive plate separation. For the two bolt test with pretension (CB-1B), failure was defined as excessive bolt force. A diagonal web member buckled in the four bolt test. The corresponding maximum load-to-working load ratios were 1.98, 2.12 and 3.11. Although, sufficient strength was found in all tests (ratios above 1.66), it is recommended that either the pretensioned two bolt or the four bolt configuration be used in actual construction.

Gravity Loading. For the two gravity load tests, correlation between predicted and measured deflections along the line of action of the applied load is excellent, within 10%, until near the failure load (Figures E.29 and E.37). Base plate/stand separations were small in both tests, less than 0.1 in. Bolt forces did change with applied load (Figures E.30 and E.38), but did not exceed the yield strength of the bolts at maximum load.

Correlation between predicted and measured column compression chord forces is again poor and is again thought to be caused by a poor instrumentation technique.

Failure in both tests was by lateral buckling of a web diagonal member. The maximum applied load-to-working load ration for the two bolt (CB-3) and four bolt (CB-4) tests were 3.19 and 2.98, respectively. From the test results, it is apparent that the number of anchor bolts has little effect on column base performance under gravity loading.

REFERENCES

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5. Yarimci, E., Yura, J.A., and Lu, L.W., "Techniques for Testing Structures Permitted to Sway," Experimental Mechanics, Vol. 7, No. 8, August 1967.

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APPENDIX A

TEE-HANGER ANALYSIS MODELS

APPENDIX A

TEE-HANGER ANALYSIS MODELS

A.1 INTRODUCTION

A number of mathematical models are available to predict the behavior of tee-hangers. Two of these models include procedures to predict bolt forces as a function of applied load and were selected for use in the research reported here.

The first method is based on the work of Struik and deBack[4] as reported in Fisher and Struik[1]. The procedure is also found in the 8th edition AISC Manual of Steel Construction[3] and for that reason is referred to herein as the "AISC method". The second method was developed by Kennedy, Vinnakota and Sherbourne[2] and will be referred to as the "Kennedy method". The two methods are very similar, differing primarily in that the Kennedy method includes the effect of shear on the flange plate plastic moment capacity and directly considers bolt bending. Both methods require iteration to obtain the optimal solution. Details of each method are now discussed.

A.2 AISC METHOD FOR TEE-HANGER DESIGN

The following assumptions are made in the development of the AISC/Struik and deBack model:

1. A resultant prying force acts at the tip of the flange as shown in Figure A.1. It is noted in Reference 1 that if the distance "a" does not exceed 1.25b, this assumption is reasonably accurate.
2. The line of action of the bolt force acts at the inner edge of the bolt hole. This assumption accounts for bending of the bolts which is the cause of the shifting of the bolt force resultant toward the tee-hanger stem.

Failure in this model is defined as (a) development of a mechanism in the flange plate or, (b) rupture of the bolts. The allowable stress design procedure in the AISC Manual includes a factor of safety of 2.0 against both failure modes.

The design equation, as given in the AISC Manual, for bolt force including prying action but exclusive of initial tightening is

$$B_c = T\left[1 + \frac{\delta\alpha}{(1+\delta\alpha)} \quad \frac{b'}{a'}\right] \tag{A.1}$$

and the equation for required plate thickness

$$t_f = \left[\frac{8B_c\ a'b'}{p\ F_y\ [a' + \delta\alpha(a'+b')]}\right]^{\frac{1}{2}} \tag{A.2}$$

where B_c = load per bolt including prying action; T = applied tension per bolt; p = length of flange, parallel to stem, tributory to each bolt; a = distance from bolt centerline to edge of tee flange but not more than 1.25b; b = distance from bolt centerline to the face of the tee stem;

$a' = a + d/2$; $b' = b - d/2$; d = bolt diameter;

and

$$\delta = 1 - d'/p \tag{A.3}$$

$$\delta = \frac{8Tb'}{p\ t_f^2\ F_y} - 1\ /\delta \qquad 0 \leq \delta \leq 1 \tag{A.4}$$

where d' = diameter of bolt hole. If desired, the prying force can be calculated from

$$Q = B_c - T \tag{A.5}$$

The AISC rules require that

$$B_c \leq B \tag{A.6}$$

where B = the allowable tension force in the bolt.

The above equations can be used to determine B_c and t_f from the applied tension T, material properties, and the geometry of the tee-hanger or the capacity of the tee-hanger can be obtained by specifying the material and geometric properties. The former calculation is iterative since B_c and t_f are interrelated thru α. An infinite number of solutions can be found which satisfy Equations A.1 and A.2; the optimal solution can be found by trial and error. A non-iterative procedure is suggested by A. Astaneh[5]. The procedure uses the same equation's as AISC method but obtains the final results directly without iterations.

A.3 KENNEDY METHOD FOR TEE-HANGER ANALYSIS AND DESIGN

Kennedy *et al.*,[2] have also proposed a method for the calculation of the capacity of tee-hangers. As with the AISC method, both bolt and plate and plate failure modes are considered. The major differences between the methods are that the Kennedy procedure includes shear yielding of the flange plate and directly includes bolt bending effects.

In the Kennedy method, depending on the geometry of the tee-hanger, the flange plate behaves as a "thick plate" or "intermediate plate" or a "thin plate". For thick plate behavior, the applied load is not sufficient to cause full yielding of the flange plate and it is assumed that no prying forces exist. In the intermediate case, the load is sufficient to cause the formation of a plastic hinge line in the flange plate adjacent to the web prior to or simultaneously with rupture of the bolt. For this case, a prying force is assumed to act at the tip of the flange and its magnitude is calculated from geometric and applied load considerations. For thin plate behavior, the ultimate condition for connections without bolt failure, two plastic hinges form in the flange plate, one adjacent to the web and one through the bolt line. For this case, a prying force is assumed to act at the flange plate tip.

It is emphasized, that the thick, intermediate or thin plate behavior is not directly related to actual plate thickness, but is a function of plate thickness, connection geometry and applied load. Assuming that the bolts do not rupture prior to the thin plate behavior limit, any tee-hanger can exhibit all three types of behavior.

By the Kennedy method, the ultimate (unfactored) capacity per bolt, T_{u1}, of a tee-hanger connection is given by the smallest of T_1, T_2 and T_3 where, using the AISC nomenclature,

$$T_1 = (M_B + C_1 M_{p_1} + C_2 M_{p_2})/b \tag{A.7}$$

with

$$C_1 = \sqrt{1 - 3(T_1/pt_f F_y)^2} \tag{A.8a}$$

$$C_2 = \sqrt{1 - 3(T_1/p' t_f F_y)^2} \tag{A.8b}$$

$$M_{p_1} = (p t_f^2/4) F_y \tag{A.8c}$$

$$M_{p_2} = (p' t_f^2/4) F_y \tag{A.8d}$$

$$p' = p - d' \tag{A.8e}$$

See Figure A.2 for definition of terms. And

$$T_2 = \frac{\sqrt{M_{p_1}^2 - (3/16)(T_2 t_f)^2} + M_B + B_u a}{a + b} \tag{A.9}$$

with $a = t_f$ if $t_f/d < 2/3$ and $a = 2t_f$ otherwise, b = distance from the centerline of the bolt hole to the face of the stem, M_B is bolt bending strength given by

$$M_B = F_{yb}\ (\pi d^3/32) \tag{A.10}$$

and B_u = ultimate bolt capacity = 2B

and

$$T_3 = B_u \tag{A.11}$$

It is noted that iteration is required to satisfy equations A.7 and A.9 and that the shear strength of the flange plate cannot be exceeded in <u>any</u> iteration, that is

$$T_1 < (p'\ t_f\ F_y)/\sqrt{3} \tag{A.12a}$$

and

$$T_2 < (p'\ t_f\ F_y)/\sqrt{3} \tag{A.12b}$$

The ultimate capacity of a four bolt tee-hanger connection is then

$$T_u = 4\ T_{u1} \tag{A.13}$$

where T_{u1} is the minimum of T_1, T_2 and T_3.

A.4 COMPUTER PROGRAM LISTINGS

A computer program was written for the IBM PC computer. The program is limited to analysis and basically provides (1) the service load capacity of a four bolt tee-hanger based on the AISC procedure and, (2) the ultimate load capacity based on the Kennedy procedure.

The program listing follows. An example output sheet is also included.

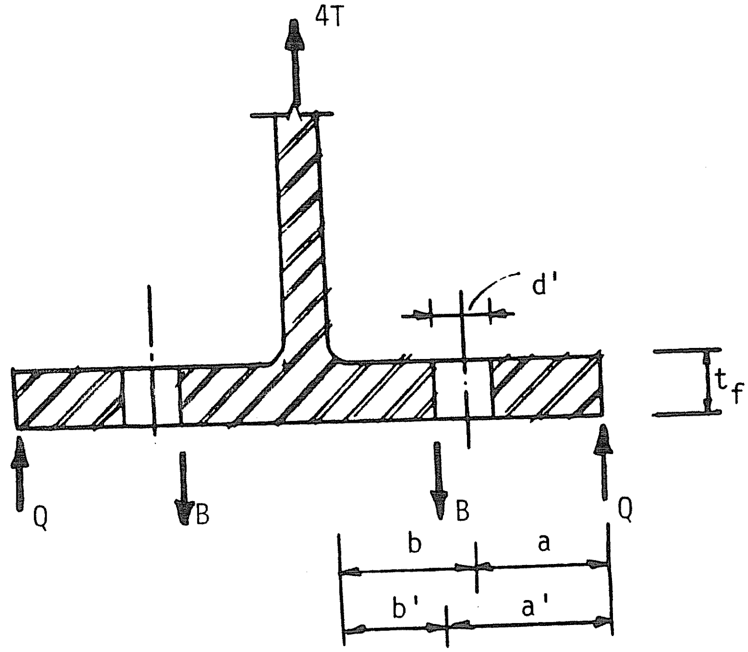


Figure 1.A AISC Model of Tee Hanger (Reference 1)

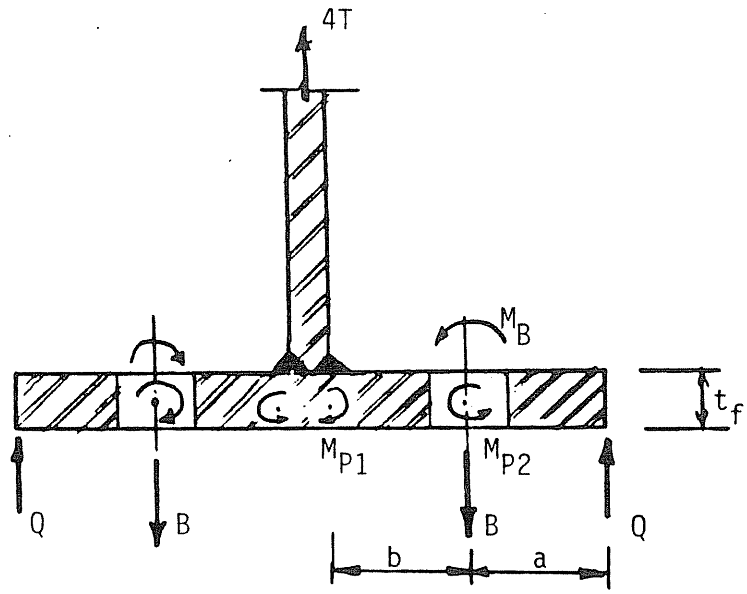
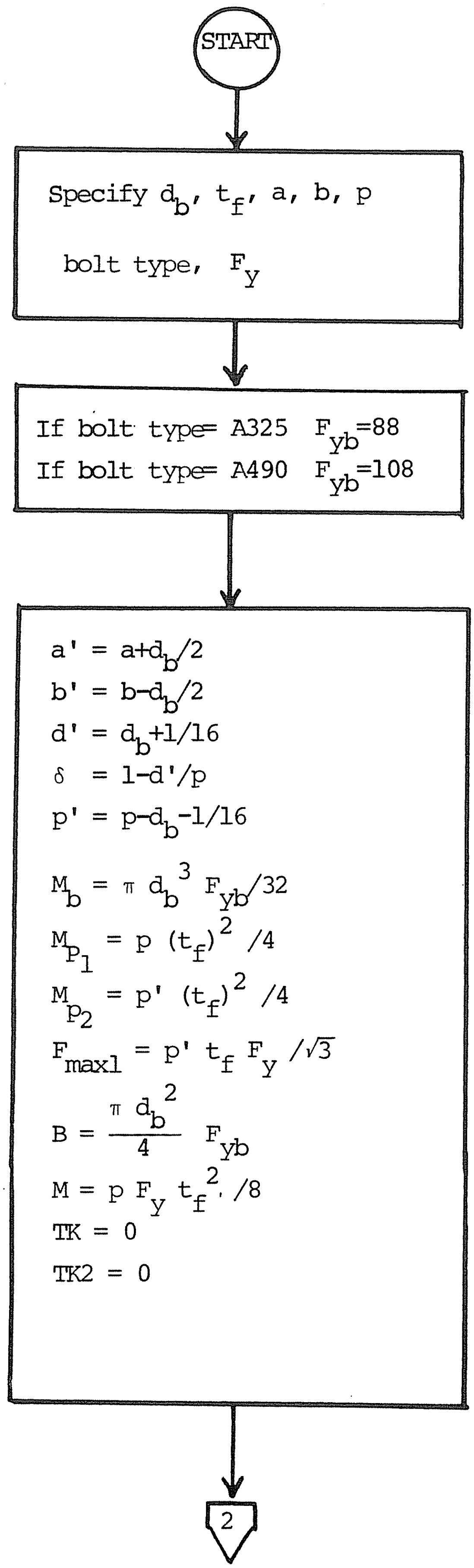
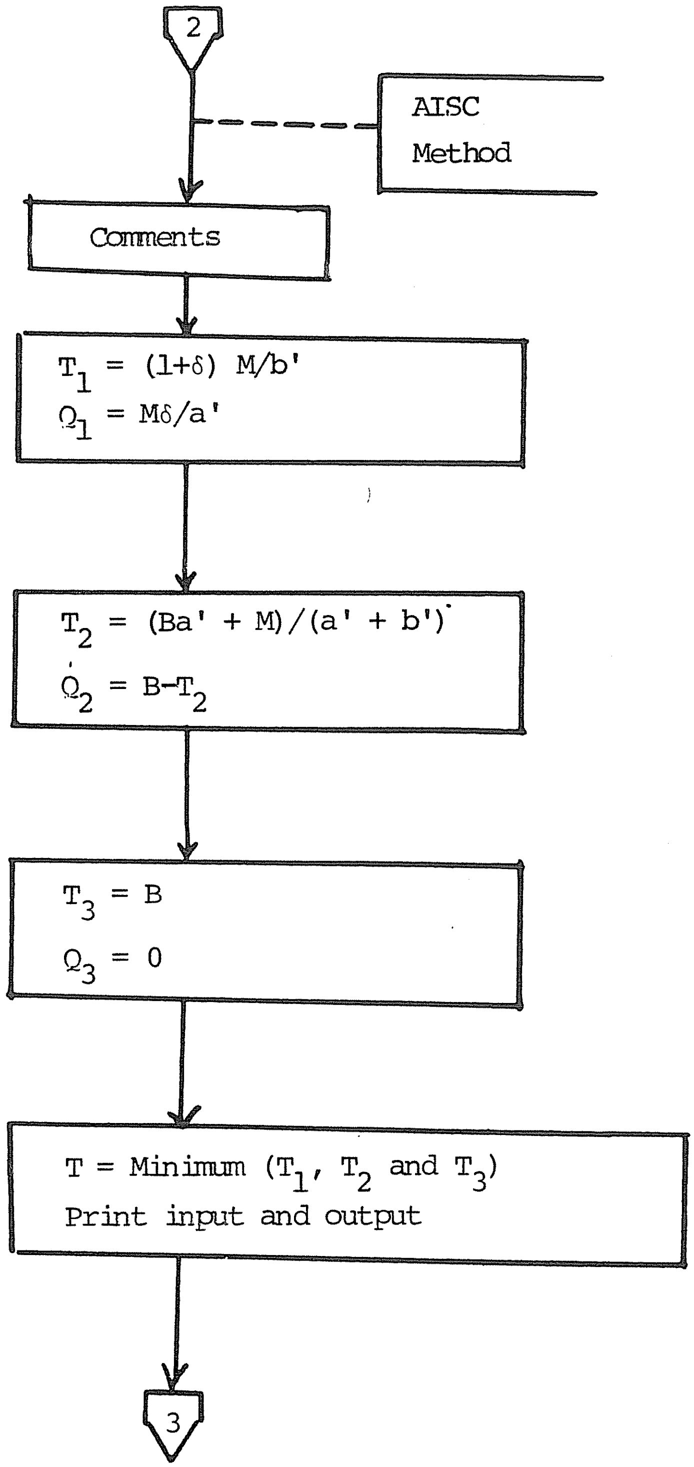
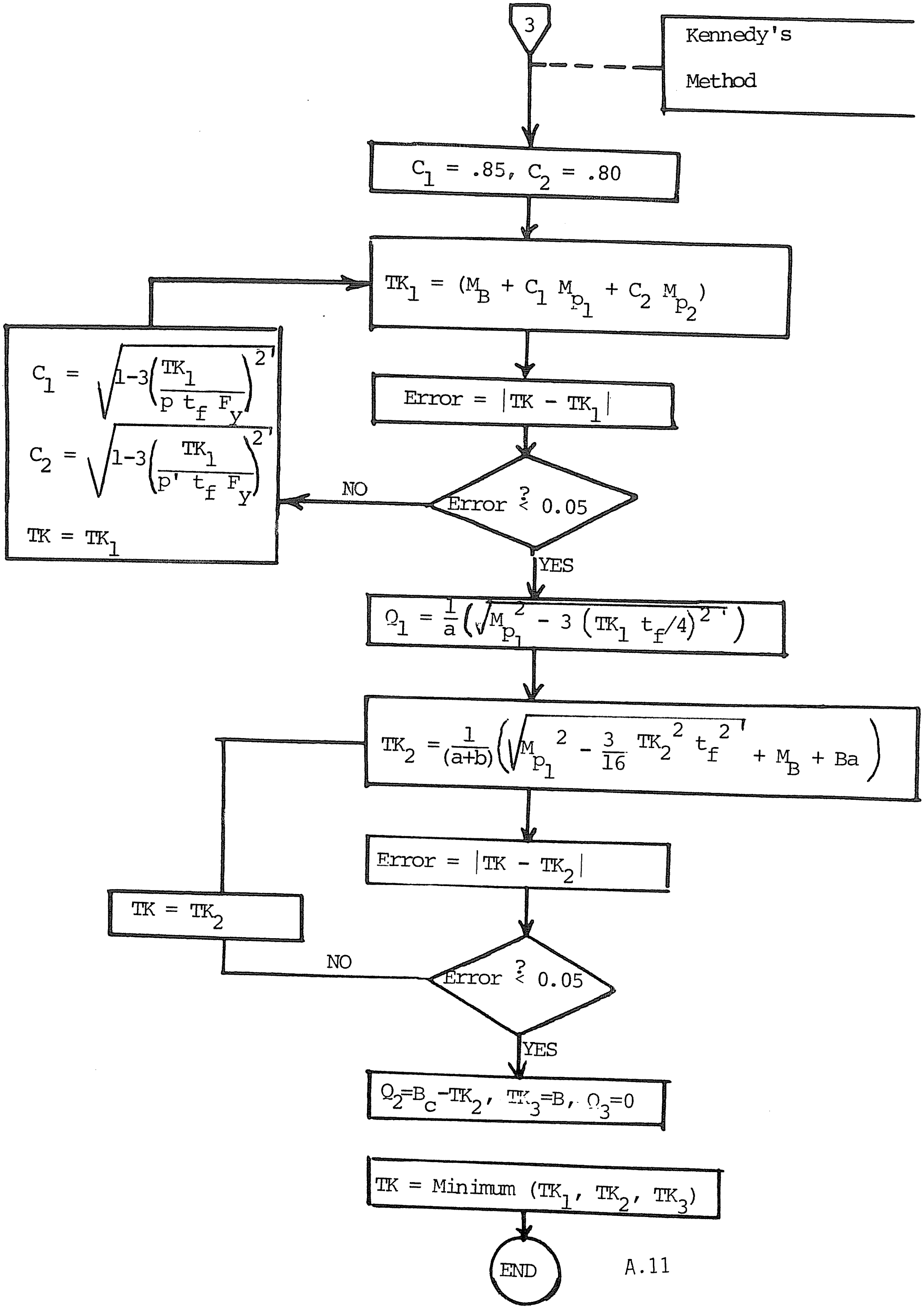


Figure 2.A Kennedy's Model of Tee Hanger (Reference 2)

Flow Chart of Program




2
AISC Method
Comments
$T_1 = (1+\delta)\ M/b'$
$Q_1 = M\delta/a'$
$T_2 = (Ba' + M)/(a' + b')$
$Q_2 = B - T_2$
$T_3 = B$
$Q_3 = 0$
T = Minimum (T_1, T_2 and T_3)
Print input and output
3


3
Kennedy's Method
$C_1 = .85,\ C_2 = .80$
$TK_1 = (M_B + C_1 M_{p_1} + C_2 M_{p_2})$
$Error = |TK - TK_1|$
$Error \overset{?}{<} 0.05$
NO
$C_1 = \sqrt{1-3\left(\frac{TK_1}{p\ t_f\ F_y}\right)^2}$
$C_2 = \sqrt{1-3\left(\frac{TK_1}{p'\ t_f\ F_y}\right)^2}$
$TK = TK_1$
YES
$Q_1 = \frac{1}{a}\left(\sqrt{M_{p_1}^{\ 2} - 3\,(TK_1\ t_f/4)^2}\right)$
$TK_2 = \frac{1}{(a+b)}\left(\sqrt{M_{p_1}^{\ 2} - \frac{3}{16}\,TK_2^{\ 2}\ t_f^{\ 2}} + M_B + Ba\right)$
$Error = |TK - TK_2|$
$TK = TK_2$
NO
$Error \overset{?}{<} 0.05$
YES
$Q_2 = B_c - TK_2,\ TK_3 = B,\ Q_3 = 0$
$TK = \text{Minimum}\ (TK_1, TK_2, TK_3)$
END

```
9 CLS:KEY OFF
10 PRINT"******************************************************************"
12 PRINT"*          PROGRAM FOR ANALYSIS AND DESIGN OF                     *"
14 PRINT"*          TEE HANGERS ACCORDING TO AISC MANUAL OR                 *"
16 PRINT"*          PROCEDURE PROPOSED BY KENNEDY et. al.                   *"
18 PRINT"*                                                                  *"
19 PRINT"*          Developed by  Abolhassan Astaneh, Ph.D. P.E.            *"
20 PRINT"*                                                                  *"
21 PRINT"*          of The University of Oklahoma                           *"
22 PRINT"*          School of Civil Engineering & Environmental Science     *"
24 PRINT"*          Fears Structural Engineering Laboratory                 *"
25 PRINT"*                                                                  *"
27 PRINT"*          Version 1.0 (11-1-1984)                                 *"
28 PRINT"******************************************************************"
29 PRINT"*  No responsibility is assumed by the developer or by the        *"
30 PRINT"*  University of Oklahoma for any errors, mistakes or mis-        *"
31 PRINT"*  representations that may occur from the use of this           *"
32 PRINT"*  computer program. All software provided are in AS IS          *"
33 PRINT"*  condition.   No warranty of any kind, whether statuary,       *"
34 PRINT"*  written, oral, expressed or implied including warranties      *"
35 PRINT"*  of fitness and merchandibility shall apply.   The legal       *"
36 PRINT"*  ownership of this program remains with the developer.         *"
46 PRINT"******************************************************************"
1000 PRINT:PRINT "Press ENTER KEY to continue";:INPUT  Z$
1010 READ DB,TF,A,B,P,B$,FY,JOB$
1011 PRINT JOB$;
1020 IF B$="A325" OR B$="a325" THEN FYB=88 :GOTO 1040
1030 IF B$="A490" OR B$="a490" THEN FYB=108     :GOTO 1040
1037 BEEP:CLS:PRINT :PRINT:PRINT:PRINT "*****************************************
*******************************"
1038 PRINT "*      GIVEN BOLT TYPE IS NOT A325 OR A490. Correct data and retry.
 *"
1039 PRINT "*********************************************************************
***" :GOTO 9998
1040 BOLT=CDBL(3.14159*DB*DB*FYB /4)
1041 BC=3.14159*DB*DB*FYB /8
1050 BP=B-DB/2: AP=A+DB/2: DP=DB+1/16: M=P*FY*TF*TF/8: DEL=1-DP/P
1070 T1=(1+DEL)*M/BP:  T2=BC*AP+M/(AP+BP): T3=BC :F=T1
1080 IF T1>T2 THEN F=T2
1090 IF F>T3 THEN F=T3
1105 IF F=T1  THEN Q=DEL*M/AP
1110 IF F=T2  THEN Q=BOLT-T2
1120 IF F=T3  THEN Q=0
1200 IF F=T1  THEN FLAG$="PLATE"
1210 IF F=T2  THEN FLAG$="BOLT"
1220 IF F=T3  THEN FLAG$="BOLT"
2000 REM          TEE HANGER DESIGN ACCORDING TO KENNEDY'S METHOD
2020 W=P
2030 WPR=W-DB-1/16
2040 MB=3.14159*DB^3/32*FYB
2050 MP01=W*TF*TF*FY/4:   MP02=WPR*TF*TF*FY/4
2055 FMAX1=MP01/(B*B+3*TF*TF/16)^.5
2060 REM FIND TK1
2061 PRINT "Iterating       TK1  "
2070 C1=.85   :  C2=.8    : FLIM=WPR*TF*FY/3^.5  : TK=0
2071 TK1=MB/B+C1*MP01/B+C2*MP02/B
2072 IF TK1 > FLIM THEN 2100
2073 IF ABS(TK-TK1) < .05 THEN 2100
2077 C1=(1-3*(TK1/W/FY/TF)^2)^.5
2078 C2=(1-3*(TK1/WPR/FY/TF)^2)^.5
2080 TK=TK1
2084 N=N+1
```

```
2085 PRINT "Iteration ", N;:PRINT " TK1=",TK1
2090 GOTO 2071
2100 PRINT"----------------------------------":REM FIND TK2
2101 PRINT"Iterating    TK2  "
2102 N=0
2110 TK2=0
2120 TK22=(1/(A+B))*(SQR(MP01^2-3/16*TK2^2*TF^2)+MB+BOLT*A)
2130 EX=ABS(TK22-TK2)
2140 IF EX < .05 THEN 2300
2150 TK2=TK22  : N=N+1:PRINT "Iteration "N;:PRINT " TK2=",TK2:GOTO 2120
2300 PRINT "-------------------------------":REM  FIND TK3
2310 TK3=BOLT :PRINT "TK3=",TK3
4000 FK=TK1
4080 IF TK1>TK2 THEN FK=TK2
4090 IF FK>TK3 THEN FK=TK3
4100 IF FK=TK1 THEN QK=1/A*(SQR(MP02^2-3*FK*FK*TF*TF/16))
4110 IF FK=TK2 THEN QK=BOLT-FK
4120 IF FK=TK3 THEN QK=0
4200 IF FK=TK1 THEN FLAGK$="PLATE"
4210 IF FK=TK2 THEN FLAGK$="BOLT"
4220 IF FK=TK3 THEN FLAGK$="BOLT"
7000 LPRINT:LPRINT:LPRINT
7230 LPRINT"          ";JOB$
7240 LPRINT"          ========="
7250 LPRINT:LPRINT
7260 LPRINT"          Given:"
7270 LPRINT"          ------"
7290 LPRINT"          tf=";USING "#.###";TF;
7291 LPRINT " in.           (Thickness of flange in tee hanger)     "
7300 LPRINT"          a =";USING "#.###"; A;
7301 LPRINT " in.           (Distance from bolt center to edge)     "
7310 LPRINT"          b =";USING "#.###"; B;
7311 LPRINT " in.           (Distance from bolt center to  web)     "
7320 LPRINT"          p =";USING "#.###"; P;
7321 LPRINT " in.           (Width of tee hanger per bolt line)     "
7330 LPRINT"          Fy=";USING "##.##"; FY;
7331 LPRINT " ksi           (Yield stress of  tee hanger plate) "
7340 LPRINT"          Bolt type=";B$;
7341 LPRINT "               (Type of bolt.  A325 or A490)       "
7342 LPRINT"          db=";USING "#.###";DB;
7343 LPRINT " in.                              (Diameter of Bolt)     "
7350 LPRINT"          B =";USING"##.##";BOLT;
7351 LPRINT " kips.                  (Ultimate capacity of bolt)     "
7360 LPRINT"          Fyb=";USING"##.##";FYB;
7361 LPRINT " kips.             (Yield stress of threaded bolt)    "
7362 LPRINT"          Ptb=";USING"##.##";.7*BOLT;
7363 LPRINT " kips.    (Pretension force in bolt=0.70 x B)     "
7370 LPRINT:LPRINT:LPRINT
7380 LPRINT"          Solution:"
7390 LPRINT"          ---------"   :LPRINT
7400 LPRINT"          According to the method outlined in Chapter 4 of the AISC ":L
PRINT
7420 LPRINT"          Manual, the results are:":LPRINT
7430 LPRINT"          Service load capacity            =";USING"###.##";4*F;
7431 LPRINT" kips."
7440 LPRINT"                                            ======"
7450 LPRINT"          Service load capacity times 2.0=";USING"###.##";8*F;
7451 LPRINT" kips."
7460 LPRINT"                                            ======"
7470 IF FLAG$="BOLT" THEN 7500
7480 LPRINT"          Predicted failure mode is  plate yielding      "
7490 GOTO 7510
7500 LPRINT"          Predicted failure mode is  bolt rupture        "
7510 LPRINT
7511 BF=(F+Q)*2
```

```
7512 IF BF > .7*BOLT THEN 7520
7513 BF=.7*BOLT
7520 LPRINT"       Predicted bolt force at factored load=";USING"###.##";BF;
7521 LPRINT" kips"
7530 LPRINT"                                                   ======"
7540 LPRINT:LPRINT
7550 LPRINT
7600 LPRINT"       According to the method outlined in   THE SPLIT-TEE ANALOGY":
LPRINT
7620 LPRINT"       IN BOLTED SPLICES AND BEAM-COLUMN CONNECTIONS, by Kennedy et
al "
7630 LPRINT
7640 LPRINT"       published in    Joints in Structural Steelwork, John Wiley &"
:LPRINT
7660 LPRINT"       Sons, NY ,1981, pp.2.138-2.157, the results are:"
7670 LPRINT
7730 LPRINT"       Elastic load capacity          =";USING"###.##";FMAX1*4;
7731 LPRINT" kips"
7740 LPRINT"                                          ======"
7750 LPRINT"       Ultimate load capacity         =";USING"###.##";4*FK;
7751 LPRINT" kips"
7760 LPRINT"                                          ======"
7770 IF FLAGK$="BOLT" THEN 7800
7780 LPRINT"       Predicted failure mode is  plate yielding     "
7790 GOTO 7810
7800 LPRINT"       Predicted failure mode is  bolt rupture       "
7810 LPRINT
7811 BFK=FK+QK
7812 IF BFK > .7*BOLT THEN 7820
7813 BFK=.7*BOLT
7820 LPRINT"       Predicted bolt force at factored load=";USING"###.##";BFK;
7821 LPRINT" kips"
8000 LPRINT"                                                    ======"
8170 LPRINT: LPRINT
8210 LPRINT: LPRINT: LPRINT :LPRINT: LPRINT
8220 LPRINT:LPRINT : LPRINT
9500 REM  db    tf    a    b    p  bolt  fy     job
9580 REM DATA .625,.375,1.25,2.25,3,"A325",36,"Test TH-1"
9590     DATA .625,.500,1.50,1.125,3,"A325",36,"Test TH-2 "
9600 REM DATA  .625,.500,1.25,2.250,3,"A325",36,"Test TH-3 "
9610 REM DATA .750,.500,2.25,2.250,3,"A325",36,"Test TH-4 "
9620 REM DATA .750,.750,1.50,1.250,3,"A325",36,"Test TH-5 "
9630 REM DATA .750,.750,1.25,2.250,3,"A325",36,"Test TH-6 "
9640 REM DATA 1.00,.750,1.50,2.50,3,"A325",36,"Test TH-7 "
9650 REM DATA 1.00,1.00,1.50,1.75,3,"A325",36,"Test TH-8 "
9660 REM DATA 1.00,1.00,1.50,2.50,3,"A325",36,"Test TH-9"
9670 REM DATA .625,.500,1.50,1.125,3,"A325",36,"Test CN-1"
9680 REM DATA .625,.500,1.50,1.125,3,"A325",36,"Test CN-1A "
9690 DATA .750,.625,1.50,1.500,3,"a325",50,"Test CN-2A"
9700 REM DATA .750,.75,1.5,1.25,3,"A325",36,"Test CN-3 "
9710 REM DATA 1.00,1.00,1.50,1.750,3,"A325",36,"Test CN-4 of VULCRAFT"
9970 GOTO 1000
9980 END
```

Sample of Program Output:

```
Test TH-2
=========


Given:
------
tf=0.500 in.          (Thickness of flange in tee hanger)
a =1.500 in.          (Distance from bolt center to edge)
b =1.125 in.          (Distance from bolt center to  web)
p =3.000 in.          (Width of tee hanger per bolt line)
Fy=36.00 ksi          (Yield stress of  tee hanger plate)
Bolt type=A325              (Type of bolt.  A325 or A490)
db=0.625 in.                           (Diameter of Bolt)
B =27.00 kips.              (Ultimate capacity of bolt)
Fyb=88.00 kips.         (Yield stress of threaded bolt)
Ptb=18.90 kips.       (Pretension force in bolt=0.70 x B)


Solution:
---------

According to the method outlined in Chapter 4 of the AISC

Manual, the results are:

Service load capacity           = 29.42 kips.
                                  ======
Service load capacity times 2.0= 58.85 kips.
                                  ======
Predicted failure mode is  plate yielding

Predicted bolt force at factored load= 18.90 kips
                                       ======


According to the method outlined in   THE SPLIT-TEE ANALOGY

IN BOLTED SPLICES AND BEAM-COLUMN CONNECTIONS, by Kennedy et al

published in    Joints in Structural Steelwork, John Wiley &

Sons, NY ,1981, pp.2.138-2.157, the results are:

Elastic load capacity           = 23.57 kips
                                 ======
Ultimate load capacity          = 46.06 kips
                                 ======
Predicted failure mode is  plate yielding

Predicted bolt force at factored load= 18.90 kips
                                        ======
```

A.5 EXAMPLE COMPUTATIONS

Example 1. Calculate allowable and ultimate capacity of a tee-hanger with the following dimensions. These dimensions correspond to test specimens TH-2.

Given:

t_f = 0.5 in. (thickness of flange in tee hanger)

a = 1.5 in. (distance from bolt center to edge)

b = 1.125 in. (distance from bolt center to web)

p = 3 in. (width of tee hanger per bolt line)

bolt dia. = .625 in. (diameter of bolt)

bolt cap. = 27 kips (ultimate capacity of bolt)

bolt type = A325 (type of bolt)

F_{yb} = 88 ksi (yield stress of A325 bolts)

F_y = 36 ksi (plate yield stress)

Solution:

$a' = a + d_b/2 = 1.5 + 0.625/2 = 1.8125$ in.

$b' = b - d_b/2 = 1.125 - 0.625/2 = 0.8125$ in.

$d' = d_b + 1/16 = 0.625 + 1/16 = 0.6875$ in.

$\delta = 1 - d'/p = 1 - 0.6875/3 = 0.771$ in.

$p' = p - d' = 3 - .6875 = 2.3125$ in.

$M_B = \pi d_b^3\ F_y/32 = \pi(0.625)^3\ (36)/32 = 0.863$ k-in.

$M_{p1} = (p\ t_f^2/4)\ F_y = 3(.5)^2\ (36)/4 = 6.75$ k-in.

$M_{p2} = (p'\ t_f^2/4)\ F_y = (2.3125)\ (.5)^2\ (36)/4 = 5.203$ k-in.

$M = p\ F_y\ t_f^2/8 = M_{p1}/2 = 6.75/2 = 3.375$ k-in.

$B = (\pi\ d_b^2/4)(F_{yb}/2) = (\pi) \times (.675^2/4)(88/2) = 13.5$ kips

(B can also be obtained from Table 1-A in Page 4-3 of AISC Manual)

$Bu = 2B = 27$ kips

$C_1 = 0.85$ (for first trial)

$C_2 = 0.80$ (for first trial)

AISC Method

The following procedure is based on rearranged AISC formulas, see Reference 7.

$T_1 = (1 + \delta)\ M/b' = (1 + 0.771)(3.375)/.8125 = 7.36$ kips

$T_2 = (Ba' + M)/(a' + b') = (13.5(1.8125) + 3.375)/(1.8125 + 0.8125) = 25.75$ kips

$T_3 = B = 13.5$

$T_{max} = \text{Min}\ (T_1, T_2, T_3) = 7.36$ kips

AISC Allowable = 4T = 4 x 7.36 = 29.4 kips

AISC x 2 = 29.4 x 2 = 58.8 kips

Kennedy's Method

$T_1 = (M_B + C_1\ M_{p1} + C_2\ M_{p2})/b$

$T_1 = (.863 + 6.75\ C_1 + 5.203\ C_2)/1.125$

$C_1 = \sqrt{1 - 3(T_1/pt_f\ F_y)^2} = \sqrt{1 - 3\ [T_1/(3 \times .5 \times 36)]^2}$

$C_2 = \sqrt{1 - 3(T_1/p't_f\ F_y)^2} = \sqrt{1 - 3\ [T_1/(2.3125 \times .5 \times 36)]^2}$

and

$T_1 < p't_f\ F_y/\ \ 3 = 24$ kips

Iteration of T_1, C_1 and C_2 results in:

$T_1 = 11.52$ kips

$$T_2 = \frac{\sqrt{M_{p_1}^{\ 2} - (3/16)\ (T_2\ t_f)^2} + M_B + B_u\ a}{a + b}$$

$$T_2 = \frac{\sqrt{(6.75)^2 - (3/16)(T_2 \times .25)^2} + 0.863 + 27 \times 1.5}{2.625}$$

and

$T_2 < (p' \; t_f \; F_y)/ \;\; 3 = 24$ kips

Iteration of above two equations results in:

$T_2 = 18.28$ kips

$T_3 = B_u = 27$ kips

$T_{u1} = \text{Min } (T_1, T_2, T_3) = 11.52$ kips

$T_u = 4 \; T_{u1} = 4 \times 11.52 = 46.06$ kips