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

FRAME FR1 TESTS

by

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CHAPTER I

INTRODUCTION

A series of tests was conducted in the Fears Structural Engineering Laboratory, School of Civil Engineering and Environmental Science, University of Oklahoma, using standard rigid frames produced and erected by VULCRAFT, a division of Nucor Corporation, hereafter referred to as VULCRAFT. The purpose of these tests was to determine the structural strength and stiffness of the rigid frames, as well as, the adequacy of the analysis/ design procedures currently employed by VULCRAFT. The frames, designated FR-1, were fabricated to the dimensions below:

Overall Span	52 ft. 10 in.
Clear Span	47 ft. 10 in.
Eave Height	15 ft. 10 7/8 in.
Clear Height	13 ft. 4 7/8 in.
Roof Slope	1/2:12
Column Taper	1 1/2 in./12 in.
Moment Splice Connections	

The FR-1 specimens consisted of clear span rigid frames with tapered open web column sections and a tapered open web rafter section, all fabricated of shop welded steel angles. A roof slope of 1/2:12 was used for these frames.

The test specimens were fabricated as part of standard production runs. The test set-up consisted of two frames spaced 24 ft. 0 in. on center, with connecting simple span joists and girts, joist bridging, chord brace angles, and rod braces as shown in Figures 1.1 and 1.2. Roof deck and sidewall panel were not installed for the tests. The rafter to column connection consisted of two flat plates and high strength bolts as shown in Figure 1.3.

Simulated dead and live load was applied using gravity load simulators. Simulated wind loading (henceforth referred to as lateral load) was applied using hydraulic cylinders attached to reaction columns. These reaction columns, Figure 1.2(b), were located outside the frames at one end but are not shown in Figure 1.1.

Four test series were conducted: unbalanced live load, lateral load only, combined unbalanced live and lateral load, and full live load. The final test was continued until failure occurred.

This report provides a detailed description of testing procedures, instrumentation and test results.

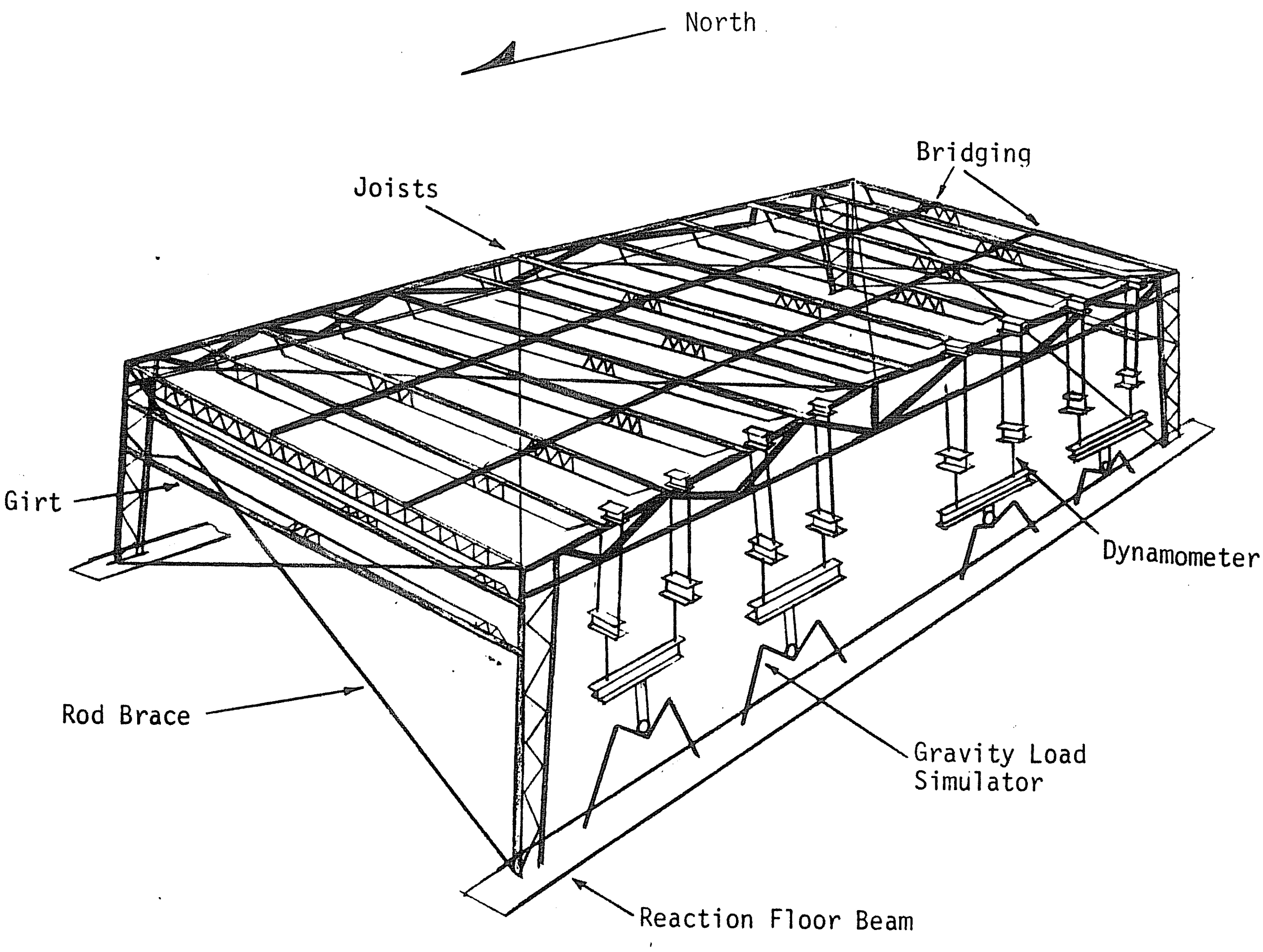
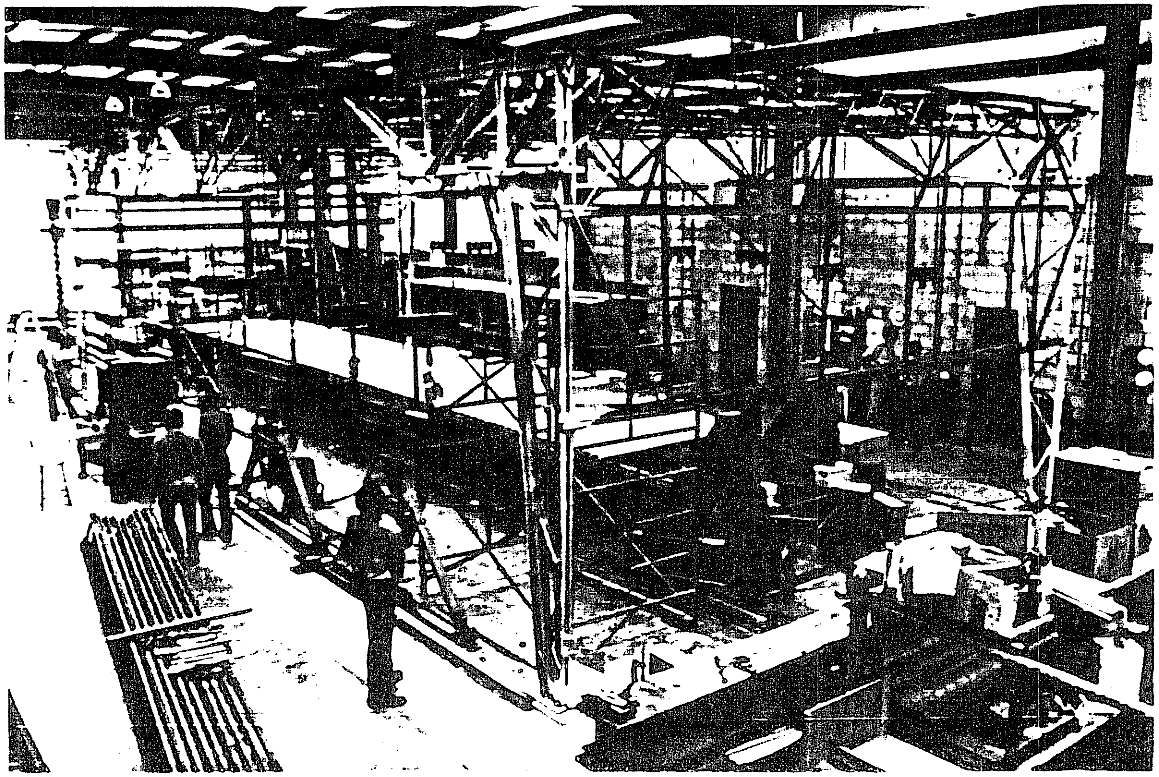
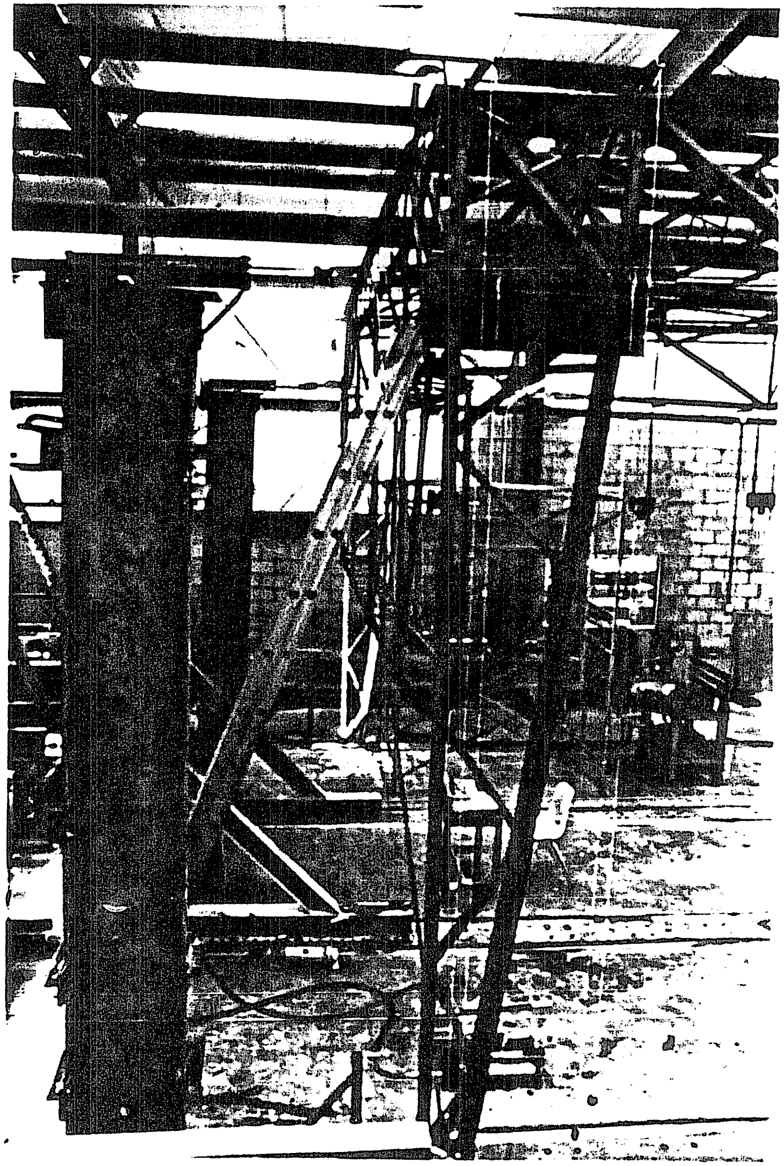


Figure 1.1 Overall View of Test Set-Up



a) Overview



b) Lateral Loading Details

Figure 1.2 Photographs of Test Setup

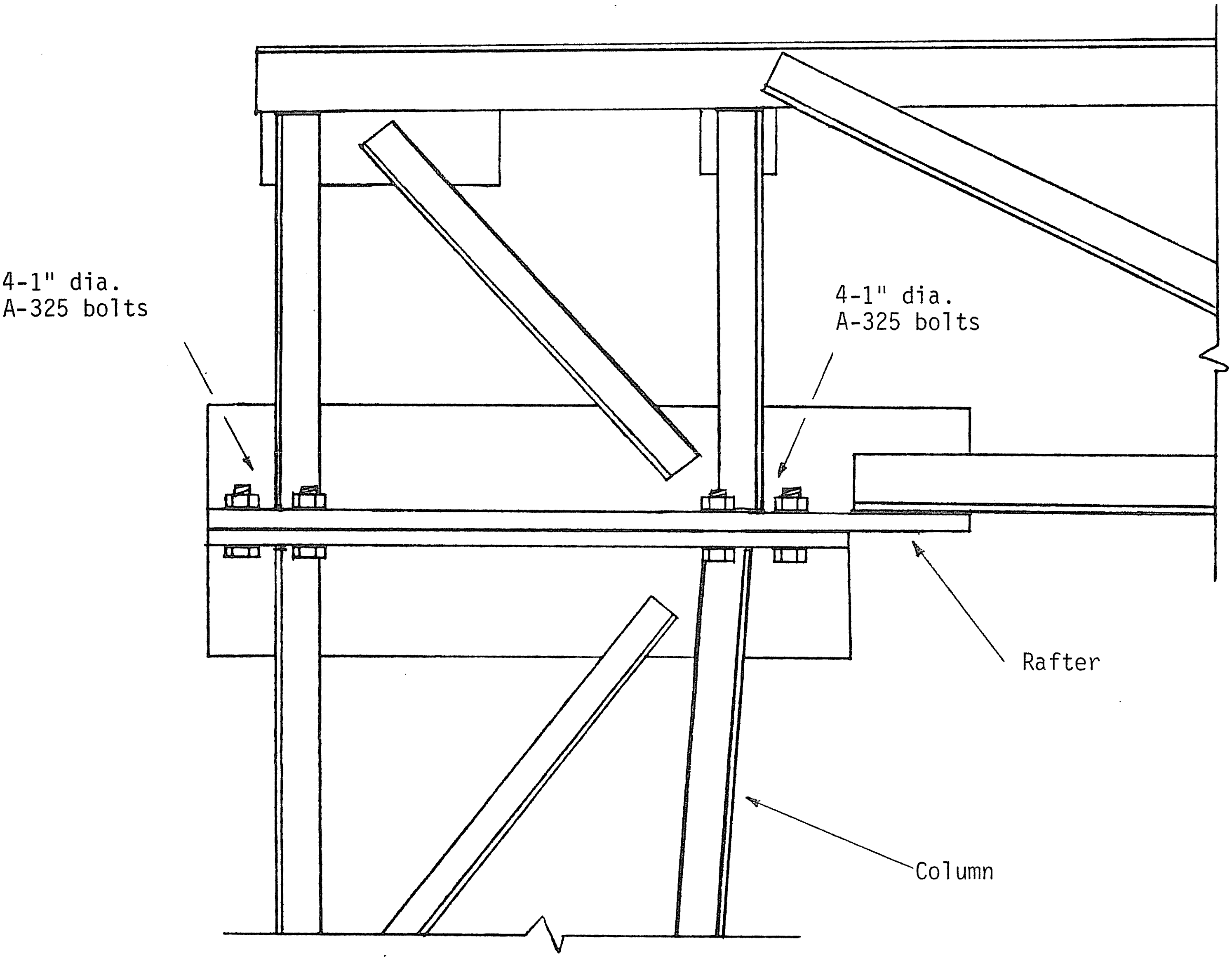


Figure 1.3 Rafter-to-Column Connection Detail

CHAPTER II

TEST DETAILS

2.1 Description of Specimens

Both the test frames and roof joists were fabricated from steel having a nominal yield stress of 50 ksi. The overall dimensions of the frames are shown in Figure 2.1, along with the member labeling used in the theoretical analyses. The dimensions and properties of the members are included with the stiffness analyses provided by VULCRAFT. Results of these analyses are found in Appendix A. Bottom chord brace locations and details are shown in Figure 2.2.

Several modifications were made to the standard components provided by VULCRAFT. Prior to any testing, the end diagonal web members of all joists adjacent to a frame bottom chord brace were reinforced as shown in Figure 2.2(b). This reinforcement consisted of an angle spot welded to the diagonal web member. The reinforcement was designed to prevent the compression failure of the joist end diagonal in the event that the lateral brace of the frame bottom chord developed a significant compressive force.

In addition, the local failure of frame members during the various loading sequences resulted in additional

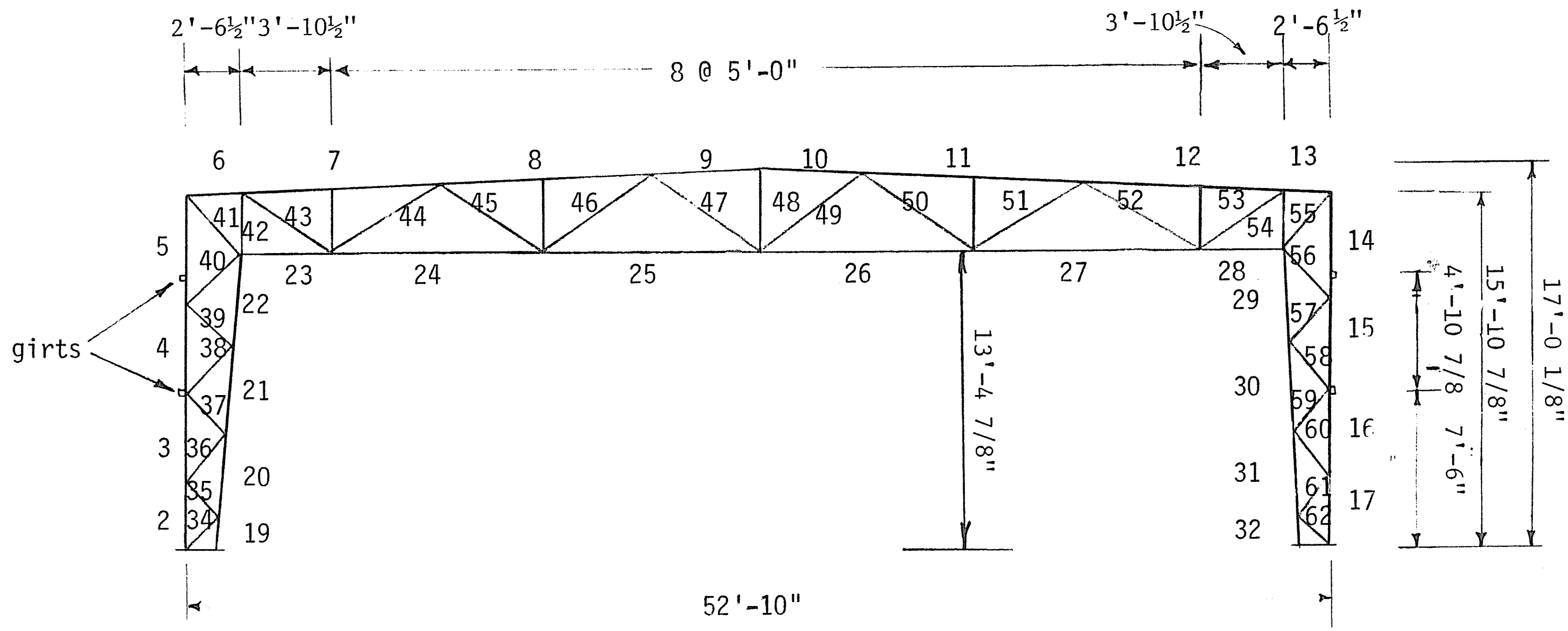
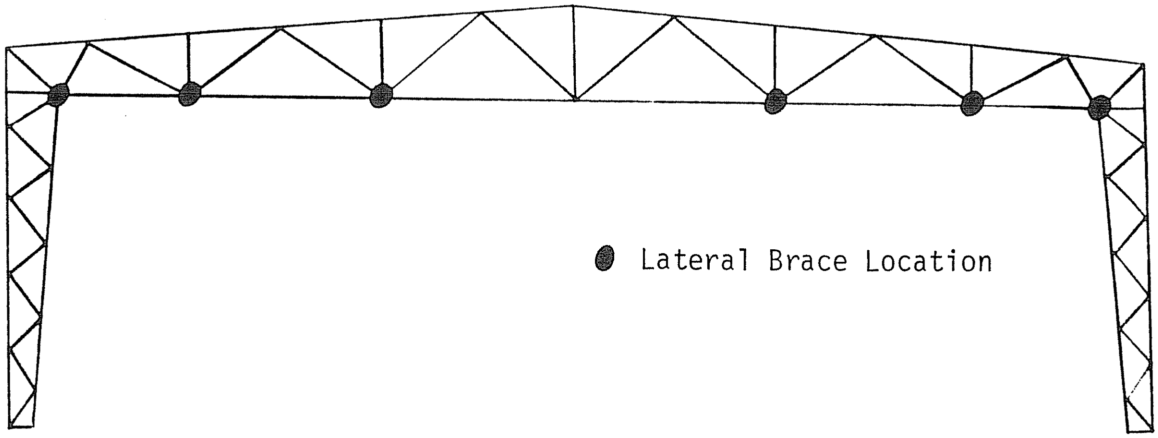
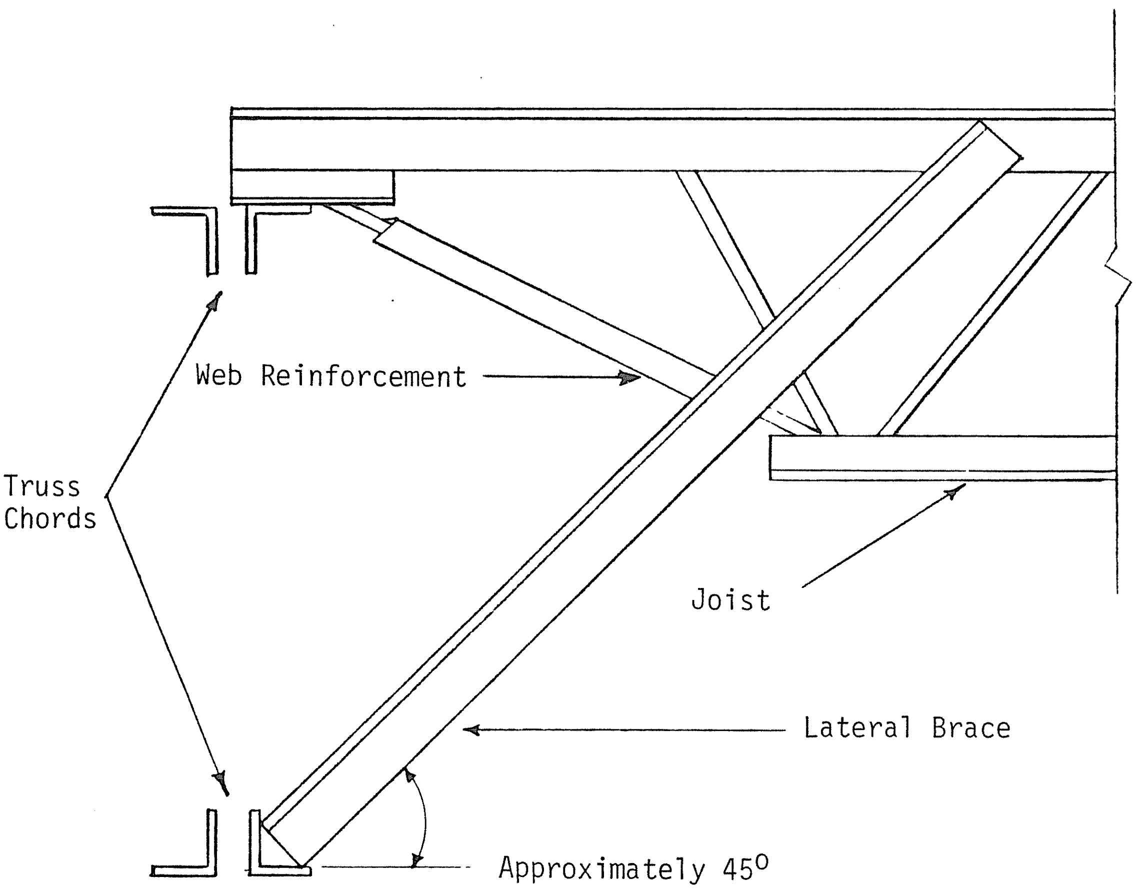


Figure 2.1 Frame Dimensions and Member Labeling



(a) Lateral Chord Brace Locations



(b) Lateral Brace Details

Figure 2.2 Chord Lateral Brace Locations and Details modifications. During the first test (full live load on the east frame), the diagonal web members at the truss centerline (members 47 and 49 of Figure 2.1) buckled prior to the working load level. Since this buckling was believed to be associated with eccentricities at the centerline, an additional bottom chord lateral brace was added. Prior to the initiation of the next test, these diagonals were further reinforced by welding angles along their lengths and the resulting cross-section of these members resembled a channel. Likewise, premature buckling of the vertical web member originating at the connection of members 50 and 51 (Figure 2.1) during the second test (unbalanced live load) required their replacement. The welds at this and the other three similar vertical members were also reinforced.

2.2 Test Set-up

The frames were erected inside the Fears Structural Engineering Laboratory on the laboratory reaction floor. The floor is a concrete slab 30 ft. by 60 ft. by 3 ft. 6 in. deep with four W36x150 steel beams embedded in concrete. The slab weighs one million pounds and is capable of reacting 320,000 lb. in any one location. The frames were erected directly over two of the embedded W36 beams, spaced 24 ft. 0 in. apart. Joists and girts at standard spacings were connected between the frames along with standard rod bracing in both the roof and side walls. Compression chord braces at the standard locations were connected between the joist and the bottom flanges of the rafters. Four runs of joist bridging, two on the top chords and two on the bottom chords, were installed the entire length of the structure as indicated in Figure 1.1.

A horizontal truss was constructed using light angles, in the plane of the top chord of the roof joists adjacent and parallel to the east frame rafter. This truss was used to simulate the diaphargm stiffness provided by standard, thru-fastener, metal roof deck.

The column base plates were bolted to the reaction floor beams as shown in Figure 2.3. The erection procedure was as near as possible to standard practice and no specific procedure was used to tighten the column base plate bolts.

2.3 Load Applications

Simulated live load was applied using loading apparatus as shown in Figure 2.4. The loading apparatus consists of a gravity load simulator, Figure 2.5, a 35 kip tension-compression hydraulic cylinder, spreader beam, two calibrated dynamometers and hanger beams and tension rods attached to the frame. The simulator is a device which permits horizontal movement of the point of load application while maintaining a vertical line of action of the applied load. For the simulator used in these tests, the point of application of the load can move left or right a maximum of 10 in. with the hydraulic ram remaining vertical.

Lateral load was applied using a reaction column erected adjacent to the frame with hydraulic cylinders and calibrated load cells positioned as shown in Figure 2.6. For all lateral load applications, load was applied to both frames simultaneously using two identical hydraulic

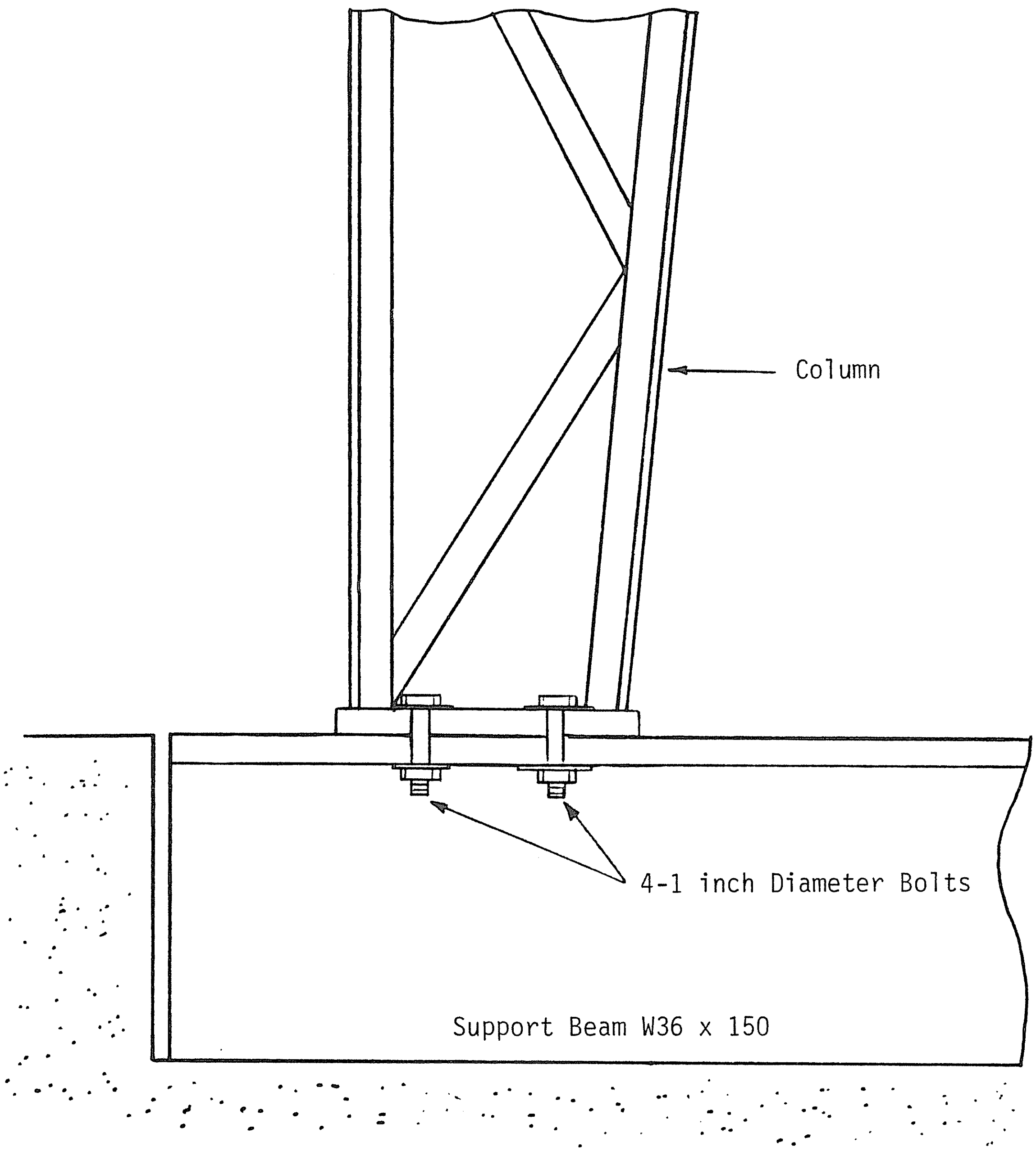


Figure 2.3 Details of Column-to-Reaction Floor Connection

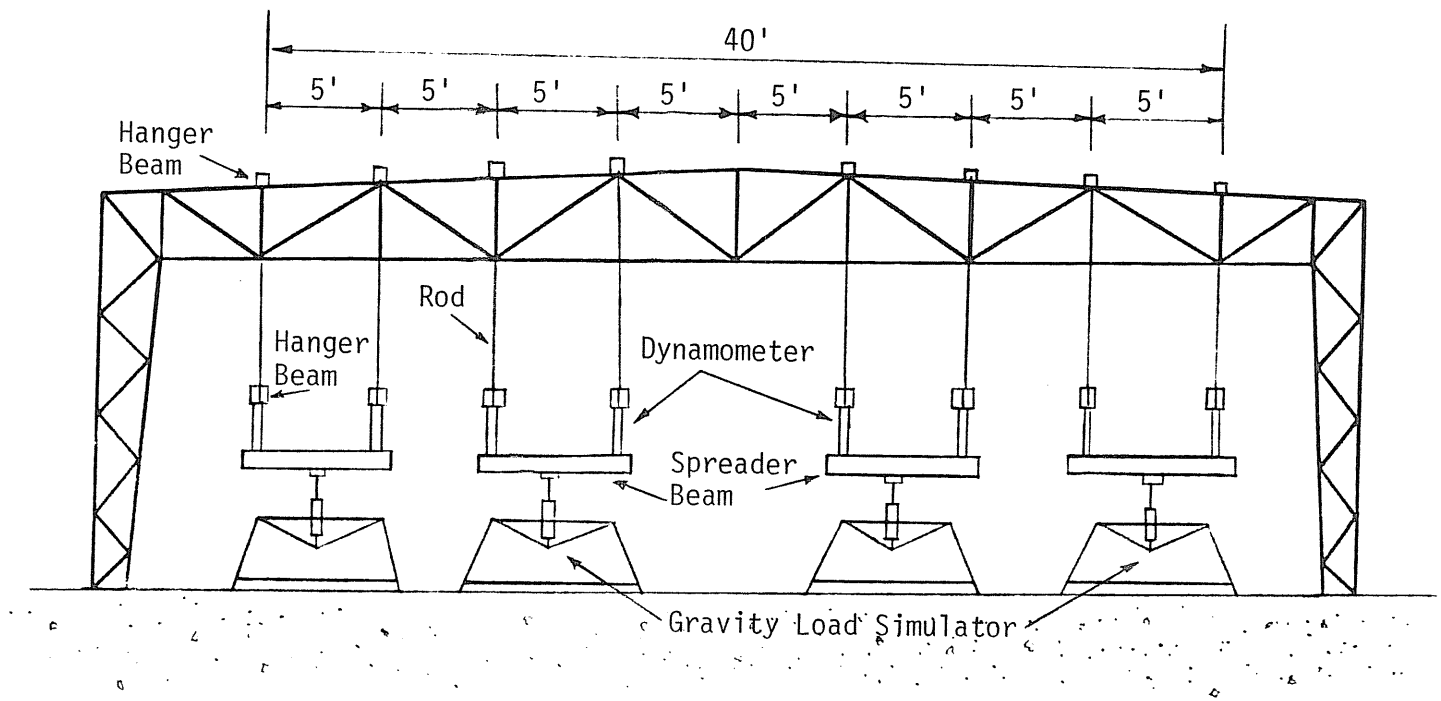


Figure 2.4 Simulated Live Loading Test Setup

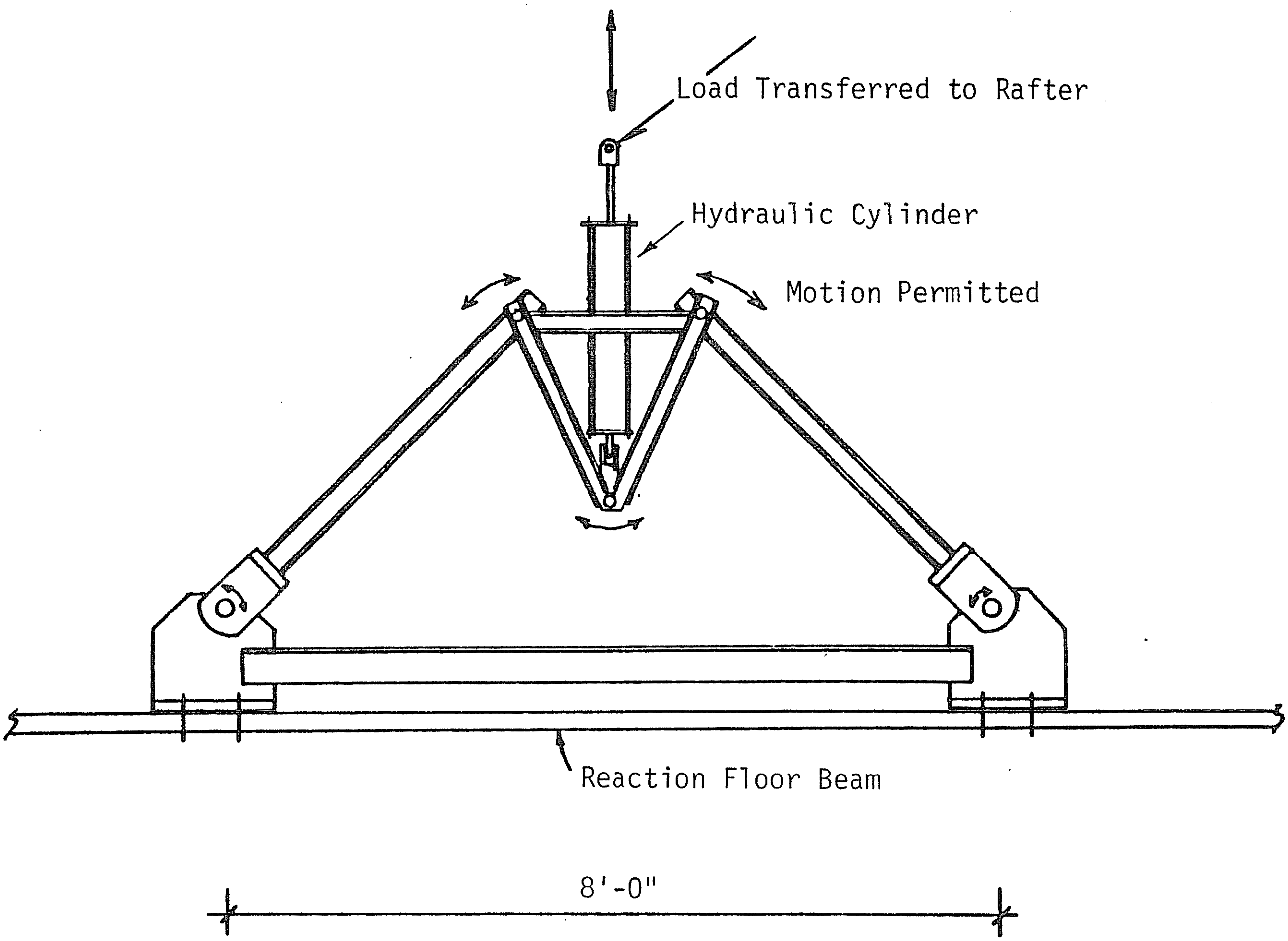


Figure 2.5 Gravity Load Simulator

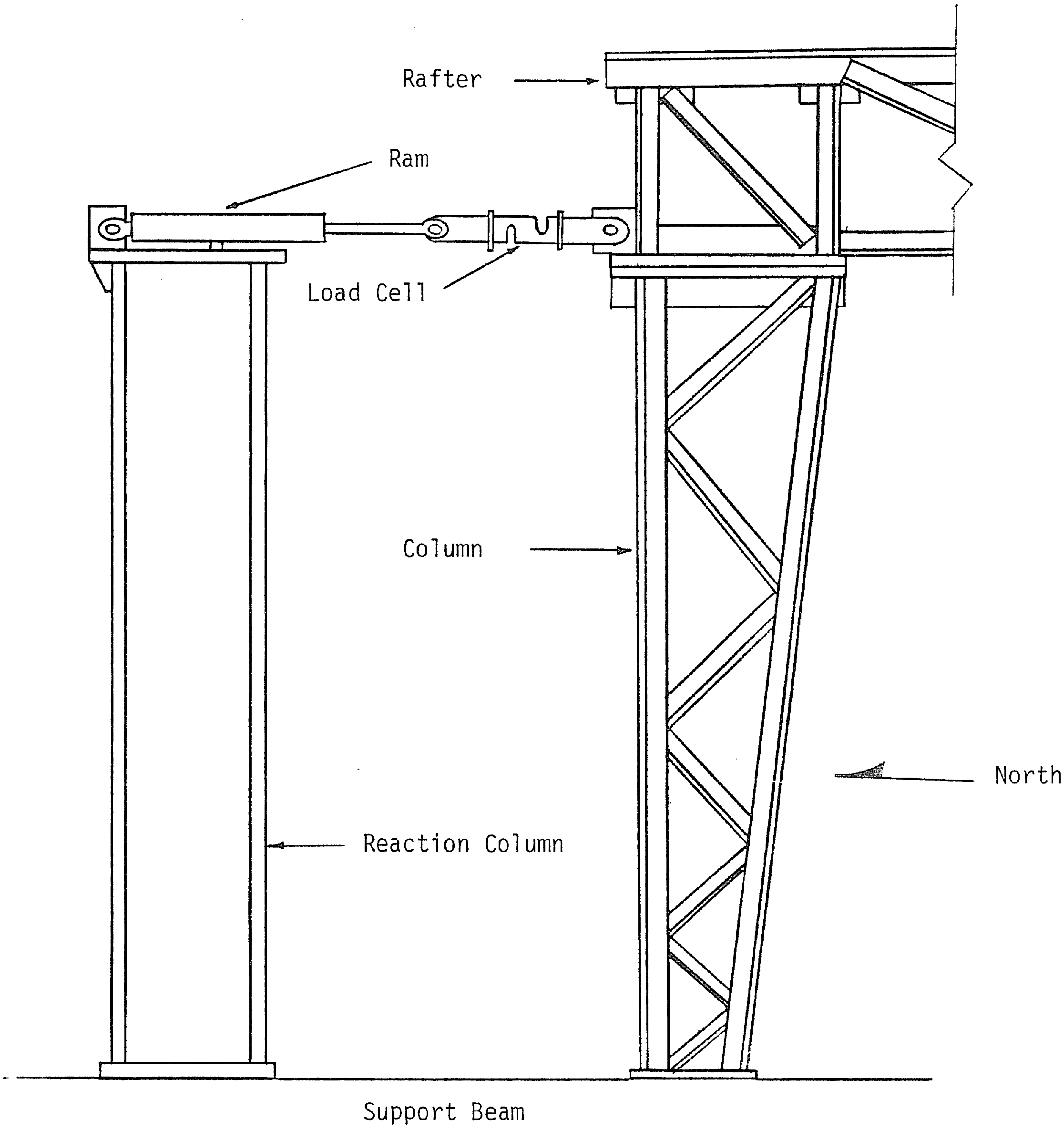


Figure 2.6 Method of Lateral Load Application

cylinders connected in series to an electric pump.

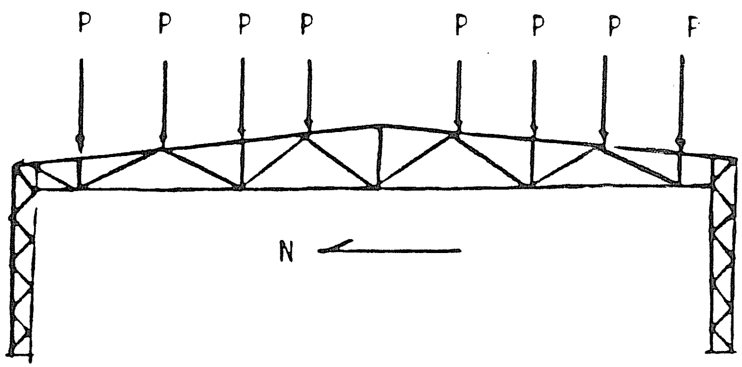
Four loading schemes were used as shown in Figure 2.7. Figure 2.7(a) shows full gravity loading applied to one frame. For this loading condition, four gravity load simulator hydraulic cylinders were connected in series to an electric pump. Figure 2.7(b) shows gravity load applied to one-half of the span to simulate unbalanced live load. For this loading, both frames were loaded simultaneously with the four gravity load simulator hydraulic cylinder connected in series. Figure 2.7(c) is lateral load only, applied as described above. Figure 2.7(d) shows combined lateral load with unbalanced live load applied on the windward side.

2.4 Instrumentation

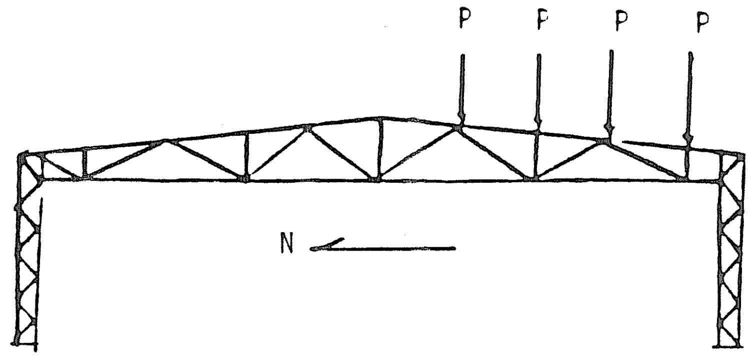
Instrumentation consisted of calibrated dynamometers, calibrated load cells, calibrated calipers and displacement transducers. In addition, strain gages were mounted inside connection bolts to monitor their tensile strains.

Gravity load was measured using the calibrated dynamometers located as shown in Figure 2.4. Lateral load was measured using the load cells positioned as shown in Figure 2.6.

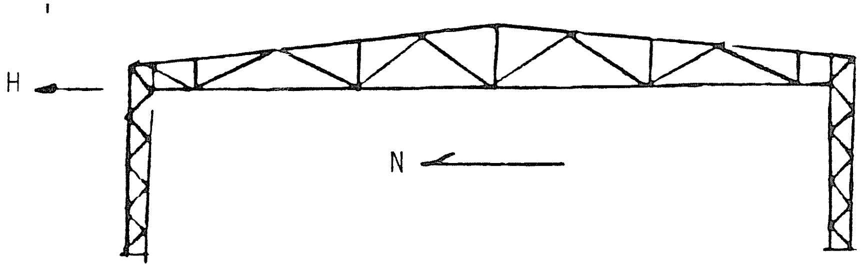
Vertical deflections at the centerline and quarter points of the frames were measured using wire-type displacement transducers attached to the bottom chords and to the reaction floor. Physical restraints imposed by the lateral load application apparatus required sidesway displacements to be measured using a horizontal scales (0.1



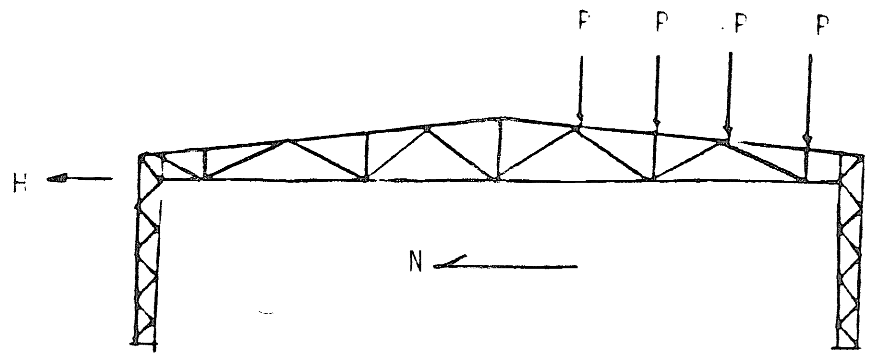
a) Full Live Load



b) Unbalanced Live Load



c) Lateral Load Only



d) Combined Unbalanced Live (wind-ward side) and Lateral Load

Figure 2.7 Loading Conditions

in. gradations) located near the top of each column. Lateral movement of the column and rafter chords were measured utilizing weighted strings suspended from horizontal angles bolted to the frame. A taunt wire running the length of the frame provided a fixed reference for the measurements. Scales were then used to measure the relative movements between the strings and wire or, equivalently, the lateral movement of the frame chords.

To measure axial elongations of the knee diagonal members, two displacement probes, capable of recording displacements as small as .001 in., were mounted on diagonals of interest, Figure 2.8. The probes were mounted on both angles comprising the member, and the average deformation of the two was taken as the deformation of the member. This deformation was then converted to strain and then stress assuming elastic material properties and a modulus of elasticity of 29,000 ksi.

To determine bolt forces, a small hole was first drilled thru the head of the bolt into the unthreaded shank. A special strain gage was then inserted into the hole and the hole filled with epoxy. After curing of the epoxy, the bolt was calibrated using a universal testing machine.

To determine connection plate separations, calibrated calipers, located as shown in Figure 2.8, were used.

2.5 Testing Procedures

Prior to any actual testing, an overall check of the testing apparatus and instrumentation was made. The

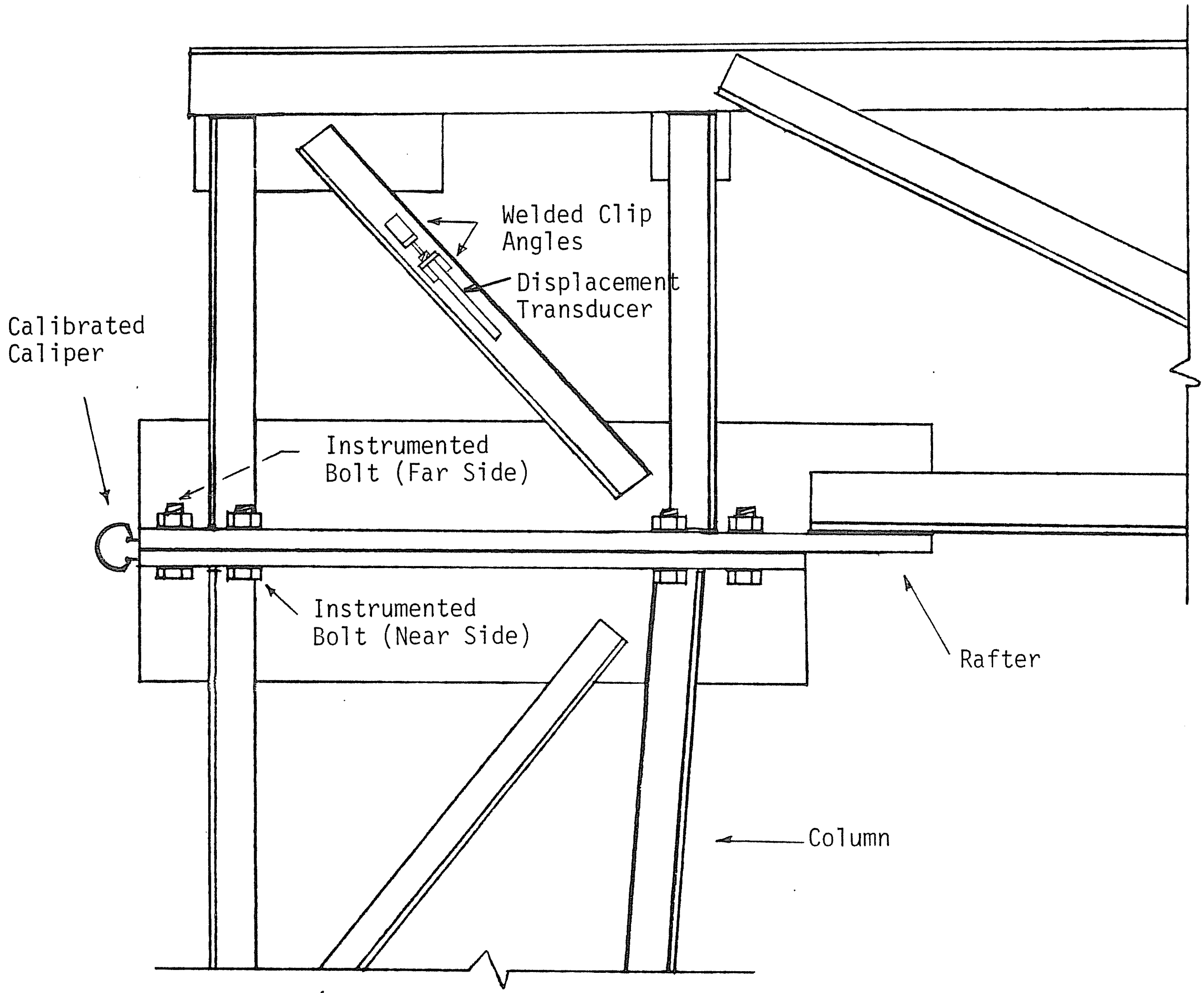


Figure 2.8 Instrumentation at Knee Area

instrumented bolts were pretensioned to 51 kips and zero readings were recorded. In general, load was applied in 1 kip increments until near the target load at which time the increment size was decreased. After each load increment, instrumentation readings were recorded and the specimens were checked for signs of yielding. Yielding was detected by flaking of the mill scale under the whitewash on the frame members. When specimens were no longer able to resist additional loading, the maximum load was recorded and then removed.

Six tests were conducted to verify the performance of the frames relative to analytical predictions for the following loading cases:

a) Full live load on the east frame, Figure 2.7(a). Loading was to the working level live load, 8.2 kips at each application point.
b) Unbalanced live load on the north slope of both frames simultaneously, Figure 2.7(b). Maximum loading was 1.5 times working level live load at each application point or 12.3 kips.
c) Lateral load applied to both frames simultaneously, Figure 2.7(c). Maximum load was 1.25 times working level wind load or 8.75 kips and was applied to the columns at the rafter bottom chord elevation.
d) Unbalanced live load on the south slope of both frames simultaneously, Figure 2.7(b). Maximum load was to be 1.67 times working level live load or 13.67 kips applied at application points on south rafters.
e) Unbalanced live load on windward side followed by

lateral load on both frames simultaneously, Figure 2.7(d). Maximum gravity load was 7 kips per application point and maximum lateral load was 8.75 kips.

f) Full live load to failure on west frame, Figure 2.7(a). Maximum load was 1.677 times working level live load or 13.77 kips at each application point.

For the case of unbalanced live load with lateral load, the simulated live load was applied first and then maintained while the lateral load was being applied in 1 kips increments.

2.6 Supplementary Tests

Upon completion of all testing, sections of undamaged column and rafter chord members were cut out of the west frame at the locations shown in Figure 2.9. The sections were then sent to the VULCRAFT, Norfolk, Nebraska facility where both angles from each member were tested using a specially designed angle test device mounted in a universal testing machine. The reported yield stress and tensile strengths from these tests are found in Section 3.8.

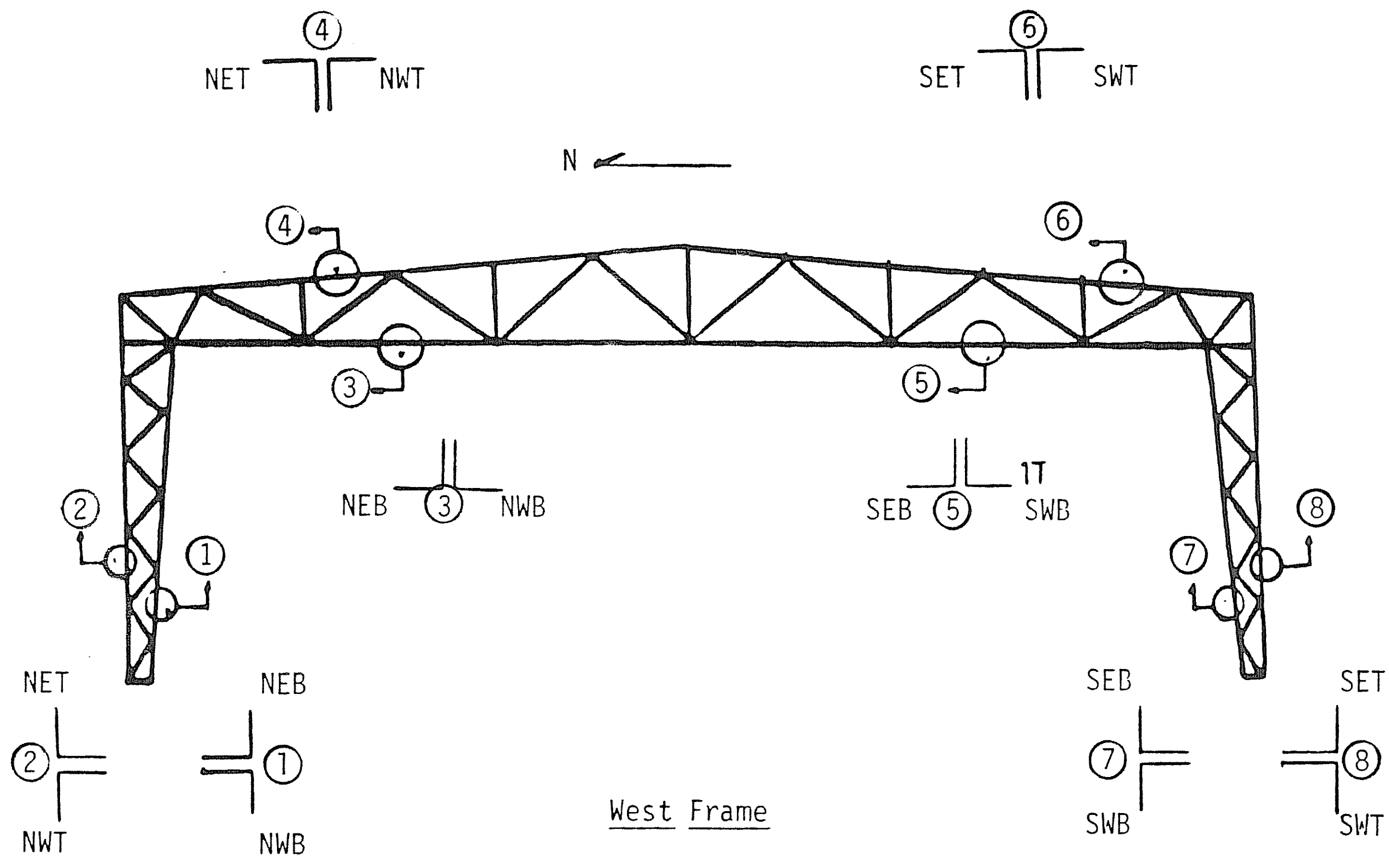


Figure 2.9 Tensile Test Coupon Specimen Locations

CHAPTER III

TEST RESULTS

3.1 Overview of Test Results

Test results consist of a test summary sheet, various load versus displacement and stress (converted from displacement measurements) plots, and a photographic record. Comparisons to predicted displacements and member forces are made using the results of stiffness analyses for each loading for each frame. Summaries of these analyses are found in Appendix A. The prediction curves are based on the assumption of a linear relationship between the applied load and the quantity plotted.

Predicted ultimate loads were obtained by multiplying the appropriate working load by 1.67. Working load levels were supplied by VULCRAFT and were based on the nominal yield stress of 50 ksi.

Detailed results are found for each loading in the appendices of this report (Appendices B thru G). Each of the appendices generally contains a test summary sheet, loading diagram, loading versus frame centerline and/or quarterpoint and sidesway deflections, load versus knee area web diagonal force, and load versus connection bolt force and connection plate separation plots.

Bolt forces are calculated from measured strains assuming a linear relationship based on the calibration curve, hence, the reported forces are not correct if bolt yielding occurs. The reported forces should be thought of as changes in strain not force.

3.2 Working Level Live Load, East Frame, Test LL

In this test full live load was applied to the east frame. The test is designated as LL. Loads were applied at eight points along the frame as shown in Figure 2.7(a). The loading sequence was initiated three times prior to the final test. Loading was halted during the first two subtests due to instrumentation problems. Once these problems were corrected load was applied and progressively increased to 7.9^k. At this point loading was terminated because local buckling was observed at the lower panel point of the diagonal web members terminating at the centerline (members 47 and 49 of Figure 2.1). In order to proceed with further loading a lateral brace was attached from the joist to the lower chord at centerline in the standard manner. Load was again applied and increased incrementally until the working level load of 8.2 kips was reached. Test results along with the theoretical predictions from VULCRAFT'S analysis program are contained in Appendix B.

As shown in Figure B.2, the final centerline load-deflection relationship was identical to the theoretical relationship prior to the application of 3 kips of test load. (The relationships for all three subtests are shown in Figure B.3). At loads greater than 3 kips the relationship remained linear but slightly greater deflections were observed than those predicted. The relationship during unloading was also linear with a permanent deflection of .039 in. remaining after complete removal of load.

In a similar manner, the vertical deflections of the frame's quarterpoints corresponded to the theoretical prediction until a load of approximately 2.5 kips was applied (Figures B.4 and B.5). At this point the observed deflections began to exceed the theoretical despite the fact that their load-deflection relationships remained linear. As expected, there was no distinguishable difference in the behavior of the north and south quarterpoints.

Data from the two instrumented connection bolts, designated northwest (NW) and southwest (SW) in Figure B.6, show no appreciable change due to loading from the 51 kips pretension force. The southeast (SE) bolt gradually increased in force with load application to a maximum force of 54.4 kips at a test load of 4.0 kips and then declined to a final force of 49.5 kips at 8.2 kips of test load. The force in the northeast (NE) bolt rapidly increased and achieved a value of 80.0 kips at the maximum test load of 8.2 kips.

The calculated forces in the knee diagonal members were in close agreement with the theoretical expectations. This force was obtained by converting the axial deformations recorded by the probe type displacement transducers to strains and, hence, to axial stress assuming a modulus of elasticity of 29,000 ksi. This stress multiplied by the member's cross-sectional area is the internal force in the member. Figure B.7 shows that the force in the north diagonal is virtually identical to the theoretical prediction. Although the force in the south diagonal is slightly greater than predicted, the difference is slight and the theoretical-experimental correlation is close, Figure B.8.

Due to instrumentation problems, plate separation data was not obtained for this test.

3.3 Unbalanced, Factored Live Load, Test 1.5ULL

For this test, the gravity load simulators were placed below the north slopes of both frames. Four concentrated loads were then applied to each slope as shown in Figure 2.7(b). The maximum loading level was 1.5 times the working load level or 12.3 kips per load application point. The test is designated as 1.5ULL and results are found in Appendix C. Two subtests were conducted.

To prevent premature failure of the diagonal web members at the centerline such as occurred in the previous test, the remaining diagonals were reinforced by welding angles along their lengths. This resulted in the cross-section of these members resembling a small channel section.

During the first subtest, load was applied in 1.0 kip increments until a load of 11 kips was attained. At this point the vertical web member originating at the connection of members 50 and 51 (Figure 2.1) buckled due to weld failure. The load was then removed and the buckled member replaced and the welds of the other three similar members were reinforced.

For this test, centerline and quarterpoint deflections were only monitored for the east frame. As shown in Figures C.2 and C.3, the measured load-centerline deflection relationships for the subtests agree closely with the theoretical predictions. Likewise, Figures C.4 and C.5 show that the measured load-deflection relationship for the north quarterpoint corresponds closely to the theoretical predictions. Such close agreement was not reflected in the results for the south quarterpoint, however. As depicted in Figures C.6 and C.7, the deflections at this point exceeded the theoretical values by a considerable margin. No explanation for the differences was found.

Figure C.8 shows that three of the instrumented bolts underwent the same slight change in tensile force during the first subtest. The northwest, southwest, and southeast bolts had final tensile forces of 51.4, 51.2, and 52.2 kips, respectively. The northeast bolt showed a substantially larger increase. At the 11.0 kip applied load level, the force was 68.4 kips. In the second subtest, the northwest and southwest bolts remained at the pretension level. The force in the northeast bolt increased immediately upon application of unblanaced loading and exceeded 80 kips at the end of the test. The southeast bolt force started to increase at approximately the 7 kips applied load level and reached approximately 65 kips at the end of the test.

The knee diagonal forces computed from the probe transducer data for the north knee are shown in Figures C.10 and C.11 for the subtests and are reasonably close to the predictions despite the erratic nature of the test load-member force relationships. At the final test load of 11.02 kips the member force was 25.8 kips which is 12% less than the 29.3 kip force predicted. Results for the south knee were erratic and are not included in Appendix C.

Finally, plate separation was virtually nonexistent for the monitored north plates. A maximum value of .001 in. was recorded (Figures C.12 and C.13).

3.4 Factored Wind Load, Test WL

In this test, simulated wind loading (lateral load) was applied by pulling both frames to the north at the level of the reentrant corner as shown in Figure 2.7(c). The maximum applied load level was 1.25 times the working load or 8.75 kips per frame. The test is designated as WL and two subtests were conducted. Test data is found in Appendix D.

The test was conducted as follows: the load was increased incrementally until the target load of 8.75 kips was applied to each frame, then removed and reapplied in the same manner. This loading did not cause noticeable distress in the frames. Vertical deflections at the centerline and the quarterpoints were negligible. However, sidesway deflections differed substantially from the theoretical predictions.

As shown in Figure D.2 both frames deflected essentially the same during the first subtest. At the maximum load of 8.75 kips, the deflections were 0.90" and 0.94" for the east and west frames, respectively. This represents increases of 67 and 75% over the predicted sidesway based on a fixed column base assumption. When the frames were unloaded a permanent deflection of 0.20 in. remained in the west frame and .23 in. in the east frame. (The erratic form of the measured curves is due to the measurement method, see Section 2.4).

Likewise, deflections in the second subtest exceeded the theoretical predictions (Figure D.3). However, the deflections of the individual frames differed substantially despite the fact that they both exhibited a greater stiffness than in the previous subtest. The deflections for the east and west frame at the maximum load of 8.75 kips were 0.78 in. and 0.66 in., respectively. These values exceed the theoretical values by 45% and 23%, respectively.

Bolt forces were measured at three locations. The data is depicted in Figure D.4 and shows the bolts undergoing a very modest but constant increase in tensile force. The final bolt forces at the maximum load of 8.75 kips were 53.1, 55.0, and 52.6 kips for the northwest, southwest and northeast bolts, respectively.

3.5 Unbalanced Full Factored Live Load, Test 1.67 ULL

This test was similar to Test 1.5ULL except that the load level was to be increased to 1.67 times working load or 13.67 kips per application point and the load was applied to the south rafters. The test is designated as 1.67ULL and the results are found in Appendix E.

The load was applied in an incremental manner as previously, however, the target load of 13.67 kips was not reached due to premature failure of a truss member. At a load of 12.6 kips the vertical web member of the west frame originating at the junction of members 45 and 46 (Figure 2.1) buckled. This buckling was accompanied by lateral movement of the adjacent top chord.

Vertical deflections were measured for the east frame. Prior to failure, the load-centerline deflection relationship was almost identical to the theoretical prediction, Figure E.2. At the failure load of 12.56 kips this deflection was 1.57 in. The deflections of the south quarter point were slightly in excess of the prediction, Figure E.3.

Sidesway displacements were recorded for both the east and west frame by reading scales attached to the frames at the north end as previously described. These deflections were substantially larger than predicted. Again, a fixed base connection was assumed in the stiffness analysis model. Examination of Figure E.4 shows that the load-lateral deflection relationship is somewhat erratic, particularly at lower loads probably due to the measurement technique. In the load interval from 6 to 12.6 kips, the load-deflection ratio is more consistent for each load increment. In this interval, the lateral stiffness is fairly consistent with respect to the theoretical prediction. It is at the lower loads (0 to 6 kips) that the unexpectedly larger deflections occurred. At 6 kips the deflections of the east and west frames were .30 and .25 in., respectively. These values exceed the theoretical prediction by 68 and 48%, respectively. Similarly, at the maximum load of 12.6 kips the displacements for the east and west frames were .49 and .44 in. which exceed the theoretical by 39 and 25%, respectively.

The bolt force data plotted in Figure E.5 indicates that the northwest and southwest bolts underwent a consistent but slight increase from the pretension force of 51 kips fo final values of 51.40 and 52.30 kips, respectively. The force in the northeast bolt also increased in this consistent, linear manner but had a final force of 56.65 kips at the maximum 12.56 kip test load. The remaining bolt (southeast) underwent an immediate and substantial decrease in force with only 3.80 kips being registered at the maximum test load. This behavior deviates so greatly from that expected that the possibility of erroneous data must be considered.

Examination of Figure E.6 shows that the calculated force in the north knee diagonal is consistent with the theoretical prediction. Although the force is slightly less than predicted for various test load increments, the final diagonal force is exactly that predicted. Such close agreement was not observed for the south diagonal, however. Figure E.7 shows the diagonal force is consistently less than that predicted. At the maximum test load the diagonal force was 25.75 kips which is 77% of that predicted.

After termination of the testing the buckled vertical member was replaced so that further tests could be executed.

3.6 Unbalanced Live Load plus Wind, Test ULL+WL

In this test gravity loading was first applied to the south rafter of each frame and then simulated wind loading was applied as shown in Figure 2.7(d). The gravity load was increased in one kip increments until a load 7 kips was achieved. The 7 kip load was then held constant while lateral loads were applied by pulling the frames toward the north at the level of the reentrant corner of the knee. When the lateral load on each frame reached 8.75 kips the test was terminated.

The test is designated ULL+WL and results are found in Appendix F. Vertical deflection measurements were made for the east frame only.

Load vs. centerline deflection data from this test shows a fairly close agreement with the theoretical prediction. As shown in Figure F.2, from 0 to 3.5 kips of gravity load, the deflections are identical to those predicted. From 3.5 to 7.0 kips there was slightly more deflection than expected. At the maximum gravity load of 7 kips the vertical centerline deflection was 0.90 which is 5% greater than predicted.

The measured centerline deflection due to the lateral load was also in good agreement with the predicted values. The maximum lateral load of 8.75 kips resulted in an additional vertical deflection of 0.042 in. over that due to the gravity load.

The close experimental-theoretical agreement for the centerline deflection was not reflected in the quarter point vertical deflections. As indicated by Figure F.3, the deflections of the south quarter point are consistently larger than those predicted. Although the deflection/ load ratio is consistent throughout the loading sequence, it has a larger value than determined theoretically for both the gravity and lateral loads. At the maximum gravity load of 7 kips the deflection was 0.89 in. which exceeds the theoretical value by 16%. The 8.75 kips of lateral load resulted in an additional deflection of 0.13 in. which is 84% greater than the expected increase. Likewise, the deflections at the north quarter point exceed the theoretical prediction, Figure F.4. With the application of 7.0 kips of gravity load the deflection was 0.85 in. which is 53% greater than the prediction. The additional deflection due to lateral loads was minimal (0.21 in.) as expected.

As in previous tests the lateral deflections of the frames substantially exceeded the theoretical predictions. Figure F.5 shows that both frames deflected approximately the same amount due to both the gravity and lateral loads. After the application of the maximum gravity load of 7 kips, the lateral deflection of the east frame was 0.32 in. and 0.27 in. was recorded for the west frame. These values exceed the theoretical by 63% and 37%, respectively. The addition of 8.75 kips of lateral load resulted in an increase of 0.72 in. in the lateral deflection of the east frame and 0.75 in. in the west frame. These values exceed the expected increase by 34% and 40%, respectively. The final total lateral deflections exceed those predicted by about 33%.

Figure F.6 shows that the southeast bolt underwent a very gradual increase in tensile force due to gravity load and then decreased very slightly during the lateral loading. The bolt force increased 1.36 kips above the 51 kip pretension force due to gravity load. During the same interval the force in the southwest bolt remained constant. With application of lateral load, the force in the southeast bolt decreased 1.0 kip, while the force in the southwest bolt remained constant. The northeast bolt showed a continuous but gradual rise in tensile force during both the gravity and lateral loadings. This bolt reached a maximum force of 58.4 kips with the final lateral load application. The remaining northwest bolt displayed a very erratic load-bolt force relationship compared to the other bolts. The rise in the bolt force due to gravity loading was gradual as in the previous bolts; a bolt force of 52.4 kips was registered at the maximum 7 kip gravity load. However, with the first increment of lateral loading there was an immediate and substantial rise in bolt force from 52.4 kips to 65.2 kips. During the five subsequent lateral load increments the bolt force increased only 0.33 kips with the application of 6 kips of lateral load, however, there was another large increase with the bolt force rising from 65.5 kips at 5 kips of lateral load to 74.3 kips at 6 kips of lateral load. This increased further to 89.0 kips at 7 kips of lateral load. For the remaining load increments the rise was more modest with a bolt force of 91.6 kips being registered at the final 8.75 kips of lateral load.

As demonstrated by Figure F.7 the test load knee diagonal force relationship for the northwest knee corresponded very closely to the theoretical prediction for both the gravity load and lateral load. In the southwest knee, however, the forces are consistently less than predicted, Figure F.8. With the application of the maximum gravity load the diagonal compressive force was 13.9 kips which is 26% less than predicted. At the maximum lateral load the force was 3.9 kips tension which differs 5.2 kips from the expected compressive force. Finally, plate separation data, available for the northwest plates only indicates a negligible separation. The maximum separation, occurring at the maximum lateral load, was 0.009 in.

3.7 Gravity Load Test to Failure of West Frame, Test 1.677

The purpose of this test was to fail the west frame under full simulated live loading, Figure 2.7(a). The test is designated 1.677LL and results are found in Appendix G.

The loading sequence used in this test differed somewhat from that previously employed. Initially, loads were applied in increments as before. However, when the test load reached 11.0 kips the load was then reduced incrementally to 6.0 kips. Then the load was once again increased until a final load of 13.77 kips was achieved. This loading sequence allowed for a more complete evaluation of the specimens behavior.

Figure G.2 indicates that the load-centerline deflection relationship conformed to the theoretical prediction prior to a test load of 8 kips. At 8 kips the deflections began to exceed those of the theoretical analysis. On reaching the 12.0 kips load level, the load-deflection relationship had become decidedly nonlinear. The 3.20 in. deflection at this point exceeds the 2.68 in. prediction by 19%. Loading was halted at this point and unloading was commenced at 1 kip increments until 6.0 kips of load remained.

The load-deflection relationship during the unloading sequence was linear. The deflections were in excess of those predicted, however, due to a permanent deflection resulting from the previous nonlinear behavior at the higher loads. Upon reapplication of the load the load-deflection relationship remained the same as that of the unloading sequence. Once the 12.0 kips test load was again reached, the load-deflection relationship began to be nonlinear. This behavior was accompanied by localized yielding in the region of the north reentrant corner as indicated by flaking of the whitewash. At 13.0 kips of test load buckling of the column flange angles beneath the reentrant corner was observed. At 13.77 kips, deflections increased without a further increase in load indicating the ultimate load level for this loading case.

The load-quarter point deflection data displayed in Figure G.3 shows that the deflections for both quarter points were virtually identical. The deflections, however, exceeded those predicted. In the test load range from 0 to 3 kips, there is close theoretical/experimental agreement. At higher loads, the deflections were greater than predicted. At approximately 12.5 kips, the deflections began to increase considerably with small increases in load. At 13.5 kips deflections increased with virtually no corresponding load increase.

The northeast, southwest and southeast bolts (Figures G.4 to G.6) underwent only modest increases in tensile force during the loading sequence. However, the northeast bolt appears to have sustained a substantial increase from 51.5 kips to 60.5 kips immediately prior to the application of the ultimate load (13.77 kips). A malfunction in the data acquisition system prevented the proper monitoring of the fourth bolt.

The forces in the knee diagonal computed from the probe data are somewhat erratic. Figure G.7 shows that the force in the north diagonal closely follows the prediction prior to a test load of 4.0 kips. At higher loads the force becomes larger than expected. At the 11 kip load, the force is once again close to the prediction. Unloading was commenced at this point and the diagonal force reduced to a level less than predicted. The phenomenon of less force than predicted for applied load was maintained throughout the reloading sequence until the final test loads were applied. This same general relationship of the experimental to theoretical forces existed in the south diagonal although it was slightly more exaggerated, Figure G.8. The measured diagonal force exceeded the theoretical throughout the first loading sequence. During the subsequent unloading and reloading sequence the forces were less than predicted.

Finally, the plate separation monitored for the north plates and depicted in Figure G.9 was minimal with a maximum value of 0.003 in. being recorded.

The maximum applied load in this test was 13.77 kips. Failure of the frame was due to lateral/local buckling of the column chord angles directly below the reentrant corner of the north column (Member 22 in Figure 2.1). Figure 3.1 is photographs of the failed member. The VULCRAFT supplied

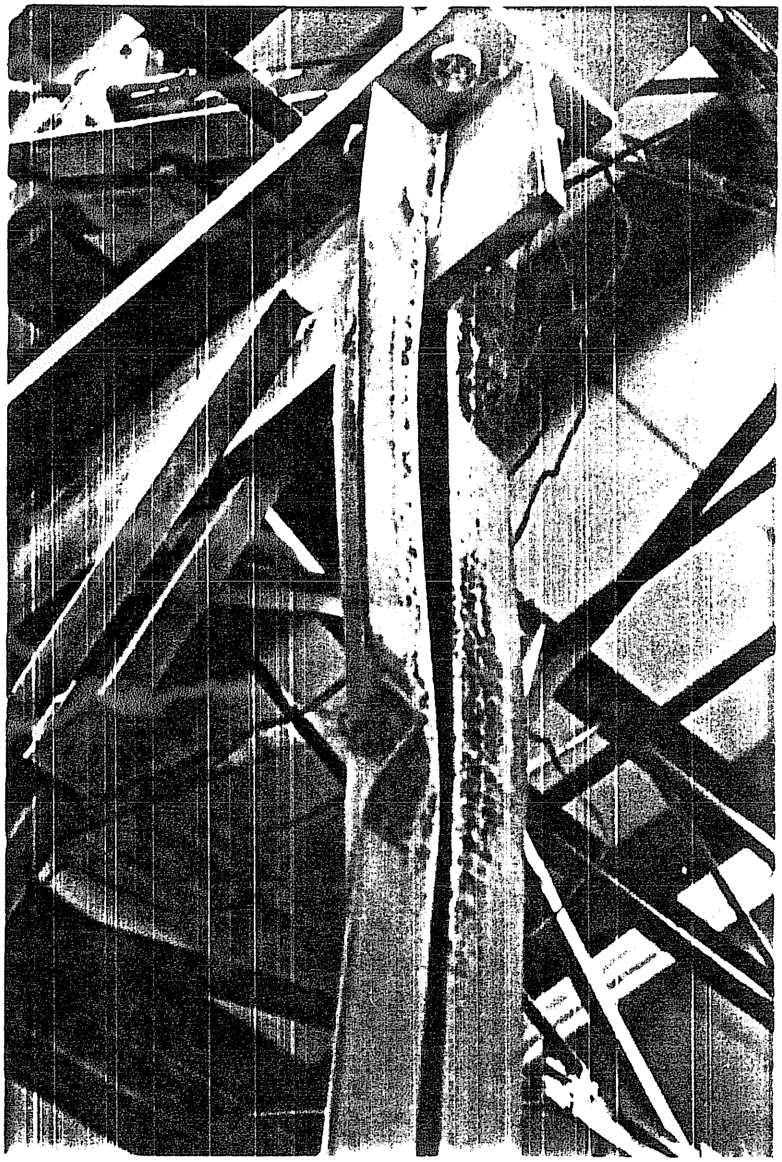


Figure 3.1 Photograph of Failed Column Chord Member

analysis for this loading case showed this member to be the controlling member. The combined stress ratio was 0.99 at the working load level of 8.2 kips. The resulting load factor is 1.68. However, the calculated combined stress ratio is based on the nominal yield stress of 50 ksi and the applied load does not include the weight of the loading apparatus approximately 0.2 kips per load application point.

3.8 Supplementary Test Results

Results of the tensile coupon tests are given in Table 3.1. For all samples, the average yield stress is 57.0 ksi and the average tensile strength is 80.3 ksi. For the inside column chords (location of failure), the average yield stress is 50.9 ksi.

Table 3.1

Tensile Coupon Test Results

Location	Nominal Section	Yield Stress (ksi)	Tensile Strength (ksi)
Outside Column Chord	L2x2x0.163	52.3	73.9
	" "	53.2	73.9
	" "	51.9	73.6
	" "	52.5	73.9
	Average	52.5	73.8
Inside Column Chord	L3x3x0.250	51.0	71.6
	" "	51.0	70.4
	" "	50.7	72.6
	" "	50.9	72.3
	Average	50.9	71.7
Bottom Rafter Chord	L3x3x0.227	64.4	83.0
	" "	64.0	94.7
	" "	64.5	90.9
	" "	64.7	92.9
	Average	64.4	90.4
Rafter Top Chord	L3-1/2x3-1/2x0.287	59.6	82.8
	" " "	60.7	87.1
	" " "	60.5	84.3
	" " "	59.0	86.8
	Average	60.0	85.3

CHAPTER IV

SUMMARY AND CONCLUSIONS

A series of tests was conducted on standard pre-engineered metal building frames fabricated by VULCRAFT, a division of Nucor Corporation. The test set-up consisted of two frames forming a single bay, 24 ft. 0 in. by 52 ft. 10 in. Joists, joist bridging, girts, chord braces and rod braces were used to construct the test set-up. The frames were subjected to a range of loadings, including unbalanced live load, lateral load, combined unbalanced live load and lateral load, and full live load. All loading conditions other than full live load were applied to the complete assembly. Full live load was applied to each frame individually.

Experimentally determined results were compared to values predicted by VULCRAFT'S stiffness analysis program. The vertical deflections at the frame's centerline predicted by the stiffness analysis agreed well with the measured deflections. In many of the tests, the agreement was "exact" and in no case was the deviation greater than 5%. The VULCRAFT analysis was less reliable in predicting the deflections at the quarter points of the frame. The analysis was reasonably accurate in predicting the quarter point deflections under full live load, as well as, the deflections of the quarter point of the loaded rafter in the unbalanced live load cases. The deflections of the quarter point of the unloaded portion of the rafter in these cases were consistently larger than predicted.

The tests clearly indicate that the frames have less lateral stiffness than indicated by the analyses which were based on the fixed base assumption. Lateral deflections exceeded the predicted values by as much as 80%.

Correlation between measured and predicted knee area diagonal member forces was very good for the first full live load test and reasonable for the second. Inconsistent results were obtained for the unbalanced live load cases. Good correlation was found in some of the unbalanced live load tests but substntial error was found in others. For the wind load case, the test load-diagonal force relationship was erratic for both of the knees monitored. The method of measuring these forces could account for the erratic results.

Only one test was conducted to failure: full live load on one frame. Both the experimental and predicted failure load and failure mode were in excellent agreement.

APPENDIX A

STIFFNESS ANALYSES

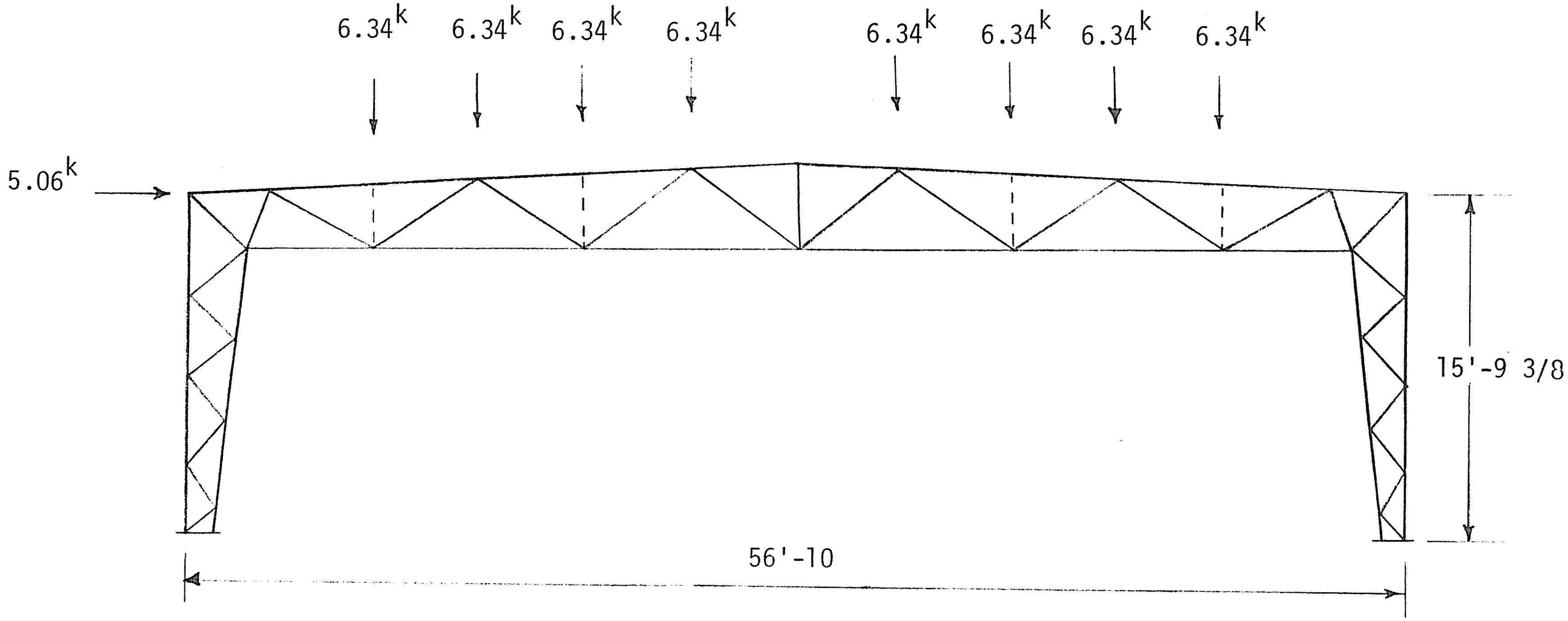


Figure A.1 Loads Used for Design

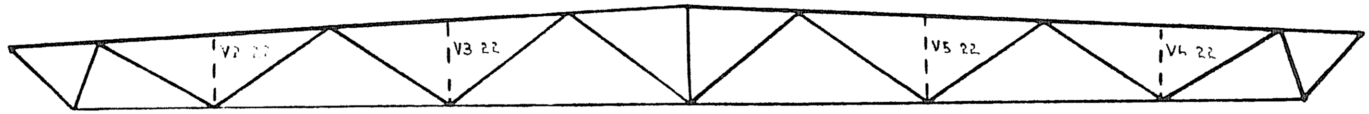
*Loads are based on 30 psf live load, 15 psf wind load, 5 psf dead load, 25'-6" bay:

Gravity Loads

$(30+5)(25.5)(56.83)/8 = 6340\#$

Wind Load

$(17.0-7.5/12)(25.5)(15) = 5068\#$



WEB DESIGN STEEL FY = 50000. PSI. MIN VERT SHEAR = 5131.8 LBS.
END REACTION = 20527.4 LBS.

WEB NO	DESCRIPTION	Q	K	KL/R	AX	REQ LOAD	DEV AX LOAD	REQ FILLET WELD SIZE	AVBL WELD	SLOT
2 *39	L2-1/2X2-1/2X.250	2	1.00	50.3	+	27537.+	71220.	17.1X0.000	12.2+	6.7B
V1 17	L1-1/2X1-1/2X.138	1	1.00	97.1	-	1831.-	6064.	2.0X0.138	4.2+	0.0
3 *31	L 2.00X2.00X0.232	2	1.00	47.3	-	19751.-	43278.	12.3X0.000	12.1+	3.4P
4 *39	L2-1/2X2-1/2X.250	2	1.00	86.6	+	41732.+	71220.	20.7X0.000	15.0+	6.7B
V2 22	L 2.00X2.00X0.1370	1	1.00	79.2	-	8205.-	9363.	4.0X0.137	4.9+	0.0
5 *43	L 3 X 3 X.250	2	1.00	74.1	-	20138.-	56450.	16.8X0.000	13.4+	6.7P
6 *22	L 2.00X2.00X0.1370	2	1.00	110.4	+	17363.+	31740.	12.7X0.000	14.8+	0.0B
V3 22	L 2.00X2.00X0.1370	1	1.00	91.8	-	8205.-	8264.	4.0X0.137	4.9+	0.0
7 *22	L 2.00X2.00X0.1370	2	1.00	114.5	-	9432.-	12049.	7.8X0.000	13.9+	0.0P
8 *22	L 2.00X2.00X0.1370	2	1.00	114.5	+	9432.+	31740.	5.7X0.000	13.9+	0.0B
V4 16	L1-1/2X1-1/2X.130	1	1.00	138.9	-	1831.-	2885.	2.0X0.130	4.2+	0.0
8R*22	L 2.00X2.00X0.1370	2	1.00	114.5	+	9432.+	31740.	5.7X0.000	13.9+	0.0P
7R*22	L 2.00X2.00X0.1370	2	1.00	114.5	-	9432.-	12049.	7.8X0.000	13.9+	0.0B
V5 22	L 2.00X2.00X0.1370	1	1.00	91.8	-	8205.-	8264.	4.0X0.137	4.9+	0.0
6R*22	L 2.00X2.00X0.1370	2	1.00	110.4	+	17363.+	31740.	12.7X0.000	14.8+	0.0P
5R*43	L 3 X 3 X.250	2	1.00	74.1	-	20138.-	56450.	16.8X0.000	13.4+	6.7B
V6 22	L 2.00X2.00X0.1370	1	1.00	79.2	-	8205.-	9363.	4.0X0.137	4.9+	0.0
4R*39	L2-1/2X2-1/2X.250	2	1.00	86.6	+	41732.+	71220.	20.7X0.000	15.0+	6.7P
3R*31	L 2.00X2.00X0.232	2	1.00	47.3	-	19751.-	43278.	12.3X0.000	12.1+	3.4B
V7 17	L1-1/2X1-1/2X.138	1	1.00	97.1	-	1831.-	6064.	2.0X0.138	4.2+	0.0
2R*39	L2-1/2X2-1/2X.250	2	1.00	50.3	+	27537.+	71220.	17.1X0.000	12.2+	6.7P

* - WEB SEC NUM, SINGLE OR DOUBLE SPECIFIED W/WO WELD OR FILLET.

BRIDGING REQUIREMENTS

TC RYY = 1.841, BC RYY = 1.640, WLEN/(TC RYY) = 356.794
WORKING LENGTH = 54.750 FT
MAXIMUM SPACING FOR BC BRACES = 32.80

Table A.1 Vertical Web Member Design

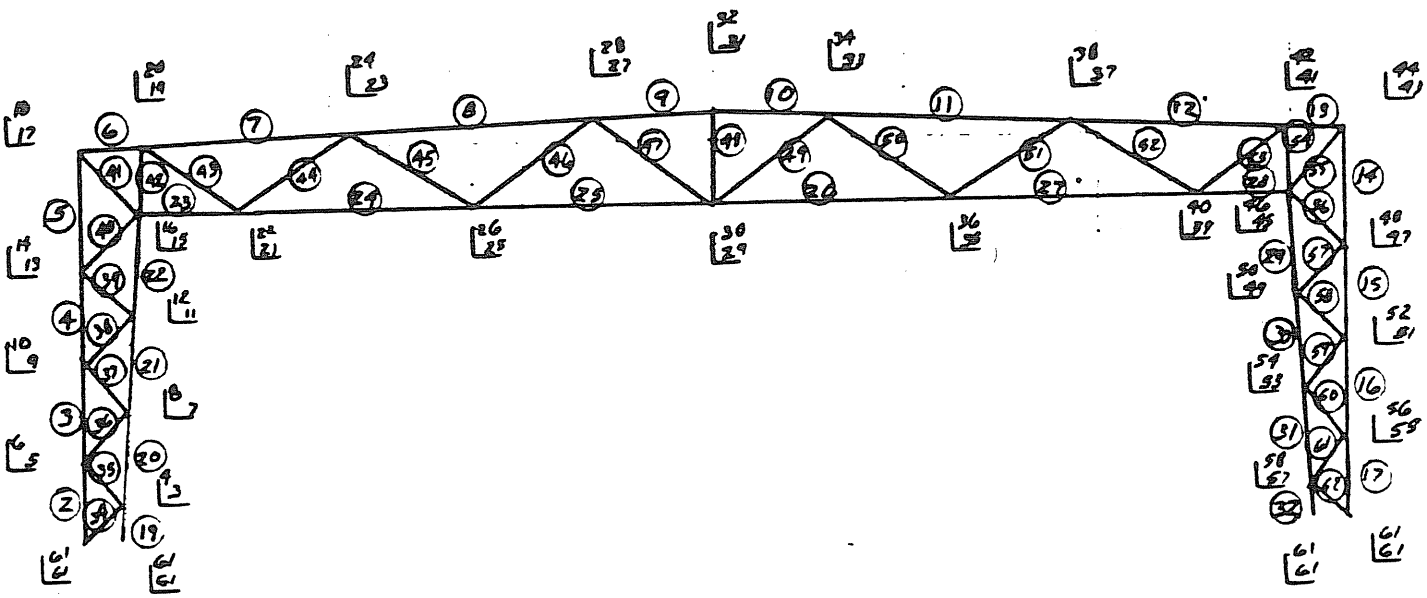


Figure A.2 Member and Degree of Freedom
Labelling

MEMBER	NP1	NP2	NP3	NP4	HORZ (IN)	VERT (IN)	LENGTH (IN)	AREA (IN**2)	SEC SIZE
2	61	61	5	6	0.00	40.87	40.87	1.250	25
3	5	6	9	10	0.00	48.00	48.00	1.250	25
4	9	10	13	14	0.00	48.00	48.00	1.250	25
5	13	14	17	18	0.00	54.06	54.06	1.250	25
6	17	18	19	20	30.50	1.27	30.53	3.854	51
7	19	20	23	24	106.50	4.44	106.59	3.854	51
8	23	24	27	28	120.00	5.00	120.10	3.854	51
9	27	28	31	32	60.00	2.50	60.05	3.854	51
10	31	32	33	34	60.00	-2.50	60.05	3.854	51
11	33	34	37	38	120.00	-5.00	120.10	3.854	51
12	37	38	41	42	106.50	-4.44	106.59	3.854	51
13	41	42	43	44	30.50	-1.27	30.53	3.854	51
14	47	48	43	44	0.00	54.06	54.06	1.250	25
15	51	52	47	48	0.00	48.00	48.00	1.250	25
16	55	56	51	52	0.00	48.00	48.00	1.250	25
17	61	61	55	56	0.00	40.87	40.87	1.250	25
19	61	61	3	4	2.10	16.87	17.00	2.874	43
20	3	4	7	8	5.97	48.00	48.37	2.874	43
21	7	8	11	12	5.97	48.00	48.37	2.874	43
22	11	12	15	16	5.97	48.00	48.37	2.874	43
23	15	16	21	22	47.00	0.00	47.00	2.620	42
24	21	22	25	26	120.00	0.00	120.00	2.620	42
25	25	26	29	30	120.00	0.00	120.00	2.620	42
26	29	30	35	36	120.00	0.00	120.00	2.620	42
27	35	36	39	40	120.00	0.00	120.00	2.620	42
28	39	40	45	46	47.00	0.00	47.00	2.620	42
29	45	46	49	50	5.97	-48.00	48.37	2.874	43
30	49	50	53	54	5.97	-48.00	48.37	2.874	43
31	53	54	57	58	5.97	-48.00	48.37	2.874	43
32	57	58	61	61	2.10	-16.87	17.00	2.874	43
34	61	61	3	4	12.10	16.87	20.76	0.888	19
35	5	6	3	4	12.10	-24.00	26.88	0.625	25
36	5	6	7	8	18.06	24.00	30.04	0.717	27
37	9	10	7	8	18.06	-24.00	30.04	0.373	16
38	9	10	11	12	24.03	24.00	33.96	0.529	22
39	13	14	11	12	24.03	-24.00	33.96	0.373	16
40	13	14	15	16	30.00	24.00	38.42	0.746	16
41	17	18	15	16	30.00	-30.06	42.47	2.374	39
42	15	16	19	20	0.50	31.33	31.34	1.434	27
43	19	20	21	22	46.50	-31.33	56.07	2.374	39
44	21	22	23	24	60.00	35.77	69.85	2.874	43
45	23	24	25	26	60.00	-35.77	69.85	1.058	22
46	25	26	27	28	60.00	40.77	72.54	1.058	22
47	27	28	29	30	60.00	-40.77	72.54	0.962	20
48	29	30	31	32	0.00	43.27	43.27	1.010	16
49	29	30	33	34	60.00	40.77	72.54	0.962	20
50	33	34	35	36	60.00	-40.77	72.54	1.058	22
51	35	36	37	38	60.00	35.77	69.85	1.058	22
52	37	38	39	40	60.00	-35.77	69.85	2.874	43
53	39	40	41	42	46.50	31.33	56.07	2.374	39
54	41	42	45	46	0.50	-31.33	31.34	1.434	27
55	45	46	43	44	30.00	30.06	42.47	2.374	39
56	45	46	47	48	30.00	-24.00	38.42	0.746	16
57	49	50	47	48	24.03	24.00	33.96	0.373	16
58	49	50	51	52	24.03	-24.00	33.96	0.529	22
59	53	54	51	52	18.06	24.00	30.04	0.373	16
60	53	54	55	56	18.06	-24.00	30.04	0.717	27
61	57	58	55	56	12.10	24.00	26.88	0.625	25
62	57	58	61	61	12.10	-16.87	20.76	0.888	19

Table A.2 Member Geometry

NP=	1	0.00
~~NP=~~	~~2~~	~~0.00~~
NP=	3	0.00
NP=	4	0.00
~~NP=~~	~~5~~	~~0.00~~
NP=	6	0.00
NP=	7	0.00
~~NP=~~	~~8~~	~~0.00~~
NP=	9	0.00
NP=	10	0.00
~~NP=~~	~~11~~	~~0.00~~
NP=	12	0.00
NP=	13	0.00
~~NP=~~	~~14~~	~~0.00~~
NP=	15	0.00
NP=	16	0.00
~~NP=~~	~~17~~	~~0.00~~
NP=	18	0.00
NP=	19	0.00
~~NP=~~	~~20~~	~~0.00~~
NP=	21	0.00
NP=	22	0.00
~~NP=~~	~~23~~	~~0.00~~
NP=	24	0.00
NP=	25	0.00
~~NP=~~	~~26~~	~~0.00~~
NP=	27	0.00
NP=	28	0.00
~~NP=~~	~~29~~	~~0.00~~
NP=	30	0.00
NP=	31	0.00
NP=	32	0.00
NP=	33	0.00
~~NP=~~	~~34~~	~~0.00~~
NP=	35	0.00
NP=	36	0.00
~~NP=~~	~~37~~	~~0.00~~
NP=	38	0.00
NP=	39	0.00
~~NP=~~	~~40~~	~~0.00~~
NP=	41	0.00
NP=	42	0.00
~~NP=~~	~~43~~	~~0.00~~
NP=	44	0.00
NP=	45	7.00
~~NP=~~	~~46~~	~~0.00~~
NP=	47	0.00
NP=	48	0.00
~~NP=~~	~~49~~	~~0.00~~
NP=	50	0.00
NP=	51	0.00
~~NP=~~	~~52~~	~~0.00~~
NP=	53	0.00
NP=	54	0.00
~~NP=~~	~~55~~	~~0.00~~
NP=	56	0.00
NP=	57	0.00
~~NP=~~	~~58~~	~~0.00~~
NP=	59	0.00
NP=	60	0.00

Table A.3 Wind Load for Analysis, kips

~~NX =~~	~~1~~	~~0.00~~
NX =	2	0.00
NX =	3	0.02
~~NX =~~	~~4~~	~~-0.01~~
NX =	5	0.09
NX =	6	0.01
~~NX =~~	~~7~~	~~0.16~~
NX =	8	-0.03
NX =	9	0.23
~~NX =~~	~~10~~	~~0.01~~
NX =	11	0.29
NX =	12	-0.04
~~NX =~~	~~13~~	~~0.35~~
NX =	14	0.01
NX =	15	0.40
~~NX =~~	~~16~~	~~-0.05~~
NX =	17	0.43
NX =	18	-0.01
~~NX =~~	~~19~~	~~0.42~~
NX =	20	-0.05
NX =	21	0.41
~~NX =~~	~~22~~	~~-0.07~~
NX =	23	0.42
NX =	24	-0.08
~~NX =~~	~~25~~	~~0.42~~
NX =	26	-0.07
NX =	27	0.41
NX =	28	-0.05
NX =	29	0.43
~~NX =~~	~~30~~	~~-0.02~~
NX =	31	0.41
NX =	32	-0.02
~~NX =~~	~~33~~	~~0.42~~
NX =	34	0.00
NX =	35	0.43
~~NX =~~	~~36~~	~~0.03~~
NX =	37	0.42
NX =	38	0.05
~~NX =~~	~~39~~	~~0.42~~
NX =	40	0.06
NX =	41	0.43
~~NX =~~	~~42~~	~~0.05~~
NX =	43	0.43
NX =	44	0.02
~~NX =~~	~~45~~	~~0.42~~
NX =	46	0.05
NX =	47	0.37
~~NX =~~	~~48~~	~~-0.00~~
NX =	49	0.31
NX =	50	0.04
~~NX =~~	~~51~~	~~0.25~~
NX =	52	-0.01
NX =	53	0.17
~~NX =~~	~~54~~	~~0.03~~
NX =	55	0.10
NX =	56	-0.01
~~NX =~~	~~57~~	~~0.03~~
NX =	58	0.01
NX =	59	0.00
~~NX =~~	~~60~~	~~0.00~~

Table A.4 Deflections from Wind Load Analysis, in.

MEMBER	AXIAL FORCE
1	0.00
2	12.10
3	-0.11
4	-6.25
5	-9.95
6	-9.55
7	-7.34
8	-2.74
9	0.81
10	0.81
11	4.56
12	9.45
13	11.79
14	12.29
15	8.11
16	1.18
17	-12.60
18	0.00
19	-18.84
20	-3.66
21	4.81
22	9.54
23	12.75
24	8.11
25	4.09
26	0.67
27	-3.59
28	-8.51
29	-11.69
30	-6.35
31	3.21
32	20.34
33	0.00
34	9.54
35	-8.19
36	6.13
37	-4.39
38	3.73
39	-2.91
40	2.63
41	13.50
42	1.56
43	-2.63
44	2.87
45	-2.49
46	2.27
47	-2.01
48	-0.07
49	2.13
50	-2.41
51	2.64
52	-3.04
53	2.79
54	-1.65
55	-16.67
56	-2.97
57	3.28
58	-4.21
59	4.96
60	-6.92
61	9.24
62	-10.77
63	0.00

Table A.5 Member Forces from Wind Load Analysis, kips

NP=	1	0.00
NP=	2	0.00
NP=	3	0.00
NP=	4	0.00
NP=	5	0.00
NP=	6	0.00
NP=	7	0.00
NP=	8	0.00
NP=	9	0.00
NP=	10	0.00
NP=	11	0.00
NP=	12	0.00
NP=	13	0.00
NP=	14	0.00
NP=	15	0.00
NP=	16	0.00
NP=	17	0.00
NP=	18	0.00
NP=	19	0.00
NP=	20	0.00
NP=	21	0.00
NP=	22	3.20
NP=	23	0.00
NP=	24	5.20
NP=	25	0.00
NP=	26	[illegible].20
NP=	27	0.00
NP=	28	6.20
NP=	29	0.00
NP=	30	0.00
=	[illegible]1	[illegible]
NP=	32	0.00
NP=	33	0.00
NP=	34	0.64
NP=	35	0.00
NP=	36	0.64
NP=	37	0.00
NP=	38	0.64
NP=	39	0.00
NP=	40	0.64
NP=	41	0.00
NP=	42	0.00
NP=	43	0.00
NP=	44	0.00
NP=	45	0.00
NP=	46	0.00
NP=	47	0.00
NP=	48	0.00
NP=	49	0.00
NP=	50	0.00
NP=	51	0.00
NP=	52	0.00
NP=	53	0.00
NP=	54	0.00
NP=	55	0.00
NP=	56	0.00
NP=	57	0.00
NP=	58	0.00
NP=	59	0.00
NP=	60	0.00

Table A.6 Unbalanced Live Loads for Analysis, kips

NX	=	1	0.00
NX	=	2	0.00
NX	=	3	-0.01
NX	=	4	-0.00
NX	=	5	-0.04
NX	=	6	-0.00
NX	=	7	-0.04
NX	=	8	-0.02
NX	=	9	-0.03
NX	=	10	0.01
NX	=	11	0.01
NX	=	12	-0.04
NX	=	13	0.06
NX	=	14	0.02
NX	=	15	0.14
NX	=	16	-0.08
NX	=	17	0.29
NX	=	18	0.05
NX	=	19	0.30
NX	=	20	-0.10
NX	=	21	0.13
NX	=	22	-0.42
NX	=	23	0.31
NX	=	24	-0.77
NX	=	25	0.19
NX	=	26	-1.03
NX	=	28	-1.10
NX	=	29	0.27
NX	=	30	-1.00
NX	=	31	0.23
NX	=	32	-0.99
NX	=	33	0.21
NX	=	34	-0.86
NX	=	35	0.31
NX	=	36	-0.65
NX	=	37	0.20
NX	=	38	-0.40
NX	=	39	0.32
NX	=	40	-0.17
NX	=	41	0.23
NX	=	42	-0.00
NX	=	43	0.23
NX	=	44	0.05
NX	=	45	0.30
NX	=	46	0.00
NX	=	47	0.30
NX	=	48	0.02
NX	=	49	0.28
NX	=	50	0.02
NX	=	51	0.24
NX	=	52	-0.00
NX	=	53	0.18
NX	=	54	0.02
NX	=	55	0.11
NX	=	56	-0.01
NX	=	57	0.03
NX	=	58	0.01
NX	=	59	0.00
NX	=	60	0.00

Table A.7 Deflections from Unbalanced Live Load Analysis, in.

MEMBER		AXIAL FORCE
~~1~~		~~0.00~~
2		-3.67
3		7.20
4		12.76
5		16.08
6		15.43
7	57.13(1.6)= [illegible]	-21.22
8	71.95	-59.19
9		-42.86
10		-42.85
11		[illegible]
13		20.91
14		21.80
15		16.13
16		6.71
17		-12.00
~~18~~		~~0.00~~
19		-14.78
20	FDEV	-28.38
21	= -65.46	-35.96
22		-40.19
23		-21.81
24		41.57
25	FDEV	49.72
26	= -59.95	27.59
27		3.90
28		-27.52
29		-29.5[illegible]
30		-22.33
31		-9.35
32		13.92
~~33~~		~~0.00~~
34	21.5, 5.8	-8.54
35		7.33
36		-5.49
37		3.93
38		-3.34
39		2.60
40		-2.36
41		-21.82
42	-35.4	-25.93
43		43.66
44		-31.63
45		12.53
46	FDEV 7	3.17
47	-12.05, 31.74	-16.55
48		3.57
49		10.20
50	-12.0, 31.7	-12.74
51		15.23
52		-18.50
53		13.43
54		-10.94
55		-29.57
56		-4.04
57		4.45
58		-5.72
59		6.73
60		-9.39
61		12.55
62		-14.62
~~63~~		~~0.00~~

Table A.8 Member Forces from Unbalanced Live Load Analysis, kips

Table A.1

VULCRAFT Steel Inventory
(Brigham City, Utah)

SEC	ANGLE	AREA	RX	RZ	RY-1	SL	SS	Y	Q	IXX	WELD
3	1.00X.109	0.206	0.3066	0.1962	0.8466	0.067	0.027	0.290	1.0000	0.019	1619
8	1.25X.111	0.265	0.3880	0.2500	0.9373	0.113	0.064	0.353	0.9841	0.060	1668
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	1.0296	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	1.0363	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	0.089	0.433	1.0000	0.094	1856
20	1.50X.170	0.481	0.4590	0.2960	1.0430	0.232	0.095	0.437	1.0000	0.101	2060
22	2.00X.137	0.529	0.6265	0.3957	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	1.2295	0.430	0.167	0.560	0.9522	0.241	1856
26	2.00X.175	0.669	0.6191	0.3944	1.2322	0.456	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	0.579	1.0000	0.306	2320
[illegible]	2.00X.232	0.876	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	0.4963	1.4279	0.819	0.317	0.695	0.9389	0.571	2320
36	2.50X.212	1.015	0.7760	0.4930	1.4309	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	0.656	2487
39	2.50X.250	1.187	0.7690	0.4910	1.4399	0.980	0.394	0.717	1.0000	0.703	2786
42	3.00X.227	1.310	0.9330	0.5930	1.6293	1.369	0.527	0.836	0.9223	1.142	2662
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	1.616	0.642	0.856	1.0000	1.378	3266
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	1.8416	2.475	0.976	0.990	0.9866	2.450	3712
56	3.50X.344	2.290	1.0781	0.6833	1.8425	2.657	1.065	1.002	1.0000	2.661	4179
57	3.50X.375	2.486	1.0700	0.6870	1.8535	2.842	1.150	1.010	1.0000	2.870	4660
59	4.00X.375	2.859	1.2300	0.7880	2.0530	3.825	1.520	1.140	1.0000	4.360	4660
61	4.00X.438	3.312	1.2300	0.7850	2.0631	4.284	1.750	1.140	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	0.9850	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	2.4864	8.542	3.512	1.457	1.0000	12.445	5568
	6.00X.500	5.750	1.8500	1.1800	2.8659	11.365	4.610	1.680	0.9607	19.900	5568
69	6.00X.563	6.439	1.8500	1.1900	2.8838	12.923	5.140	1.710	1.0000	22.100	5568
70	6.00X.625	7.109	1.8400	1.1900	2.8963	13.938	5.660	1.730	1.0000	26.200	5568
71	6.00X.688	7.772	1.8430	1.1800	2.9061	16.930	6.160	1.750	1.0000	26.200	5568
72	6.00X.750	8.438	1.8300	1.1700	2.9226	19.843	6.660	1.780	1.0000	28.200	5568

APPENDIX B

WORKING LEVEL FULL LIVE LOAD

(TEST LL)

VULCRAFT FRAME TEST SUMMARY

Project: Vulcraft

Test No.: FR-1

Test Date: 7/25/84

Purpose: Test of working level live load

Maximum Test Load: 8.19

Failure Mode:

Discussion:

- The loading sequence was initiated three times prior to the final test run. Loading was halted during the first two due to instrumentation problems. Once these problems were corrected, load was applied and progressively increased to 7.94 kips. At this point loading was terminated because local buckling was observed at the lower panel point of the diagonal web member at the centerline. This buckling was accompanied by substantial lateral deflection (.40 in.).

- In order to proceed with further loading a lateral brace was attached to the lower chord at the centerline. Once this improvement was made, load was applied and increased incrementally until the working load level of 8.2 kips was reached. Centerline vertical deflection data from this test run shows the experimental load-deflection relationship to be identical to theoretical up to 3 kips.

- At loads greater than 3 kips the relationship remained linear but had slightly less slope than the theoretical. The load deflection relationship during unloading was also linear with a permanent deflection of .039 in. remaining after the removal of load.

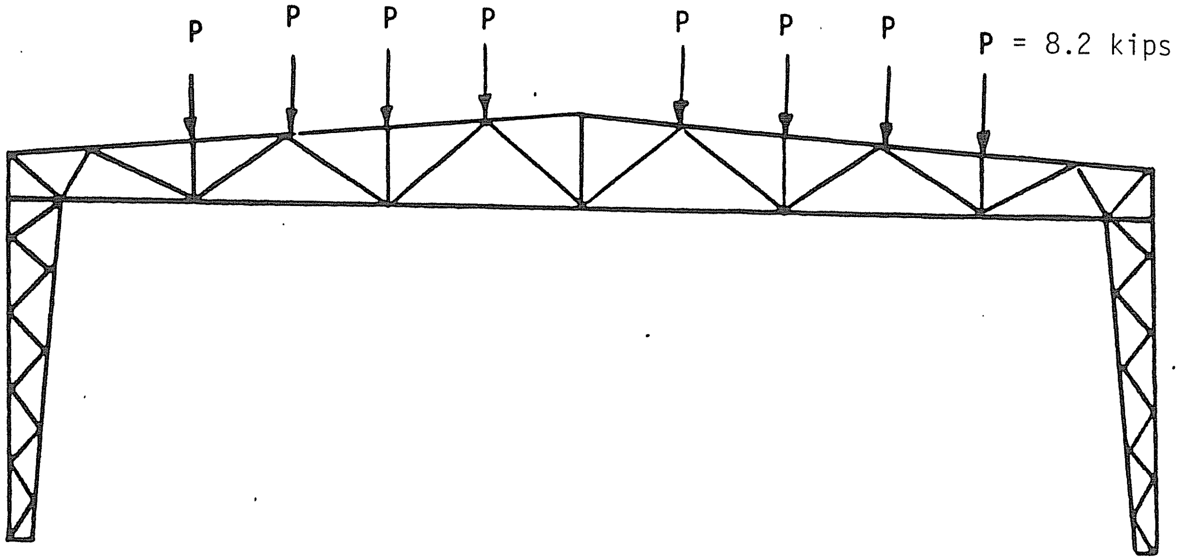


Figure B.1 Full Live Load

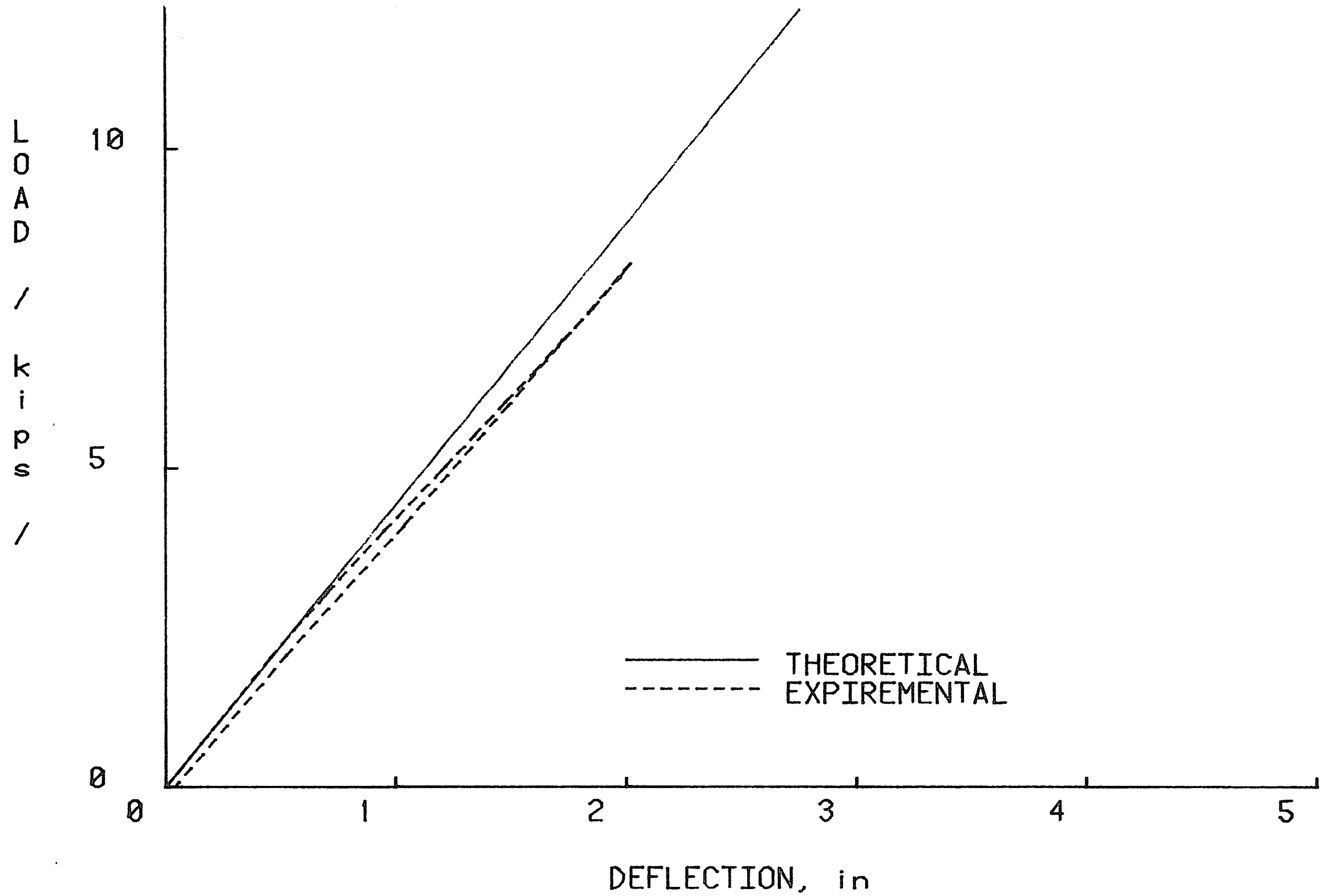


Figure B.2 LOAD vs CENTERLINE DEFLECTION, LL

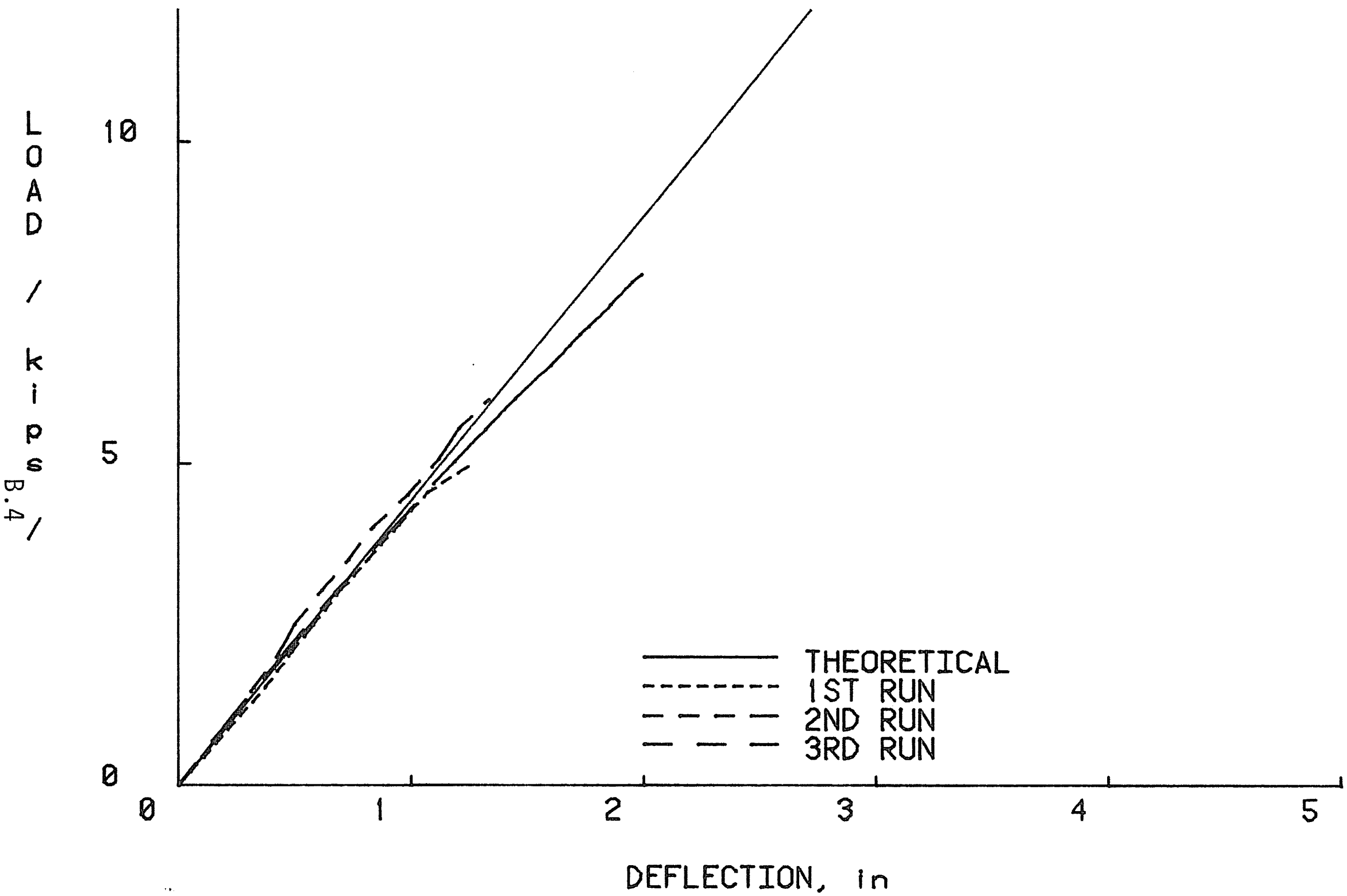


Figure B.3 LOAD vs CENTERLINE DEFLECTION, LL

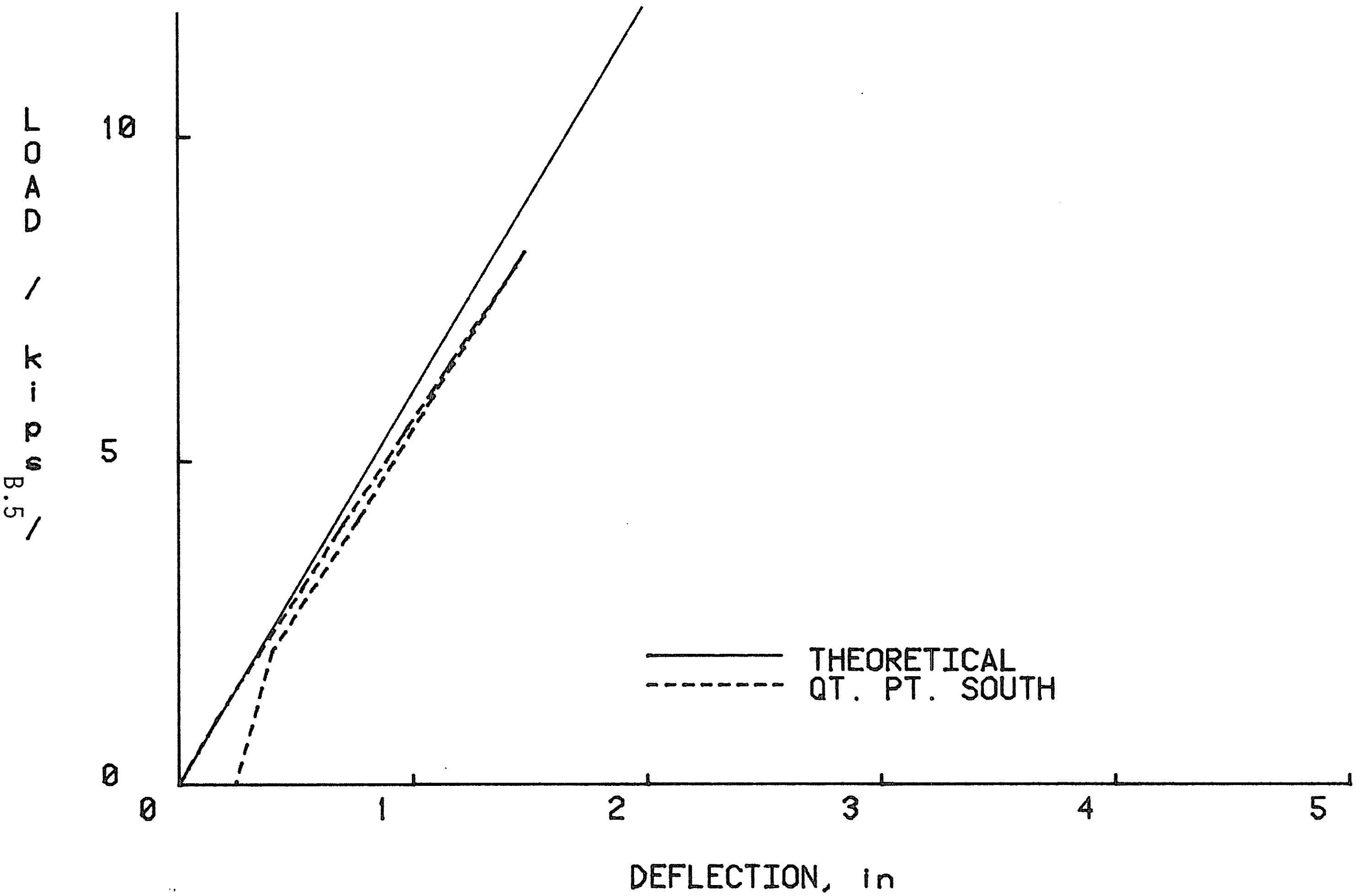


Figure B.4 LOAD vs QUARTERPOINT DEFLECTION, LL: SOUTH

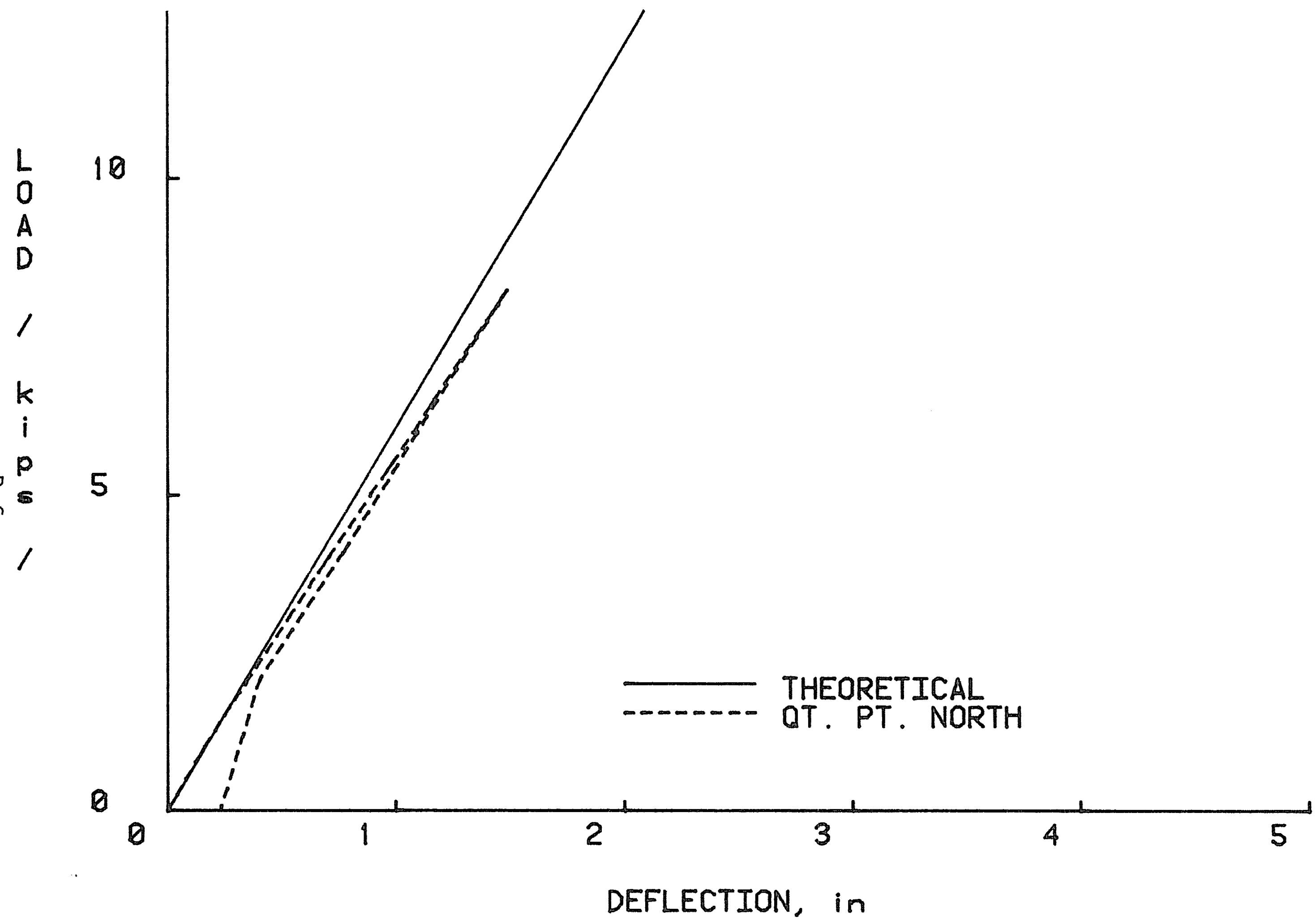


Figure B.5 LOAD vs QUARTERPOINT DEFLECTION, LL: NORTH

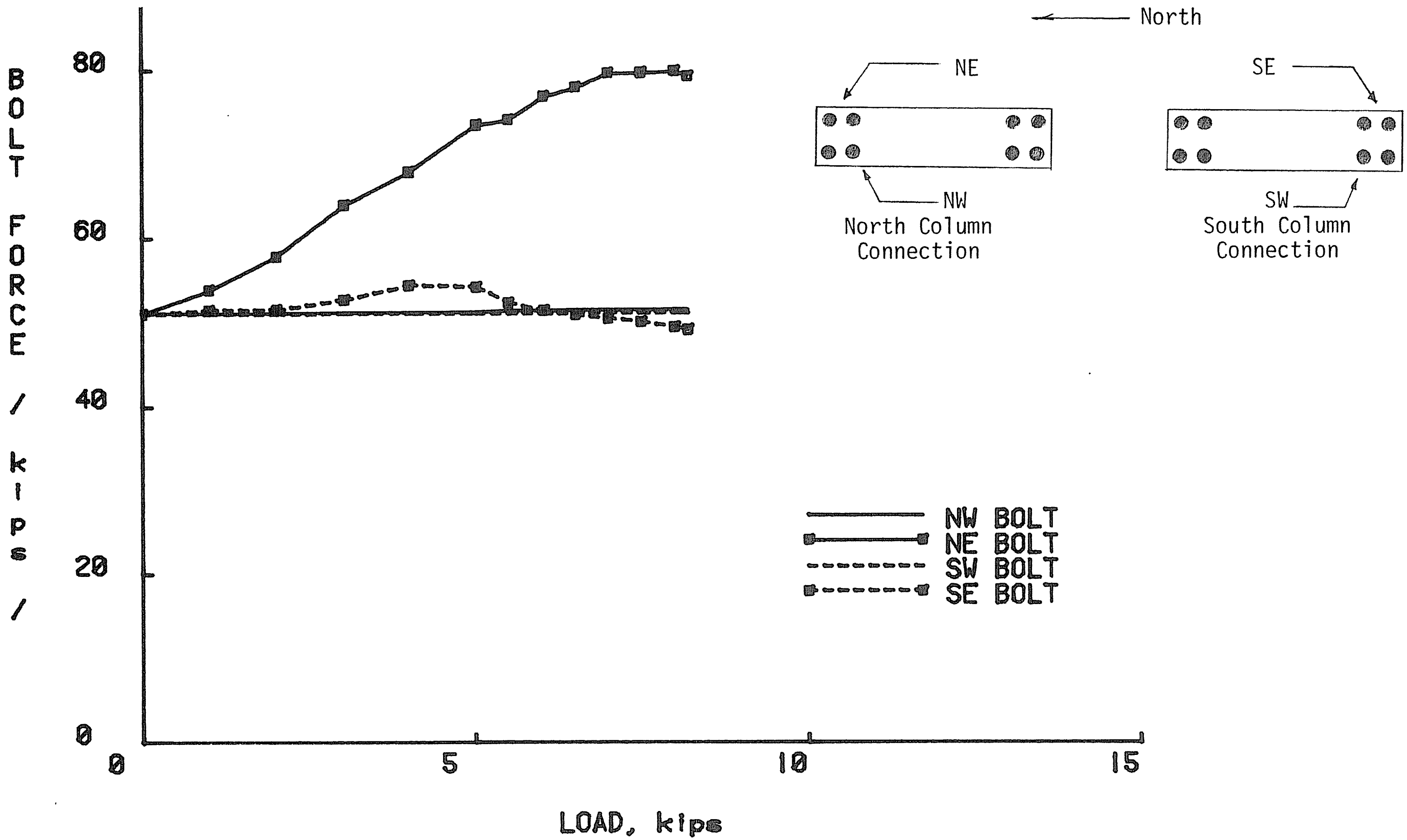


Figure B.6 BOLT FORCE vs LOAD, LL

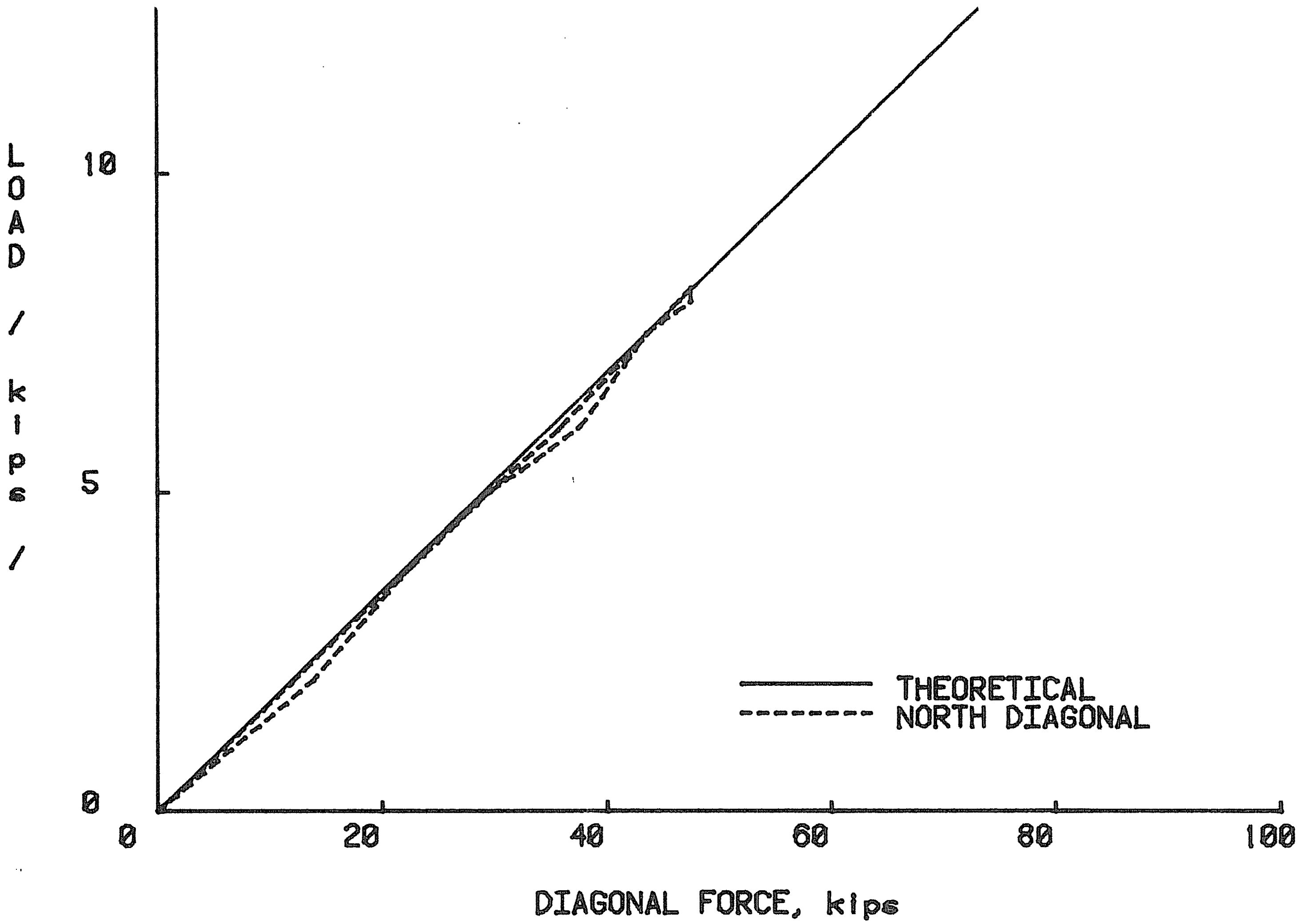


Figure B.7: LOAD vs KNEE DIAGONAL FORCE, LL: NORTH

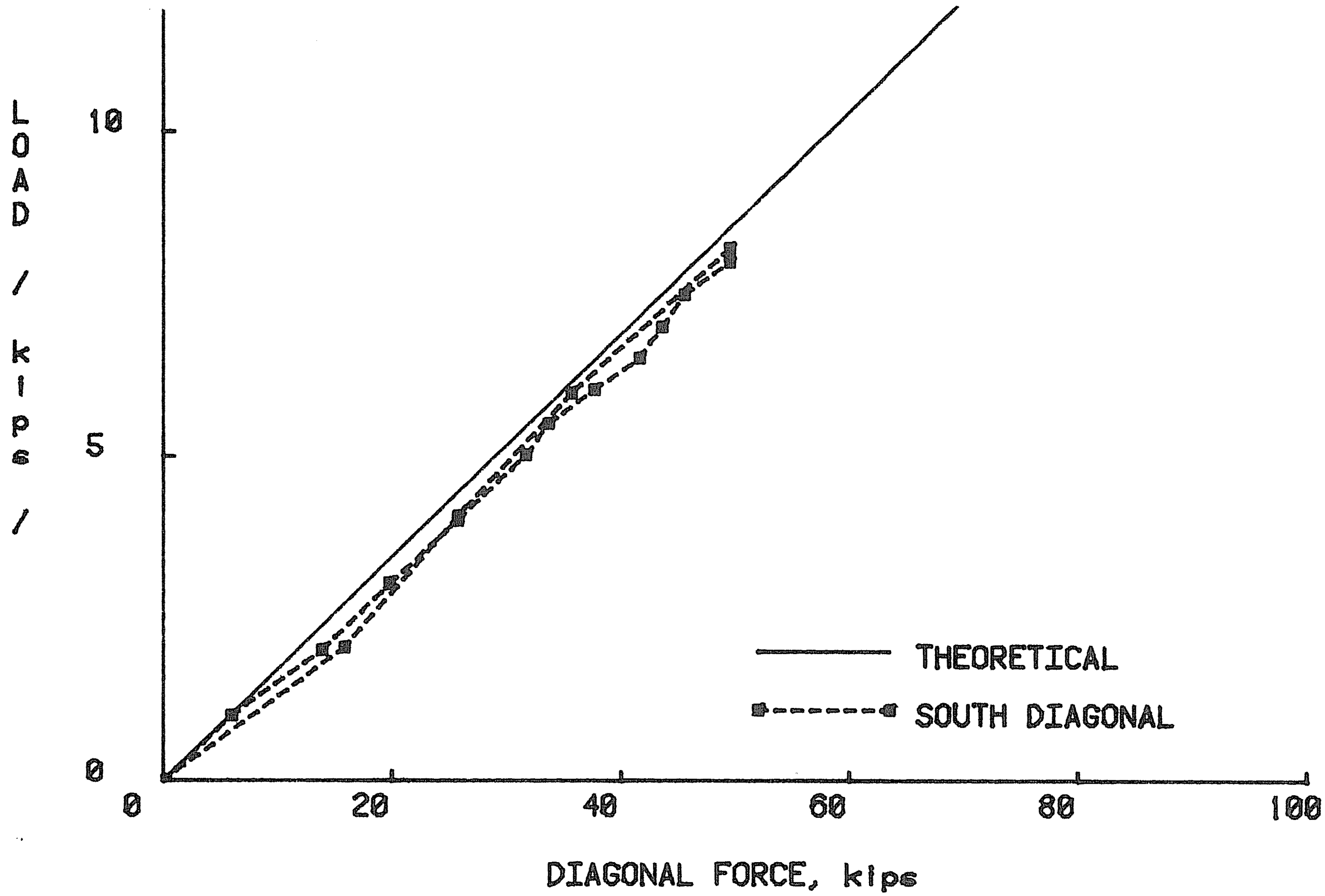


Figure B.8 LOAD vs KNEE DIAGONAL FORCE, LL: SOUTH

APPENDIX C

UNBALANCED, FACTORED LIVE LOAD

(TEST 1.5ULL)

VULCRAFT FRAME TEST SUMMARY

Project: Vulcraft

Test No.: FR-1

Test Date: 7/30/84

Purpose: Test of 1.5x working level unbalanced live load (DL + ULL)

Maximum Test Load: 12.40 kips

Failure Mode:

Discussion:

- To prevent premature failure such as that occurring in the live load test the centerline diagonals were reinforced by welding angles along their lengths. Still, unexpected failure of a vertical truss member required the execution of two test runs. For the 1st run centerline deflection data indicates that the load displacement-relationship is in close agreement with the theoretical prediction prior to the failure of the vertical member. When the load reached 11 kips, the vertical member originating at the connection of members 50 and 51 (Figure 1) buckled. Loading was then halted to effect repairs; the buckled member was replaced and the welds of all such members were improved.

- The loading sequence was again initiated and pursued until the target load was achieved. Data from this loading also indicates close agreement between the experimental load-deflection relationship and the theoretical.

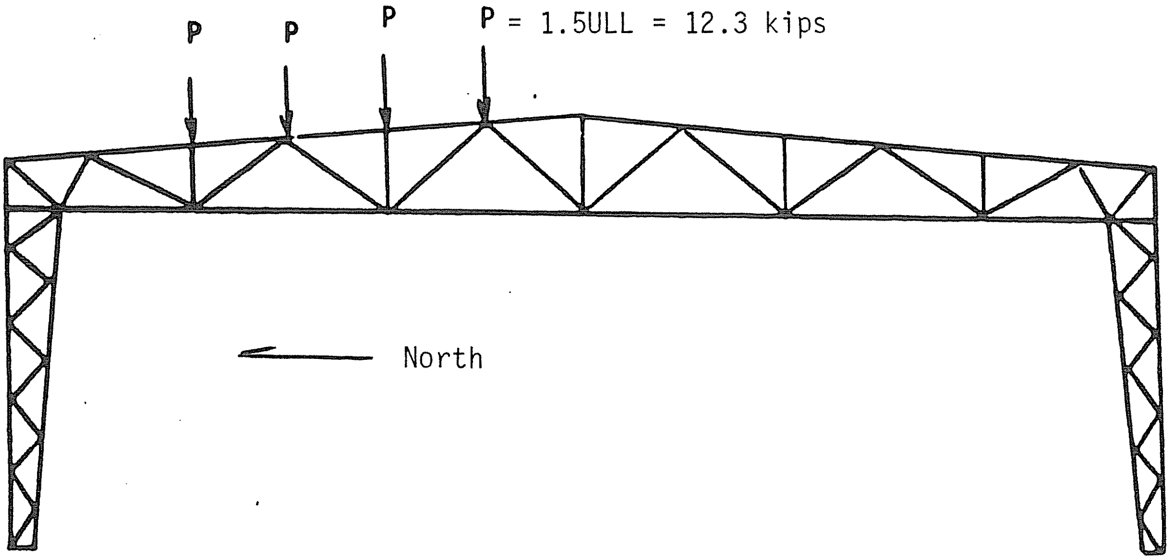


Figure C.1 Unbalanced Live Load

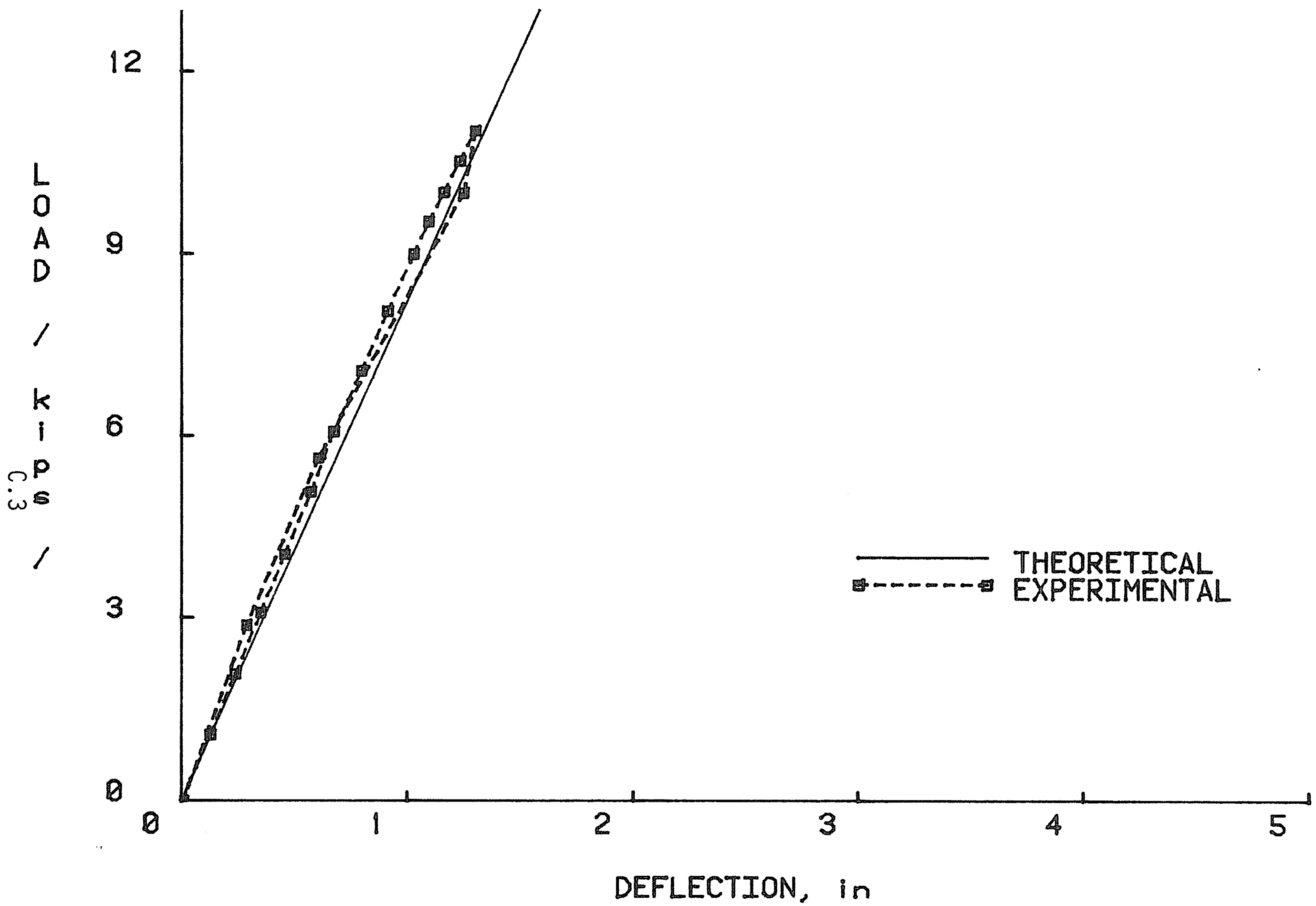


Figure C.2 LOAD vs CENTERLINE DEFLECTION, 1.5ULL: 1st run

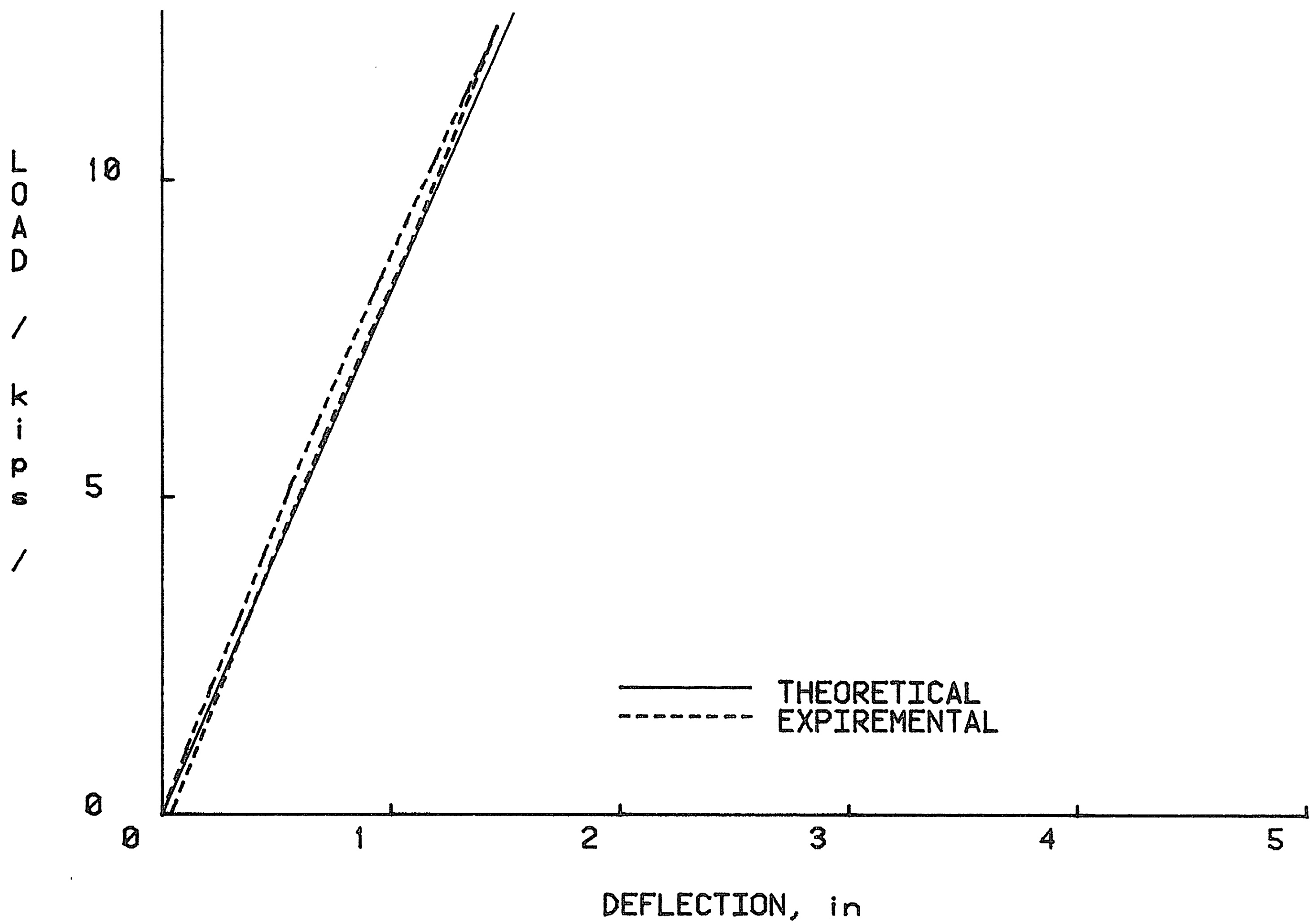


Figure C.3 LOAD vs CENTERLINE DEFLECTION, 1.5ULL: 2nd run

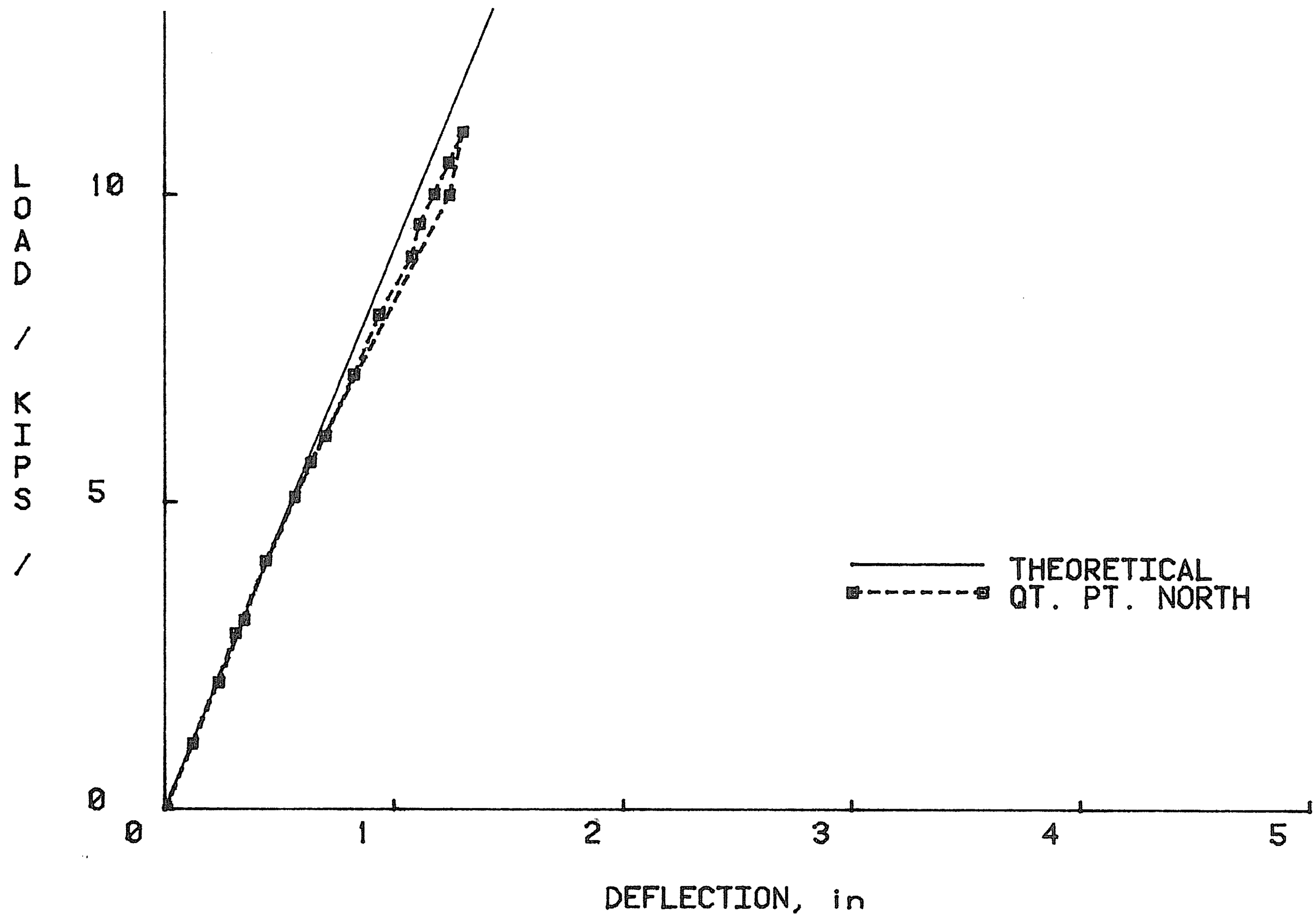


Figure C.4 LOAD vs QUARTERPOINT DEFLECTION, 1.5ULL: 1st run

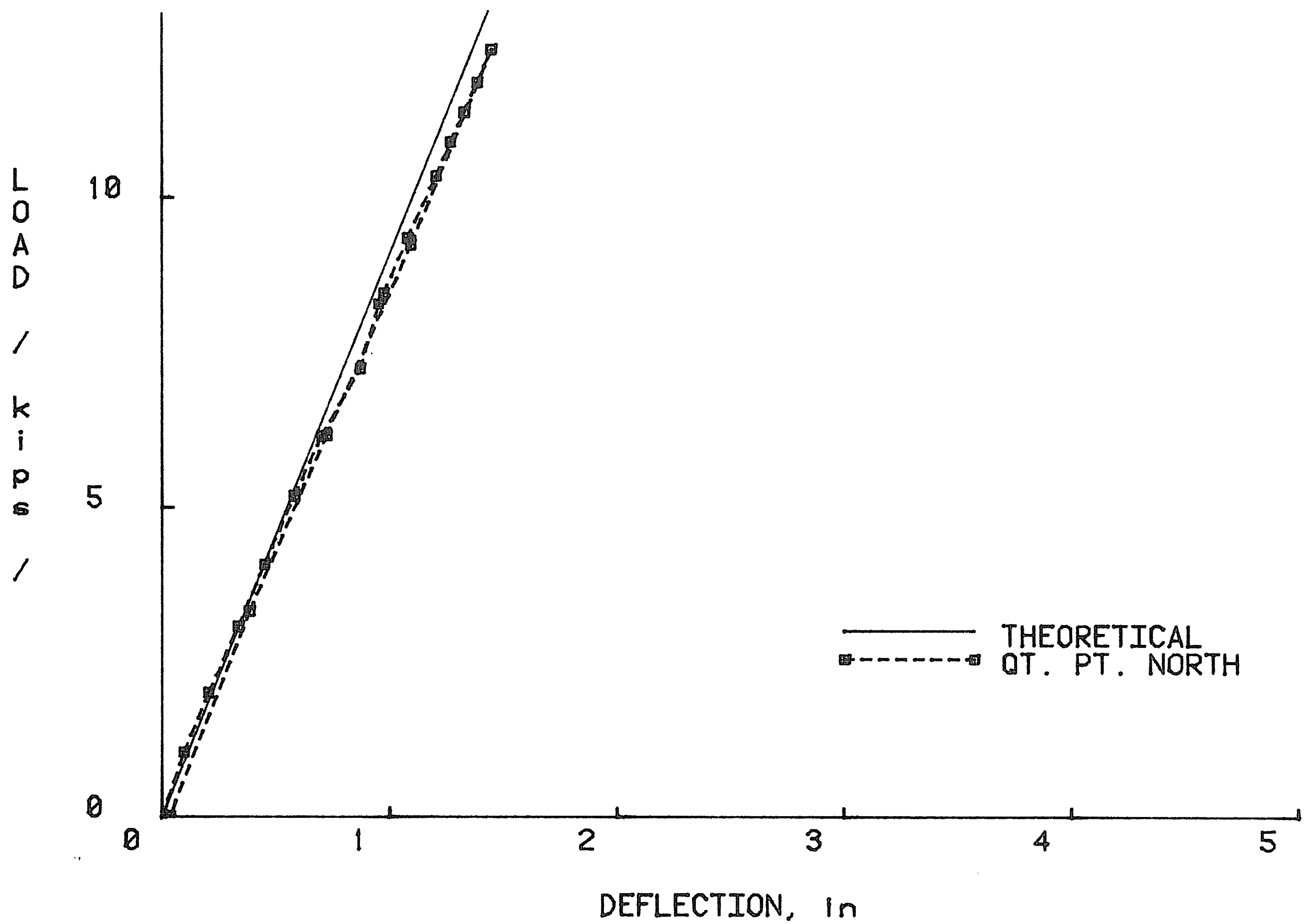


Figure C.5: LOAD vs QUARTERPOINT DEFLECTION, 1.5ULL: 2nd run

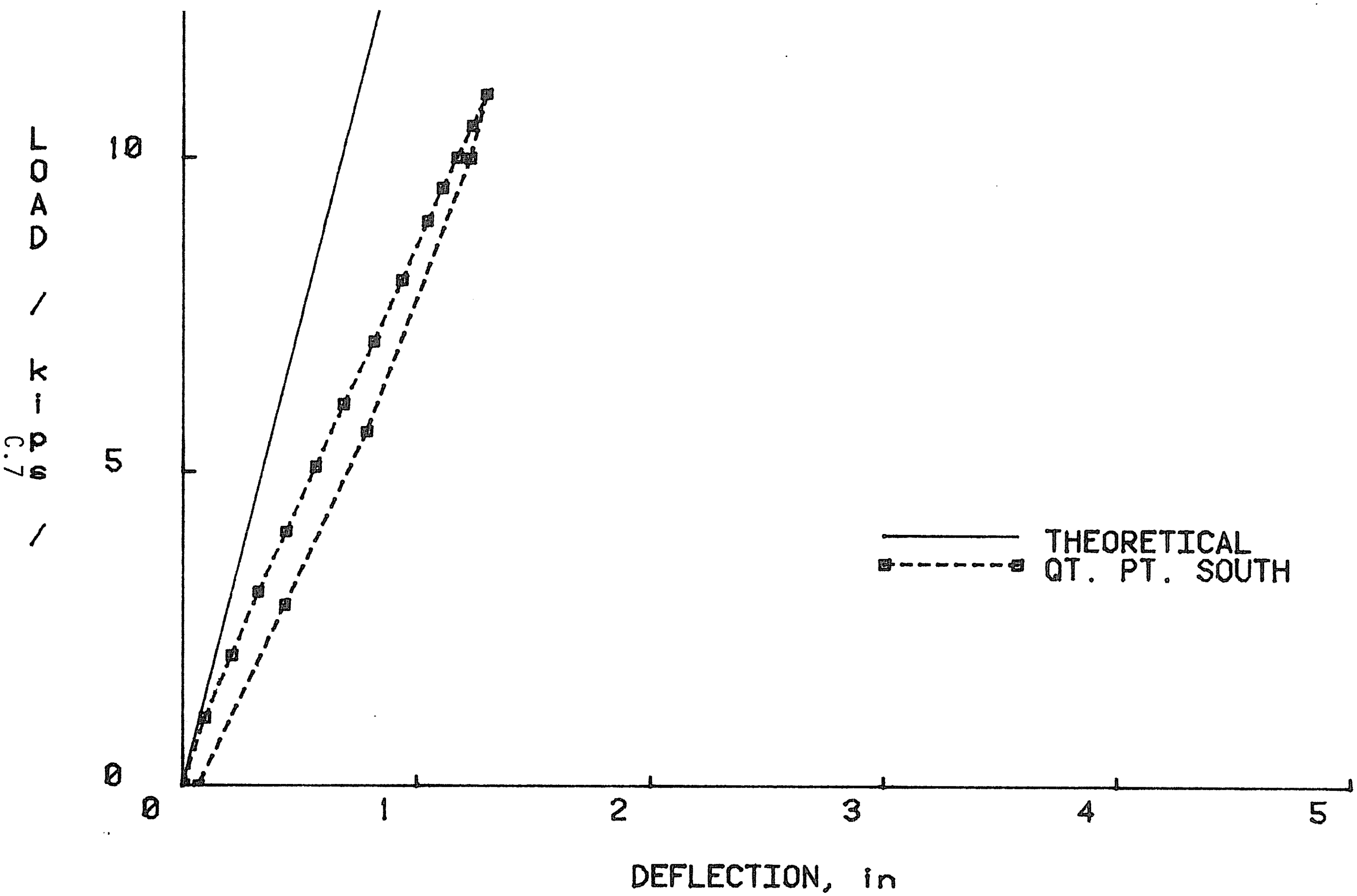


Figure C.6 LOAD vs QUARTERPOINT DEFLECTION 1.5ULL: 1st run

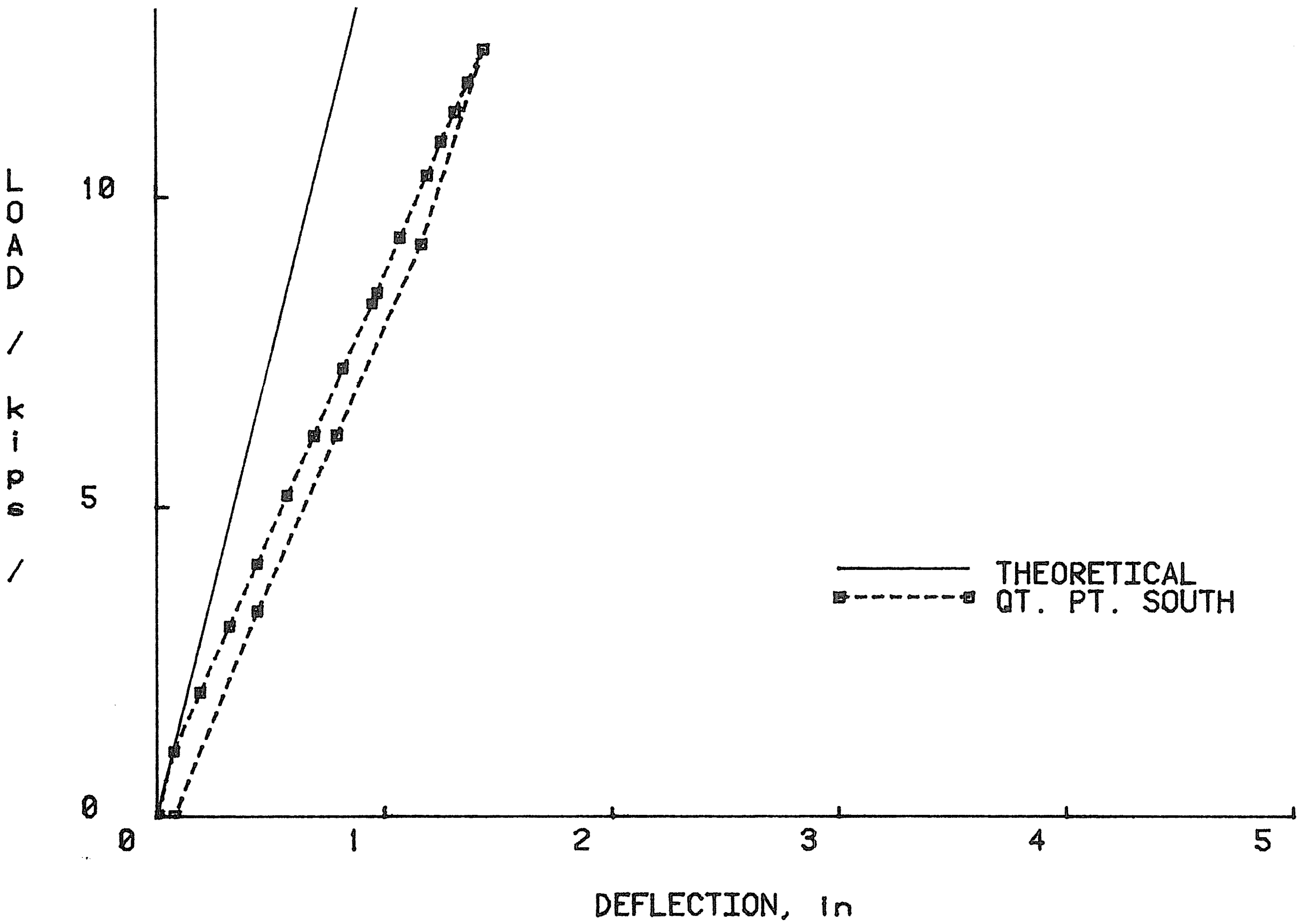


Figure C.7 LOAD vs QUARTERPOINT DEFLECTION 1.5ULL: 2nd run

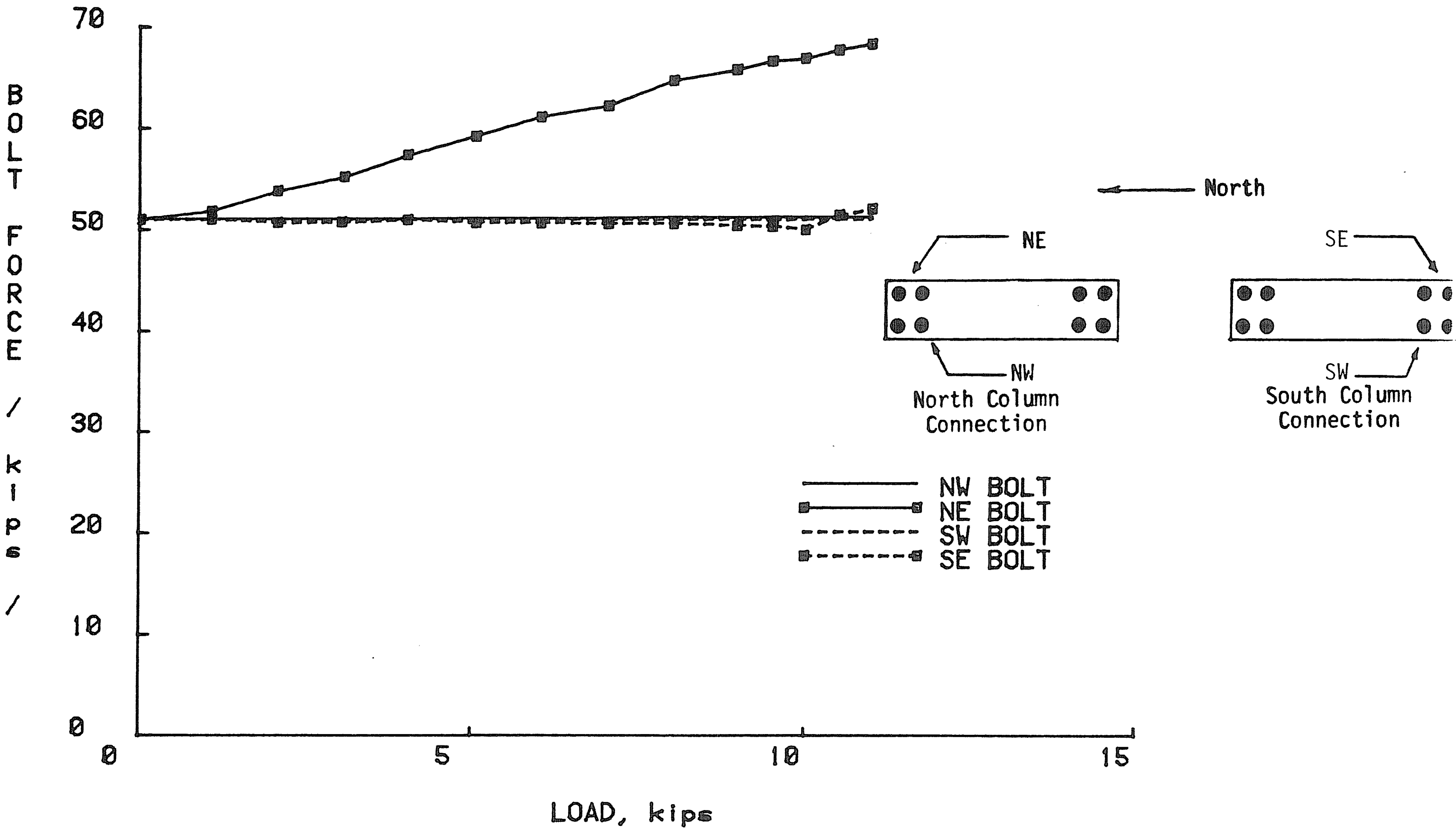


Figure C.8 BOLT FORCE vs LOAD, 1.5ULL: 1st run

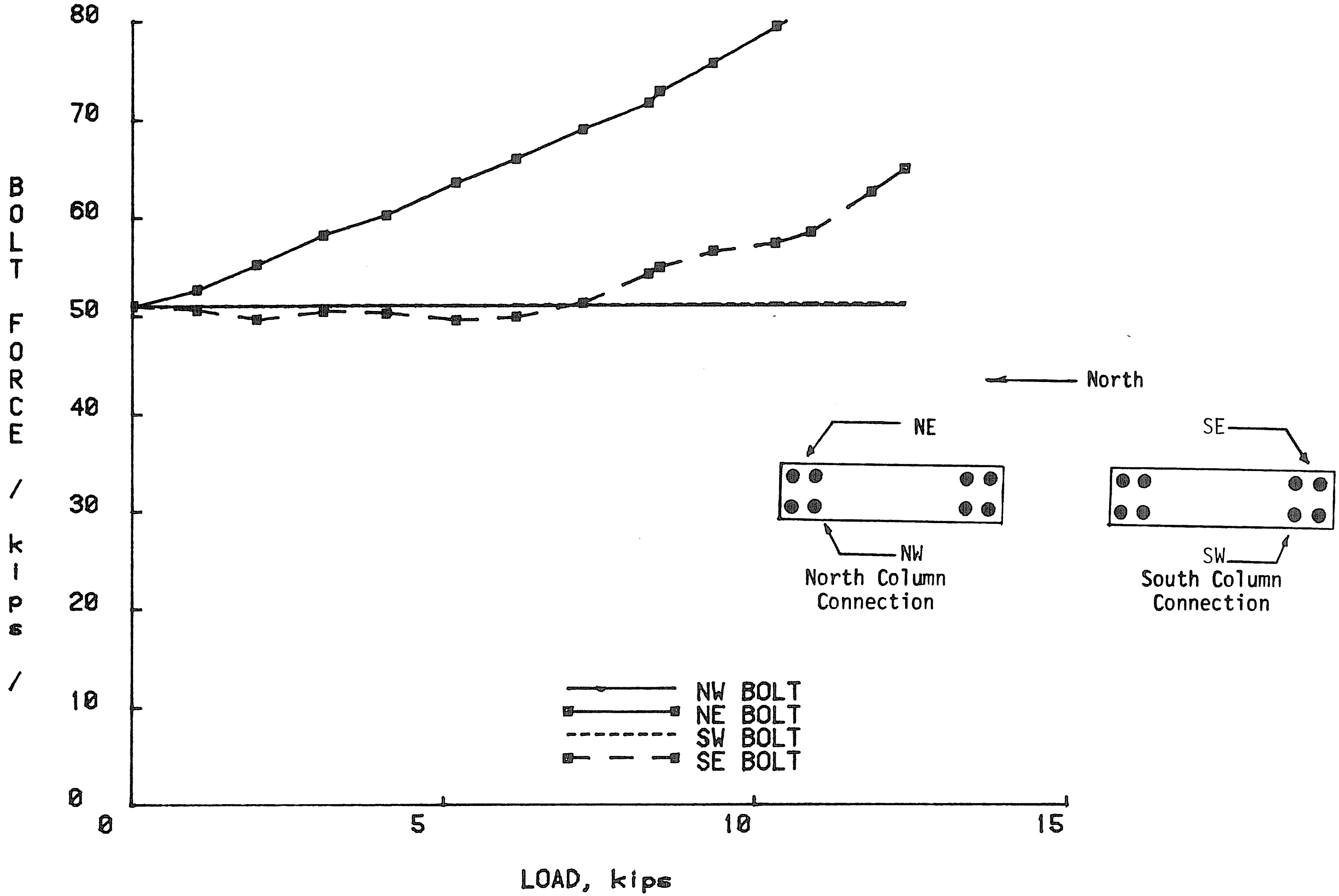


Figure C.9 BOLT FORCE vs LOAD, 1.5ULL: 2nd run

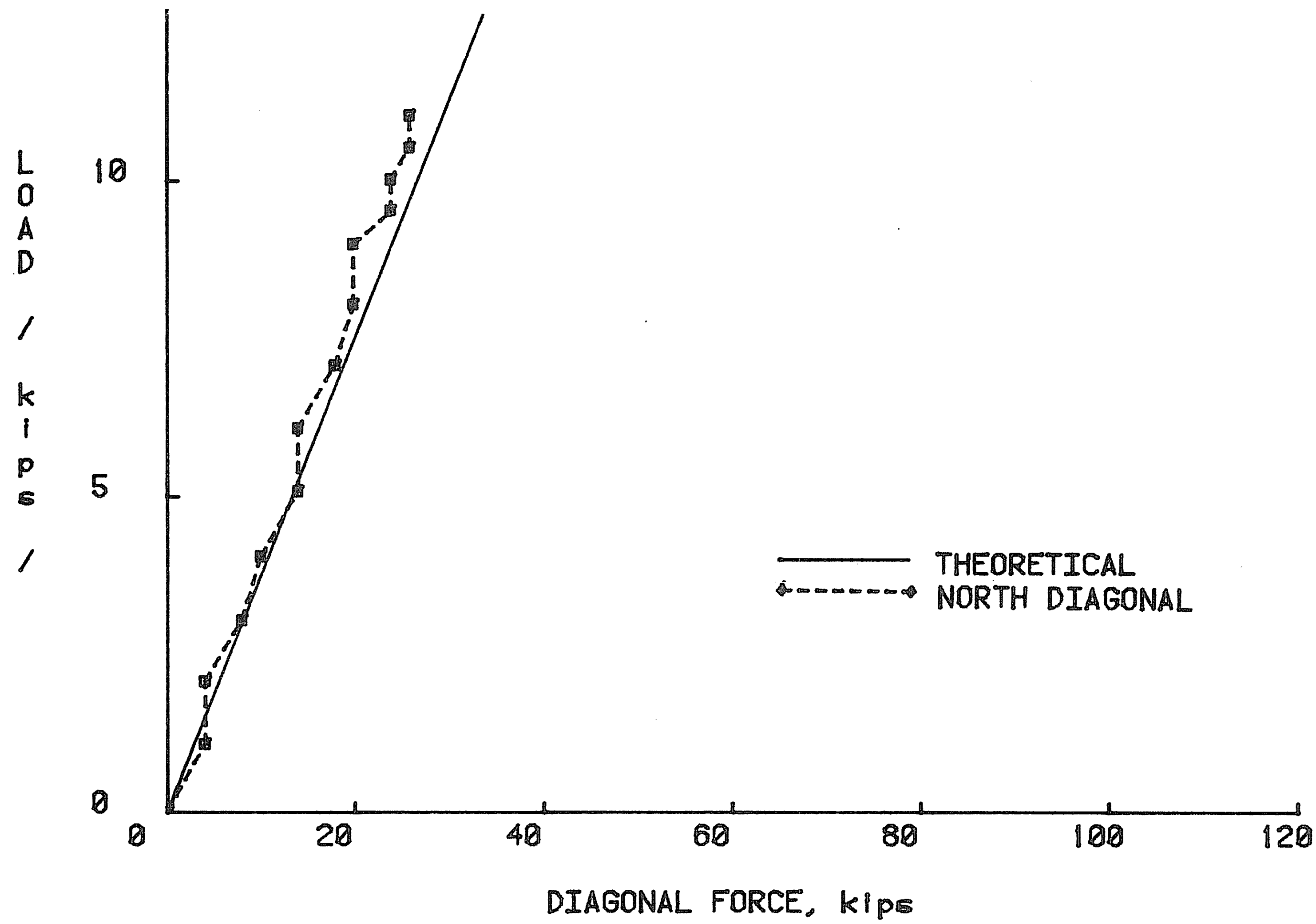


Figure C.10 LOAD vs KNEE DIAGONAL FORCE, 1.5ULL: 1st run

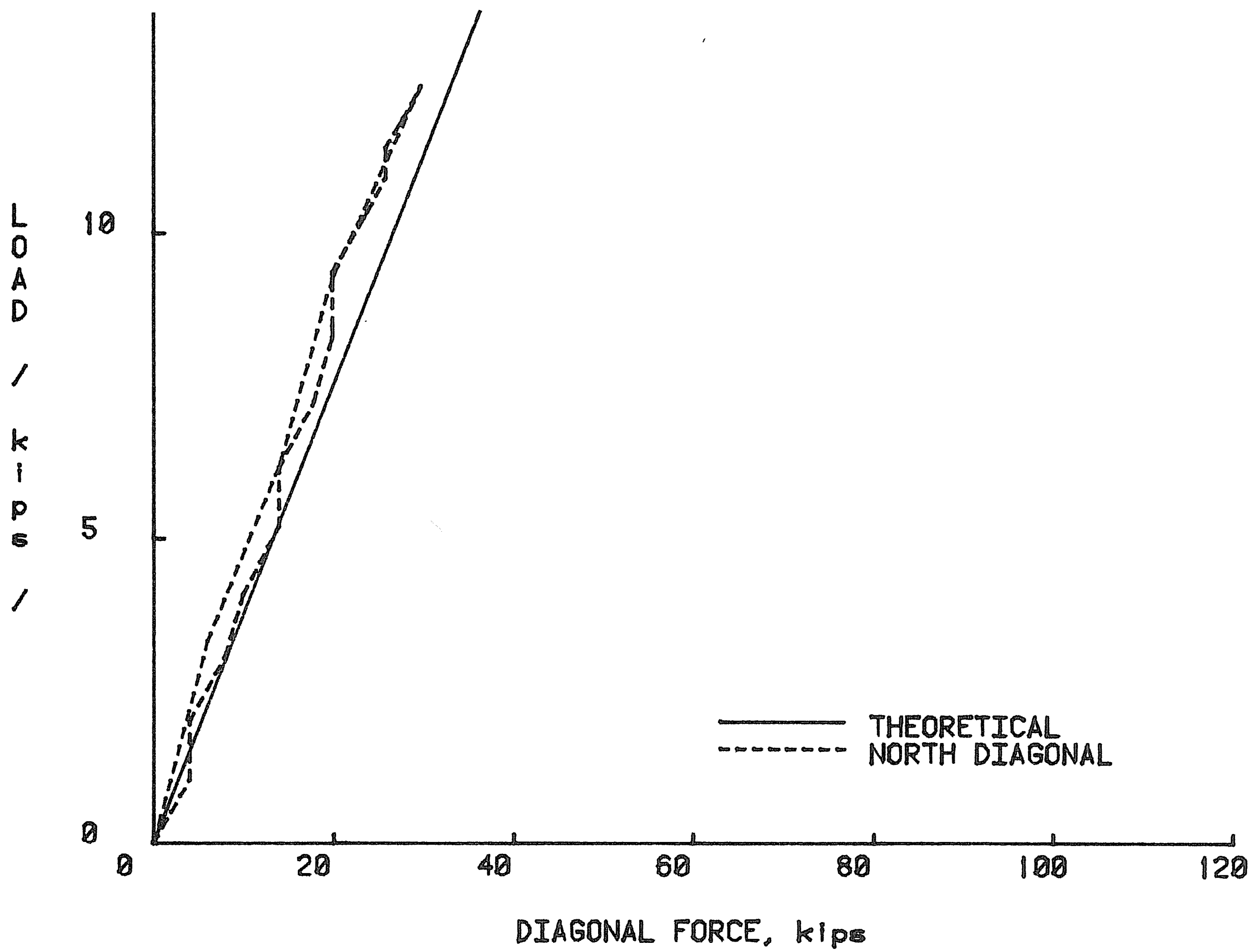


Figure C.11 LOAD vs KNEE DIAGONAL FORCE, 1.5ULL: 2nd run

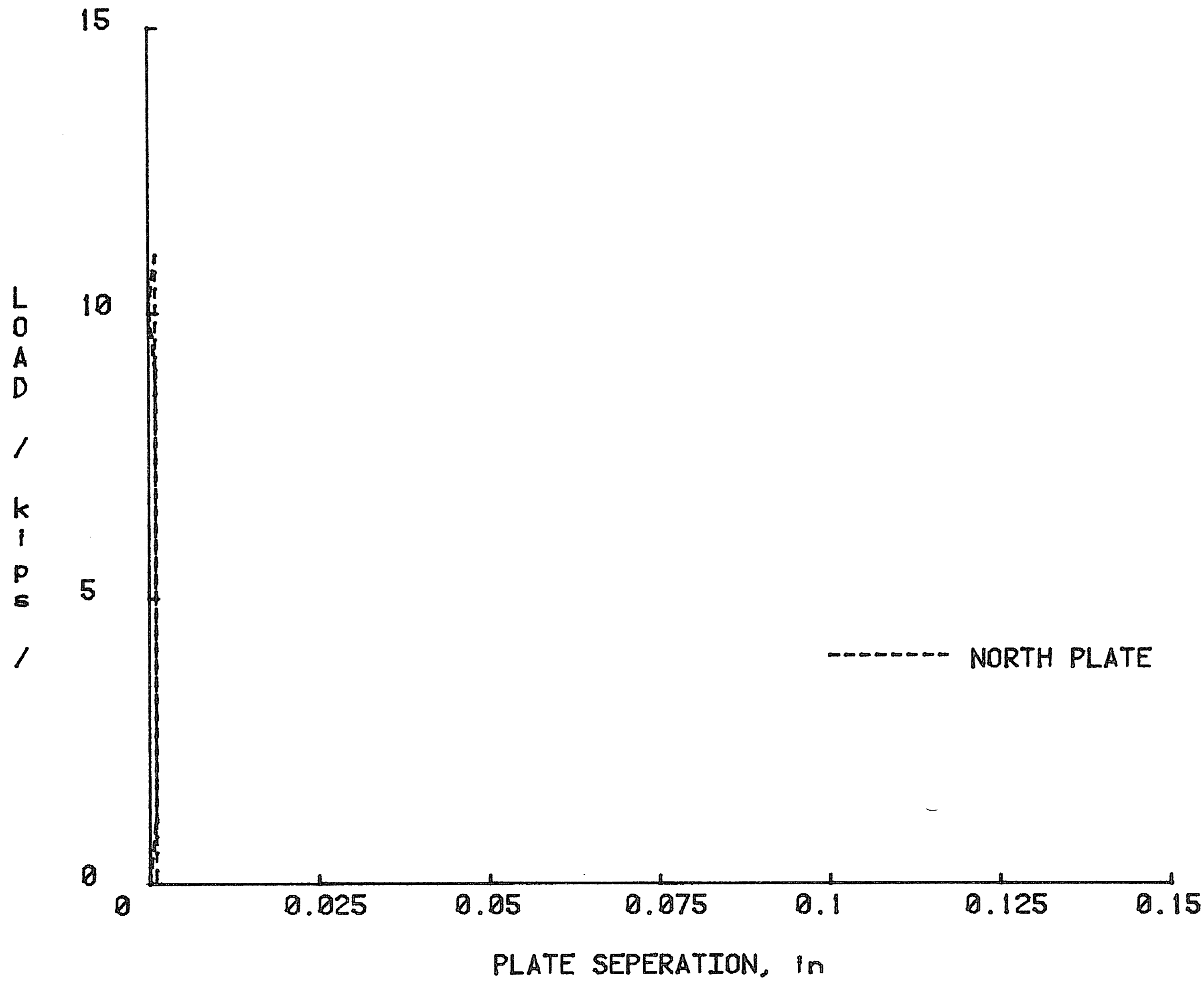


Figure C.12 LOAD vs PLATE SEPARATION, 1.5ULL: 1st run

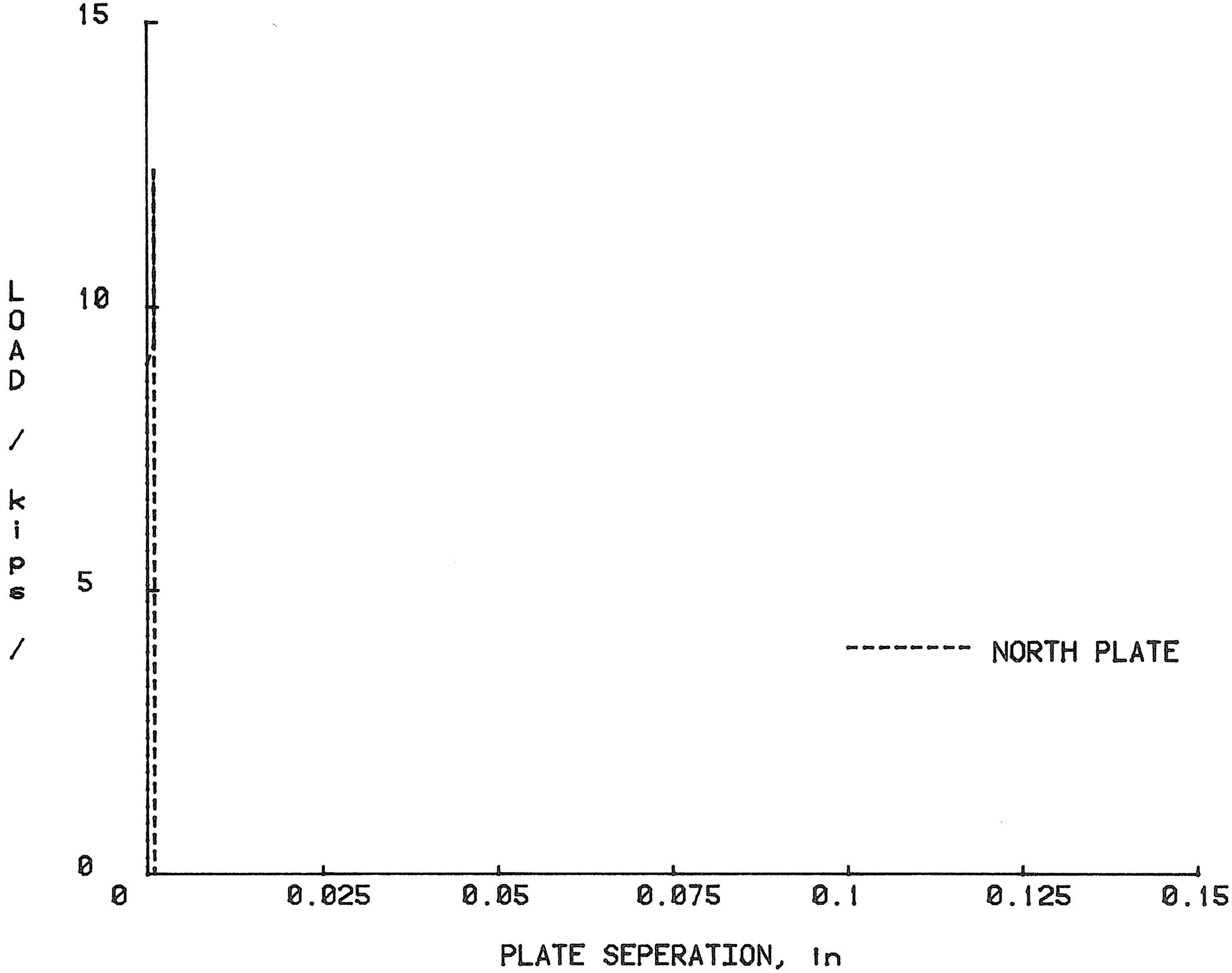


Figure C.13 LOAD vs PLATE SEPARATION, 1.5ULL: 2nd run

APPENDIX D

FACTORED WIND LOAD

(TEST WL)

VULCRAFT FRAME TEST SUMMARY

Project: Vulcraft

Test No.: FR-1

Test Date: 8/1/84

Purpose: Test of 1.25 working level wind load

Maximum Test Load: 8.75 kips

Failure Mode:

Discussion:

- Lateral load was applied by pulling both frames toward the north at the level of the reentrant corner. The load was increased incrementally until a force of 8.75 kips was applied to each frame. Subsequently, the load was removed and, in the previous manner, a load of 8.75 kips was again applied. This magnitude of lateral load did not cause any noticable distress in frame. There were no vertical deflections at the centerline of the span and quarterpoint deflections were negligable as expected.

- Lateral deflections, however, were substantially greater than theoretically determined values. During the first loading sequence both frames deflected essentially the same amount. At 8.75 kips the deflections were .90" and .94" for the east and west frames, respectively. This represents increases of 67 and 75% over the expected sideway. Likewise, in the second loading sequence lateral deflections exceeded the theoretical values albeit to a lesser degree. At the maximum load of 8.75 kips the sideways of the west frame was .66" or 22% greater than predicted. For the east frame these respective values were .78" and 41%. In addition, plate separation data indicates substantially more plate separation (.31") than in any of the other fram tests.

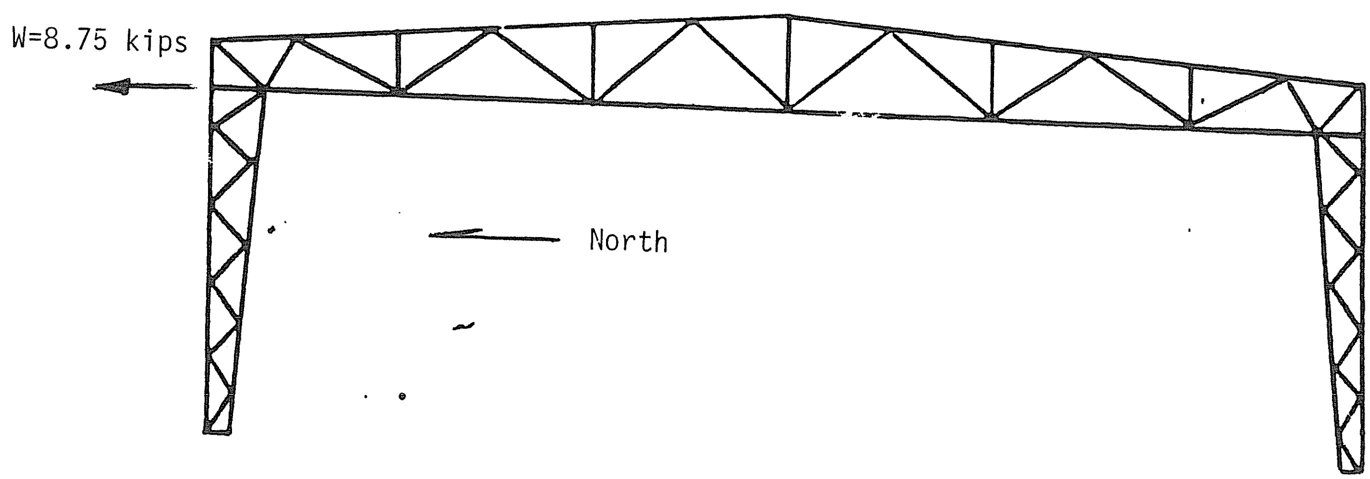


Figure D.1 Wind Load

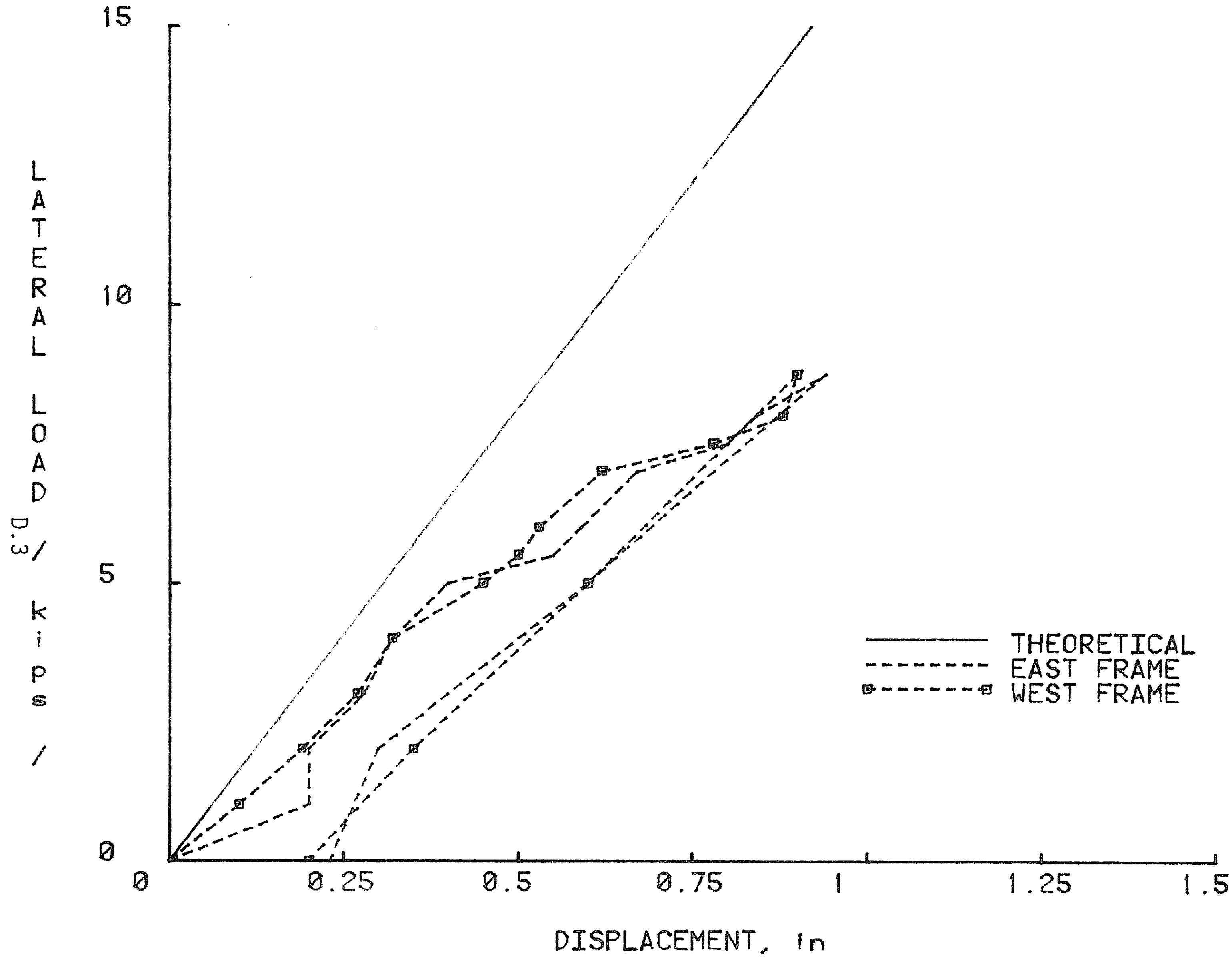


Figure D.2 LATERAL LOAD vs SIDESWAY, W: 1st run

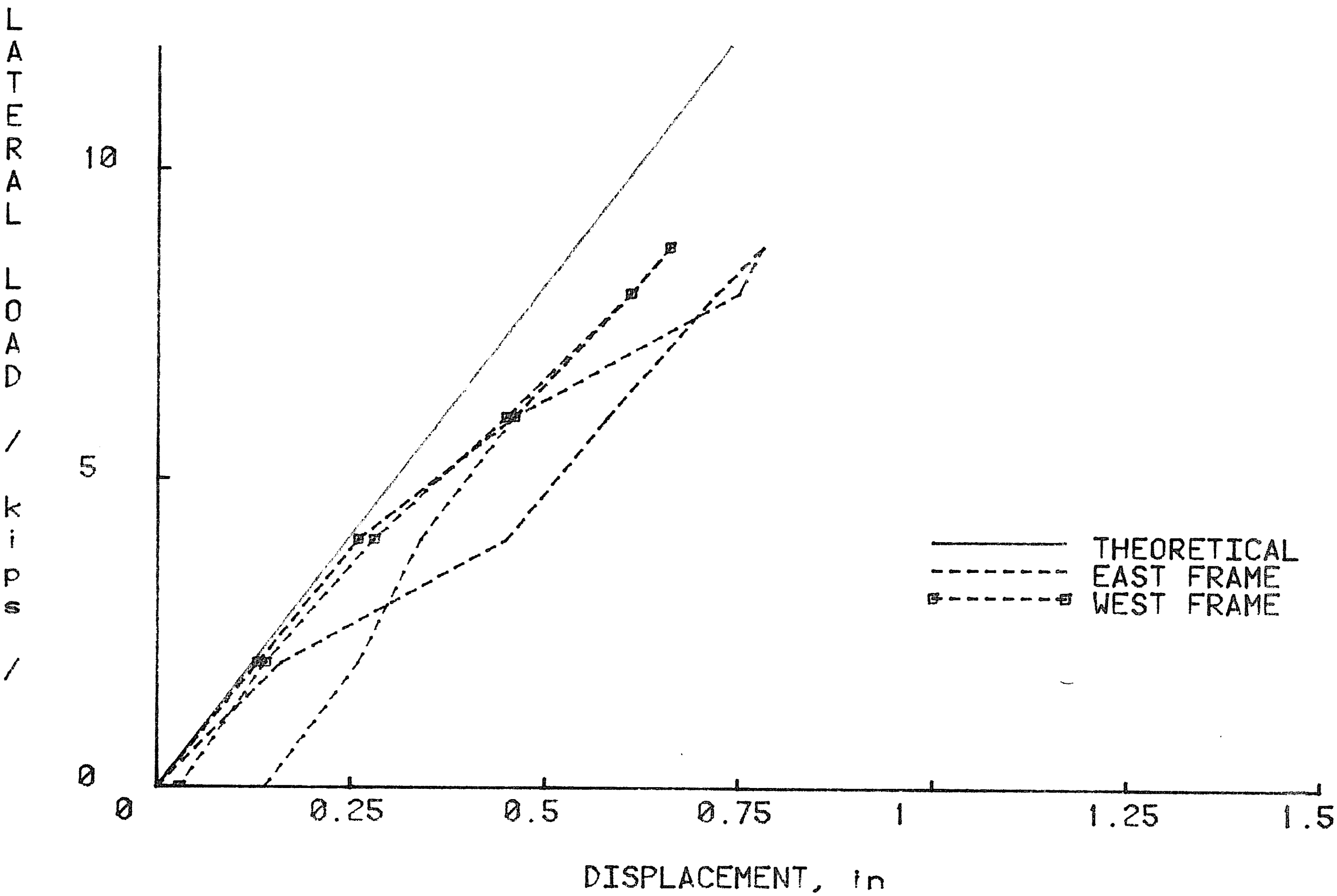


Figure D.3 LATERAL LOAD vs SIDESWAY, W: 2nd run

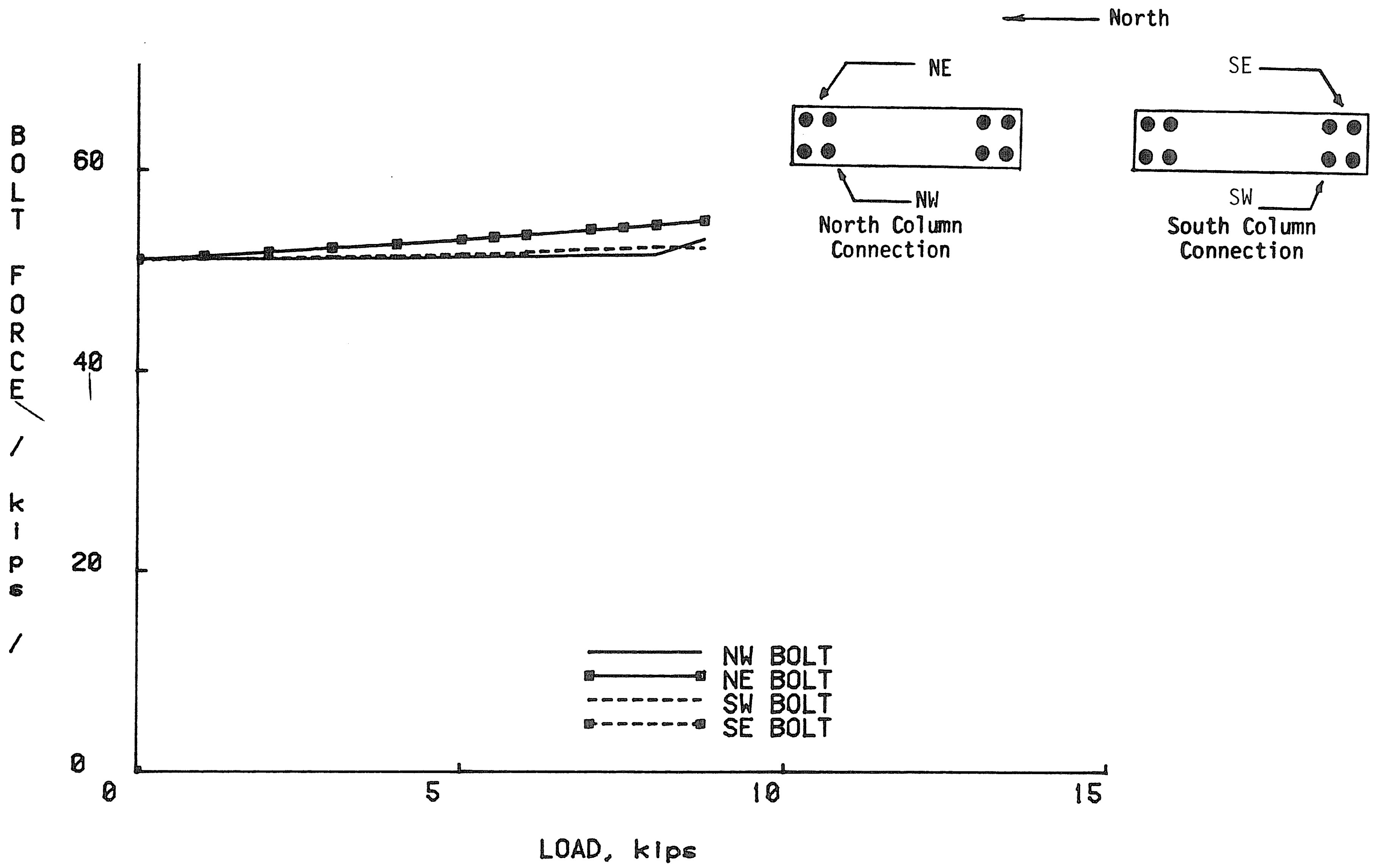


Figure D.4 BOLT FORCE vs LOAD, W

APPENDIX E

UNBALANCED, FULL FACTORED LIVE LOAD

(TEST 1.67ULL)

VULCRAFT FRAME TEST SUMMARY

Project: Vulcraft Frame

Test No.: FR-1

Test Date: 8/1/84

Purpose: Test of 1.67 working level unbalanced live load

Maximum Test Load: 12.56 kips

Failure Mode: Buckling of vertical web member accompanied by lateral buckling of adjacent top chord

Discussion:

- Gravity load was applied to the south run of both frames. The target load of 13.67 (1.67 ULL) was not achieved due to premature failure of truss members. As before, load was applied incrementally with data being acquired at each application. This data indicates that the load-centerline deflection relationship is almost exactly that predicted by the stiffness analysis. Likewise, the load-quarter point deflection data is close to the theoretical despite slightly larger deflections than expected. In addition, the frames underwent significant lateral deflections. Transit readings of scales positioned at the tops of the frames on the north end recorded movements exceeding the theoretical by 30% and 45% for the west and east frames, respectively, at the maximum test load.

- At a load of 12.56 kips the vertical web member originating at the junction of members 45 and 46 buckled. This buckling was accompanied by lateral buckling of the adjacent top chord. Failure was then considered to have occurred and testing was terminated.

$P = 1.67ULL = 13.67^k$

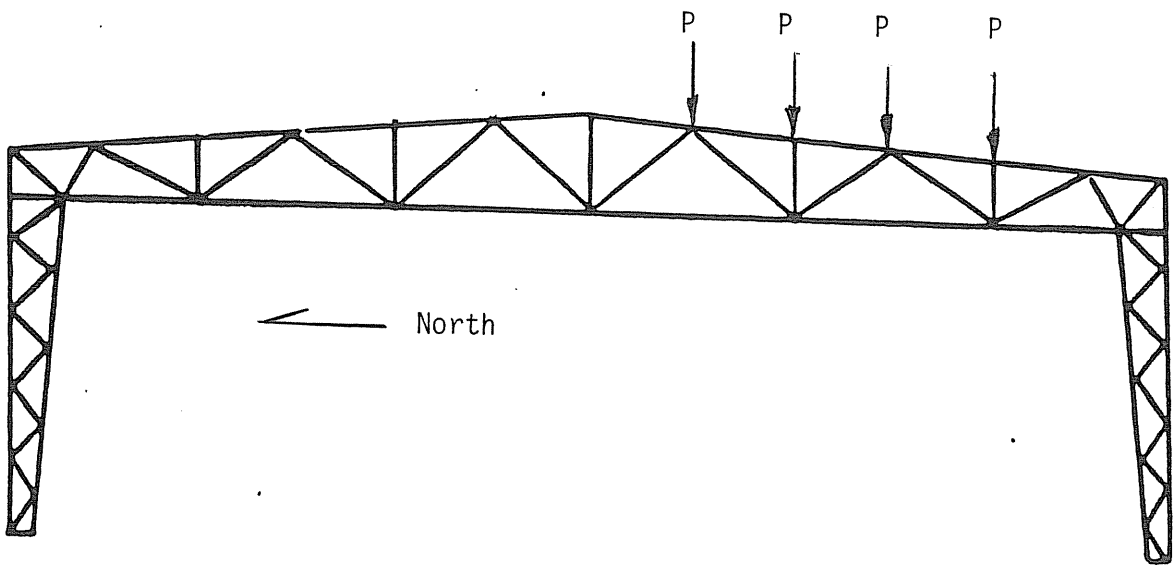


Figure E.1 Unbalanced Live Load

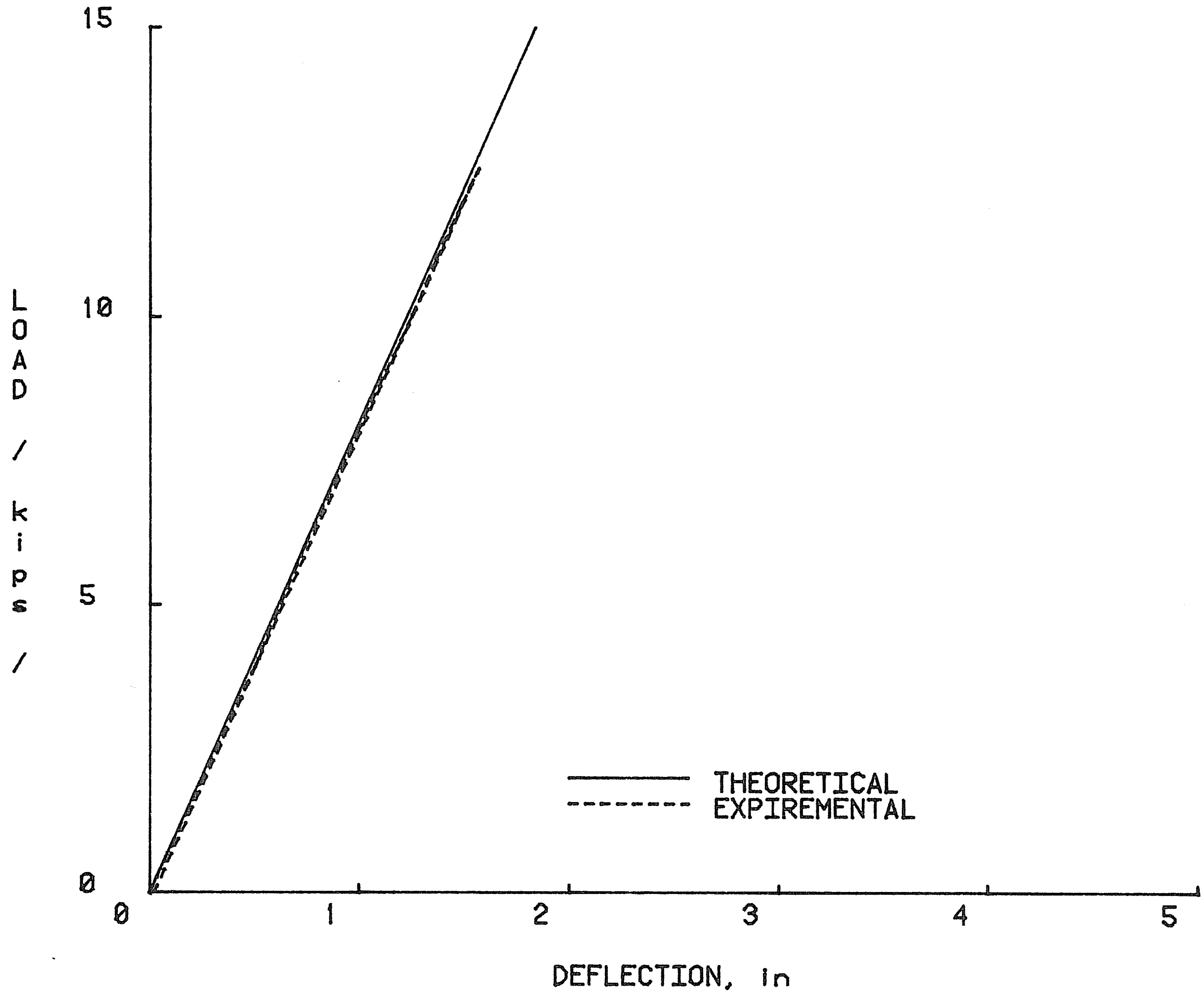


Figure E.2 LOAD vs CENTERLINE DEFLECTION, 1.67ULL

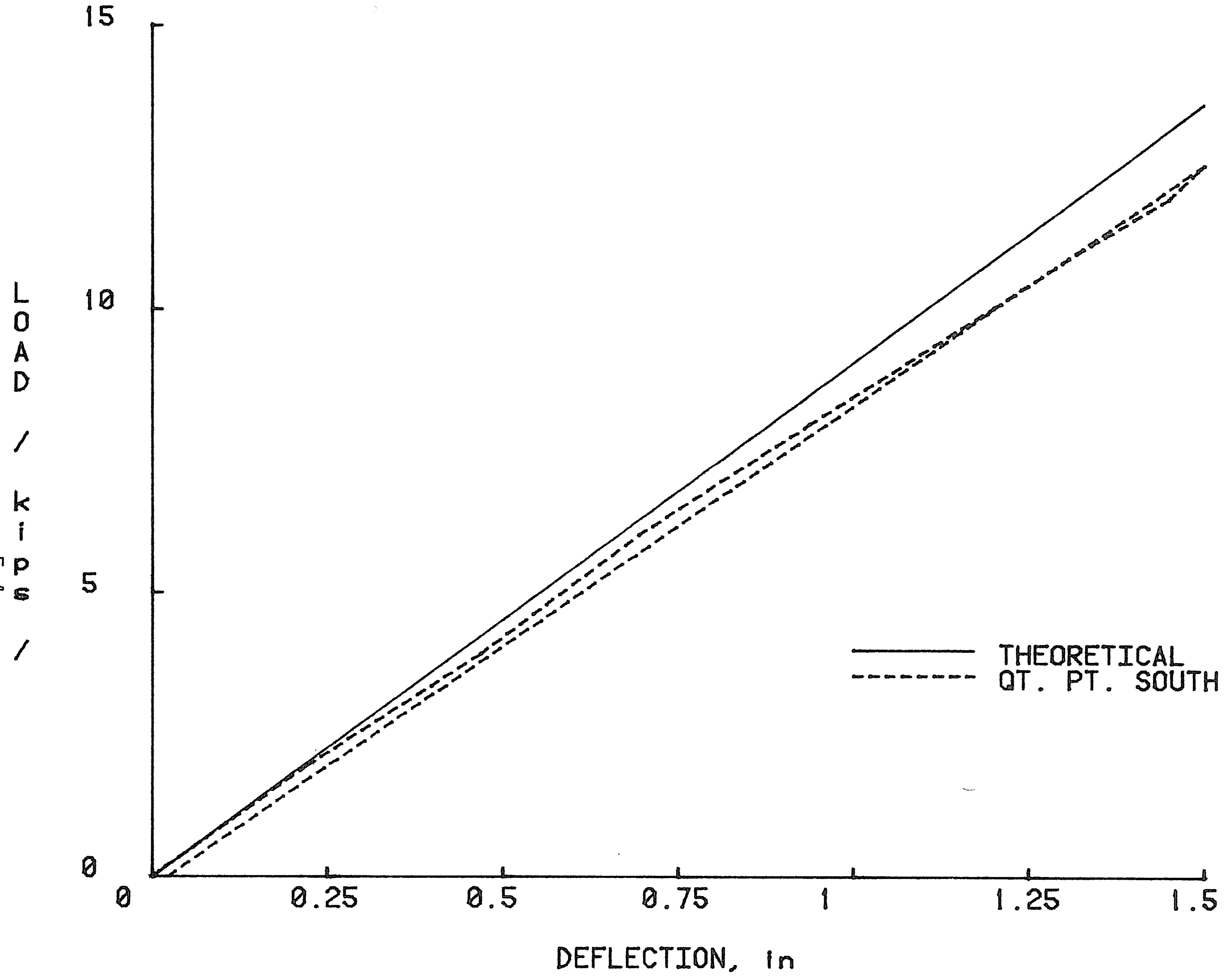


Figure E.3 LOAD vs QUARTERPOINT DEFLECTION, 1.67ULL

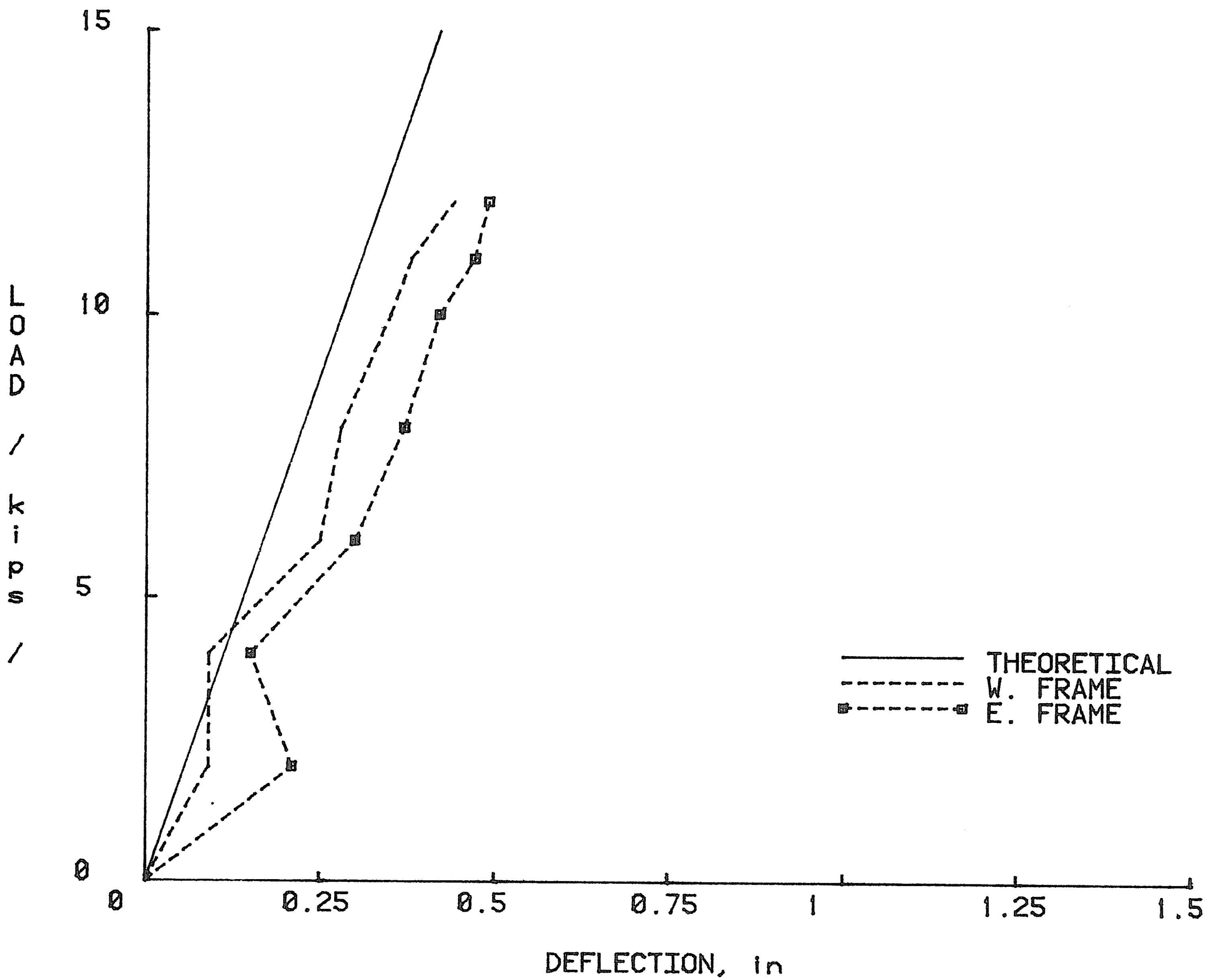


Figure E.4: LOAD vs LATERAL DEFLECTION, 1.67ULL

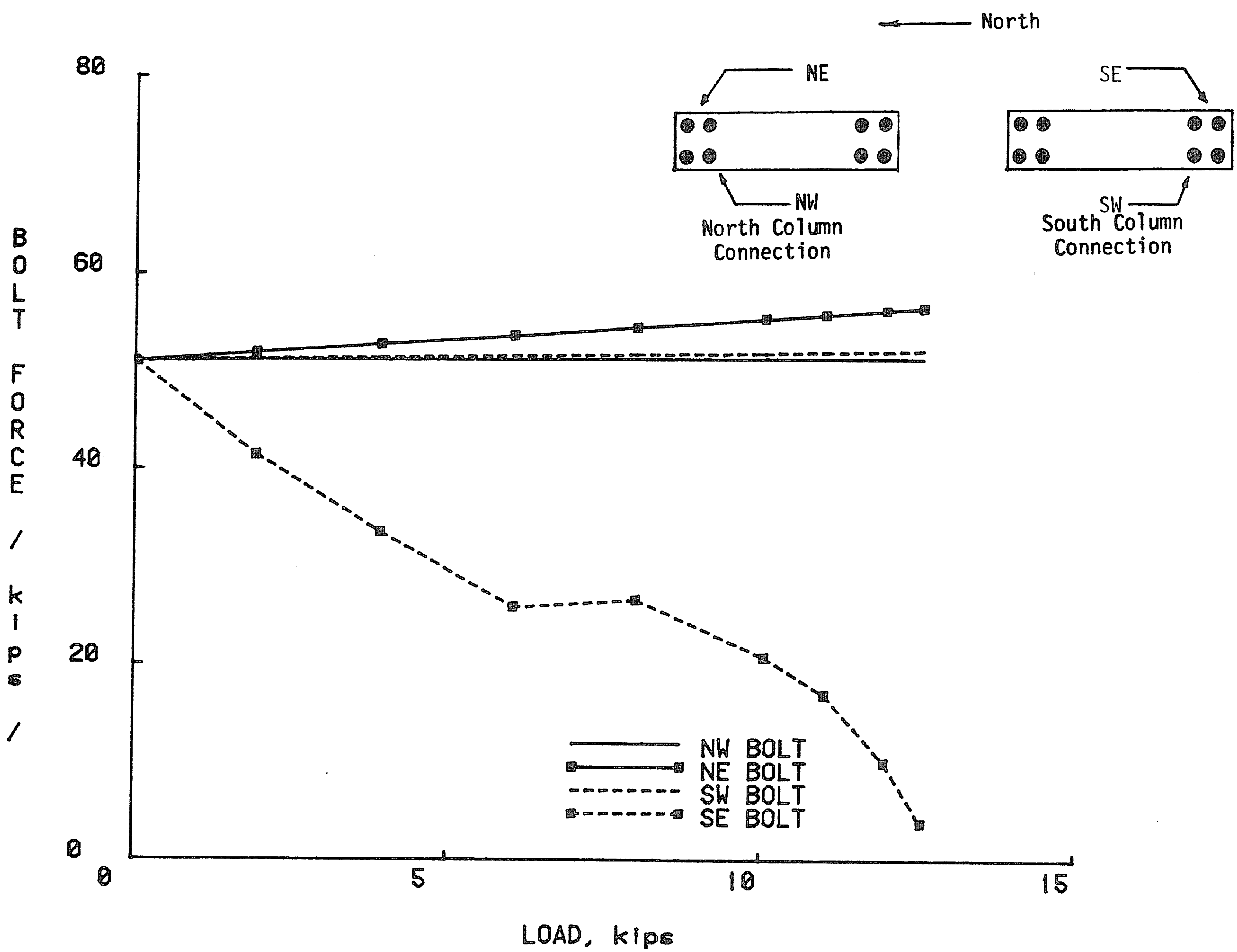


Figure E.5 LOAD vs BOLT FORCE, 1.67ULL

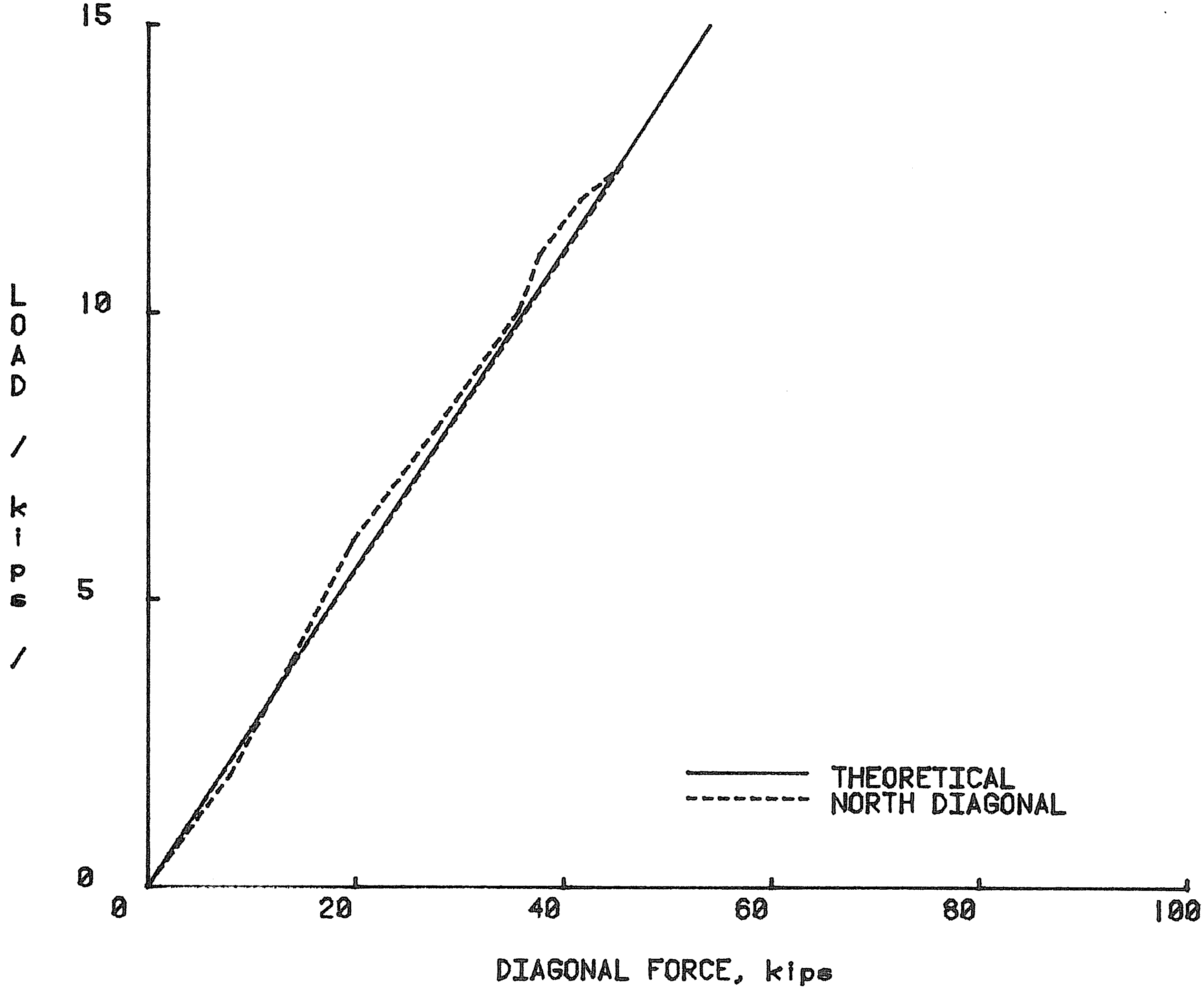


Figure E.6 LOAD vs KNEE DIAGONAL FORCE, 1.67ULL: NORTH

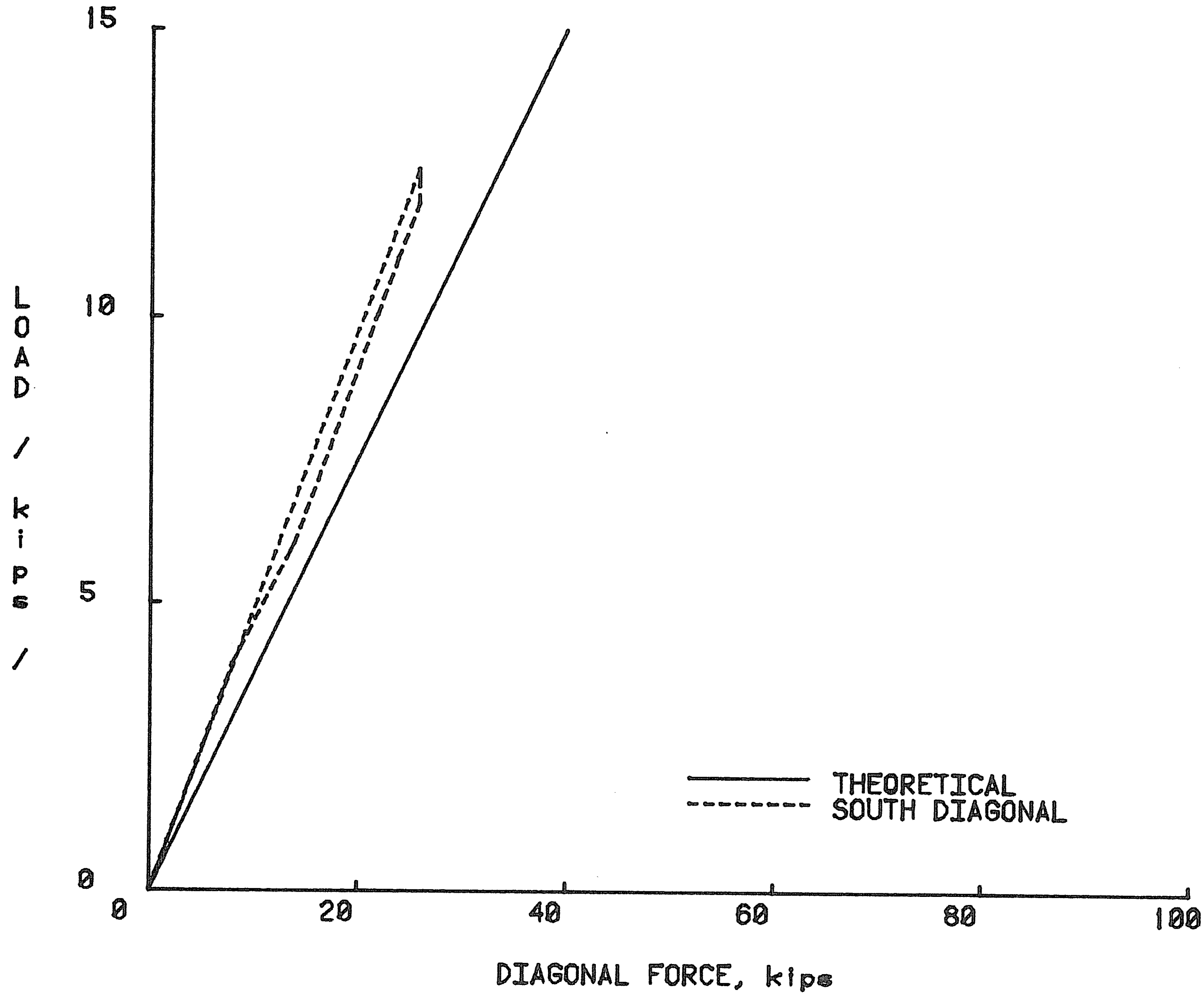


Figure E.7 LOAD vs KNEE DIAGONAL FORCE, 1.67ULL: SOUTH

APPENDIX F

UNBALANCED LIVE LOAD PLUS WIND LOAD

(TEST ULL+WL)

VULCRAFT FRAME TEST SUMMARY

Project: Vulcraft

Test no.: FR-1

Test Date: 8/2/84

Purpose: Test of 8.75 kips working level unbalanced live load plus 1.25 working level wind load

Maximum Test Load: Unbalanced live load: 7 kips; Wind load: 8.75 kips

Failure Mode:

Discussion:

- A gravity load was applied to the south run of both frames and was increased in one kip increments until a load of 7 kips was achieved. Subsequently, this load was maintained while a lateral load was applied by pulling the frames toward the north. When the lateral load on each frame reached 8.75 kips the test was terminated.

- Load vs. centerline deflection data from this test indicates reasonable agreement between the theoretical and experimentally determined values. From 0 to 3.5 kips of gravity load the load-deflection curves are identical. In the interval from 3.5 to 7.0 kips there was slightly more deflection than expected although agreement is still close. As expected, the centerline deflection due to lateral loading is negligibly small. A large variance was observed between the experimental and theoretical span quarterpoint deflections, however. Deflections for both quarterpoints were in excess of the expected values with the north quarterpoint having substantially larger values (170% of theoretical).

- Likewise, lateral deflections exceeded the expected values. Deflections due to gravity loads exceeded the theoretical values by 63% for the east frame and 37% for the west. When combined with the large deflections due to lateral load the total deflection exceeds the theoretically expected deflection by about 33%.

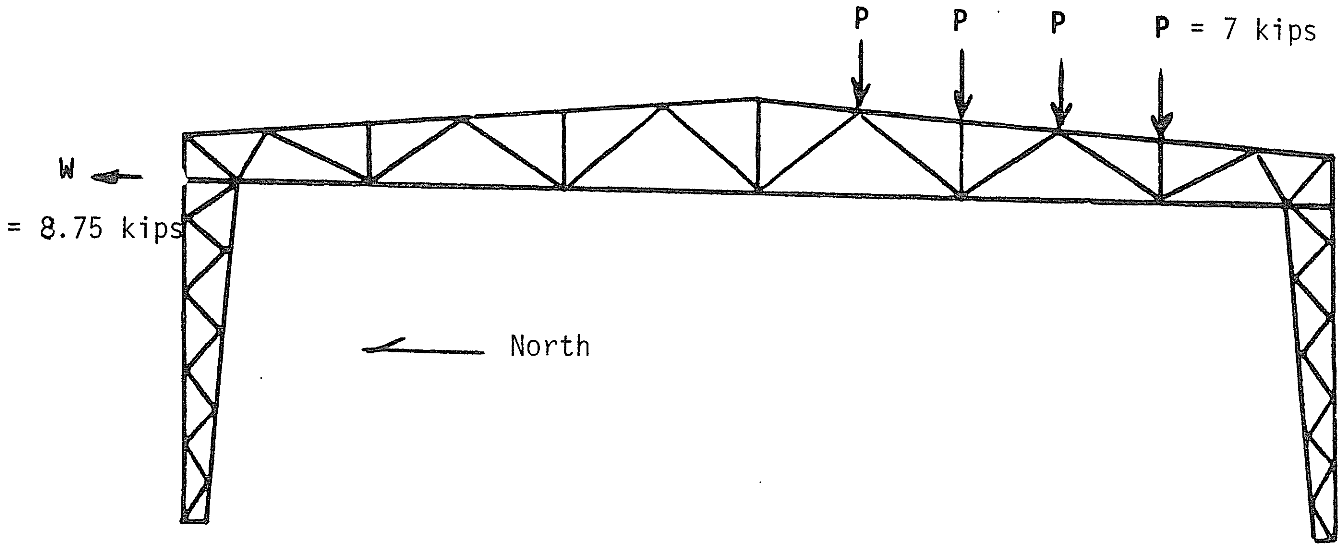


Figure F.1 Unbalanced Live Load

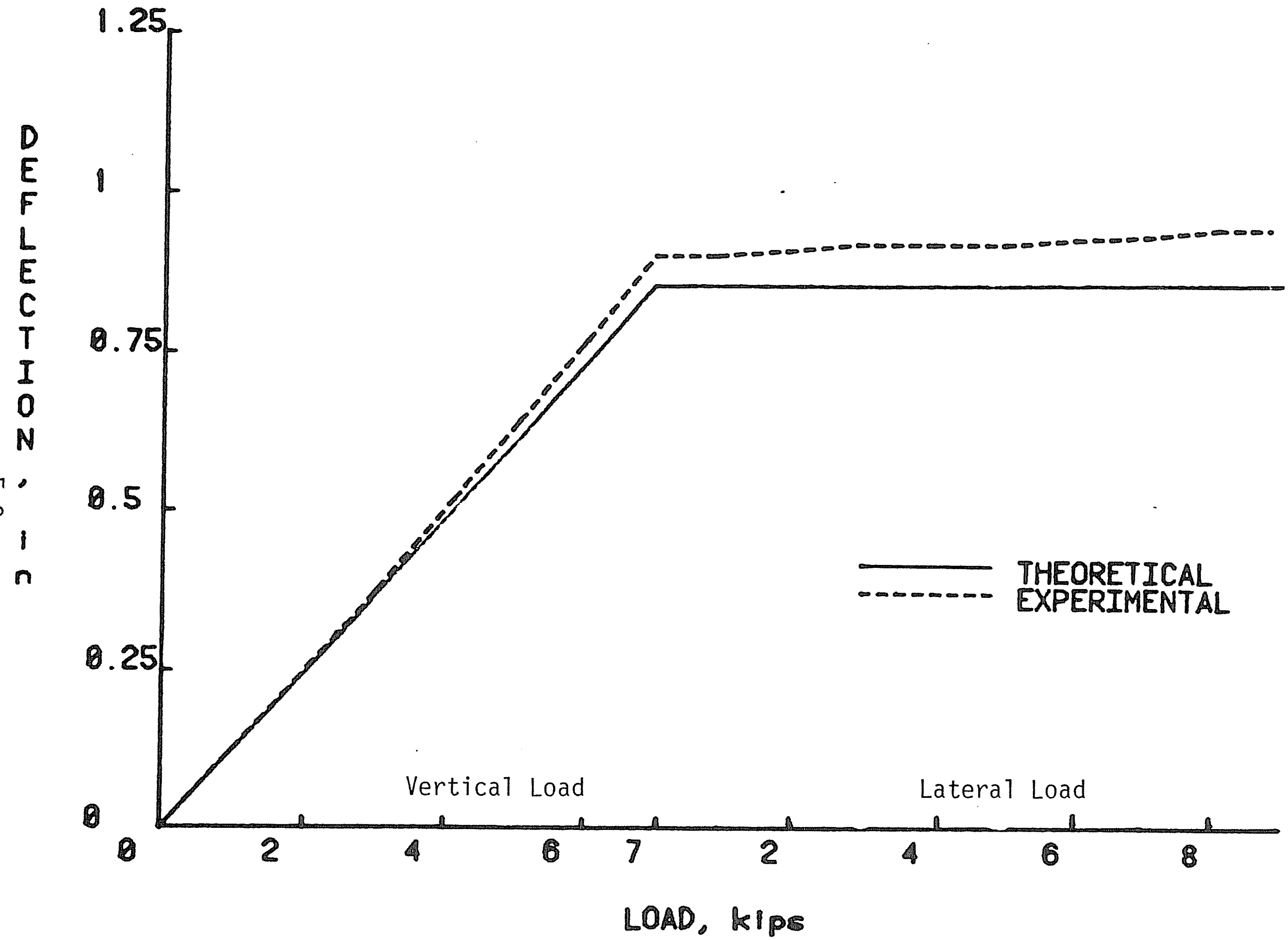


Figure F.2 CENTERLINE DEFLECTION vs LOAD, ULL+WL

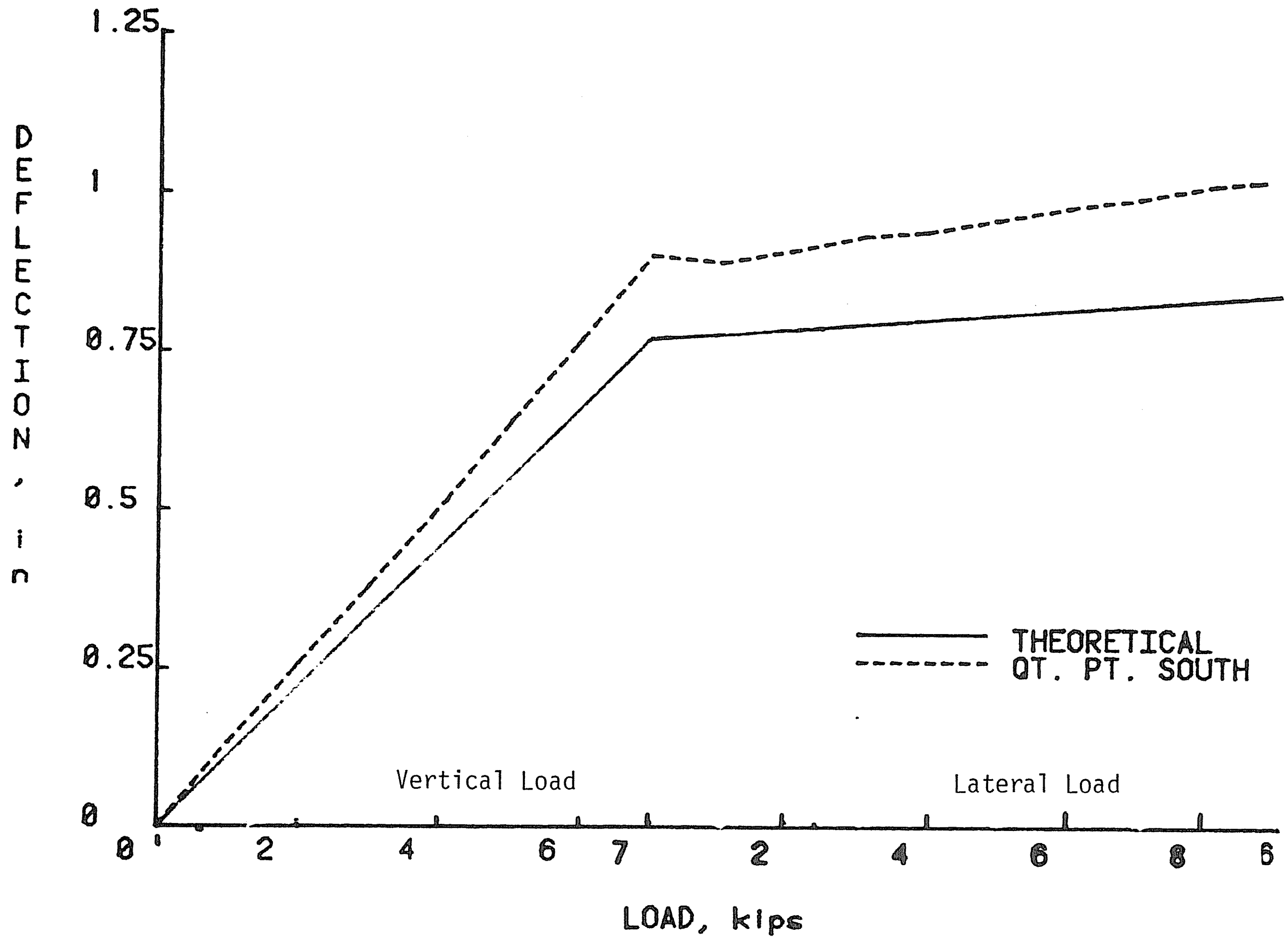


Figure F.3 QUARTERPOINT DEFLECTION vs LOAD, ULL+WL

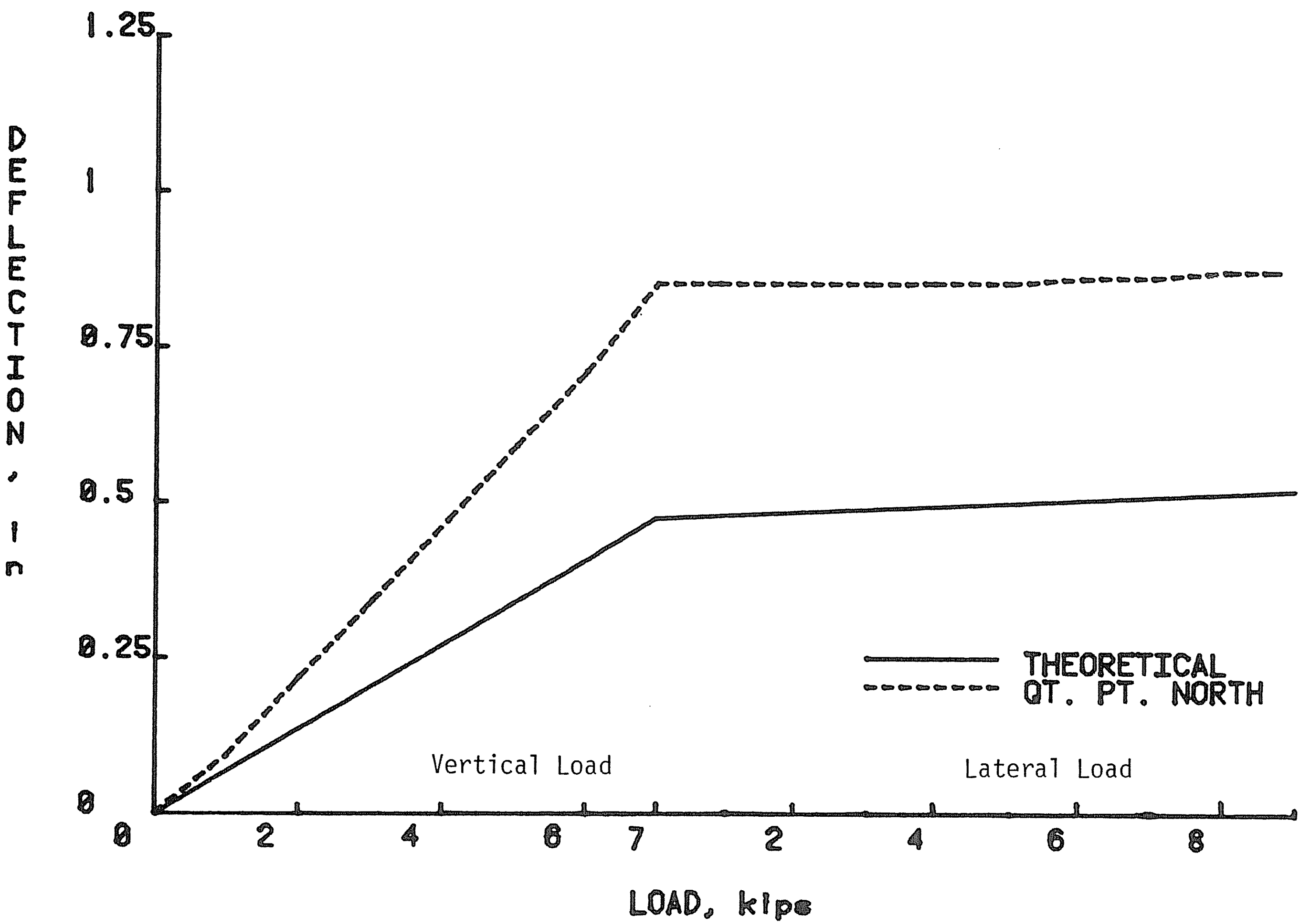


Figure F.4 QUARTERPOINT DEFLECTION vs LOAD, ULL+WL

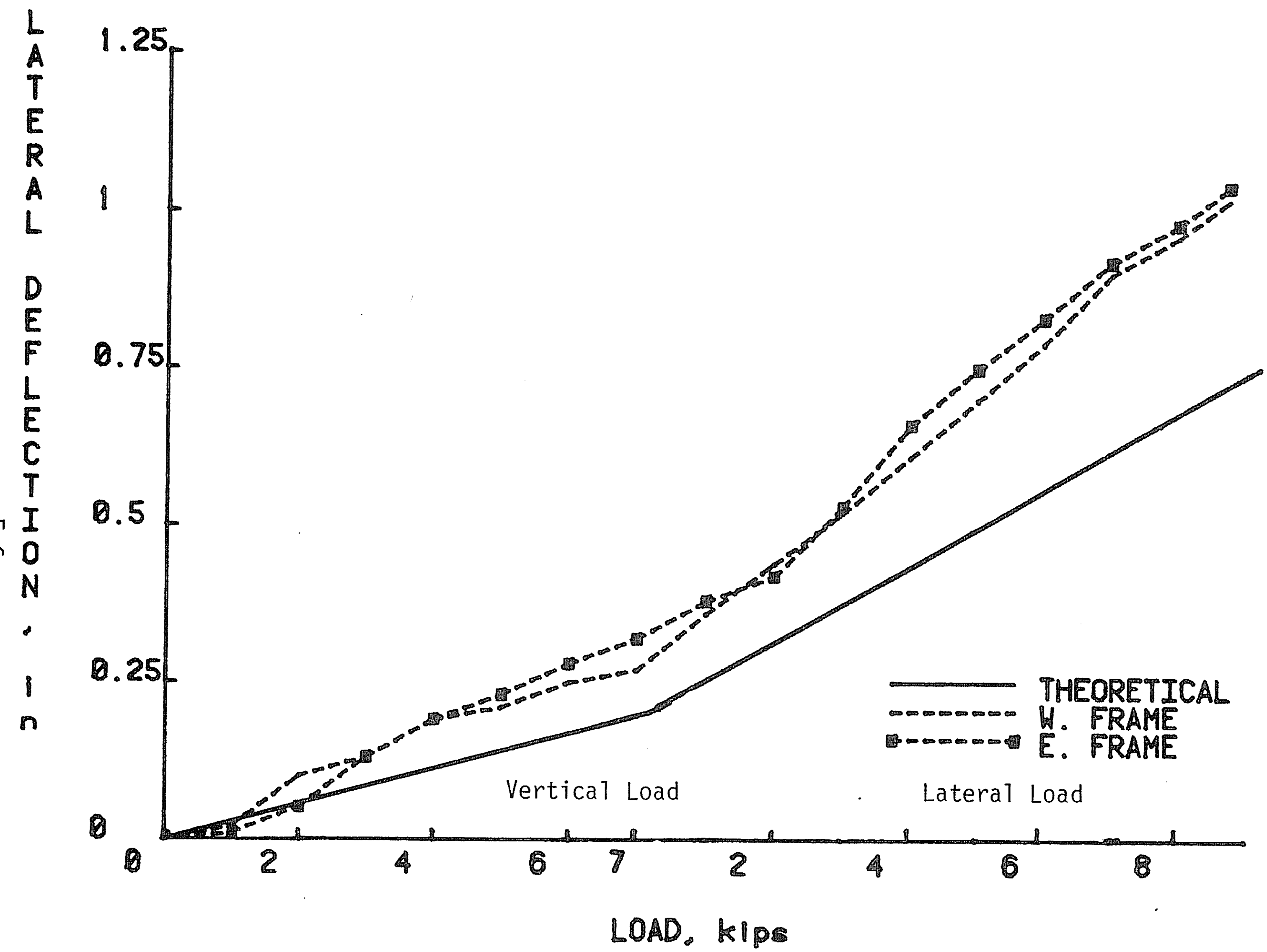


Figure F.5 LATERAL DEFLECTION vs LOAD, ULL+WL

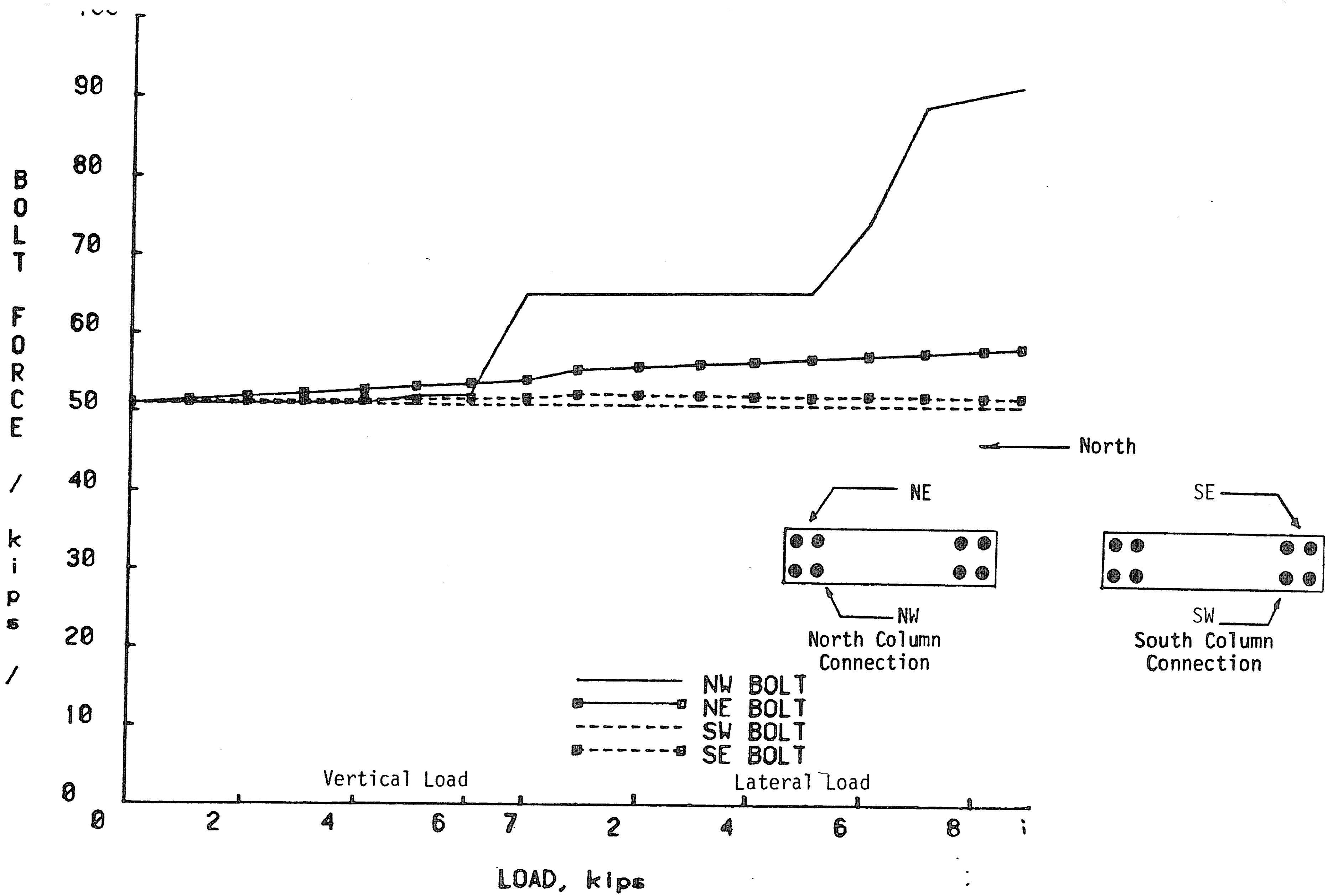


Figure F.6 BOLT FORCE vs LOAD, ULL+WL

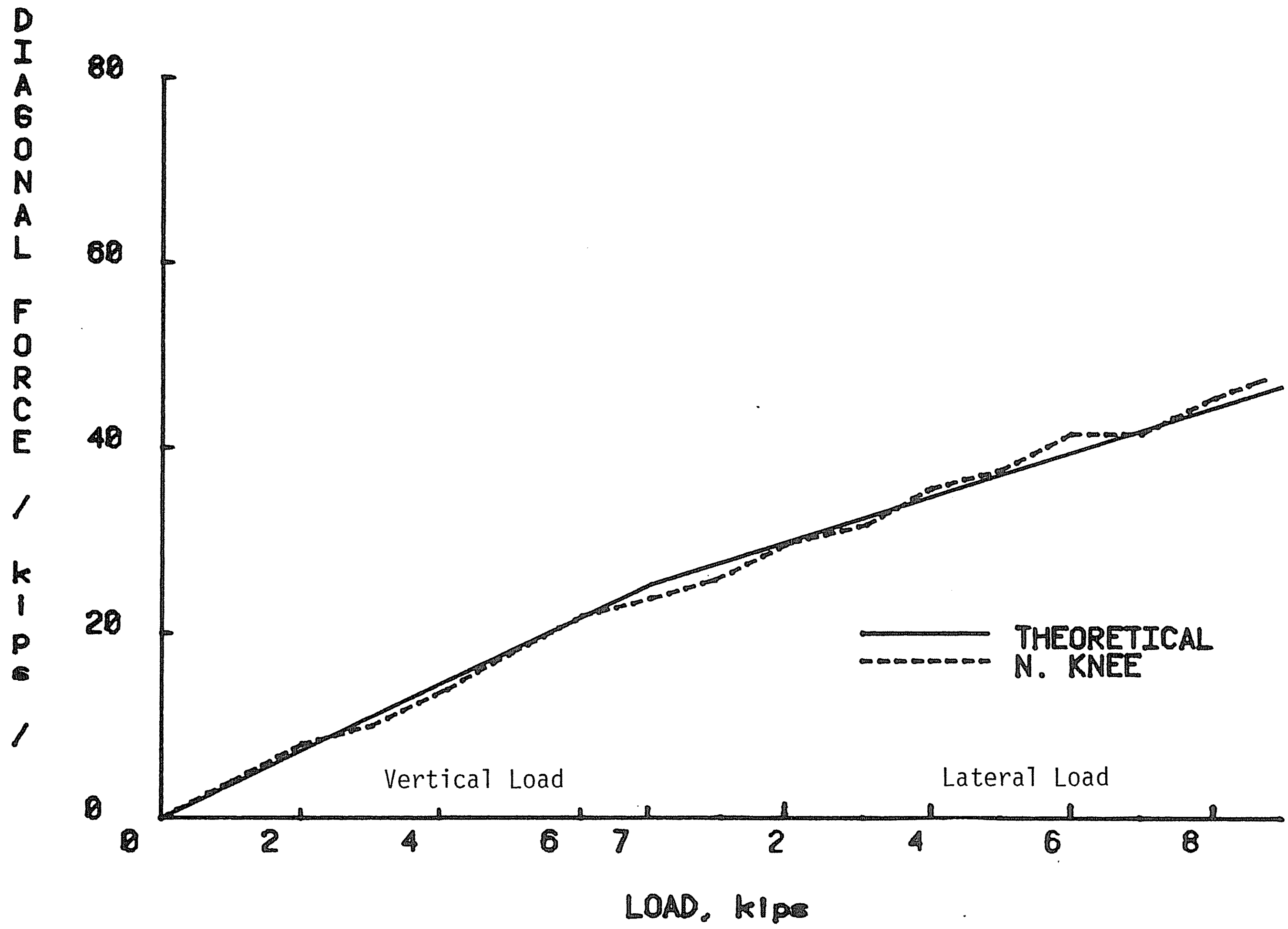


Figure F.7 KNEE DIAGONAL FORCE vs LOAD, ULL+WL: NORTH KNEE

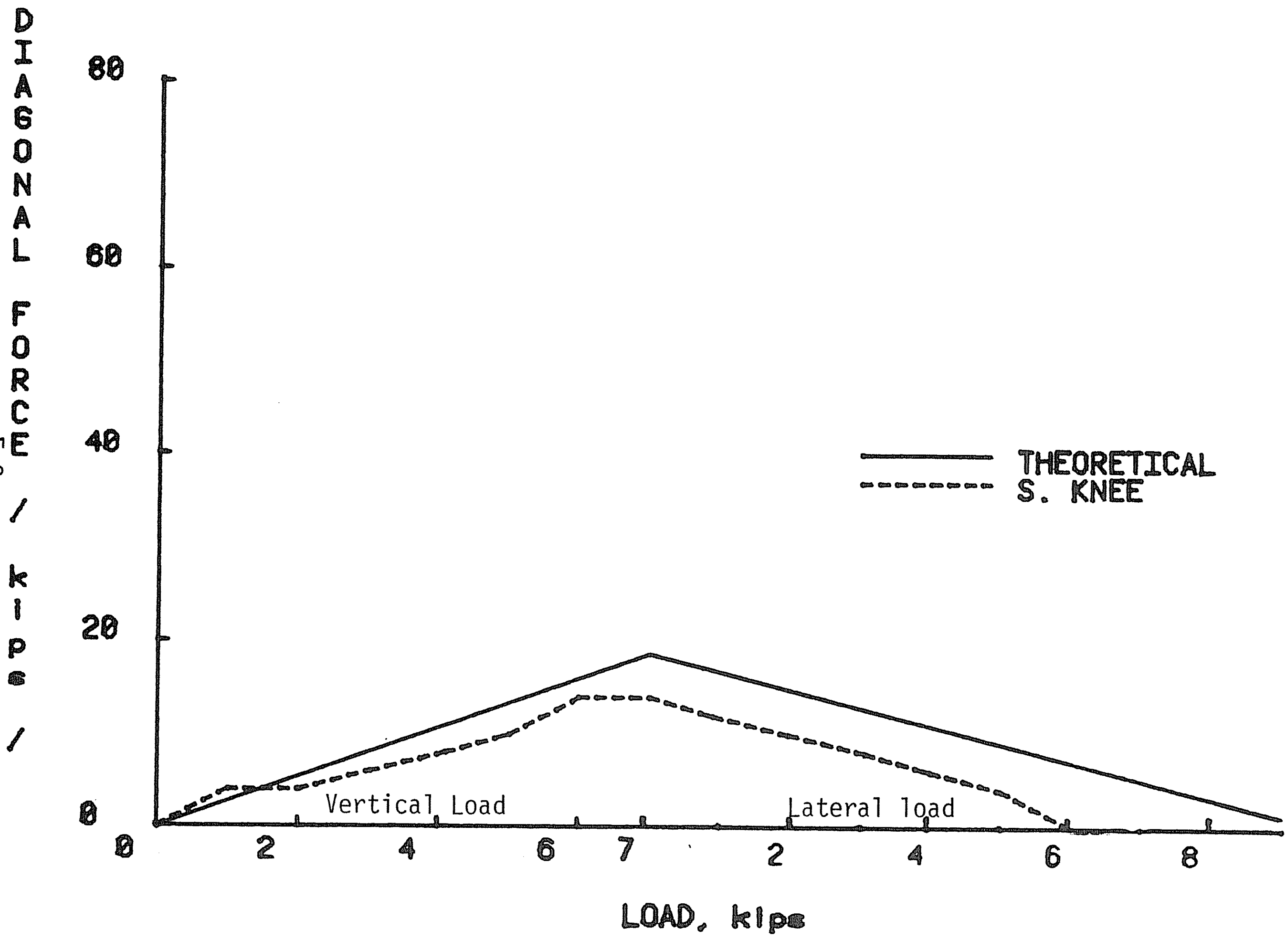


Figure F.8 KNEE DIAGONAL FORCE vs LOAD, ULL+WL: SOUTH KNEE

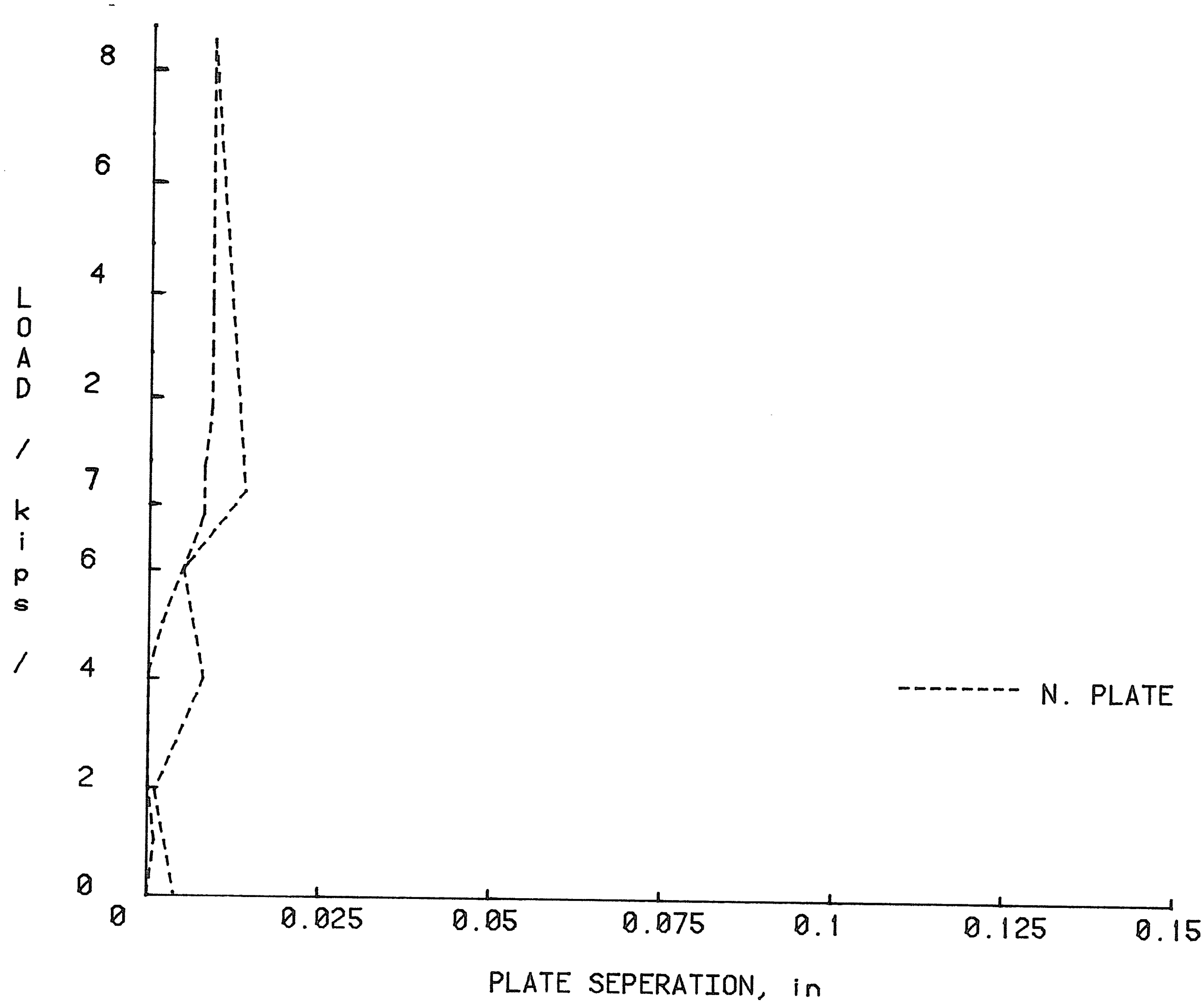


Figure F.9 LOAD vs PLATE SEPARATION, ULL+WL

APPENDIX G

GRAVITY LOAD TEST TO FAILURE OF WEST FRAME

(TEST 1.677LL)

VULCRAFT FRAME TEST SUMMARY

Project: Vulcraft Frame

Test No.: FR-1

Test Date: 8/6/84

Purpose: Test of 1.67 x full live load

Maximum Test Load: 13.77 kips

Failure Mode: Buckling of column flange

Discussion:

- The load deflection relationship during this test was linear with slightly greater deflections occurring than theoretically predicted.

- At 8 kips, the linear nature of the load-deflection relationship began to degrade. At 12 kips, loading was halted and incremental unloading of the frame was initiated. During the unloading sequence the load-deflection relationship remained linear. After the frame was unloaded to 4 kips the process was reversed and the load was again increased. During this loading sequence the load-centerline deflection relationship remained linear until a load of 11 kips was applied. At this point a gradual degradation of the line began.

- Small localized areas of yielding were indicated by flaking of the white-wash at 12 kips. At a slightly higher load severe yielding was observed in the column flange angles beneath the reentrant corner. At 13 kips, buckling of this flange was observed. With this buckling the load- deflection curve became virtually horizontal. At 13.77 kips the buckling was so severe that the frame lost its structural integrity and unloading began.

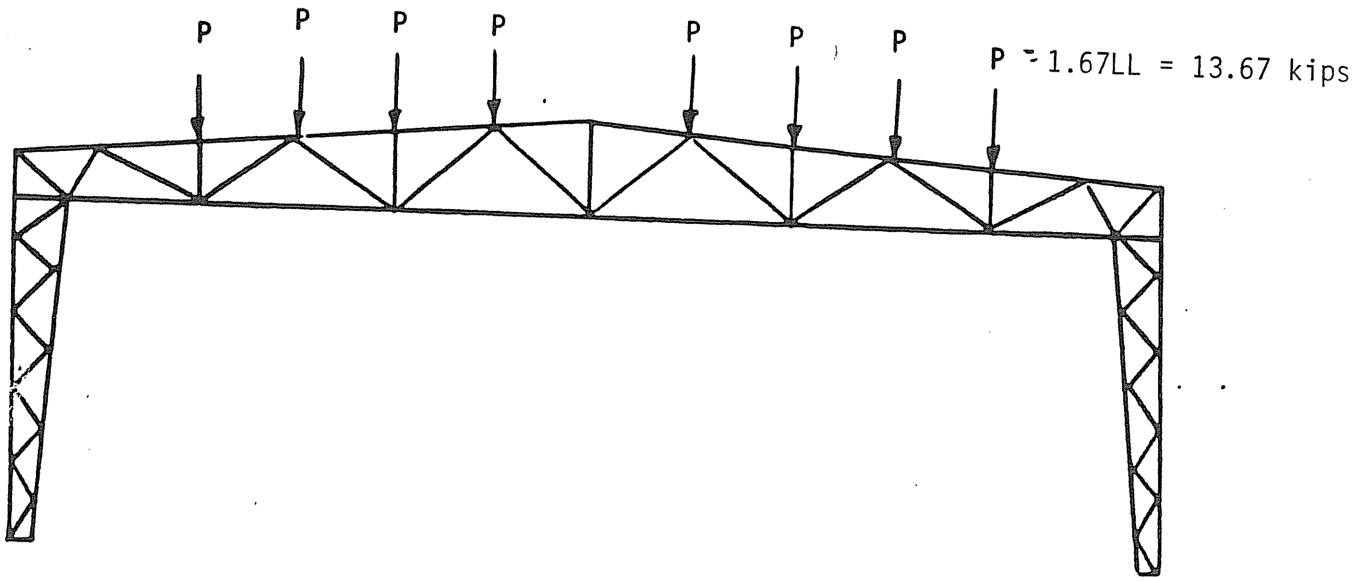


Figure G.1 Full Live Load

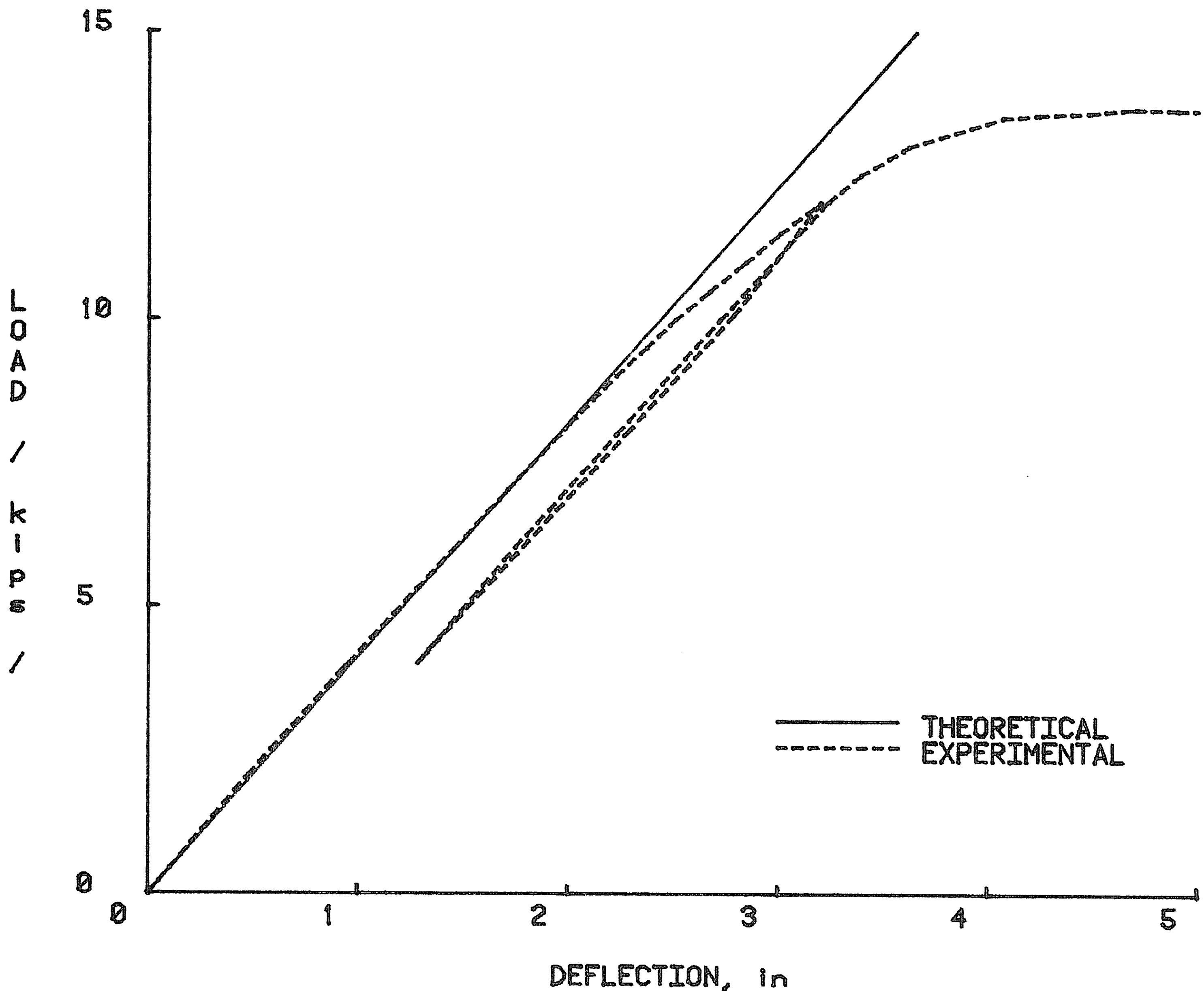


Figure G.2 LOAD vs CENTERLINE DEFLECTION, 1.67LL

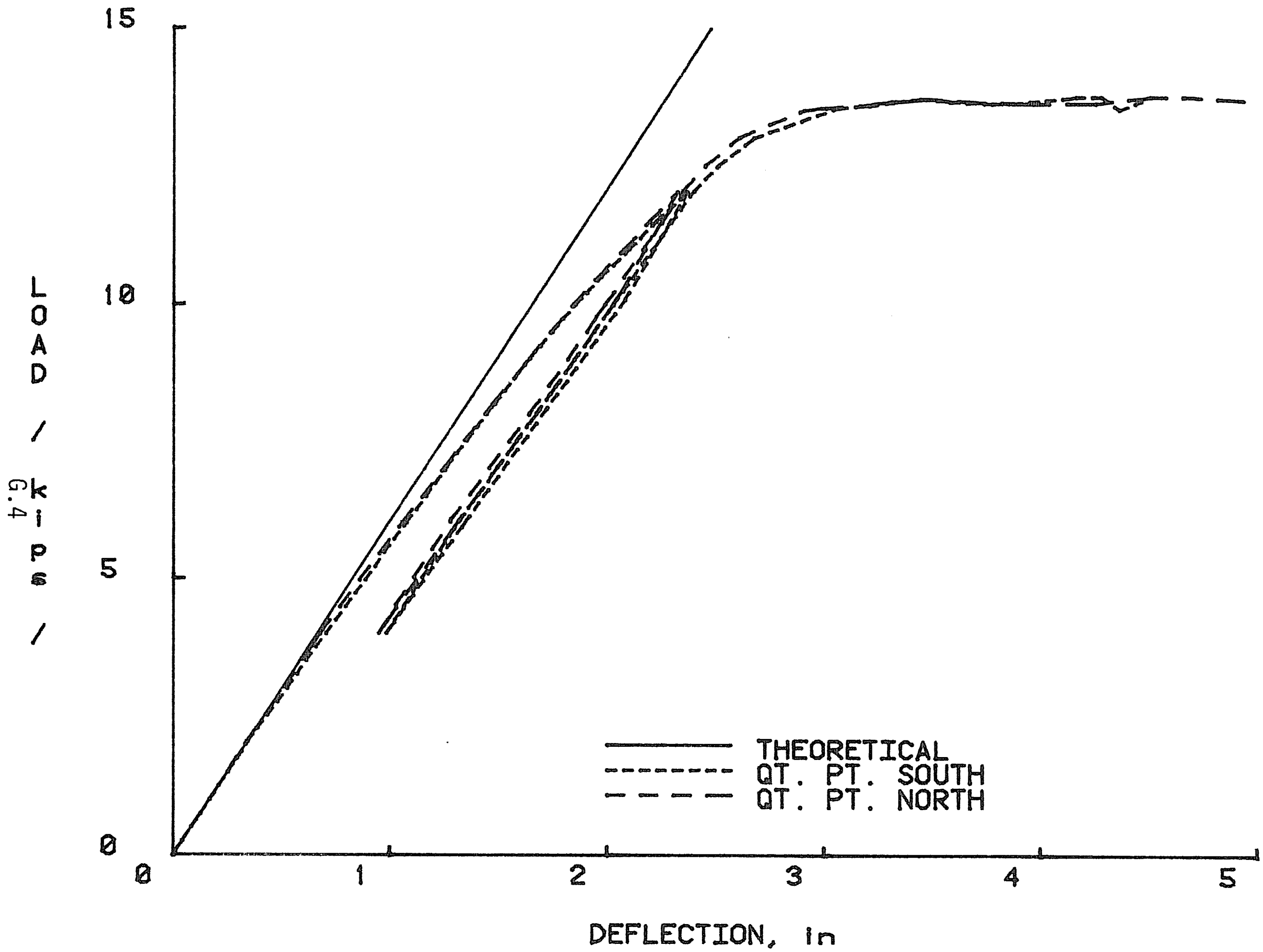


Figure G.3 LOAD vs QUARTERPOINT DEFLECTION, 1.677LL

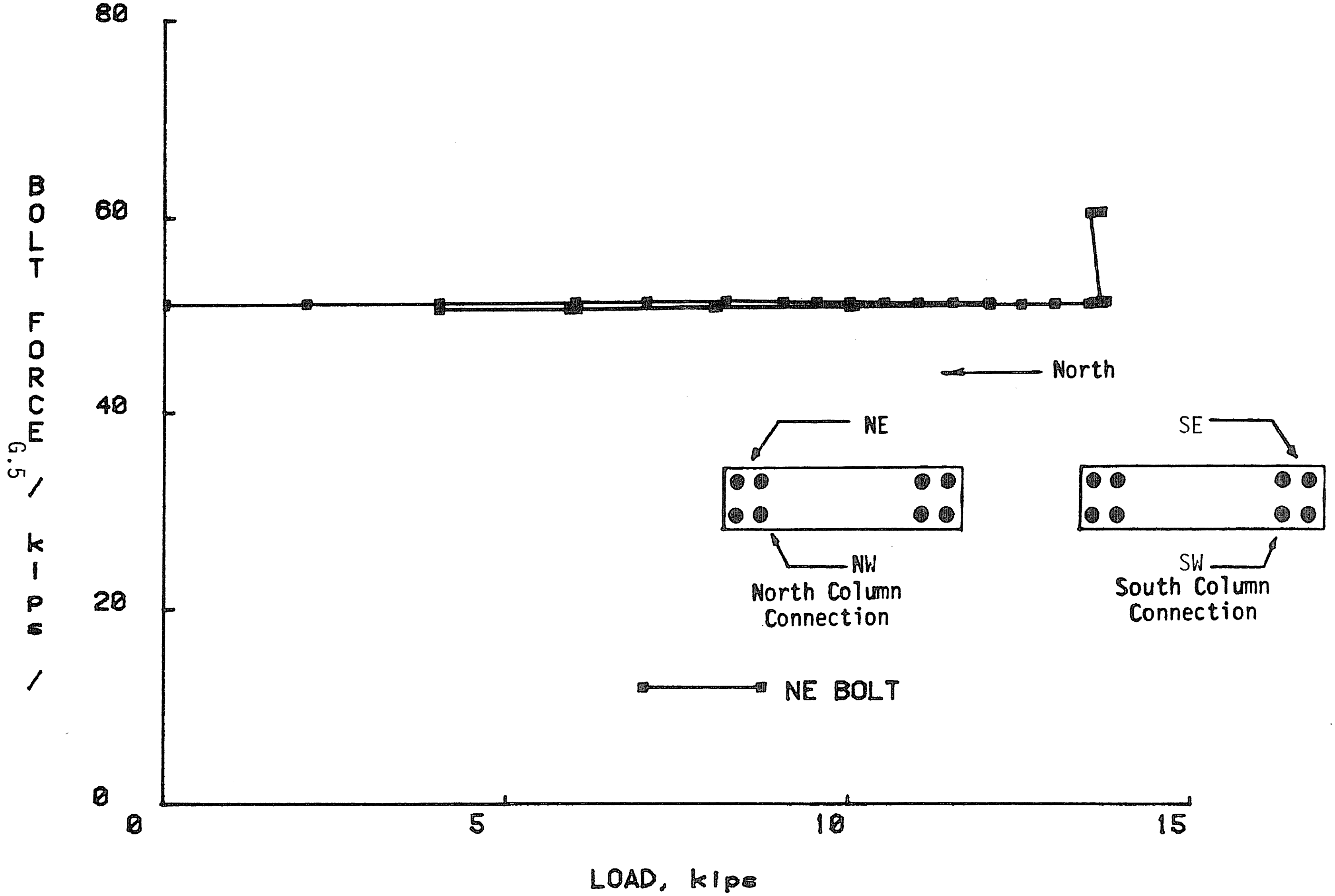


Figure G.4 BOLT FORCE vs LOAD, 1.67LL: NE BOLT

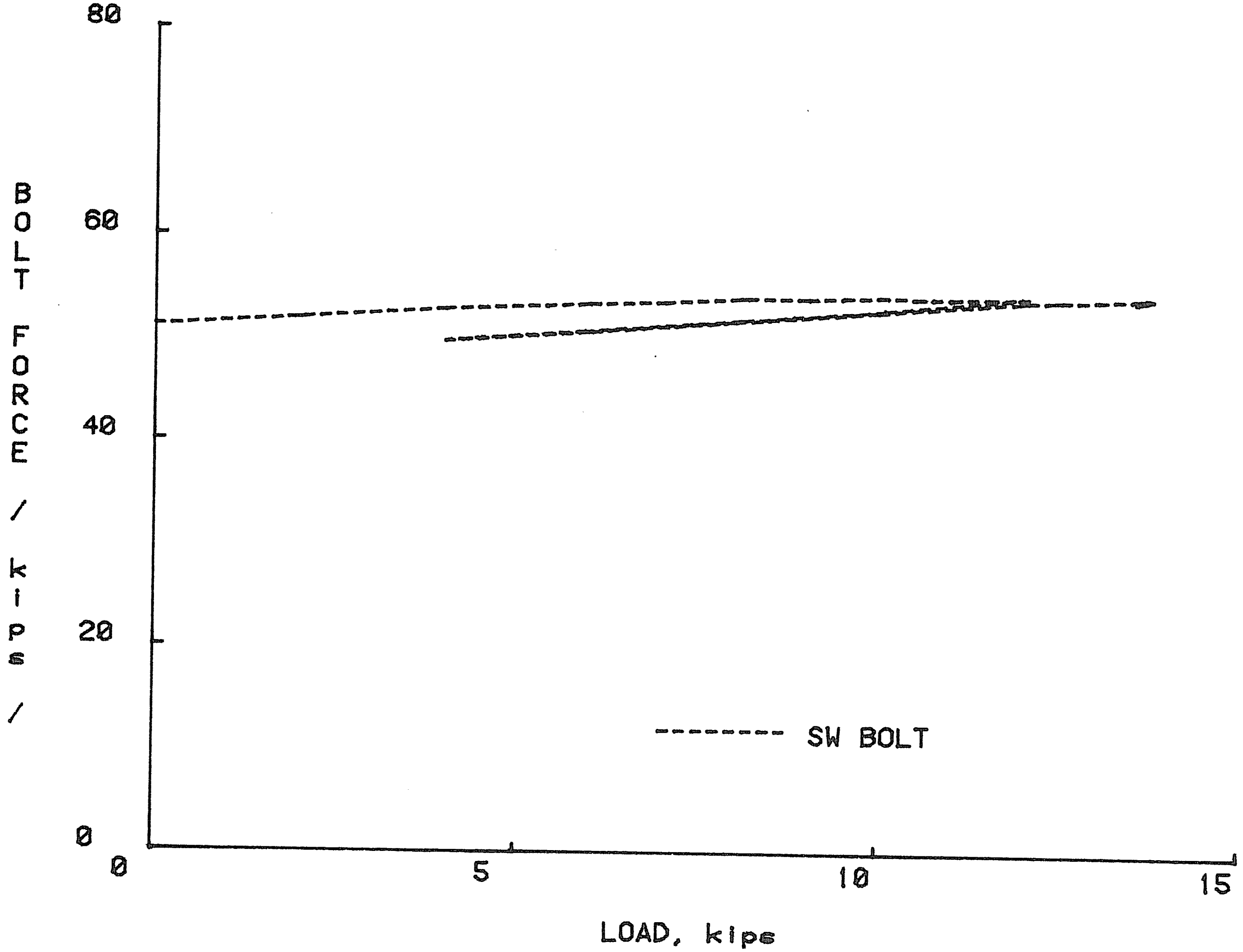


Figure G.5 BOLT FORCE vs LOAD, 1.67LL: SW BOLT

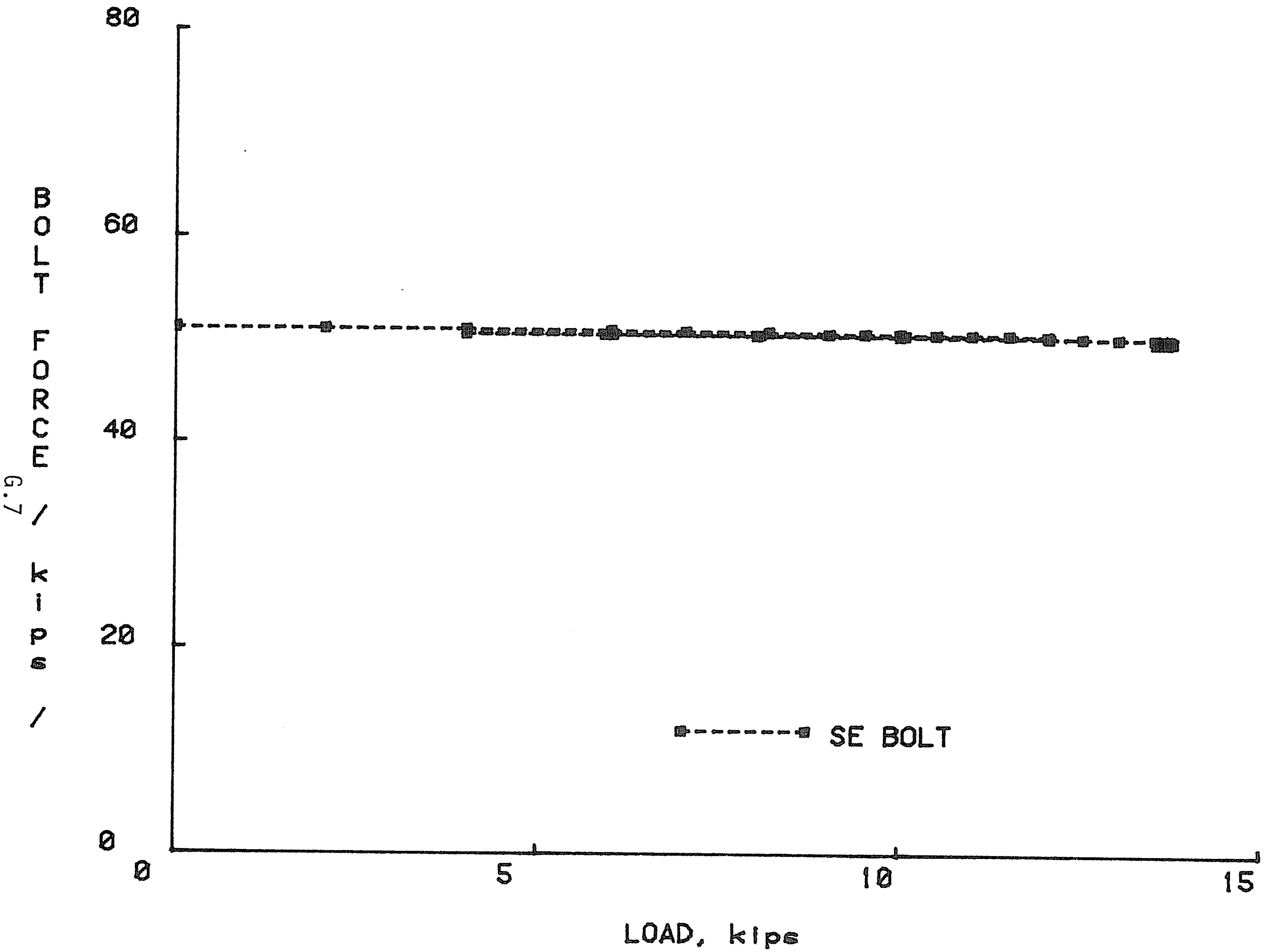


Figure G.6 BOLT FORCE vs LOAD, 1.67LL

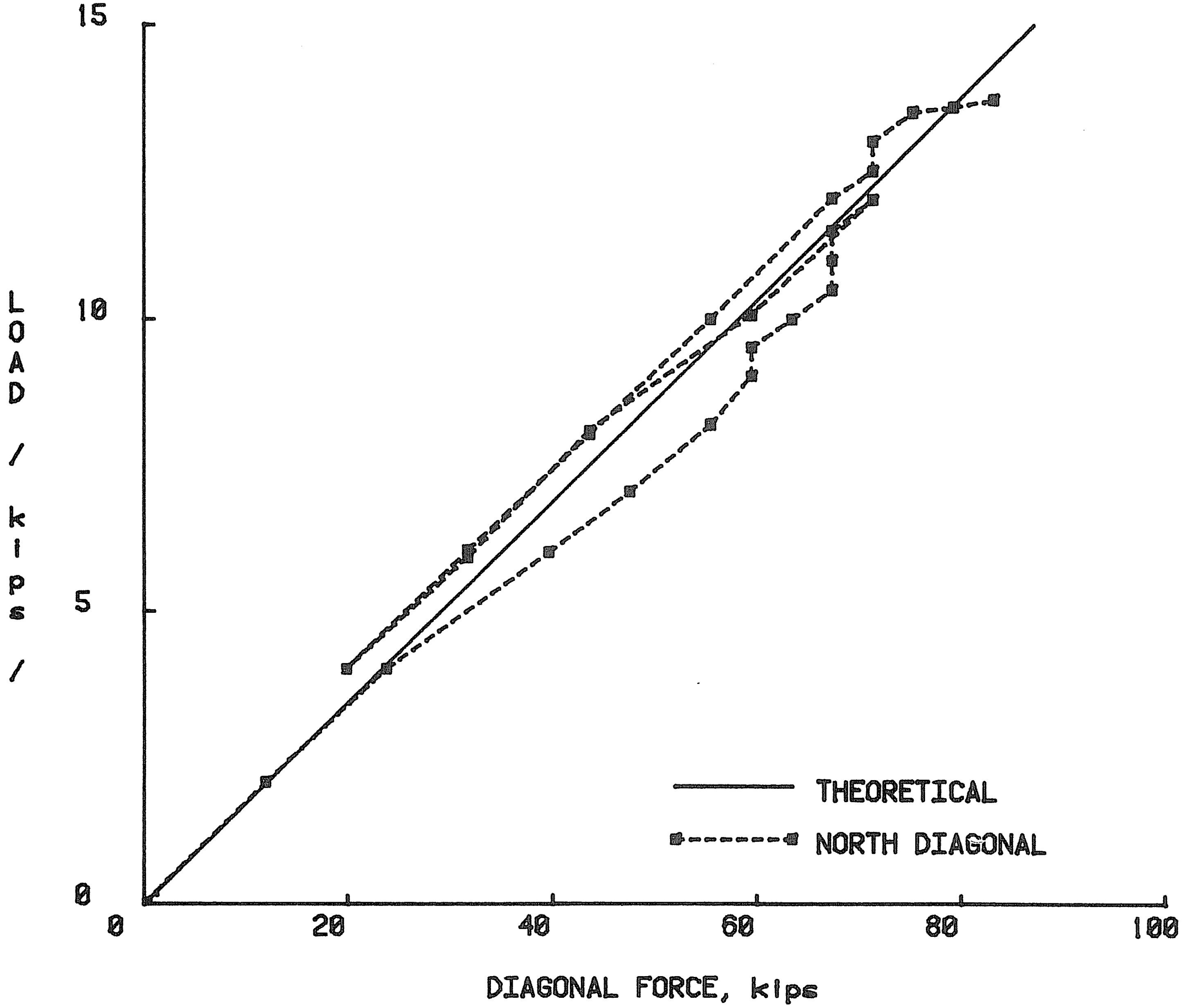


Figure G.7 LOAD vs KNEE DIAGONAL FORCE, 1.67LL: NORTH

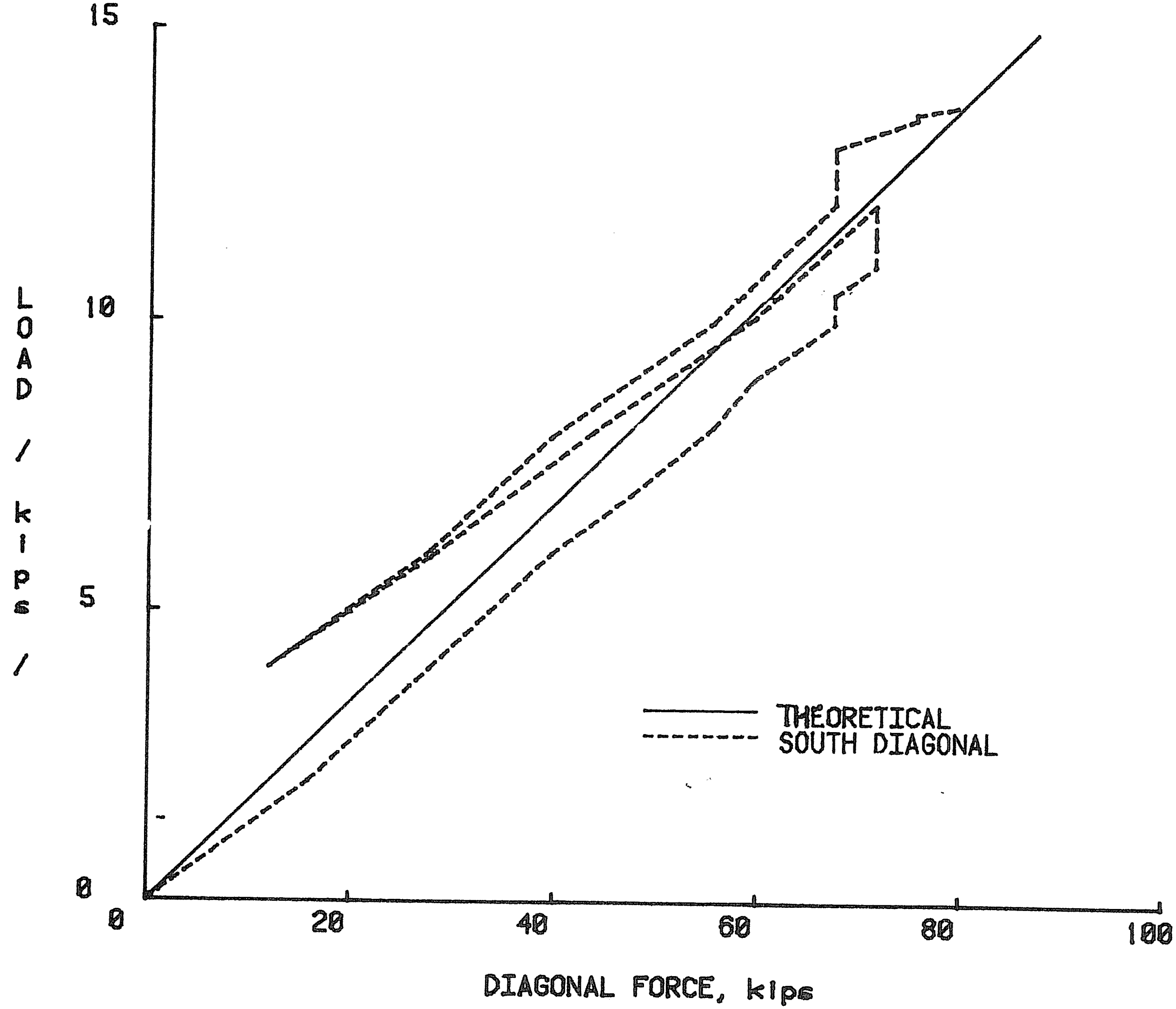


Figure G.8 LOAD vs KNEE DIAGONAL FORCE, 1.67LL: SOUTH

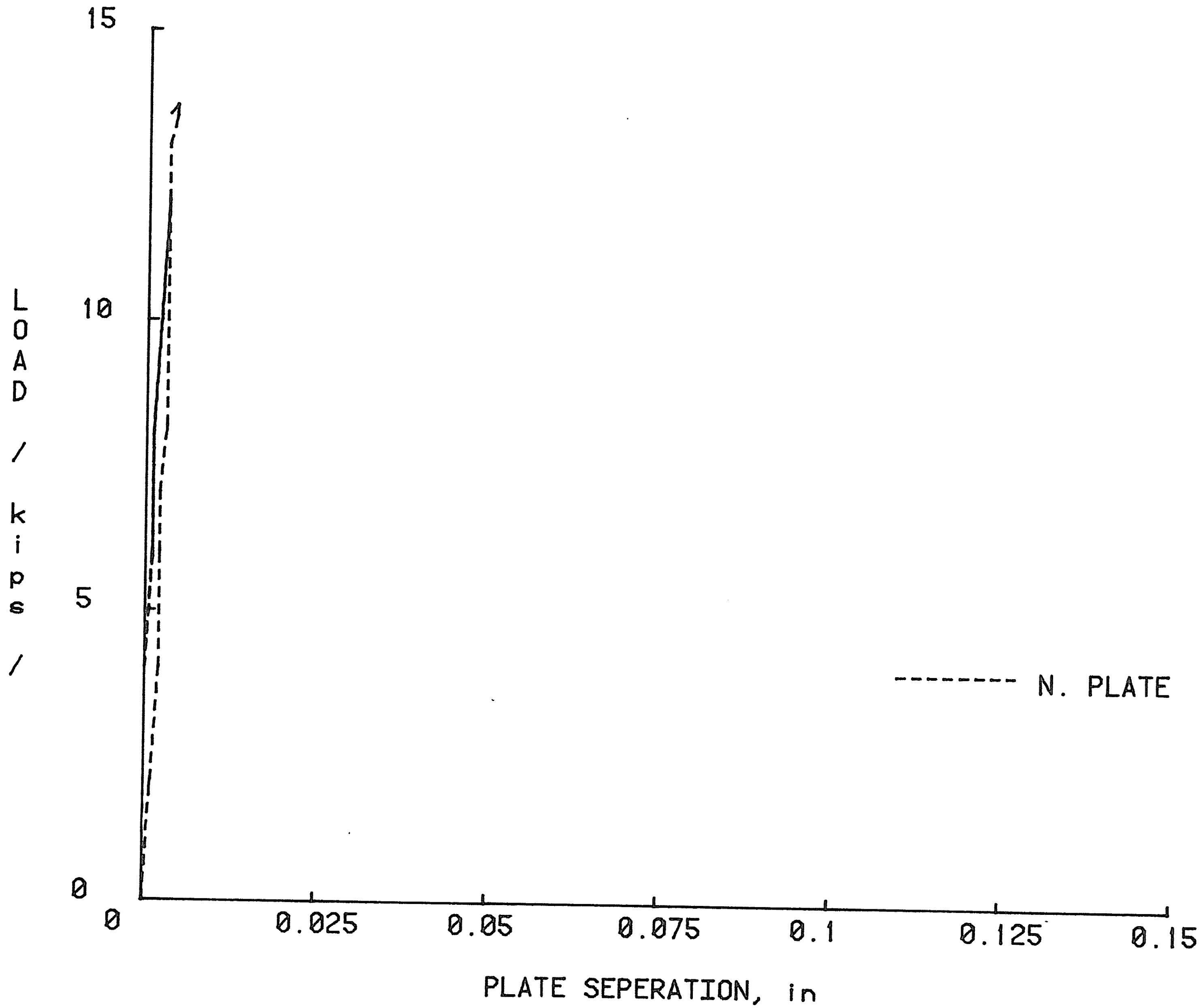


Figure G.9 LOAD vs PLATE SEPARATION, 1.67LL