DUCTILITY OF SINGLE PLATE FRAMING CONNECTIONS

by

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29 Apr. 75
Date

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ABSTRACT

Research on the strength and ductility of single web plate framing connections was conducted through the use of the finite element method of analysis. Mathematical models were constructed to simulate experimental test apparatus designed for the study of these connections. This analysis illustrates that the finite element models give essentially the same results as the actual experimental tests yielded.

Used in this finite element analysis is the establishment of a shear versus deformation response for a single bolted lap joint. This response was determined through a test fixture designed specifically for the study of this lap joint connection. The parameters obtained from this response were employed in the finite element analysis. The investigation of this connection also confirmed the premise that the ductility of the single web plate connection comes from the plate distortion at the bolts and not from shear deformation of the bolt itself for the plate sizes studied in this research.

CHAPTER 1

INTRODUCTION

Modern day shear framing connections are of various types, including double angle web connections, which have been the standard in the past, and single framing angles, single web plates, and structural tees which have been used in more recent designs as shown in Figures 1 and 2.

Single web plates and single framing angles offer several advantages over other types of shear framing connections. They provide for easier fabrication and erection, especially when they can be shop welded to the carrying beam or girder and field bolted to the beam. The single web plate also offers an economical advantage in that it requires less material than framing angle and structural tee connections. When comparing the double framing angle connections (field and shop bolted) to the shop welded, field bolted web plate, it can be seen that the relative cost for the double angle can be from 5 to 24% higher than that of the welded plate (Steel Committee, 1973, pp. 3-4). (However, it should be noted that the beam loading can influence the economics of the single plate connection through increments in weld size due to large loads.)

The single web plate, which has been extensively used in recent years, is designed against vertical shear only and is oftentimes referred to as a shear tab. Although the shear tab has several advantages

and an excellent performance record to justify its extensive use, there has been little research work done on the strength and ductility of this connection to back up its performance history.

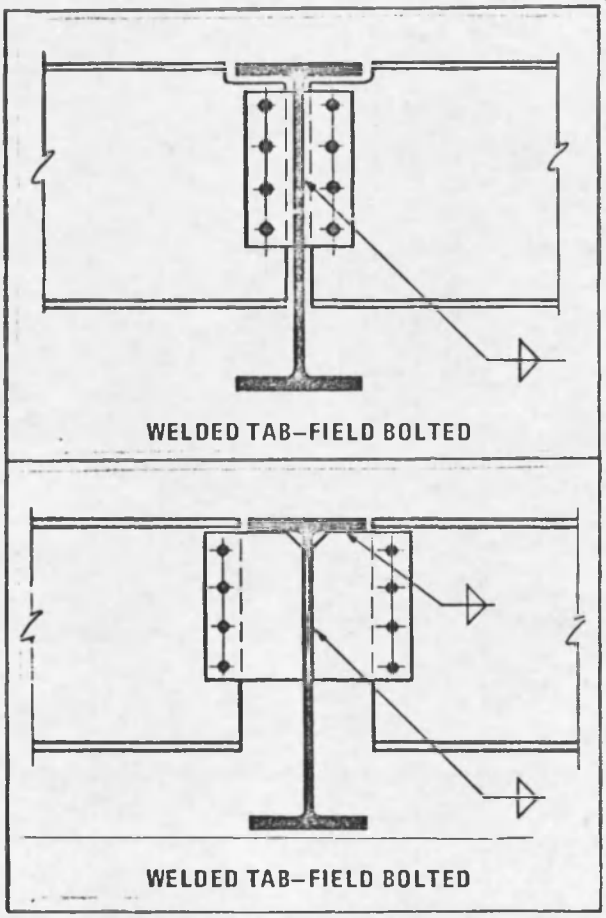


Figure 1. Single Web Plates.

Source: (Steel Committee, 1973, p. 2)

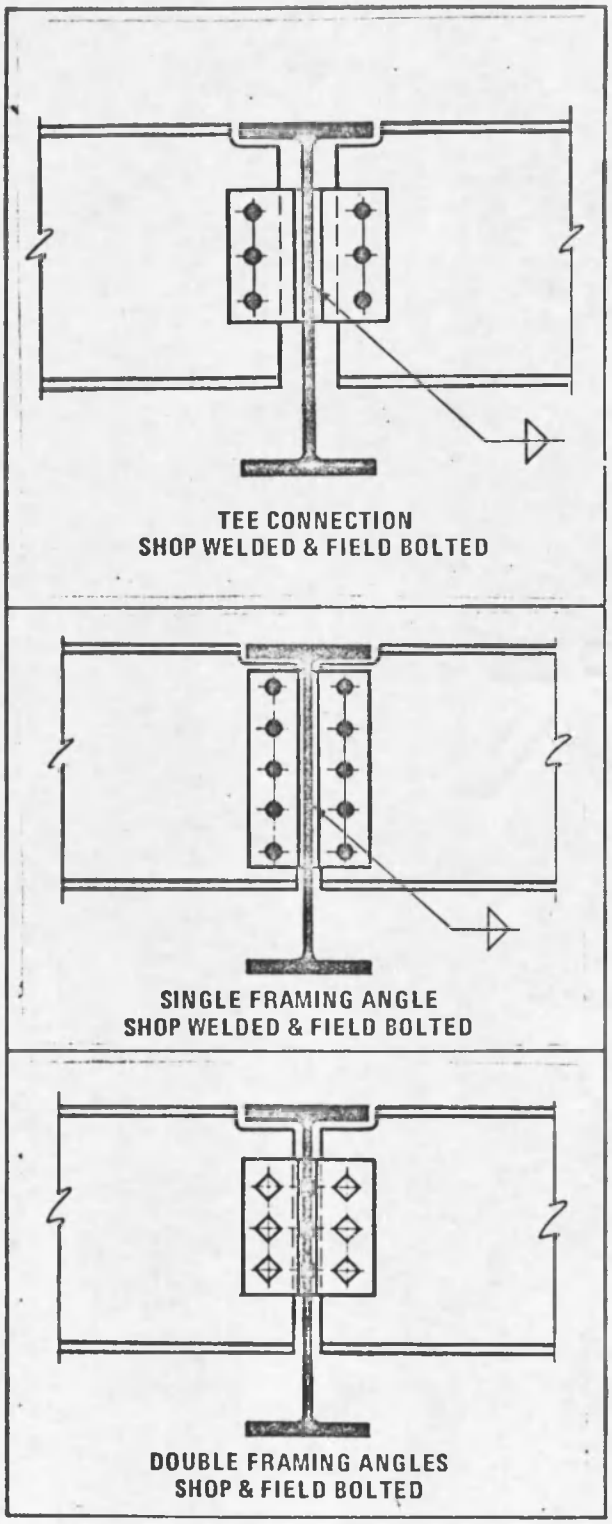


Figure 2. Framing Angle and Structural Tees.
Source: (Steel Committee, 1973, pp. 2-3).

CHAPTER 2

OBJECTIVES

The main objective of this research was to simulate analytically the experimental results obtained by Professor Samuel L. Lipson, M.S., in his report, "Single-Angle and Single-Plate Beam Framing Connections" (Lipson, 1968). If this could be accomplished, the strength and ductility of this type of connection could be generally established.

In order to achieve this main objective, two things had to be accomplished. First, the load versus deformation response was needed for a single bolt. This was required because in a shear tab connection the load on every bolt varies with respect to the C.G. (center of gravity) of the bolt pattern with the bolt load increasing as its distance from the C.G. increases.

The second objective was the implementation of finite element models which would efficiently replicate the experimental beam apparatus results obtained by Professor Lipson.

Through the use of program INELAS (Richard, 1968), the same experiments run by Professor Lipson could be run analytically, using the shear-deformation parameters obtained.

Upon completion of the analytical studies, a comparison could be made between the results that had been obtained in actual experimentation and the results obtained through INELAS; appropriate conclusions could then be drawn.

CHAPTER 3

TEXT FIXTURE

In order to achieve a representative bolt shear load versus deformation response, a test fixture was needed that would simulate the lap joint connection that occurs when a shear tab is bolted to the web of a beam.

This test fixture needed to be devised so that it would replicate the lap joint that occurs in Professor Lipson's experiments and also so that it would eliminate, or practically eliminate, any problems that may have arisen during test proceedings. First, the text fixture needed to be stiff enough so that its deformations and stresses would be small and fixture failure would not be a problem. Second, the fixture had to provide torsional rigidity for the test plates as the flanges on a beam provide for the web. Third, it was necessary that the fixture be designed so that there would be no eccentricity problems caused by the mounting of the device onto the Tinius Olson 200 kip testing machine which was to be used in the test. Fourth, it was essential to know exactly how much shear was being applied to the bolt at all times; and fifth, what the deformation was that occurred at any particular load value. Sixth, the fixture had to be set up so that a series of tests could be run with it. Not only did it have to accomodate for the interchanging of plates, but also for plates of varying thicknesses,

so that the shear force versus deformation response could be observed with these guidelines established.

The fixture as shown in Figure 3 was designed.

The original formulation of this text fixture called for the top and bottom heads and the right hand leg to be fabricated from 1" thick steel plate, but a check on the fixture's weight showed that it would weigh approximately 80 pounds. A reduction in thickness of these plates to 3/4" thickness would cause a decrement in fixture weight of 25%, or 20 pounds, which would make it more suitable for handling. It was believed that the 3/4" plates would still provide the rigidity the fixture needed.

The fixture's heads are connected to the right hand leg by pins (see Figure 3) in order that the moments in the test fixture to be zero are at these two points. The right hand leg and the left hand leg (formed by the test plates and bolts) are both equidistant from the fixture's center line which assured a load of one-half the total load (P) in the test leg. Although the separation of the left and right legs at the top head is slightly greater than that at the bottom head, the variation is small and has negligible effects on the P/2 load value in the left leg.

The apparatus was mounted using brackets or clevises as shown in Figure 4. These clevises passed through the heads of the Tinius Olson 200 kip machine and were pinned to the text fixture by 1" tooled steel pins. The load was then applied to the horizontal plates of the mounting brackets which transmitted the entire applied load to the test fixture.

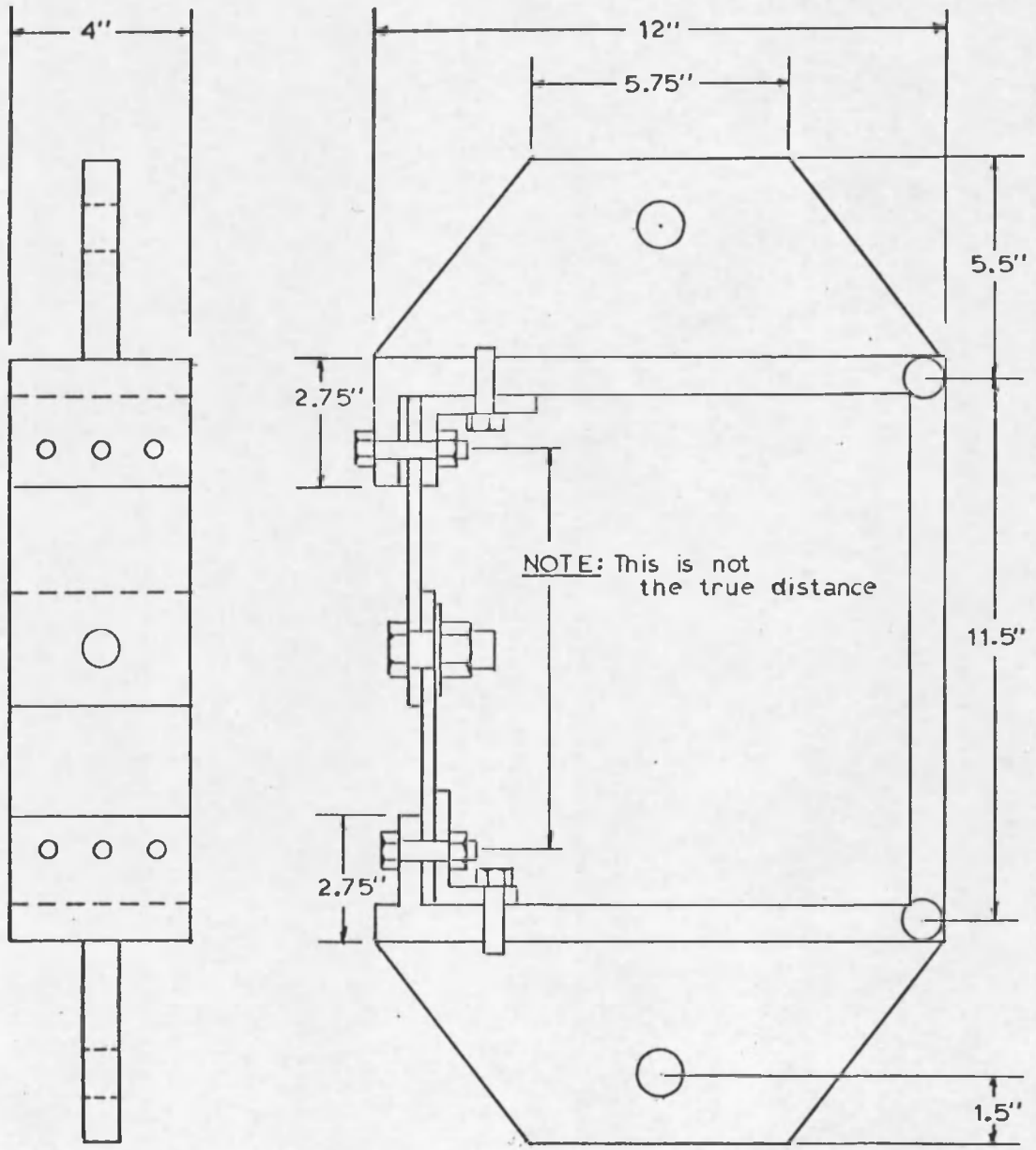


Figure 3. Test Fixture.

Figure 4. Clevis.

The test plates had a width of 4" and were of varying thicknesses. These are bolted to the test apparatus by three 1/2" steel bolts through the use of fixed ears and movable angles which are bolted down to the fixture's heads as shown in Figure 5. The connection at the top head also used a shim to the left of the test plate when needed to align plates of different thickness. This 4" test plate width and the clamping effect achieved through bolting these plates in place gave the lap joint formed the torsional rigidity it required. The plates could then be bolted together, thus completing the lap joint.

The lap joints consisted of two steel plates bolted together with a 3/4" A325 steel erection bolt. One set of plates under experimentation consisted of 3/8" and 1/4" plates which models the type of lap joint achieved by Professor Lipson. In Lipson's (1968) experiments, he used 1/4" single web plates bolted with A325 bolts to a WF 21 x 62 which has a thickness of 0.4" (3/8" = 0.375"); then these bolts were torqued to 356 ft.-lbs. Similarly, the single bolted lap joint in the test fixture was torqued to 350 ft.-lbs. (the maximum capacity of the torque wrench available).

To measure the deformation that occurred in the lap joint, it was decided that the change in distance between Points A and B shown on Figure 6 was to be used. This would yield an approximate measure as to how the center of the bolt moved from its original position due to shearing and bearing deformation. Plate elongation and distortion are conceivable cause for error in this measurement. The elongation due to the tensile loading and surface frictions amounted to less than 2% of the total deformation and could therefore be neglected. The distortion

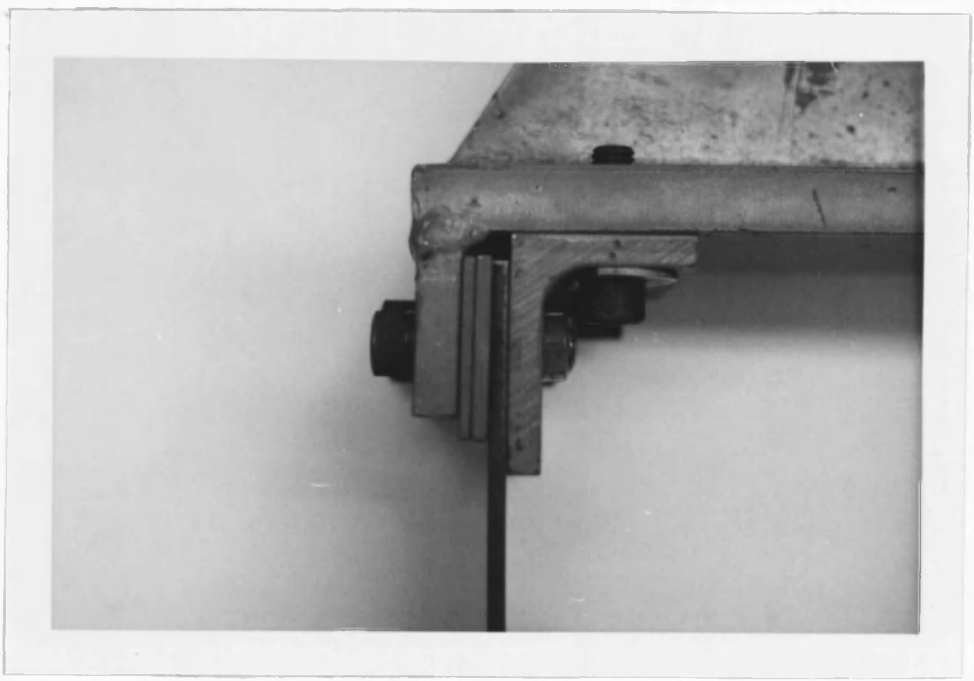


Figure 5. Attaching of Test Plate to Test Fixture.

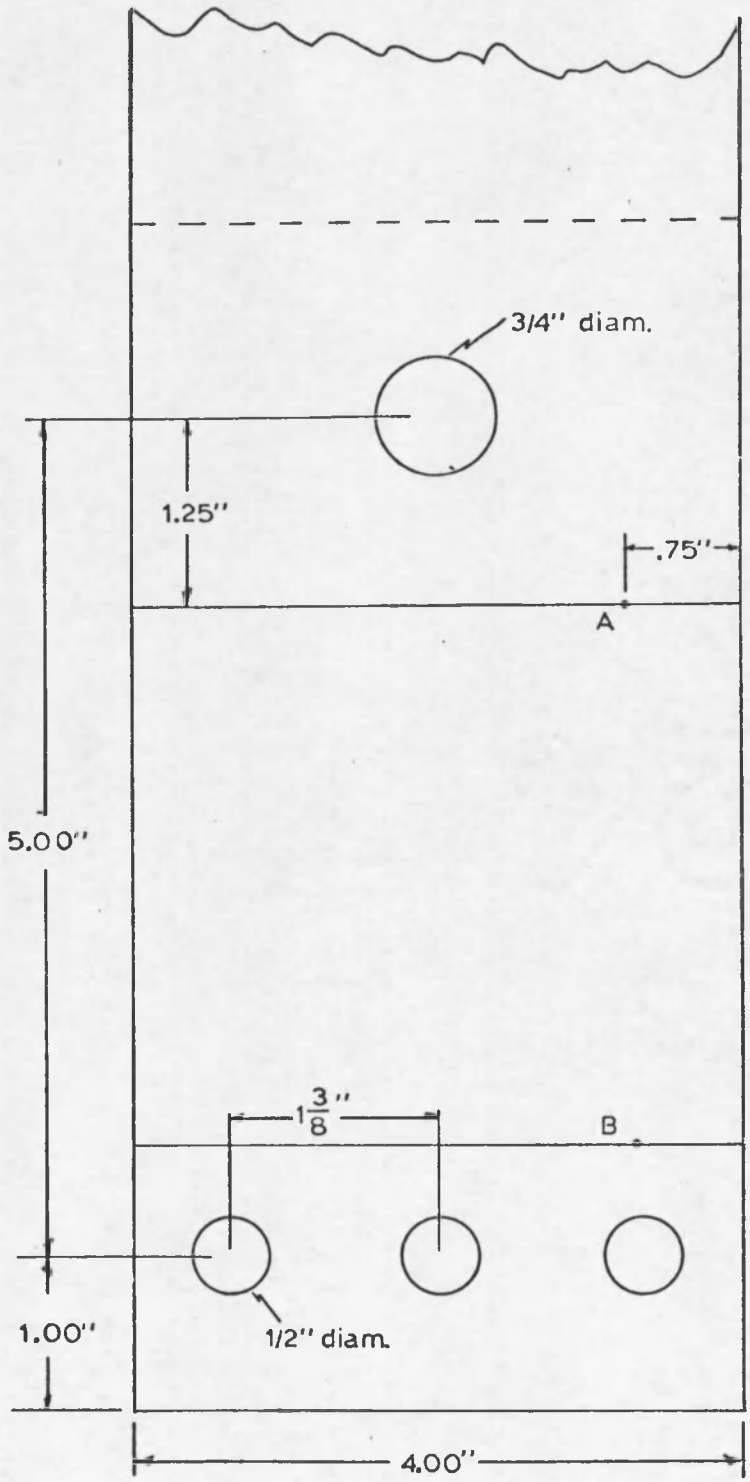


Figure 6. Test Plates.

along the line of the 1/2" attaching bolts was also found to be negligible by inspection of test plates.

In order to make this measurement, a set of 10" calipers and a Lufkin dial gauge were used as shown in Figure 7. The gauge had a total range of 0.2" with 0.0001" graduations and was mounted on the lower arm, half-way between the calipers' pivot pin and measuring tip. The gauge was mounted so that it unwound as the calipers opened. This was done as a precaution against having the gauge slam shut in case of complete plate or bolt failure. If such failure occurred, the gauge would just pop open and would undergo no damage.

The measuring device was then mounted on an adjustable stand to support it at its proper elevation for gathering of data.

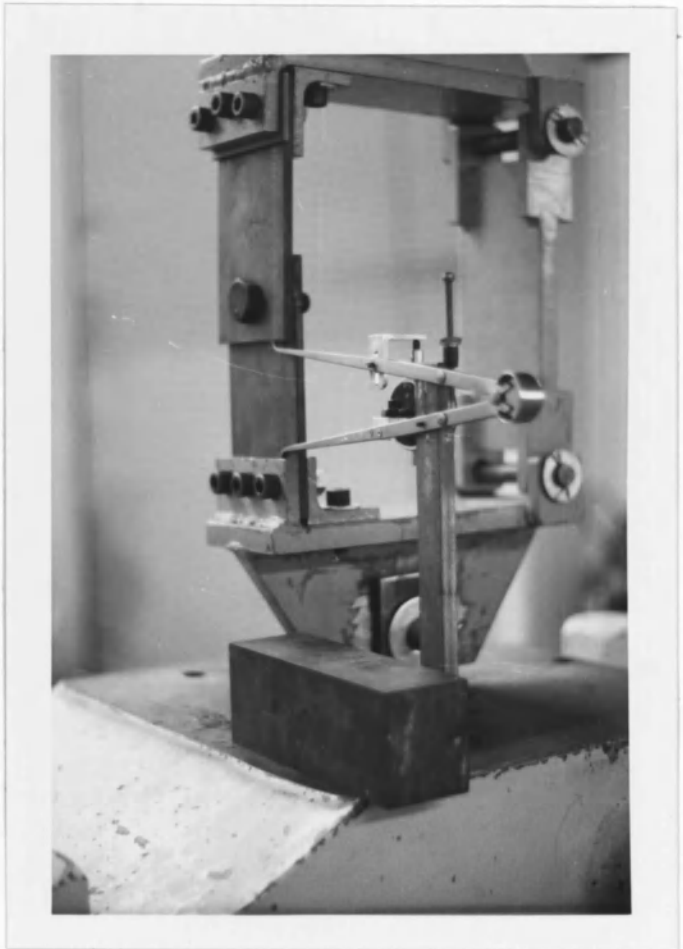


Figure 7. Measuring Device.

CHAPTER 4

COMPUTER PROGRAMS

The computer program INELAS (Richard, 1968) is used for static inelastic analysis of a three-dimensional structure comprising two dimensional finite elements. For isotropic elements, the nonlinear response, which is based upon incremental plastic theory, uses a numerical algorithm that yields practically identical results to the Von Mise's yield criterion and associated flow rule. To implement the nonlinear analysis a three parameter uniaxial stress-strain relation is used. The nonlinear differential equations that describe the structure's nonlinear response can be solved by two techniques:

(1) The first order Euler method, and

(2) The fourth order Runge Kutta method.

Boundary displacements and loads can be specified wherever desired or needed. The program is currently being run on the Control Data Corporation (CDC) 6400 computer.

PARAM (Abbott, 1974) is a program written for the Hewlett Packard (HP) 9100. This program gives the stress-strain parameters mentioned in INELAS based on the modulus of elasticity (E), the plastic modulus (Ep), and appropriate secant moduli. The parameters obtained are n, a function of the curve's shape and σ_o, a reference stress.

NETPLOT (Malvick, 1968) is a program which utilizes the CDC 6400 computer and the CALCOMP plotter. The program plots the finite element grid network from any chosen vantage point in space.

CHAPTER 5

TEST RESULTS

The test fixture was loaded gradually and the separation of Points A and B (see Figure 6) was recorded at 2500 pound intervals; therefore, a deformation value for every 1125 pounds of shear was obtained. Upon completion of the shear test, all gauge deformation values were doubled to obtain the actual separation of Points A and B and then plotted against the corresponding shear values in order to obtain the shear versus deformation response. (For tabulated experimental data, see Appendix A.)

Four sets of plates were tested and the results studied. These sets consisted of 3/16" vs. 3/16", 1/4" vs. 1/4", 1/4" vs. 3/8", and 5/16" vs. 5/16" thick plate.

The first set of plates under observation consisted of two 1/4" steel plates. Four tests were run, each test using a new set of plates and a new A325, 3/4" bolt. The data was plotted and the results can be seen in Figure 8.

Initially the elastic (K_e) response taken was 4.2×10^5 lb./in. as indicated in Figure 8, but the finite element analysis results of the pure-moment apparatus showed a lack of initial stiffness in comparison to those of Professor Lipson's (1968) tests. This meant that what appeared to be the obvious K_e value for the shear-deformation was in error.

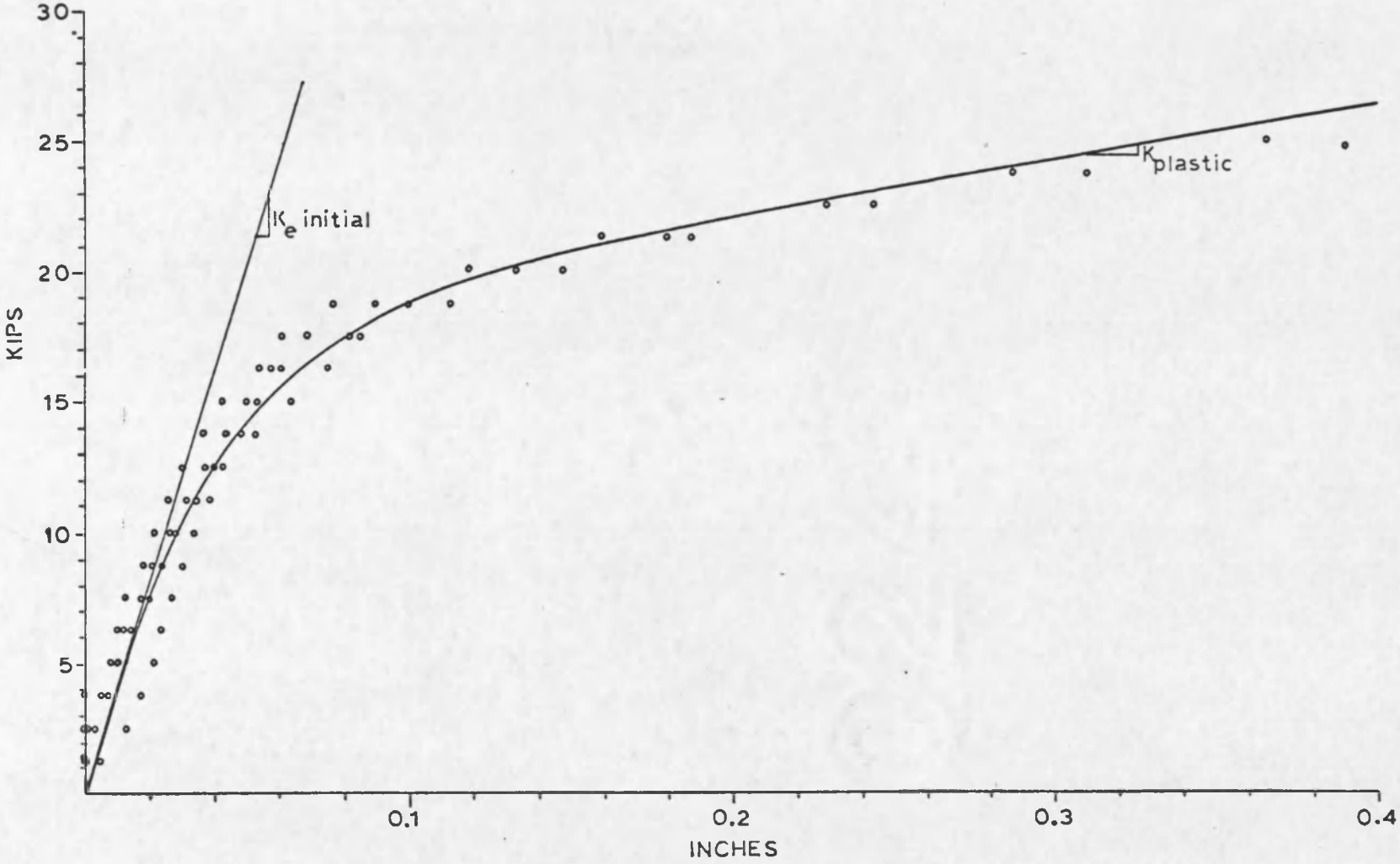


Figure 8. Initial Shear vs. Deformation Response (1/4" plates).

Upon consideration of this problem, it became apparent that slippage between the bolts and the plates was the cause of the initial flexibility. When measured with a micrometer, the 3/4" holes averaged 0.758" in diameter while the 1/2" holes averaged 0.503" and the bolts were 0.001 to 0.002" undersized. This meant that there could be a 0.015" of movement in each plate or that a total 0.03" of slippage could occur in a set of plates.

From the test data (Appendix A), it can be seen that major slips occur, as shown in Test No. 2, for 1200, 2500, and 3750 pounds of shear and at 7500 pounds of shear in Test No. 3. The test data also illustrates that in some cases the K_e value initially appears to be extremely high because no measurable deformation occurred until the bolt shear load reached a value as high as 3750 pounds, as shown in Test No. 1. Also, once deformation begins and appears to increment without the occurrence of major slip, it is possible that the measured deformations are a combination of actual bolt-plate deformation and minor slippage.

Through studies of unpublished Chance Vought tests for single fastener lap joint stiffnesses (see Appendix B), Professor R. M. Richard determined that a K_e could be established by the following equation:

$$K_e = \frac{2E(t_1 t_2)}{(t_1 + t_2)}$$

for two plates of the same material in which E is Young's Modulus, and t_1 and t_2 are the plate thicknesses.

This yielded a K_e value of 7.25×10^6 lb./in., an elastic response almost 175 times stiffer than the observed value from the load-deformation curve which included slip. This response value proved to be

effective in giving the necessary initial stiffness in the finite element model. This topic is further discussed in Chapter 3.

Through the application of program PARAM, the curve shown in Figure 9 was adopted as the load deformation response for the 1/4" plates bolted together with a 3/4" A325 bolt torqued to 350 ft.lbs.

It is important to note that the response discussed is not solely the response of an A325 3/4" bolt under shear, but the response of that bolt due to shearing forces applied through 1/4" steel plates. The test results show that under these conditions no visible marks of shear deformation occur in the bolt. Instead, there is yielding of the plate material around the bolt hole as shown in Figure 10. Thus the shear-deformation response obtained is not only a function of bolt type and diameter, but also of the type of plates it joins together.

The situation where one plate is 1/4" and the other is 3/8" closely simulates Professor Lispon's (1968) 1/4" shear tab attached to a beam web of 0.4". Three tests were run for this set of plates, all yielding acceptable data.

The K_e value calculated for this set of plates was 819×10^6 lb./in. K_p (the plastic response) was chosen from the response curve to be approximately 1.25×10^4 lb./in. (see Figure 11). The solid line that can be seen in Figure 11 is the curve given by program PARAM (Abbott, 1974) for the data points using the K parameters discussed.

The resulting shear-deformation response shows little difference from the response obtained in the two 1/4" plate study (Figure 14, p. 26). It appears that in this second plate set the 3/8" plate gives the lap joint a small additional amount of initial stiffness, but the yield

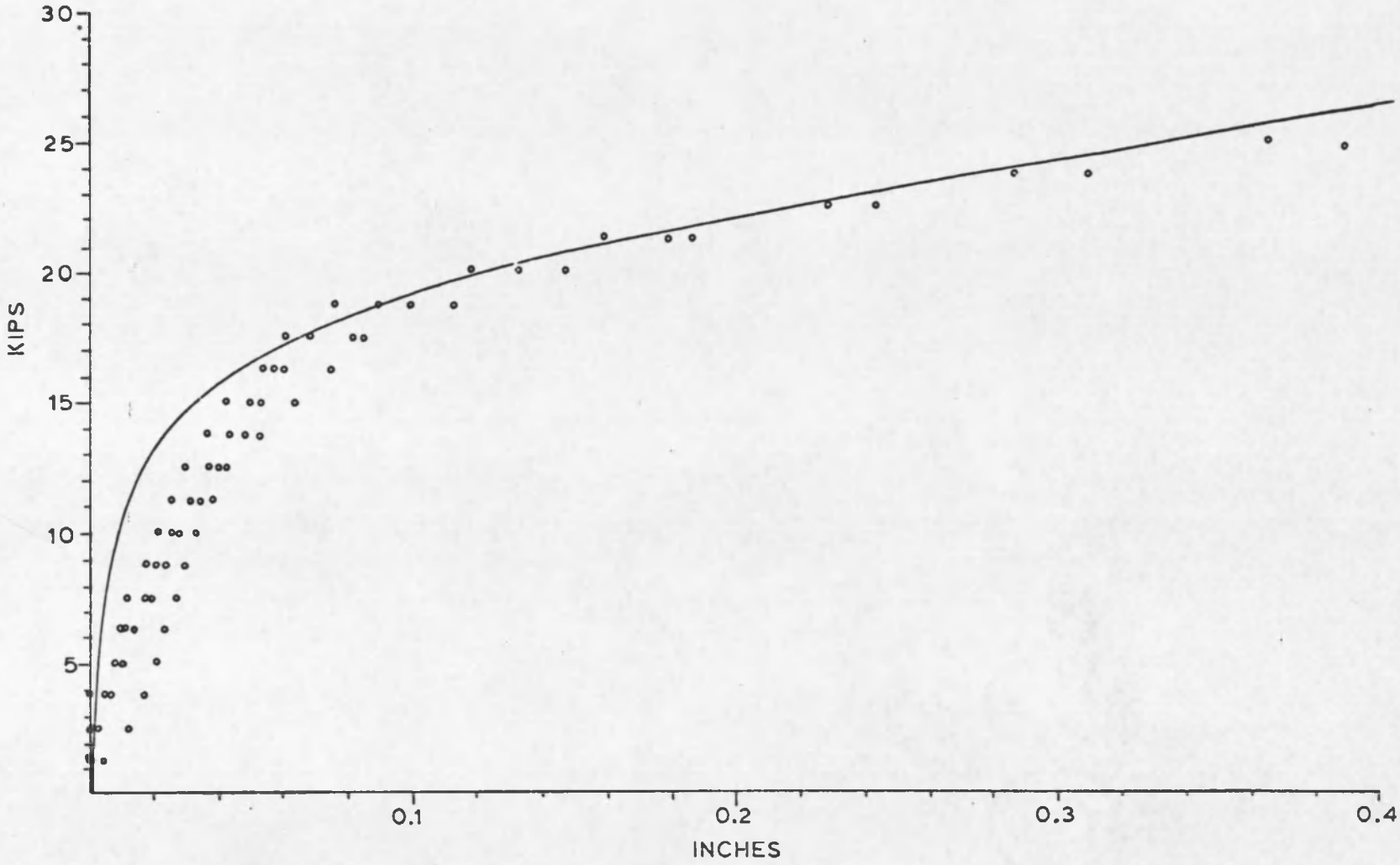


Figure 9. Final Shear vs. Deformation Response (1/4" plates).

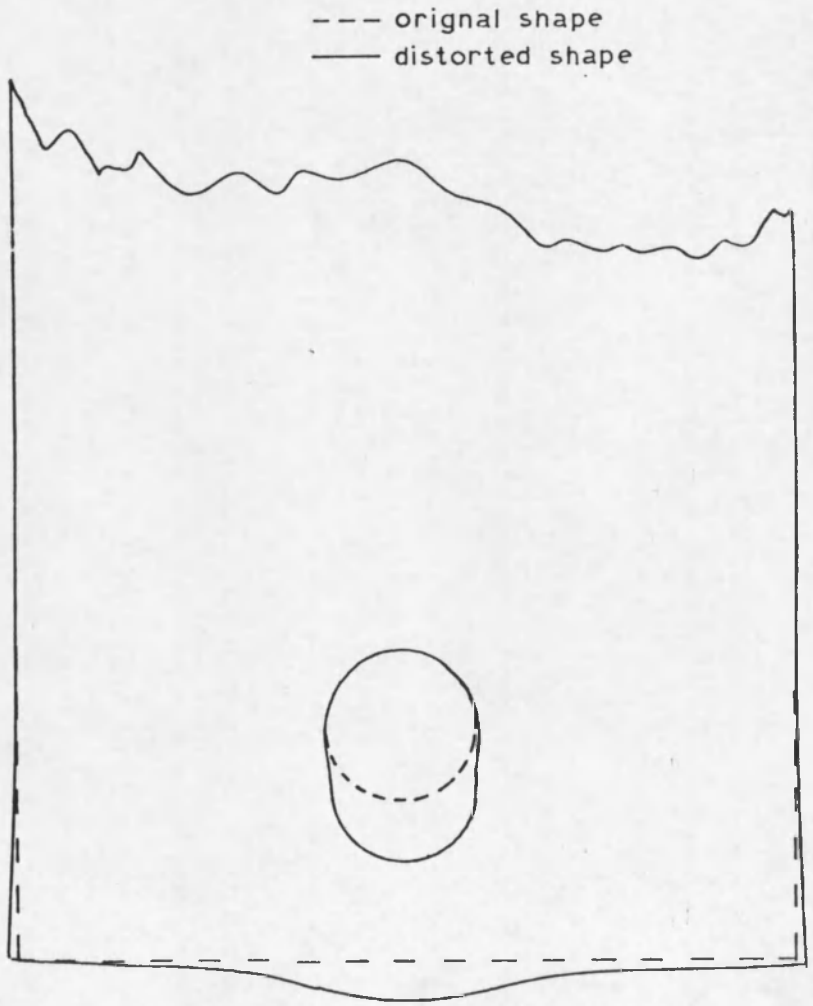


Figure 10. Distorted Test Plate.

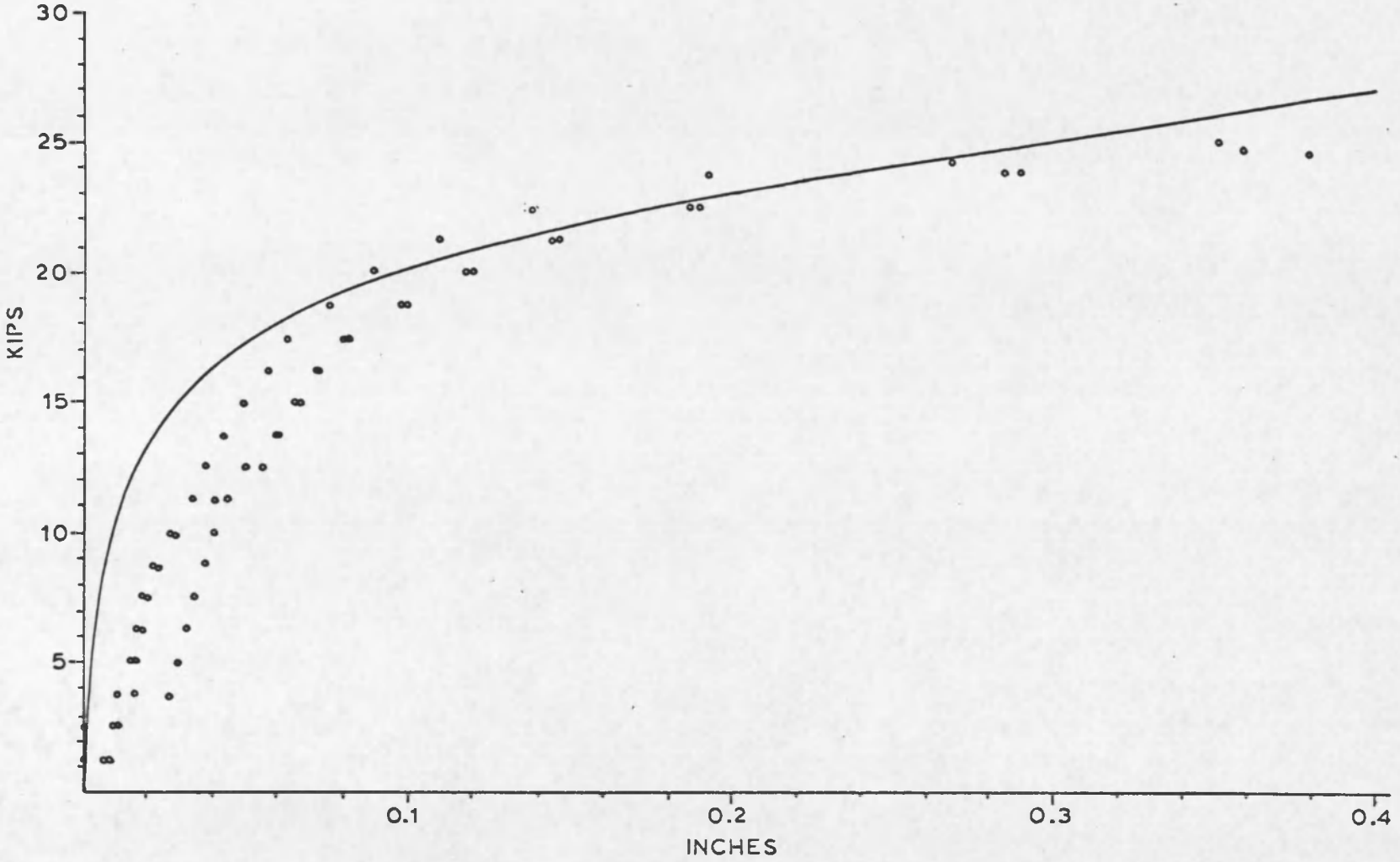


Figure 11. Shear vs. Deformation Response (1/4" and 3/8" plates).

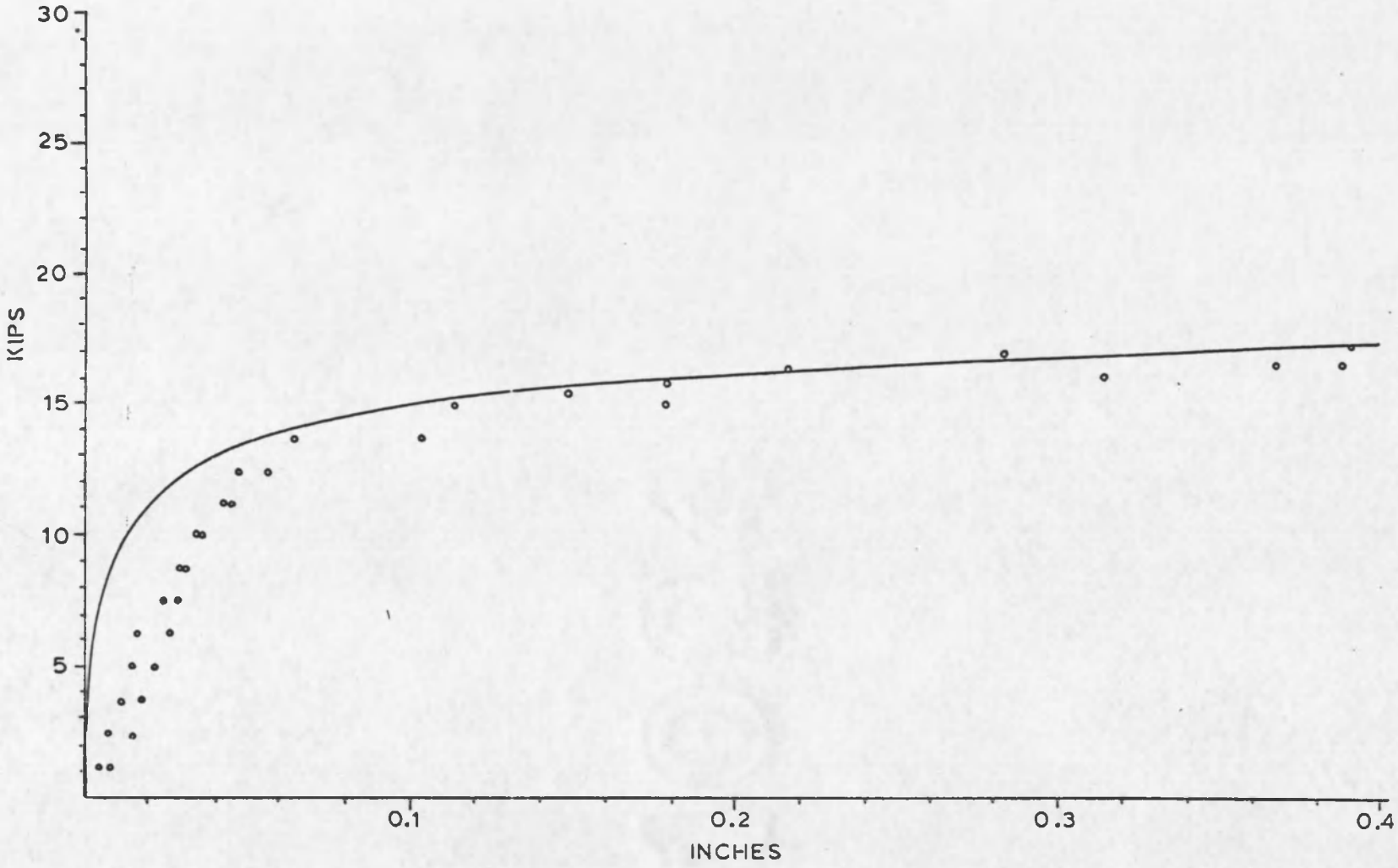


Figure 12. Shear vs. Deformation Response (3/16" and 3/16" plates).

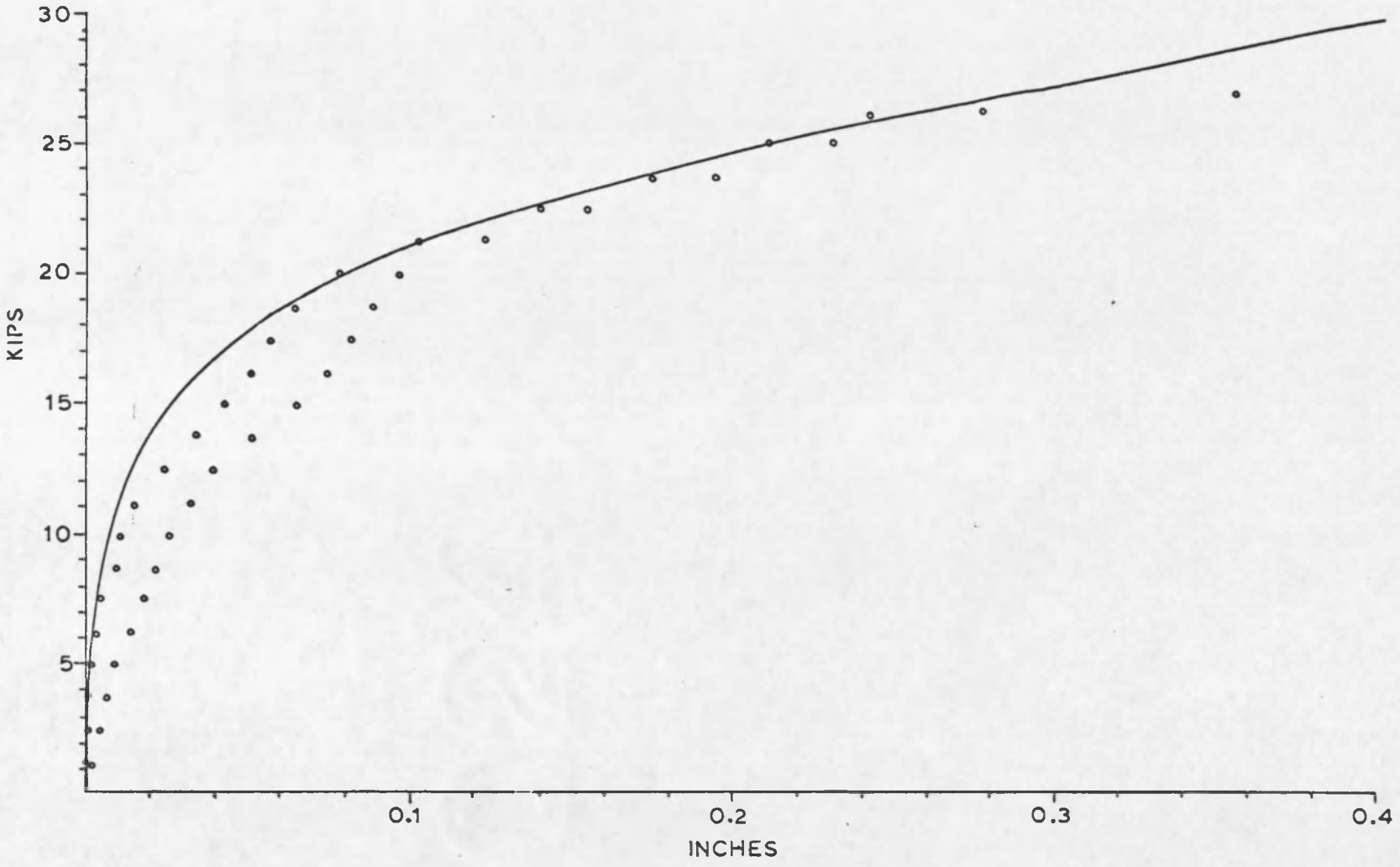


Figure 13. Shear vs. Deformation Response (5/16" and 5/16" plates).

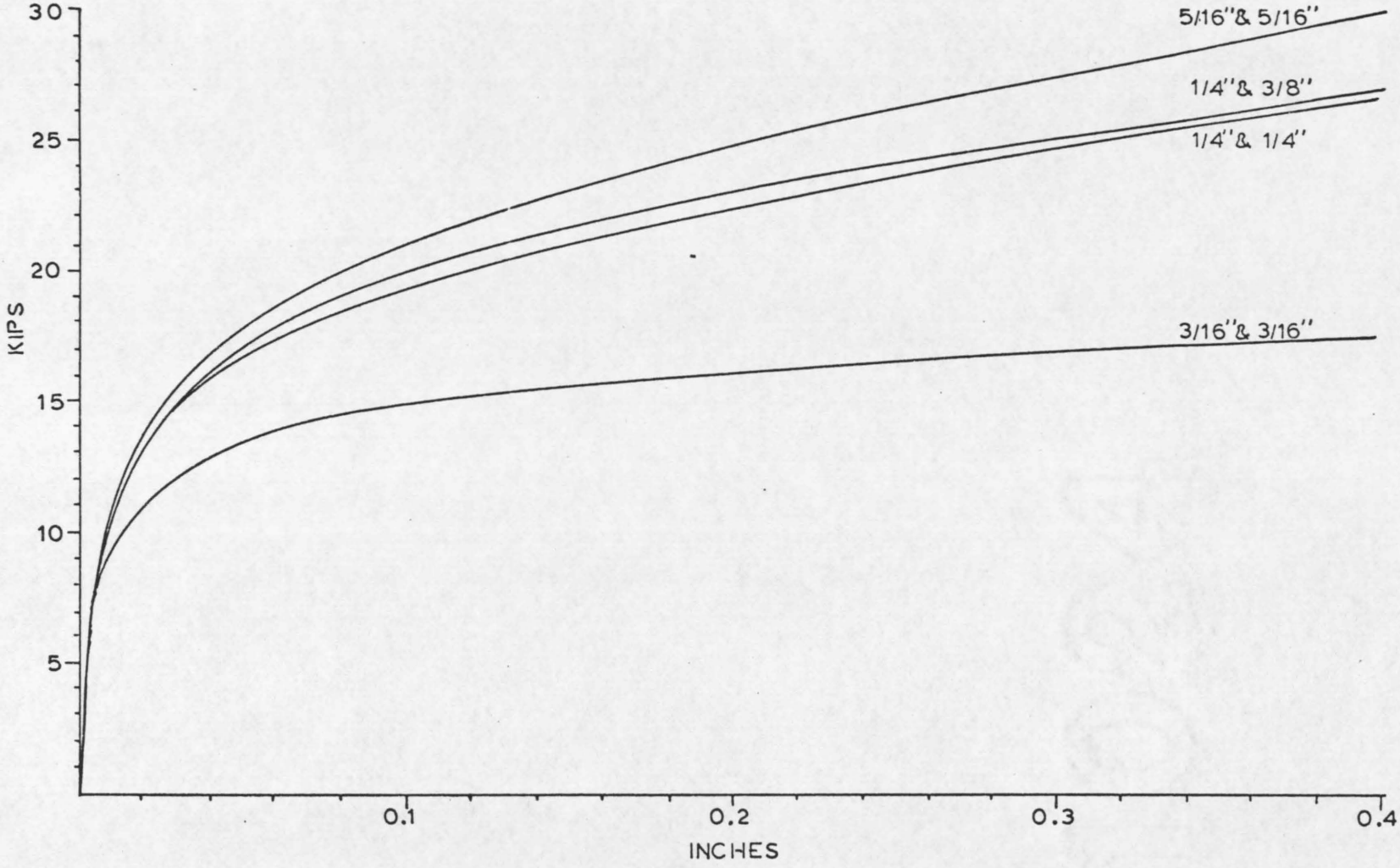


Figure 14. Shear vs. Deformation Response (all plate sets).

in bearing of the 1/4" plate at the bolt hole again becomes the controlling factor.

Two other sets of plates were briefly studied in order that some comparisons could be made. The first of these two sets consisted of two 3/16" plates and the second was comprised of 5/16" plates. Two tests were run for each set and all gave acceptable data (see Figures 12 and 13).

K_e for the 3/16" set is 5.4×10^6 lb./in. with $K_p = 6.94 \times 10^3$ lb./in. whereas for the set of 5/16" plates is $K_e = 9.1 \times 10^6$ lb./in. with $K_p = 2.0 \times 10^4$ lb./in. and using the data response curves shown in Figures 12 and 13, the PARAM responses could be calculated and the results are shown in Figure 14.

Comparing the 3/16" set, 1/4" set, and 5/16" set, it is immediately observed that as the plate thickness increases, the strength of the shear tab connection increases; however, it is at a decreasing rate, and the plastic moduli also become stiffer (increase in magnitude).

It is expected for the increments in joint strength and plastic response to occur as the plate thickness increases, but the cause for the decreasing rate of the strength increment is not immediately obvious.

Although the plate thicknesses were increased in steps of 1/16", the total cross sectional area increments grew smaller. That is for 3/16" to 1/4" (or 4/16" the area increases 33%, but from 1/4" to 5/16" the area is only 25% greater. This is one cause for the decreasing plate strength rate.

Secondly, there is the effect of a couple that occurs at the bolt in the lap joint due to the single shear condition. This coupling

effect tends to rotate the lap connection as shown in Figure 15. The resistance to this bending moment is dependent on the plate's moment of inertia. As plate thicknesses increase at a steady rate the moments of inertia will increase at a decreasing rate as shown here: $I_{3/16}$ = 0.0022in.^{4}, $I_{4/16}$ = 0.0052in.^{4} (136% greater than $I_{3/16}$), and $I_{5/16}$ = 0.0102in.^{4} (96% greater than $I_{4/16}$).

With thinner plates having greater rotation at the bolt there will be greater bearing on the edge of the bolt holes instead of an even load distribution across the thickness of the plate. The increase in bearing on the bolt hole edge causes a decrease in the effective area the bolt is bearing on, and this will cause yielding effects to show earlier than might be anticipated. This effect can be seen in the response for the 3/16" plate shown in Figure 14.

Figure 15. Couple Effect At Bolt.

CHAPTER 6

FINITE ELEMENT ANALYSIS

In order to analytically simulate the actual test experiments run by Professor Lipson (1968), finite element models were needed that would sufficiently represent the test configurations.

Professor Lipson employed two basic kinds of test apparatus. Both tests utilized a WF 21 x 62 steel beam and a 1/4" single plate web shear connection. The first test configuration subjected the connection to pure moment, the second to both shear and moment (see Figures 16 and 17). From the pure moment apparatus, Professor Lipson obtained a set of response curves for the moment versus the bolt line rotation and from the shear-moment condition, the shear in the bolt line versus its vertical deflection response was obtained.

The numerical analysis was accomplished through the use of the finite element program INELAS (Richard, 1968) which has the capacity to handle six different types of finite elements: (1) bars, (2) isotropic triangles, (3) nonisotropic triangles, (4) rectangles, (5) quadrilateral elements, and (6) discrete fasteners. Four of these elements were used in the modeling of Professor Lipson's (1968) experimental test appartus.

A general pure moment model was designed to handle a shear tab that varied from two to six fastening bolts (see Figure 18). The flanges of the beam were modeled by the use of bar elements, the web through the use of isotropic triangles and rectangles (quadrilateral elements

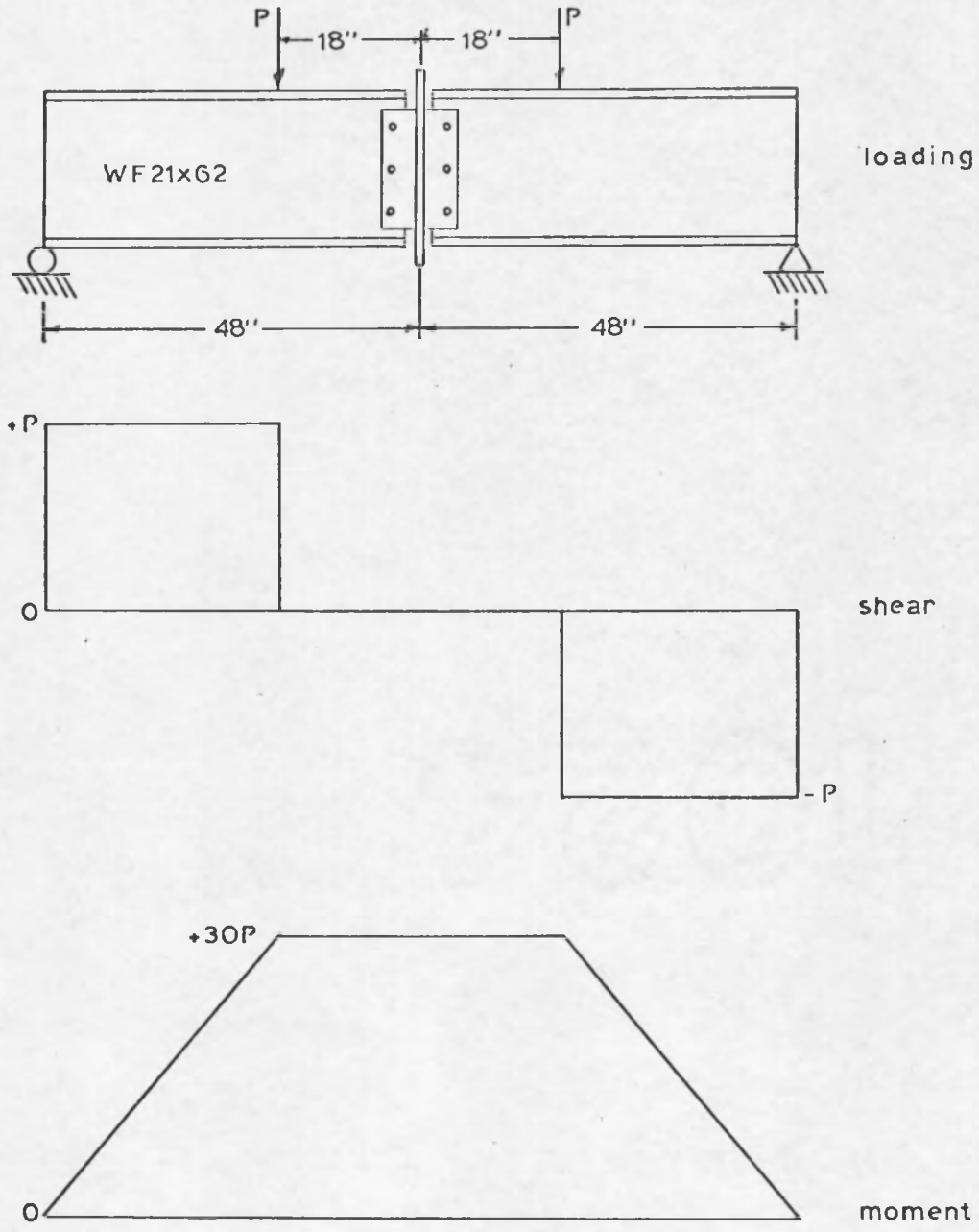


Figure 16. Experimental Pure-Moment Apparatus.

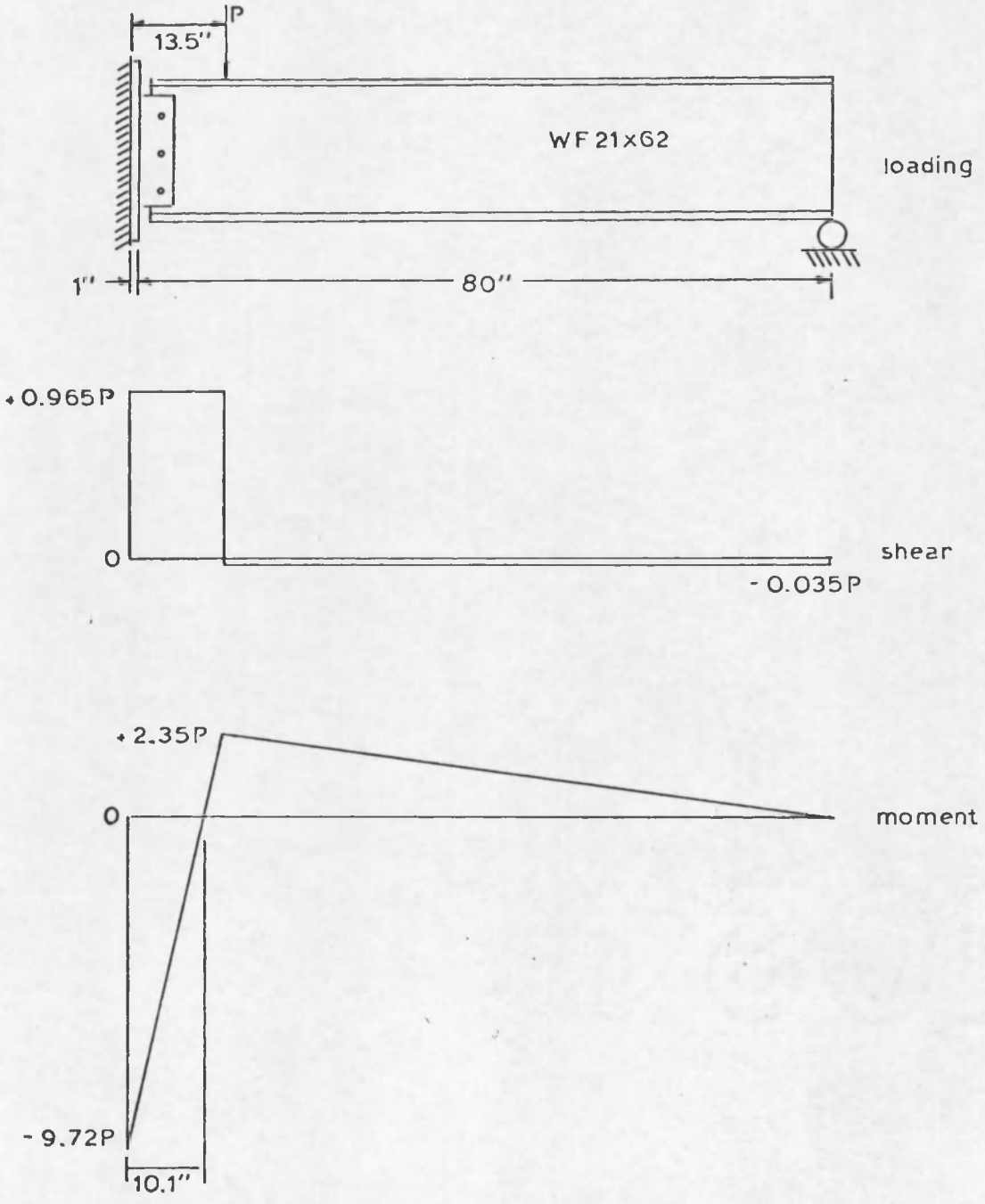


Figure 17. Experiment Shear-Moment Apparatus.

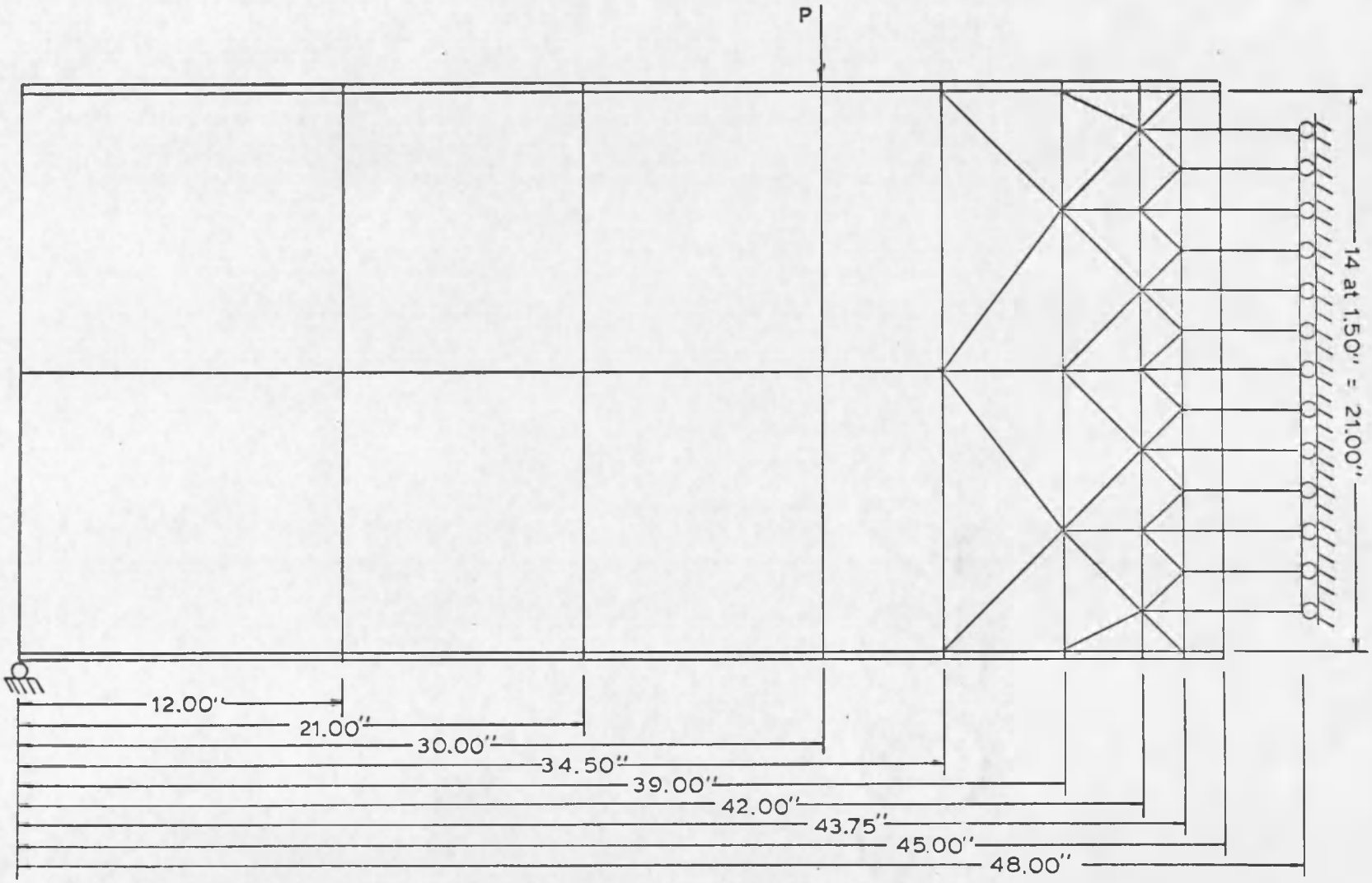


Figure 18. Pure-Moment Finite Element Model.

could also have been used), and the bolts were modeled as discrete fasteners.

The symmetry of Professor Lipson's (1968) pure moment apparatus allowed the finite element models to require only half the total beam length when proper support conditions were applied. Both models were supported in the vertical direction by a roller at the end opposite the single web plate, and the web plate itself was fully supported against horizontal movement by a vertical wall to which the plate was attached through the use of rollers. The application of a single point load 18" from the vertical wall now yields the same pure-moment condition studied by Professor Lipson (1968). This simplification has economized on computer memory space and run time due to far fewer elements.

A general shear-moment model was designed on the same fundamental basis used for the pure-moment model (see Figure 19). For this shear-moment model there was no symmetry that could be taken advantage of, so the entire test configuration used by Professor Lipson had to be modeled. The right end of the model attained vertical support from a roller, while the cantilevered left end attained complete horizontal support through hetuse of rollers on a vertical wall except at the center line of the web where the shear tab was pinned to the wall. This pin provided the vertical constraint required to obtain fixity.

On the general pure-moment and shear-moment models the depth of the shear tab was always 17.5", the required dimension for a six-bolt single plate connection regardless of whether or not the shear tab had six bolts. This was allowable because the depth of the shear tab is not

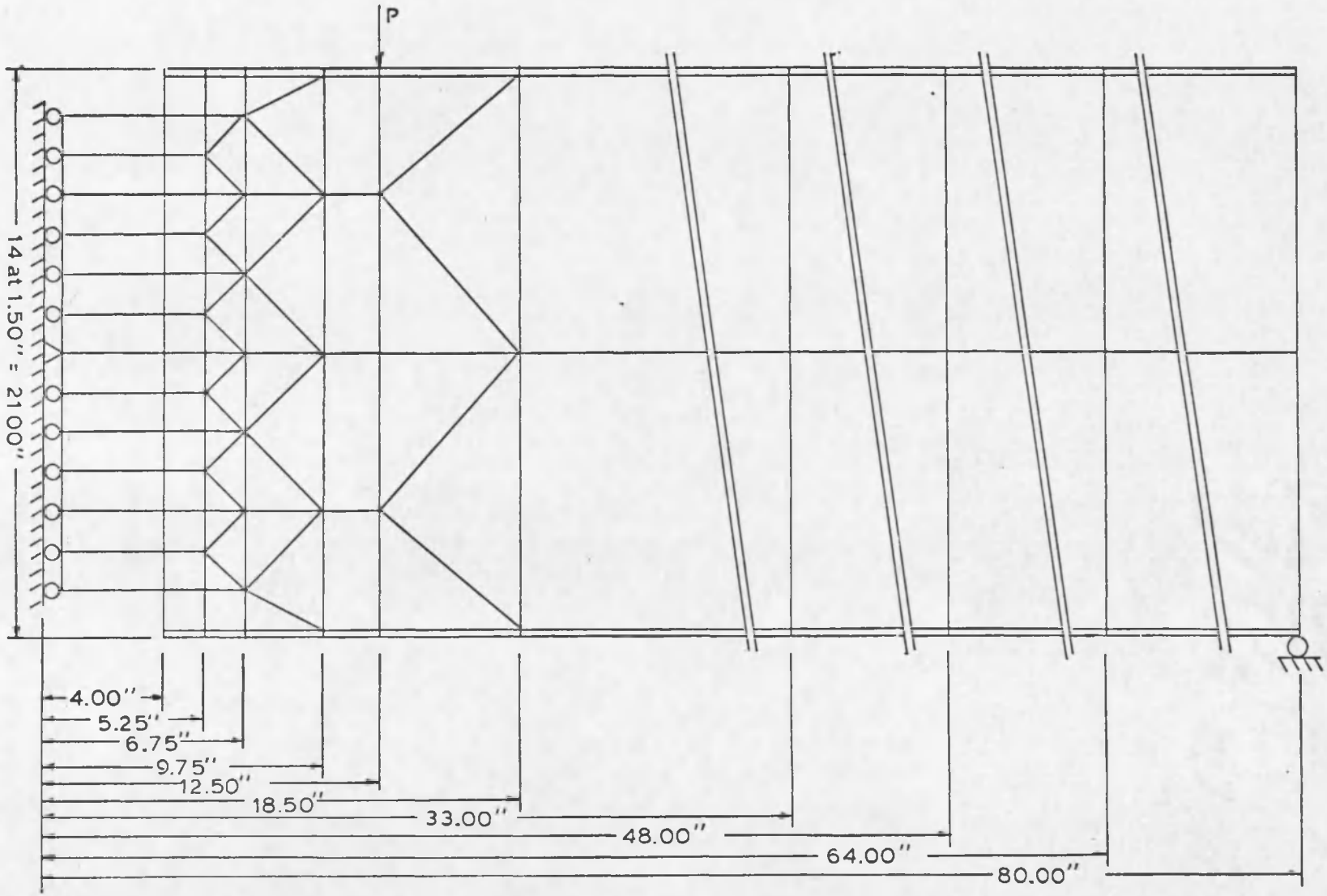


Figure 19. Shear-Moment Finite Element Model.

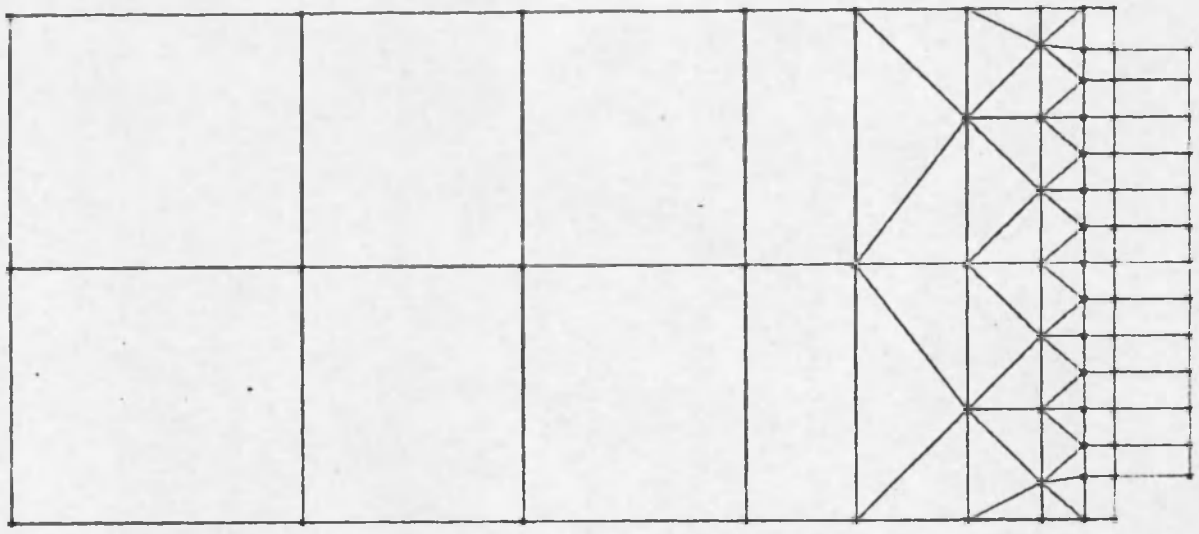
crucial as long as the minimum vertical edge distance of 1-1/4" is observed.

Both general models were dimensioned from the experimental apparatus of Professor Lipson (1968) and loaded exactly as he had done. The shear tabs for the pure-moment models were 4.25" long and for the shear-moment model the tab length was 5.25". The horizontal minimum edge distance was also 1.25" measured from the edge of the plate to the bolt line.

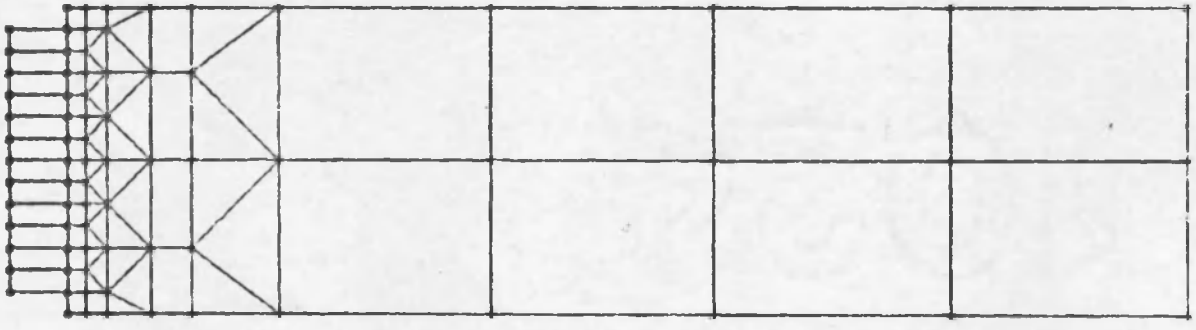
For both general models the short portions of the beams that overlapped the shear connections were omitted. This simplification cut down on the number of total elements, which economized on computer memory space and run time. The effect of omitting this short portion of the beam is negligible because all of the interaction between the beam and web connection occurs analytically at the location of the bolts.

The bolted lap joint shear-deformation results obtained in the laboratory were used as fastener parameters. Through INELAS all of the lap joint nonlinearity could be concentrated into the fastener element. The power of this concept lies in the fact that the fastener will react exactly as the experimental bolted lap joint, thus not only allowing for bolt deformation but also for the failure of the shear tab material surrounding the bolt holes, giving the complete shear-deformation response.

All keypunched data decks which defined the geometry of the finite element models were run on the CALCOMP plotter through the use of program NETPLOT (Malvick, 1968). The plotted results obtained showed the geometric configuration as it was defined by the data deck. The plots are presented in Figure 20.



GENERAL PURE-MOMENT MODEL



GENERAL SHEAR-MOMENT MODEL

Figure 20. Plots from NETPLOT.

The total load applied to each model varied, depending on whether the test was pure-moment or shear-moment and the number of bolts in the shear tab. This value applied was chosen according to the moment-rotation and shear-deformation graphs shown in Professor Lipson's (1968) work. The total load (P) was then applied in three increments of 0.6P, 0.25P, and 0.15P using a fourth order Runge Kutta scheme. In some cases the number of increments was increased to four using the values 0.4P, 0.2P, 0.2P, and 0.2P. This incremental application of the total load allowed for the presentation of moment-rotation or shear-deformation data at intermediate points so that a complete response could be plotted. All total loads were of the magnitude required to cause complete failure of fasteners by loading them into their plastic state.

CHAPTER 7

INTERPRETATION OF FINITE ELEMENT ANALYSIS OUTPUT DATA

The program INELAS (Richard, 1968) provides an abundance of information for each load value applied to any particular model. The output data basically consists of nodal point displacements, stresses in each element, loads in fasteners and load direction, a factor of safety for each element based upon the reference stress σ_o, external reactions, an equilibrium check, and residual stresses and strains.

In order for the moment-rotation curves to be plotted for the analytical results, it was necessary to locate the pivot point of the bolt line. This point always occurred at the center of gravity of the bolt line. The rotation of the bolt line was always symmetrical about this pivot point; therefore, in order to determine the amount of rotation, the horizontal displacement of the node directly above the center of gravity nodal point was divided by their vertical separation. This yielded the bolt line rotation in radians. This value could then be plotted against the appropriate moment in the web connection.

For determination of the analytical shear-deformation response, the vertical displacement of the node directly below the bolt line at the bottom of the modeled beam was plotted against the shear in the bolt line at that instant.

CHAPTER 8

FINITE ELEMENT MODEL RESULTS

When initiating the experimental testing, there was no 3/8" plate immediately available. It was decided that the first set of plate tests would be run with two 1/4" plates, the thickness of Professor Lipson's (1968) shear tabs. From this set of experiments K_e and K_p values could be obtained for use in the finite element analysis. It was believed that the results obtained would still be acceptable because although there would be some variance in the parameters obtained from two 1/4" plates versus one 1/4" and one 3/8", the values would still be of the proper magnitude, assuring similar analytical results.

Initially the linear response of the shear-deformation curve for the 1/4" plates was chosen directly from the plot of experimental data as $K_e = 4.2 \times 10^5$ lb./in. However, upon plotting the moment-rotation curve for the two bolt shear tab through use of the two-bolt pure-moment model, it was seen that the initial portion of the analytical curve was softer (less stiff) than the experimental curve obtained by Professor Lipson (1968) (see Figure 21). It was believed that the problem lay with the chosen K_e value, that it really did not represent the linear response of the bolt shear deformation curve.

Upon the reexamination of this value and the adoption of the formula $K_e = 2E(t_1t_2)/(t_1+t_2)$, a new value for K_e was calculated to be 8.9×10^6 lb./in. using thicknesses of 0.25" and 0.40". Using this K_e

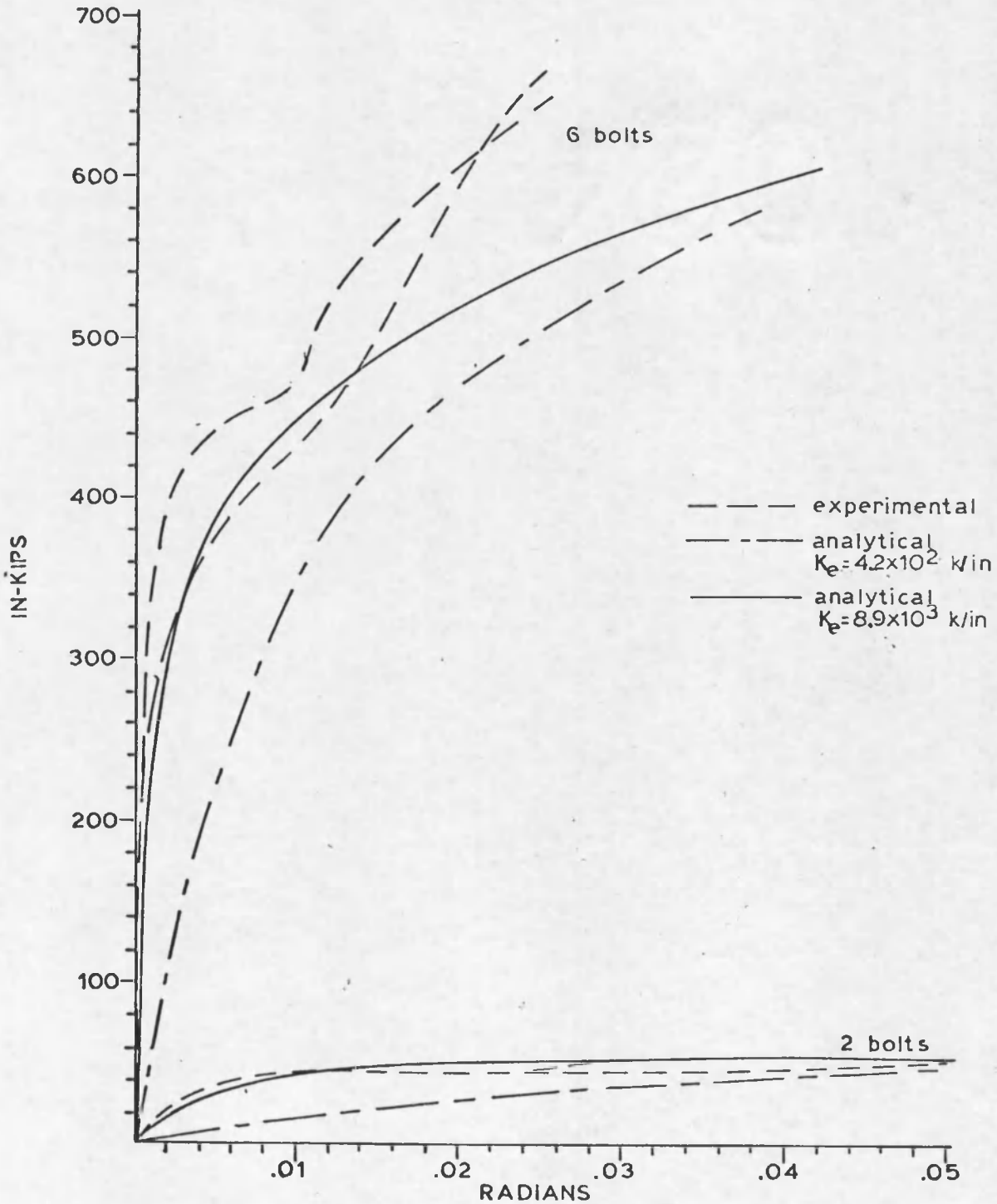


Figure 21. Moment vs. Rotation Responses (Comparing Initial and Final K_e Values).

value and other associated parameters obtained through program PARAM, the resulting analytical response improved significantly, yielding excellent correlation with the experimental results shown by Professor Lipson (1968) (see Figure 21).

To further illustrate the softness of the original $K_e = 4.2 \times 10^5$ lb./in., associated parameters were used in an analysis of the six-bolt pure-moment model. Once again the resulting response (see Figure 21) lacks initial stiffness which is obtained when the same model is analyzed again using $K_e = 8.9 \times 10^6$ lb./in. parameter.

Figure 22 shows the analytical moment-rotation responses superimposed on Professor Lipson's (1968) experimental results, which show two curves for each type of shear tab. "Each pair of curves represents the two joints tested in one loading operation" (Lipson, 1968, p. 148) (see Figure 16). Apparently the differences that occur in the pair of curves are due to variances in bolt hole clearances upon completion of joint fabrication.

Professor Lipson's results are comprised of two types of responses, the first being one of increasing bolt line rotation at constant moment and the second being increased rotation with increasing moment, but at a decreasing rate (see Figure 22). The first action is due to slippage of all connecting bolts and the absence of this type of response means that at least one bolt is in bearing (Lipson, 1968).

The analytical responses obtained closely approximate the actual experimental responses. These curves are smooth and show no signs of slippage because the mathematical models used make no such allowances, all fasteners (or bolts) being in bearing at all times.

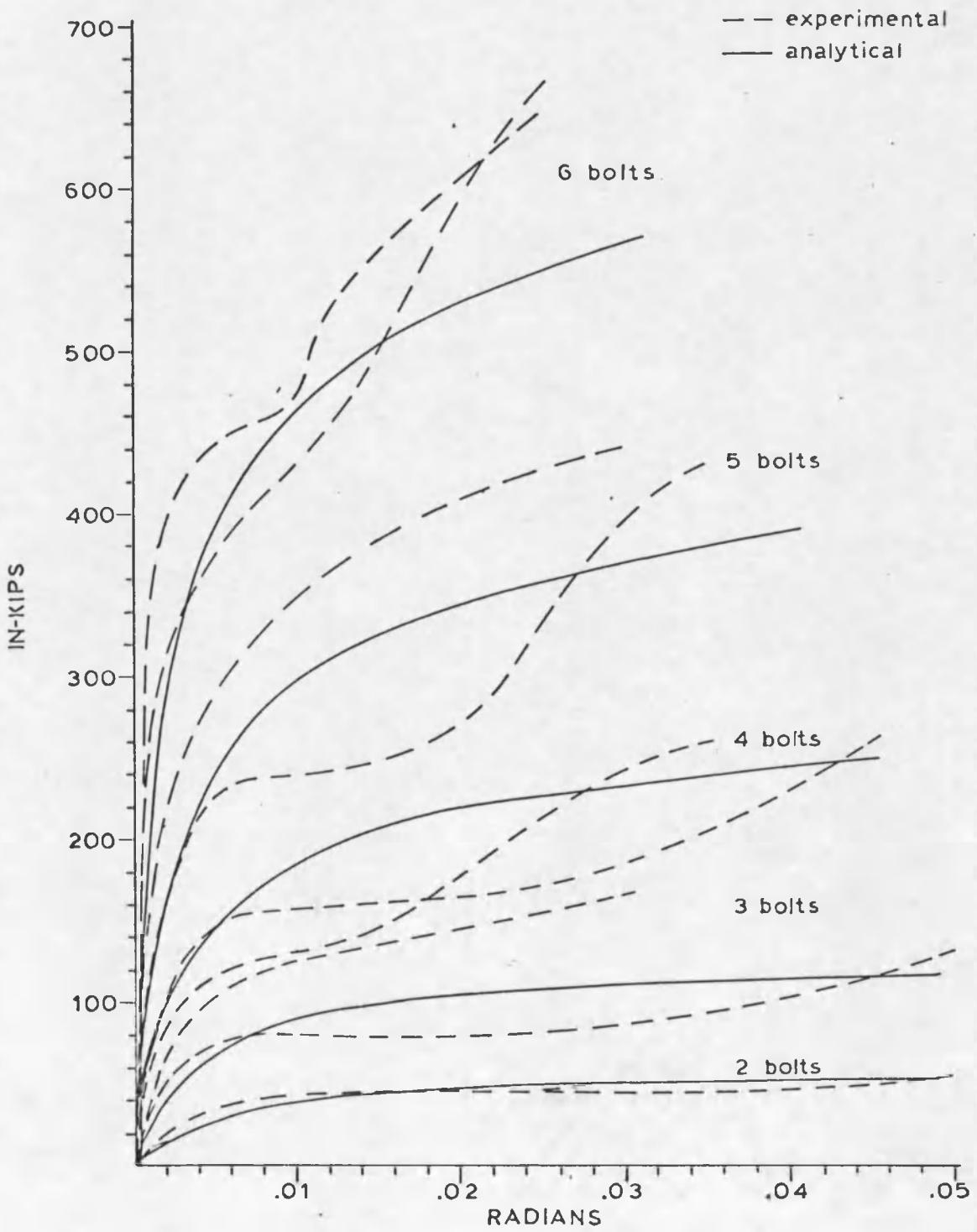


Figure 22. Moment vs. Rotation Responses.

These finite element responses show three distinct regions, the first being an initial near-linear response. The second region illustrates gradual yielding of the fasteners. The third shows the connection's plastic response where all the fasteners have exceeded their plastic limits.

The pure moment analysis shows that as each joint enters its region of yielding, the outermost fasteners will go plastic first as theoretically predicted. When the outermost fasteners go plastic, all additional load must be carried by the innermost bolts; this will continue to happen until all fasteners have reached the plastic state. It is also noted that the lines of action for the fasteners is horizontal if the web plate is assumed to be infinitely stiff; however, the finite element results show that the line of action initially deviates as much as 7" (in the six bolt connection) because in reality the stiffness that exists is finite.

The analytical bolt-line shear versus deformation response comparison is shown in Figure 23. However, immediate inspection does not reveal the excellent analytical to actual response correlation seen in the pure-moment responses. Upon further inspection, one notices that all of the actual responses (with the exception of the two-bolt connection) show major slippage occurring in the joints, which again cannot be accounted for in the analytical models. When a comparison of initial response slopes and plastic response slopes is made, it can be seen that these values are relatively close.

Therefore, if slippage did not occur, the actual response would continue to soften as the shear increased and the comparison of

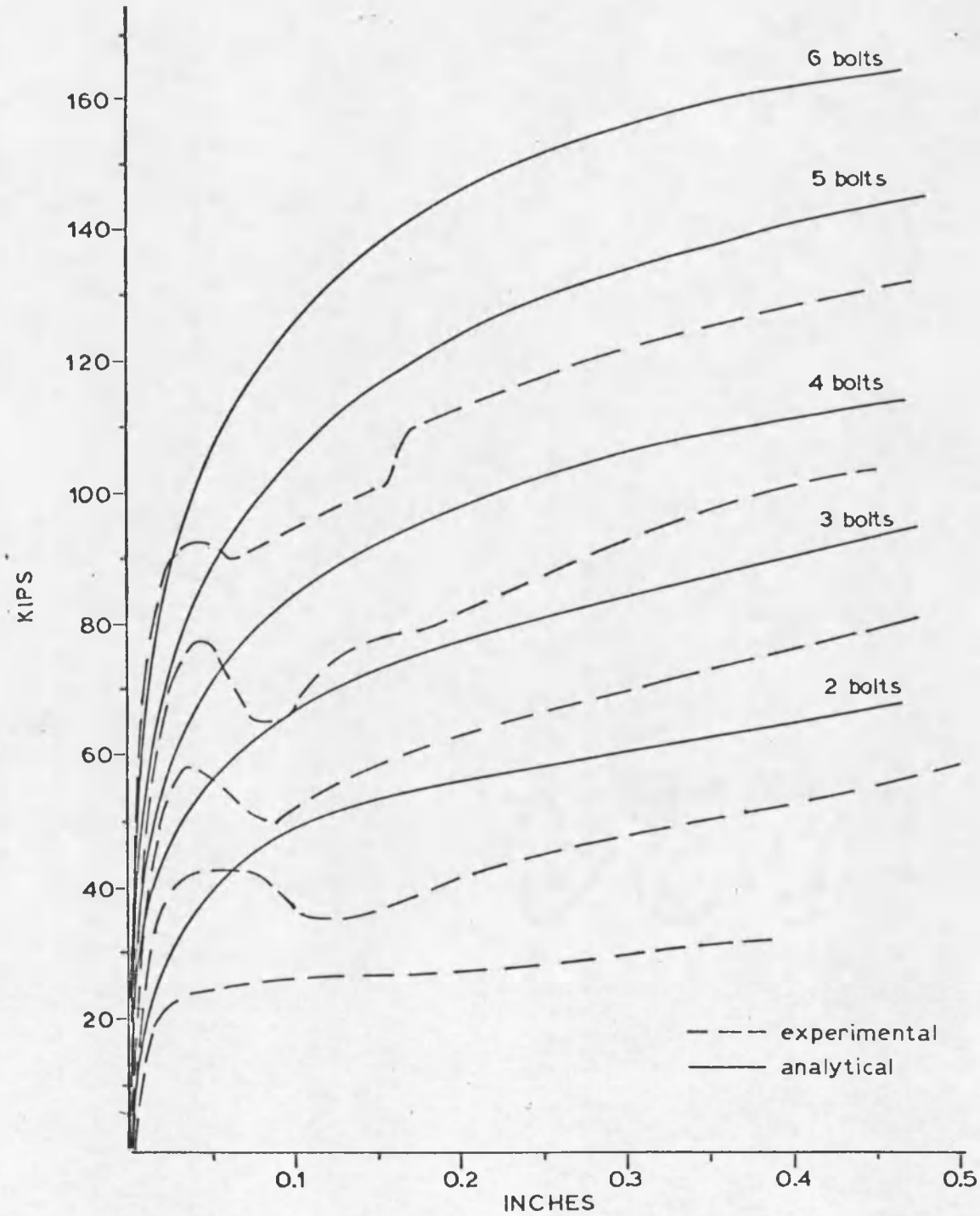


Figure 23. Shear vs. Deformation Responses.

the finite element to experimental results would have shown closer correlation.

Also contributing to the correlation problems is the fact that analytically it is possible to obtain a true fixed connection, but in actuality 100% fixity is not possible. This would cause the finite element model to be stiffer than the experimental apparatus, yielding stiffer response data than the actual experiments did.

For the shear-moment condition it is important to note that the lines of action for the bolt loads deviate as much as 4° (for the six bolt connection) as the applied beam load increases. This effect is the result of the interaction between the shear and bending moment stresses in the web and it can be seen in all cases with the exception of the two bolt connection.

Professor Lipson (1968) also mentions that the center of rotation for the bolt line was never more than 0.8" from the centroid of the bolt pattern in the direction of the compressive edge. This, too, was found to be the case analytically with the maximum distance being only 0.23" from the centroid towards the compressive edge. Again slippage occurring in actual experiments causes the large amount of discrepancy between these two values.

CHAPTER 9

CONCLUSIONS

The following conclusions were drawn from the results of this analytical study:

1. The obvious linear shear-deformation response (K_e) of the bolted lap joint is in error due to existing slack. K_e can be better approximated through use of the formula $K_e = 2E(t_1 t_2)/(t_1+t_2)$.

2. The single plate framing connection can be effectively modeled by concentrating the nonlinearity of the joint into discrete fasteners.

3. Shear tab ductility comes from distortion of the bolt holes, providing the bolts are of the necessary strength.

4. Finite element models provide the capability to view the elastic and inelastic responses for the single plate framing connection.

5. The resultant bolt forces tend to rotate as the shearing force in the bolt line and moment in the connection interact.

CHAPTER 10

RECOMMENDATIONS

The following recommendations are made for extensions to this study:

1. If larger plates (meaning larger loads) are to be studied, the test fixture will need to be strengthened in certain areas. Presently the major weak points in the fixture are the 1" tooled steel pins that attach the test apparatus to the clevises. They have taken 50K of load in double shear (25K/shear surface), but they also show large permanent disfiguration. Also, as the total fixture load is increased, the pins attaching the right leg to the upper and lower heads may have to be increased in size or can be changed to tooled steel (or some similar material).

2. At this time the 1/2" bolts (in double shear) that attach the plates to the test fixtures have approximately three times the total shear surface that the 3/4" bolt has. Any increases in test bolt diameter (from 3/4") may require an increment of these attaching bolt diameters in order to maintain a factor of safety and the deformation measurement error due to distortion in this area minimal.

3. This analysis only studied a gauge distance of 2-1/2" for the pure-moment case and 4" for that of shear-moment. In both studies the pitch distance was always 3". Therefore, investigation of bolt

pitch and gauge distance variations would make a natural extension to this work.

4. The effect of punching holes versus the effect of drilling holes (studied in this work) may be another possible extension.

5. The effect of multi-bolt lines versus the single bolt line.

APPENDIX A

TEST RESULTS

Strength Values for Materials Used Obtained from Test Coupons.

	Yield Strength	Ultimate Strength	Failure Strength
Lot 1	Not Used for Testing		
Lot 2	49.8 ksi	72.8 ksi	60.0 ksi
Lot 3	No Coupon Available		
Lot 4	48.0 ksi	64.0 ksi	53.3 ksi
Lot 5	45.1 ksi	66.2 ksi	54.4 ksi

Plate 1: 1/4" - Lot 2 Plate 2: 1/4" - Lot 2 Test # 1

Total Load (Lbs.)	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
2,500	1784	0	0	1,250
5,000	1783	1	2	2,500
7,500	1783	0	2	3,750
10,000	1773	10	22	5,000
12,500	1742	31	84	6,250
15,000	1723	19	122	7,500
17,500	1693	30	182	8,750
20,000	1675	18	218	10,000
22,500	1652	23	264	11,250
25,000	1633	19	302	12,500
27,500	1603	30	362	13,750
30,000	1568	35	432	15,000
32,500	1513	55	542	16,250
35,000	1480	33	608	17,500
37,500	1335	145	898	18,750
40,000	1188	147	1,192	20,000
- - - - - - - - - - - - - - - END OF TEST - - - - - - - - - - - - - - -				

Plate 1: 1/4" - Lot 2 Plate 2: 1/4" - Lot 2 Test # 2

Total Load (Lbs.)	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
2,500	1875	25	50	0
5,000	1838	37	124	1,250
7,500	1813	25	174	2,500
10,000	1794	19	212	3,750
12,500	1783	11	234	5,000
15,000	1765	18	270	6,250
17,500	1752	13	296	7,500
20,000	1733	19	334	8,750
22,500	1706	27	388	10,000
25,000	1684	22	432	11,250
27,500	1635	49	530	12,500
30,000	1583	52	634	13,750
32,500	1523	60	754	15,000
35,000	1475	48	850	16,250
37,500	1400	75	1000	17,500
40,000	1233	167	1334	18,750
42,500	1000	233	1800	20,000
45,000	754	246	2292	21,250
47,500	465	289	2870	22,500
- - - - - - - - - - - - - - - - END OF TEST - - - - - - - - - - - - - - - -				

Plate 1: 1/4" - Lot 2 Plate 2: 1/4" - Lot 2 Test # 3

Total Load (Lbs.)	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
0	1950	0	0	0
2,500	1950	0	0	1,250
5,000	1935	15	30	2,500
7,500	1915	20	70	3,750
10,000	1900	15	100	5,000
12,500	1876	24	148	6,250
15,000	1863	13	174	7,500
17,500	1842	21	216	8,750
20,000	1815	27	270	10,000
22,500	1792	23	316	11,250
25,000	1762	30	376	12,500
27,500	1732	30	436	13,750
30,000	1702	30	496	15,000
32,500	1662	40	576	16,250
35,000	1542	120	816	17,500
37,500	1386	156	1128	18,750
40,000	1211	175	1478	20,000
42,500	1011	200	1878	21,250
45,000	732	279	2438	22,500
47,500	400	332	3100	23,750
49,500	0	400	3900	24,750
- - - - - - - - - - - - - - - - END OF TEST - - - - - - - - - - - - - - - -				

Plate 1: 1/4" - Lot 2 Plate 2: 1/4" - Lot 2 Test # 4

Total Load (Lbs.)	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
0	1910	0	0	0
2,500	1908	2	4	1,250
5,000	1907	1	6	2,500
7,500	1888	19	44	3,750
10,000	1867	21	86	5,000
12,500	1849	18	122	6,250
15,000	1813	36	194	7,500
17,500	1790	23	240	8,750
20,000	1767	23	286	10,000
22,500	1737	30	346	11,250
25,000	1712	25	396	12,500
27,500	1667	45	486	13,750
30,000	1637	30	546	15,000
32,500	1608	29	604	16,250
35,000	1567	41	686	17,500
37,500	1425	142	970	18,750
40,000	1278	147	1264	20,000
42,500	1110	168	1600	21,250
45,000	878	232	2064	22,500
47,500	560	318	2700	23,750
50,000	88	472	3644	25,000

Plate 1: 1/4" - Lot 2 Plate 2: 3/8" - Lot 3 Test # 5

Total Load (Lbs.)	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
0	1900	0	0	0
2,500	1900	0	0	1,250
5,000	1900	0	0	2,500
7,500	1820	80	160	3,750
10,000	1810	10	180	5,000
12,500	1805	5	190	6,250
15,000	1795	10	210	7,500
17,500	1775	20	250	8,750
20,000	1755	20	290	10,000
22,500	1695	60	410	11,250
25,000	1618	77	564	12,500
27,500	1595	23	610	13,750
30,000	1570	25	660	15,000
32,500	1540	30	720	16,250
35,000	1500	40	800	17,500
37,500	1405	95	990	18,750
40,000	1305	100	1190	20,000
42,500	1170	135	1460	21,250
45,000	945	225	1910	22,500
47,500	470	475	2860	23,750
48,900	0	470	3800	22,450

Plate 1: 1/4" - Lot 2 Plate 2: 3/8" - Lot 3 Test # 6

Total Load (Lbs.)	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
0	1817	0	0	0
2,500	1770	47	94	1,250
5,000	1764	6	106	2,500
7,500	1678	86	278	3,750
10,000	1665	13	304	5,000
12,500	1655	10	324	6,250
15,000	1643	12	348	7,500
17,500	1624	19	386	8,750
20,000	1609	15	416	10,000
22,500	1594	15	446	11,250
25,000	1565	29	504	12,500
27,500	1514	51	606	13,750
30,000	1484	30	666	15,000
32,500	1455	29	724	16,250
35,000	1404	51	826	17,500
37,500	1314	90	1006	18,750
40,000	1214	100	1206	20,000
42,500	1074	140	1486	21,250
45,000	870	204	1894	22,500
47,500	365	505	2904	23,750
50,000	155	210	3324	25,000

Plate 1: 1/4" - Lot 2 Plate 2: 3/8" - Lot 3 Test # 7

Total Load (Lbs.)	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
0	1802	0	0	0
2,500	1763	39	78	1,250
5,000	1755	8	94	2,500
7,500	1745	10	114	3,750
10,000	1731	14	142	5,000
12,500	1715	16	174	6,250
15,000	1704	11	196	7,500
17,500	1685	19	234	8,750
20,000	1664	21	276	10,000
22,500	1630	34	344	11,250
25,000	1605	25	394	12,500
27,500	1580	25	444	13,750
30,000	1554	26	496	15,000
32,500	1518	36	568	16,250
35,000	1484	34	636	17,500
37,500	1420	64	764	18,750
40,000	1350	70	904	20,000
42,500	1250	100	1104	21,250
45,000	1104	146	1396	22,500
47,500	835	269	1934	23,750
48,500	455	380	2694	24,250
49,500	0	455	3604	24,750

Plate 1: 3/16" - Lot 4 Plate 2: 3/16" - Lot 4 Test # 8

Total Load (Lbs.)	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
0	1935	0	0	0
2,500	1915	20	40	1,250
5,000	1900	15	70	2,500
7,500	1879	21	112	3,750
10,000	1864	15	142	5,000
12,500	1855	9	160	6,250
15,000	1815	40	240	7,500
17,500	1790	25	290	8,750
20,000	1763	27	344	10,000
22,500	1715	48	440	11,250
25,000	1655	60	560	12,500
27,500	1415	240	1040	13,750
30,000	1040	375	1790	15,000
32,500	360	680	3150	16,250
33,200	100	260	3670	17,500
33,400	0	100	3870	18,750

ULTIMATE STRENGTH
21,300# Shear

Plate 1: 3/16" - Lot 4 Plate 2: 3/16" - Lot 4 Test # 9

Total Load (Lbs.)	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
0	1955	0	0	0
2,500	1915	40	80	1,250
5,000	1877	38	156	2,500
7,500	1864	13	182	3,750
10,000	1845	19	220	5,000
12,500	1825	20	260	6,250
15,000	1814	11	282	7,500
17,500	1804	10	302	8,750
20,000	1775	29	360	10,000
22,500	1750	25	410	11,250
25,000	1718	32	474	12,500
27,500	1634	84	642	13,750
30,000	1385	249	1140	15,000
31,000	1210	175	1490	15,500
32,000	1060	150	1790	16,000
33,000	872	188	2166	16,500
34,000	540	332	2830	17,000
34,700	0	540	3910	17,350

END OF TEST

Plate 1: 5/16" - Lot 5 Plate 2: 5/16" - Lot 5 Test # 10

Total Load (Lbs.)	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
0	1900	0	0	0
2,500	1887	13	26	1,250
5,000	1875	12	50	2,500
7,500	1865	10	70	3,750
10,000	1855	10	90	5,000
12,500	1830	25	140	6,250
15,000	1814	16	172	7,500
17,500	1793	21	214	8,750
20,000	1770	23	260	10,000
22,500	1738	32	324	11,250
25,000	1695	43	410	12,500
27,500	1645	50	510	13,750
30,000	1572	73	656	15,000
32,500	1525	47	750	16,250
35,000	1494	31	812	17,500
37,500	1457	37	886	18,750
40,000	1414	43	972	20,000
42,500	1285	129	1230	21,250
45,000	1125	160	1550	22,500
47,500	920	205	1960	23,750
50,000	740	180	2320	25,000
52,500	510	230	2780	26,250
54,300	115	395	3570	27,150

END OF TEST

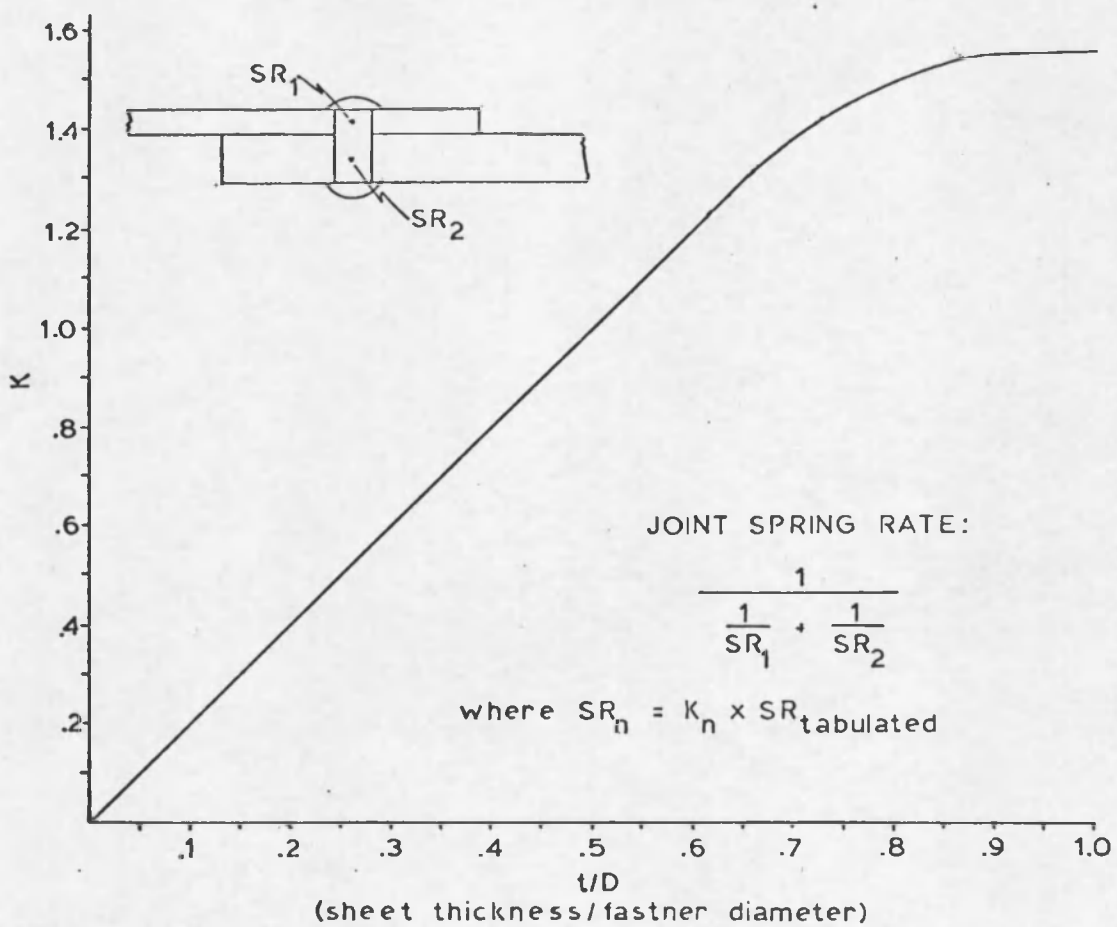
Plate 1: 5/16" - Lot 5 Plate 2: 5/16" - Lot 5 Test # 11

Total Load	Gauge Reading (10^{-4} in.)	Change in Gauge Reading (10^{-4} in.)	Length Change Between Pts. A & B (10^{-4} in.)	Shear in Bolt (Lbs.)
0	1800	0	0	0
2,500	1780	20	40	1,250
5,000	1778	2	4	2,500
7,500	1775	3	10	3,750
10,000	1770	5	20	5,000
12,500	1763	7	34	6,250
15,000	1755	8	50	7,500
17,500	1743	12	74	8,750
20,000	1725	18	110	10,000
22,500	1703	22	154	11,250
25,000	1655	48	250	12,500
27,500	1608	47	344	13,750
30,000	1565	43	430	15,000
32,500	1520	45	520	16,250
35,000	1493	27	574	17,500
37,500	1455	38	650	18,750
40,000	1388	67	784	20,000
42,500	1264	124	1032	21,250
45,000	1075	189	1410	22,500
47,500	900	175	1760	23,750
50,000	720	180	2120	25,000
52,500	560	160	2440	26,250

END OF TEST

APPENDIX B

EFFECTIVE SPRING RATES IN SINGLE SHEAR JOINTS (CHANCE VOUGHT)



	attachment diameter and SR x 10^{-6}									
sheet	1/8	5/32	3/16	1/4	5/16	3/8	7/16	1/2	7/16	5/8
ALUM	.163	.203	.244	.325	.406	.487	.563	.650	.732	.813
STEEL	3.62	4.53	5.44	7.25	9.06	10.9	12.6	14.5	16.3	18.1
OTHER	—— same as for steel x $(E_{other} / E_{steel})^3$ ——									

To Determine SR_{joint}:

1. Calculate t/D for each sheet
2. Determine K for each sheet from curve
3. Determine SR for each sheet from table
4. Calculate SR_{joint} using above formula

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