AN INVESTIGATION FOR THE EARTHQUAKE RESISTANT DESIGN OF LARGE-SIZE WELDED AND BOLTED GIRDER TO COLUMN CONNECTIONS

by

J. G. Bouwkamp [1]

I. ABSTRACT

The results of a detailed experimental study on four large full-size girder-to-column connections are presented. Each connection is composed of a riveted, 36 in. by 36 in., built-up column with bolted T-sections and welded plates which act as moment resistant supports for 42 in. deep welded girders.

The stiffness and strength characteristics of these joints were evaluated under static loads. The stress distributions in the welded plates were evaluated experimentally for different load conditions in order to determine the need for stress-relieving. The results presented proved that the design of these large-size girder-to-column connections are structurally feasible for an earthquake resistant design.

II. INTRODUCTION

The new Health Sciences Instruction and Research Buildings for the University of California, San Francisco Medical Center as designed by Ried and Tarics[2], are from many points of view unusual. This complex consists of two main buildings, one of 16 stories and one of 15 stories, to house teaching and laboratory facilities. An elevator tower and two mechanical service towers, one for each building, are structurally completely separate from these main structures. Figure 1 shows one of the main buildings and the two associated service towers.

Since no two floors in either building had the same floor plan it was virtually impossible to design a consistant interior column arrange-

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(1) Associate Professor of Civil Engineering, University of California, Berkeley, California

(2) Architects and Engineers, San Francisco, California

ment. Therefore the structural engineers considered the possibility of a column free interior with 93 ft. long girders. The design of this framing system proved to be quite feasible since the space for the very extensive mechanical and electrical equipment which had to be housed in the floors required a construction depth of 42 in. independent of the length of the span. The square plan of the towers together with the 42 in. girder depth requirement was thus ideal for a two-way framing system without interior columns. This structural framing scheme was adopted when it showed a saving of $100,000 as compared to the cost of a short-span scheme with interior columns. This selected girder arrangement is most effective since it transfers practically each floor load to all except the corner columns. Since no shear walls are present the lateral loads due to wind and earthquake are resisted only by the moment-resisting action of columns and girders.

The steel used in the design of girders and columns is a combination of A7 and A373. The long span welded girders with many openings in the web to accommodate passage of the utility ducts, are reinforced by inclined web stiffeners to increase the flexural stiffness of these girders. Girder-to-girder connections are made by means of H.S. bolted flange and web plates.

The girder-to-column connections of the 36 in. columns and 42 in. deep girders are unusual due to their size and subsequent associated behavior under load. Therefore a program of research was developed to investigate the actual load transfer and deformations of these large-size connections. Figures 2 and 3 show the two types of specimens, representing a third and fourteenth floor girder-to-column connection at the girder-flange level.

The prime objectives of this investigation as set by the engineers were twofold. The first objective was to determine the stiffness of the T-sections which accomodate the flanges of the 93 ft. long floor girders. This information was important in order to evaluate the end-restraining effect (rotation) of the large-span girders and to increase if necessary the rigidity of these connections in order to reduce the floor deflections. Furthermore, an acceptable rigidity would favorably contribute to the overall lateral stiffness of these buildings. The second objective was to determine the yield and ultimate strength of the 1 1/2 in. and 2 in. thick welded plates between the column flanges and web. These plates are 30 in. wide and H.S. bolted to the flanges of the 42 in. deep girders spanning the short distance between the columns in the exterior frames. Since no previous information was available about the strength of these welded plates as affected by residual stresses due to welding two geometrically identical specimens of the types shown in Figures 2 and 3 were to be investigated, one non-stress relieved and one stressed relieved. Should the tests indicate the need for stress relieving, all girder-to-column connections were to be treated to relieve the residual stresses.

Considering the important additional information which could be obtained from a program carried out on the specimens of such a large size, the author proposed a more complete program of research. This would not only provide information with respect to the original objectives as stated above, but also give a record about the more detailed behavior of these joints. The program was therefore expanded to include an extensive instrumentation in order to record the load distribution in the T-sections, welded plates, and welds under several types of loading.

III. TEST SPECIMENS

The two types of test specimens are shown in Figures 2 and 3. Since it was practically impossible to test a complete girder-to-column connection it was decided to study the behavior of only half of such a connection as formed by the T-section and welded plates at one level. The joints for the 3rd and 14th floors were selected as typical specimens. The difference between these two types is formed by different thickness in column flanges and welded plates. Although in the actual structure the large T-sections only appear on the inside face of the column, it was necessary to make the specimens symmetrical for test purposes. These considerations led to the type of specimen shown in Figure 4. This figure shows the filler plates and blocks on the inside of the column flanges as well as the extra blocks on top of the T-section flange. These latter plates were added to investigate the influence of these blocks on the stiffness of the T-section as well as on the load transfer in the H.S. bolted connection.

In order to meet the originally set objectives as far as the T-section stiffness was concerned, the movement of these T-sections as measured between points T - see Figure 4 - was recorded by means of a 1/10,000 in. dial gage. This deformation was measured at both ends of the T-section.

To obtain information about the load transfer through the welded plates an extensive instrumentation of photostress and electric-resistance strain gages was applied to each specimen. A number of strain gages were placed on the stem of the T-section to determine the stress distribution between the stem and flange of the T-section. Figure 5 shows part of the strain gage instrumentation as well as the dial gage arrangement to measure the movement between two opposite T-sections.

IV. TEST PROCEDURE

The tests were carried out in the 3,000,000 lb. Southwark-Emery Universal Testing Machine at the University of California in Berkeley. This machine does not allow the simultaneous application of loads to both the T-sections and the welded plates. Therefore each specimen was first tested under a tensile load applied to the stems of the T-sections. This loading was increased in 100k intervals and was to be terminated when either pronounced yielding of the flanges of the T-section would occur or when a load of 900k was reached, whichever occurred first. The design load for these sections as determined primarily by the negative D.L. + L.L. moments in the floor girders, was approximately 430k. The second loading in each specimen consisted of a tensile load applied to the welded connection plates. Under this condition the loads were increased stepwise in 100k intervals until ultimate failure would occur. The design load for these welded plates is primarily determined by the earthquake design criteria and is approximately 230k according to the Uniform Building Code and 700k under El Centro conditions.

Large bolted plates were used to extend the actual specimen in order to mount the connection in the large testing machine. Figure 6 shows the test set-up in the machine and the photo-stress on the right half of the specimen.

With the loads applied to the T-sections, the movement between the flanges of opposite T-sections was recorded at both ends of these sections. In order to determine the stress distribution in the stem of the T-section as well as in the welded plates, strain gage readings were taken at each load level.

With the loads applied to the welded plates, only the strain gages on the moment plates were observed at each load level along with the deformations between the welded plate and the column flange. (See Figure 4). This deformation is recorded by means of a deflectometer and provides information about the localized behavior of one of the welds between the flange and plate.

The testing program was carried out in the following order:

1. Specimen 1. (Type I, third-floor connection, non-stress relieved)
2. Specimen 2. (Type I, third-floor connection, stress relieved)
3. Specimen 3. (Type II, fourteenth-floor connection, non-stress relieved)
4. Specimen 4. (Type II, fourteenth-floor connection, stress relieved)

Each specimen was first subjected to the T-section loads and afterwards brought to failure by a load applied to the moment plates. In Specimen 4 one of the welds between the column web and the moment plates was inactivated prior to failure by cutting the moment plate just above the praticular weld. In this way it was possible to determine the ultimate shear strength of the side welds between the column flanges and the welded plate.

V. TEST RESULTS

T-section loading:

For specimen 1, which was not stress relieved, the maximum load applied to the T-section was 900k. At this load the average total deflection between the flanges of the T-section was 0.027 in.

Figure 7 shows a plot of the average deflections under increasing load as recorded on the two dial gages mounted between the flanges of the T-sections.

The maximum load applied to the T-sections of specimen 2, the stress-relieved connection, was 1000k. The average deflection between the flanges of the T-section of this specimen at 900k was only 0.022 in. This smaller deflection as compared to the 0.027 in. deflection recorded for specimen 1 is probably due to the presence in specimen 2 of blocking plates on top of the T-section flanges. These plates were omitted in specimen 1 in order to investigate their relative influence on the stiffness of the T-section connection. In observing the load deflection curves for specimens 1 and 2 in Figure 7, one can conclude that the influence of these blocks is rather limited, particularly at the design-load level of 430 kip.

For specimens 3 and 4 the maximum applied T-section loads were 1100k and 1000k respectively. The average load-deflection curves for the movement between the flanges of these T-sections are recorded in Figure 7. In specimen 4, the stress-relieved connection, the T-section flanges were stiffened by heavy blocking plates, similar to those used for specimen 2. Comparing the deformations one notices again relatively little difference between the two connections. Also the fact that the column flange plates, to which the T-section flanges were bolted, were reduced from 3 1/2 in. for specimens 1 and 2 to 2 in. for specimens 3 and 4 seems to have no significant influence on the stiffness of these T-connections.

During the T-section tests it was noticed by measuring the change in length of the 1 1/4 in. diameter H.S. bolts between the T-sections and the column flanges that the bolts located near the center of the connection elongated more than the bolts near the end of the section. This would indicate that the load transfer was more concentrated near the center of the section. This assumption was indeed supported by the stress distribution in the stem of the T-section as based on the strain gage information. Figure 8 shows a diagram of the average normal and shear stress distribution along section A-A located in the stem just above the filet of the T-section. It seems from observing these graphs that the presence of the blocking plates in specimen 4, as compared to specimen 1, resulted in a slightly better load distribution in the center portion of the T-section. The distribution in the outer portions are rather similar.

The load transfer in the welded plate due to a load applied to the T-section has also been studied and shows some interesting results.

The stress distribution in the two sections B-B and C-C as shown in Figure 9 has been evaluated for each specimen as based on the strain-gage data. The general shape of the distribution of the tensile stresses along section B-B agrees completely with the earlier discussed occurrence of a concentrated load transfer through the center portion of the T-section. This transfer results in the development of high stresses in the moment plates near the inside end of the flange welds. It can be further noticed that the stresses in section B-B are larger for the stress-relieved specimens 1 and 4 than for the non-stress relieved specimens 1 and 3. This is probably partly caused by the fact that the transverse stiffness of the moment plates is smaller for the non-stress relieved specimens 1 and 3, than for the stress relieved specimens 2 and 4. This is due to the presence of transverse residual stresses in the non-stress relieved joints, which therefore permit the development of local yielding in the moment plates at relatively low loads. This results subsequently in a loss of overall stiffness and a transfer of the load from the moment plate to the column web. Another factor which has contributed to a higher load transfer through the moment plates in specimens 2 and 4 as compared to specimens 1 and 3 might have been the presence of the blocking plates on the T-section flanges in the first two specimens (2 and 4).

Another conclusion from the stress distribution along section B-B can be drawn regarding the influence of the relative influences of flange and blocking plates. We observe that for specimen 1 approximately 40% of the T-section load is transferred through the welded plate, while for specimen 2 this percentage amounts to about 50%. However for specimens 3 and 4, with the relatively weak 2 in. thick column flanges these percentages are about 35% and 55% respectively. It seems that the stiffness of the column flanges improve the load transfers through the moment plate under T-section loading when blocking plates are omitted (40% against 35%). However the relative influence of the blocking-plates in combination with the weaker flange of specimen 4 seem to be considerable as indicated by the increased percentage of the load transferred through the welded plates. Despite the presence of a weaker flange in this case, specimen 4 shows a 55% load transfer through the moment plate against 50% for specimen 2.

Welded-plate loading:

With the loads applied to the non-stress relieved welded plates of specimen 1, the response was inelastic. Figure 10 illustrates this behavior in a plot of the deformation between the welded plate and the column flange (see also Figure 4). It can be assumed that localized yielding started almost immediately in several small areas where stress-concentrations occurred. The major yielding started at about 1800k simultaneously near the weld between the plate and column web and over larger areas of the connection plates around the welds near the outside edges of the column-flanges. Under increasing load the yielding progressed over the entire area of the connection plate and terminated at approximately 2600k. The ultimate load carried by this connection was 2805k with failure taking place in the welded connection plate. Initial cracking of the welds near the edge of the column flanges was noted at 1800k and all four welds showed crack growth as the load increased. The final failure was initiated at one of these cracks and extended instantaneously across the connection plate to the opposite weld (Figure 15). It should be noted that the weld at which the final failure was initiated had a large slag inclusion (Figure 16).

For specimen 2 - which was stress relieved - the load-deformation curve was fairly linear up to 1400k (see Figure 10). At this load the first crack was noted along the weld between the welded plate and the column flange just under the deformeter gage. This crack caused a sudden increase of the deformation at this load, as recorded by this gage and produced in Figure 10. The weld under the deformeter showed a crack growth as the load was increased. The overall yielding over large areas of the welded plate started at approximately 2200k. At that load the three remaining so far uncracked, welds started to crack also. It should be noted that at 2000k the deformation of this connection as measured by the deformeter gage was approximately 2.5 times that of specimen 1. This difference can probably be explained by the fact that for specimen 1 the deformation gage was placed over a weld which, although cracked, had no slag inclusion and therefore a less pronounced crack propagation. For specimen 2, however, the deformation happened to be measured over a cracked weld which had a considerable slag inclusion, and therefore a larger crack propagation.

The overall yielding in specimen 2 - which started at 2200k - had not yet come to an end when failure occurred at a load of 2780k. This failure again started at a weld with a large slag inclusion, which was in this case the weld over which the deformation gage was initially placed. As in the case of specimen 1, the crack traveled instantaneously across the plate and caused complete failure.

In order to improve the weld near the edges of the column flanges it was suggested to extend the beveled weld about one inch beyond the face of the column flange to allow a better cleaning and, if necessary, gouging of this weld prior to laying the next bead. This would prevent slag inclusion and separate the transition zone between weld and base material from the directly highly stressed area near the edge of these column flanges.

It should be noted further that after the first crack in the weld had occurred, the load on specimen 2 was cycled several times from 10 to 1000k and from 10 to 2000k. As noted from figures 10 and 11, the crack increased progressively in size after each of these load cycles. The load level at which this progressive failure took place was well above the design load of 230k.

With the loading in the welded plate, specimen 3 did not show a definite yield point as noted from figure 10, which is a plot of the deformation between the welded plate and the column flange. The connection appears to be yielding throughout the whole cycle as was also noted for specimen 1. The ultimate load carried by this connection was 2,192 k. with failure taking place in the welded plate. Initial cracking of the lower welds was noted at 1600 kip. No readings were obtained after approximately 1600 kip as the connection was yielding so extensively. Initial yielding started at 1400k.

For specimen 4 a fairly linear load-deformation curve was obtained up to 600 kip as noted in Figure 10. At approximately 1300 kip the first crack in the weld was noted. It will be observed that the deformation of this connection at 1000 kip is approximately 1.4 times that of specimen 3. This was probably due to earlier cracking of the welds, which in turn must have been caused by weld inclusions near the cracks. After the 1600 kip load was reached the specimen was unloaded and the welded plate close to one of the welds between the plate and the column web was cut.

With the one web weld cut and the load applied to the connection plate, specimen 4 showed a linear load-deformation curve between column flange and welded plate up to approximately 700k as noted from Figure 10. It should be observed, however, that even though this is a higher load level than was attained before when all welds were fully active, the deformation at 600k in this case is about 2.0 times larger than recorded previously for a non-cut connection at that same load level. For a load of 1000k the deformation of this cut-web connection is approximately 4.4 times that of the non-cut connection. The ultimate load carried in shear only was 1,342k. Failure took place due to shearing in the welds between the welded plate and column flanges on the side of the cut web.

Figures 12 and 13 show for each of the specimens the average stress distribution for two sections, located close to the welds between the loaded plate and the column section. This stress distribution is based on an applied load of 600k.

An important conclusion can be drawn from these results. It seems that the load carried through section D-D is larger for the two non-stress relieved specimens 1 and 3. Consequently the total shear force transferred along sections E-E is smaller for the non-stress relieved specimens as compared to the stress relieved ones. The results are as follows:

	end weld (D-D)	two side welds (E-E)
Specimen 1:	64%	36%
Specimen 2:	53%	47%
Specimen 3:	62%	38%
Specimen 4:	49%	51%

Assuming that the applied load should be transferred proportional to the weld lengths the above percentages should be 44% and 56% respectively. We immediately note that the end welds are actually 20 to 50% higher stressed than expected. This agrees with the observed early yielding which developed in the welded plate near the two ends of the end weld. The reason for the higher loads in the end welds for specimens 1 and 3, as compared to 2 and 4, is probably due to the residual stresses, particularly near the side welds. These stresses reduce, under increased loading, the stiffness near these welds and force the load transfer to concentrate towards the end weld. Also the fact that the applied load was led into the plate close to the welds resulted in a concentrated load transfer towards the end weld. Should the load have been transferred further away from the welds the side welds would have carried more load.

Another interesting observation can be made with respect to the shear-stress distribution. It seems that the high biaxial tensile stresses in the plate and weld near the edge of the column flanges does not permit the development of shearing stresses in that area. It further seems that the shear-stress distribution in the middle portion of the welds is more evenly distributed for the stress relieved specimens than for the non-stress relieved specimens.

Figure 14 shows a typical elastic stress-trajectory pattern. This pattern is derived from the isoclines which resulted from the photostress studies. The stress-trajectories clearly indicate a concentrated stress flow towards the end weld.

VI. CONCLUSIONS

The experimental study of two pair of large-size girder-to-column connections has provided valuable information with respect to the behavior of these connections under different loads. The 36 in. by 36 in. built-up columns were designed to support 42 in. deep girders. The connecting elements were T-sections, which were H.S. bolted (28 - 1 1/4 in. bolts) to the column flanges, and connection plates which were welded between the web and column flanges

The results of loads applied to the T-sections indicated that the stiffness of the T-section-to-column connection was not improved very effectively by the blocking plates on top of the T-section flanges (see Figure 7). The stress distribution in the stem varied from approximately 0.80f average at the side to 1.20f average near the center portion of the section (see Figure 8). The stress distribution in the welded plates under a load applied to the T-section was affected by the stress relieving of the welded plate, the thickness of the column flanges and the relative thickness of the blocking plates as compared to the thickness of the column flanges. The stresses in the welded plate due to this T-loading were maximum (1.5 to 2.0 f average) near the inside end of the weld between the column flange and welded plate.

The following yield and ultimate loads were recorded for the welded plates:

Specimen 1. non-stress relieved; welded plate 30 x 2 in.^{2}

$P_y = 1800k$, $f_y = 30$ ksi; $P_{ult} = 2805k$, $f_{ult} = 47$ ksi

Specimen 2. stress relieved; welded plate 30 x 2 in.^{2}

P_y = 2200k, f_y = 37 ksi, P_{ult} = 2750k, f_{ult} = 46 ksi

Specimen 3. non-stress relieved, welded plate 30 x 2 in.^{2}

P_y = 1400k, f_y = 31 ksi, P_{ult} = 2200k, f_{ult} = 49 ksi

Specimen 4. stress relieved; welded plate 30 x 1 1/2 in.^{2}. After load of 1600k was reached end weld was cut.

P_{ult} = 1.324k, $f_{shear\ ult}$ = 1.324/(2 x 10 x 1 1/2) = 44 ksi

The results for P_y indicated that the stress distribution in the moment plates was improved by stress relieving. Similar conclusions were derived from the stress distribution along critical sections close to the welds. These welds indicated that the distribution of the load over the several welds was better for the two stress relieved connections than for the non-stress relieved specimens.

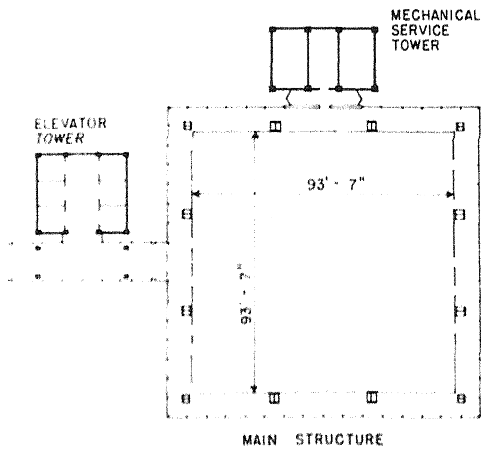

FIG I HEALTH SCIENCES COMPLEX
UNIVERSITY OF CALIFORNIA, SAN FRANCISCO

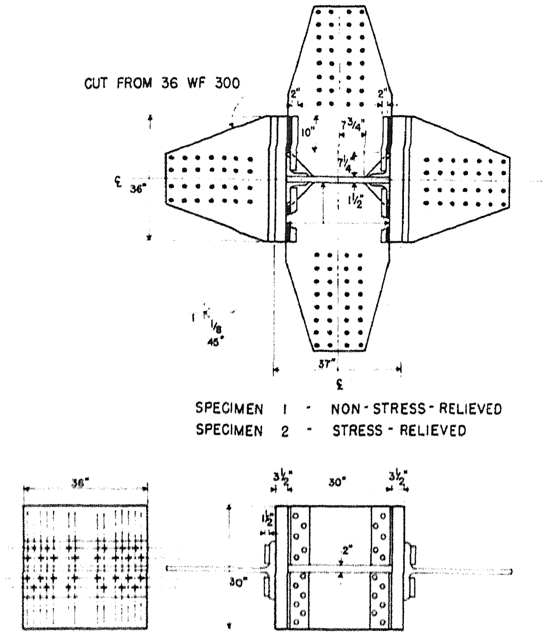

FIG. 2. TEST SPECIMEN TYPE I
(3rd FLOOR BEAM-COLUMN CONNECTION)

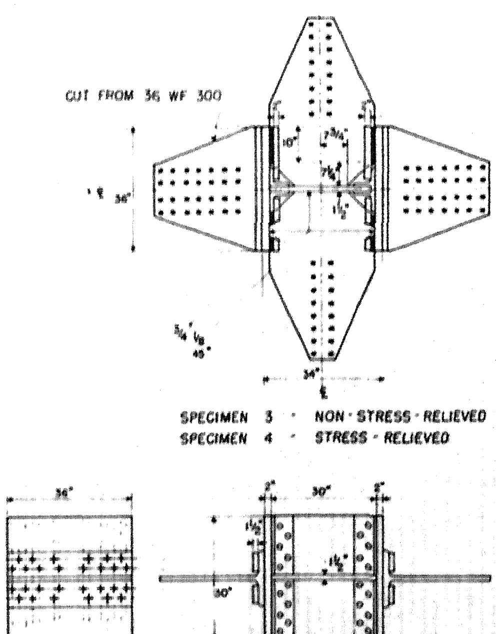

FIG. 3. TEST SPECIMEN TYPE II
(14th FLOOR BEAM-COLUMN CONNECTION)

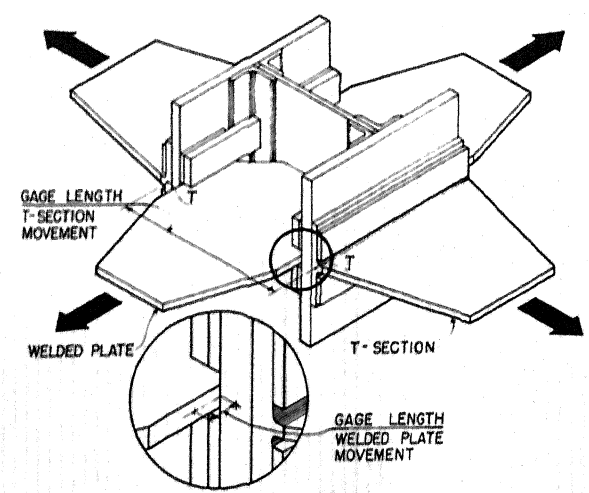

FIG. 4. LOADS ON SPECIMEN (SCHEMATIC)
OPPOSING LOADS APPLIED IN SEPARATE PAIRS ONLY.

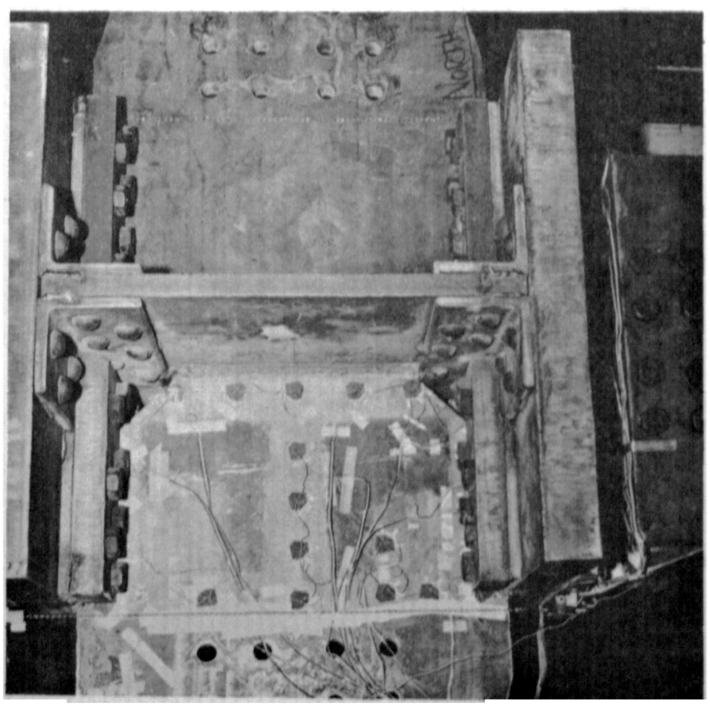

FIG. 5 PART OF INSTRUMENTATION.

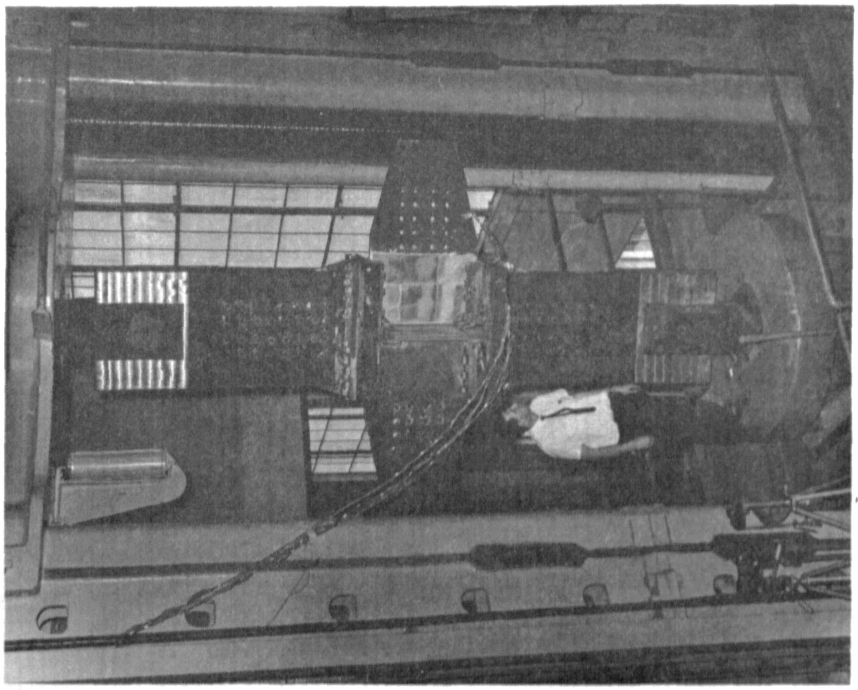

FIG. 6 TEST ARRANGEMENT

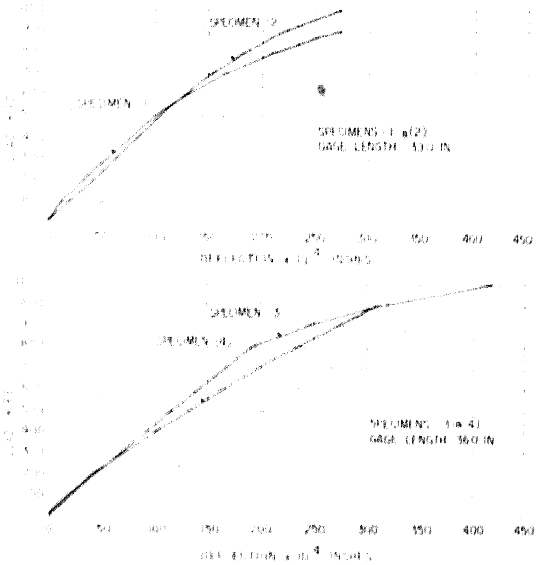

FIG. 7 MOVEMENT BETWEEN T-SECTIONS FOR SPECIMENS 1, 2, 3, & 4

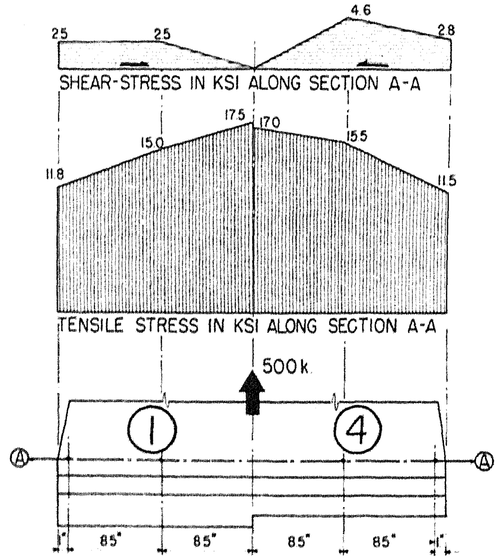

FIG. 8. AVERAGE STRESS DISTRIBUTION IN T-SECTION UNDER T-SECTION LOAD OF 500k. FOR SPECIMENS 1 & 4.

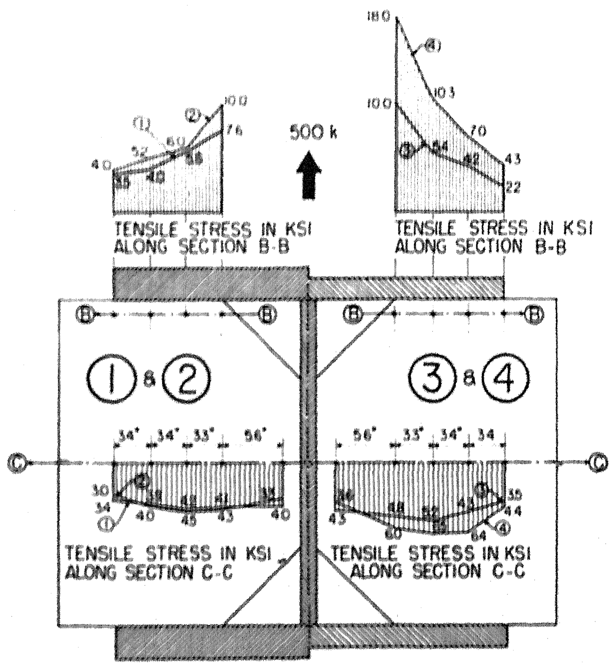

FIG. 9 AVERAGE STRESS DISTRIBUTION IN WELDED PLATE UNDER T-SECTION LOAD OF 500k. FOR SPECIMENS 1, 2, 3, & 4

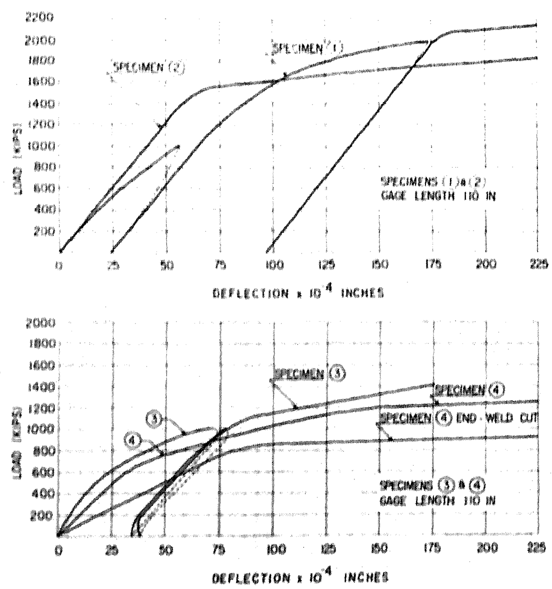

FIG. 10 MOVEMENT BETWEEN COLUMN FLANGE AND WELDED PLATE FOR SPECIMENS 1, 2, 3, & 4

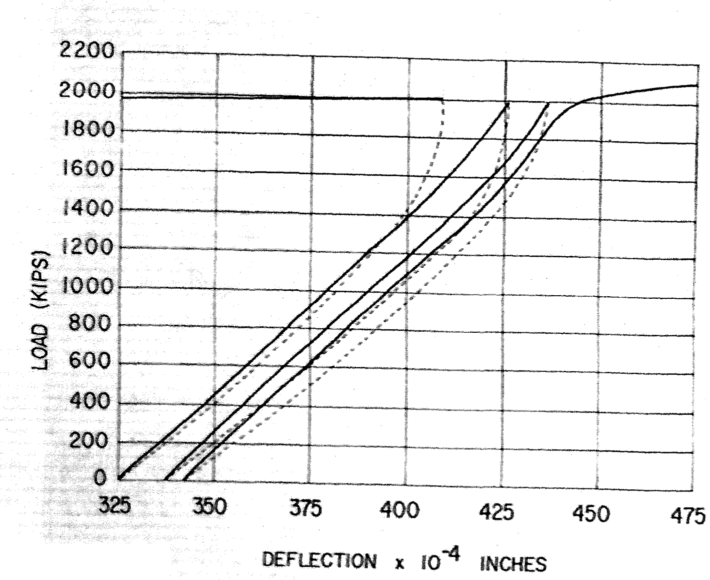

FIG. 11. MOVEMENT BETWEEN COLUMN FLANGE WELD AND WELDED PLATE AFTER INITIAL CRACK IN SPECIMEN 2.

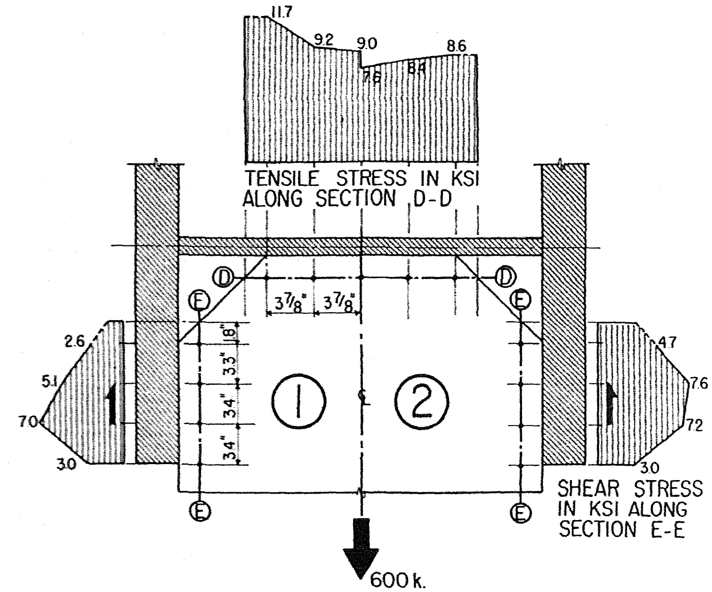

FIG. 12. AVERAGE STRESS DISTRIBUTION IN WELDED PLATE FOR LOAD OF 600 k. (SPECIMENS 1 & 2)

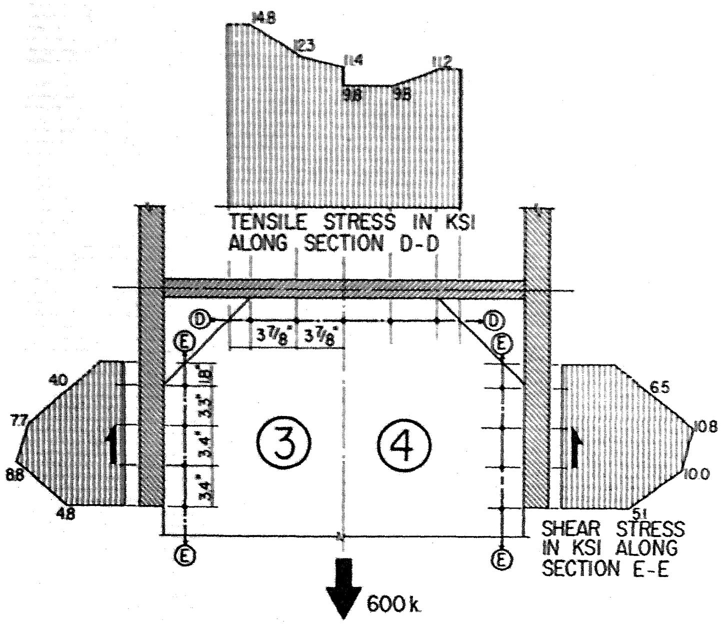

FIG. 13. AVERAGE STRESS DISTRIBUTION IN WELDED PLATE FOR LOAD OF 600k. (SPECIMENS 3 & 4)

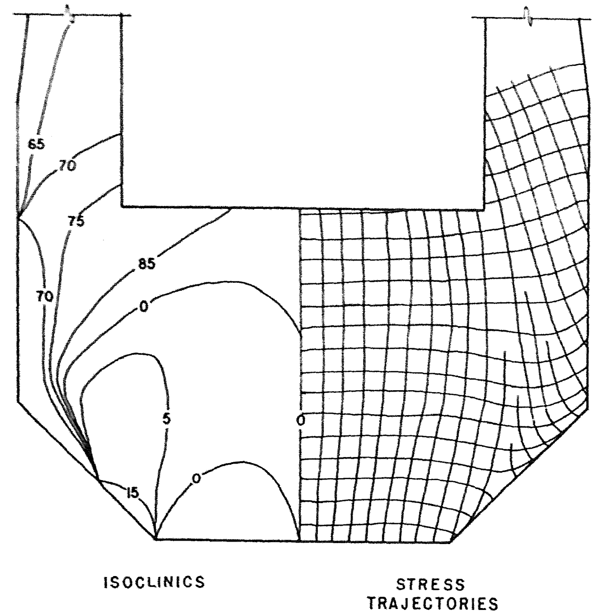

FIG. 14. ISOCLINICS AND STRESS TRAJECTORIES FOR WELDED PLATE UNDER AXIAL LOAD

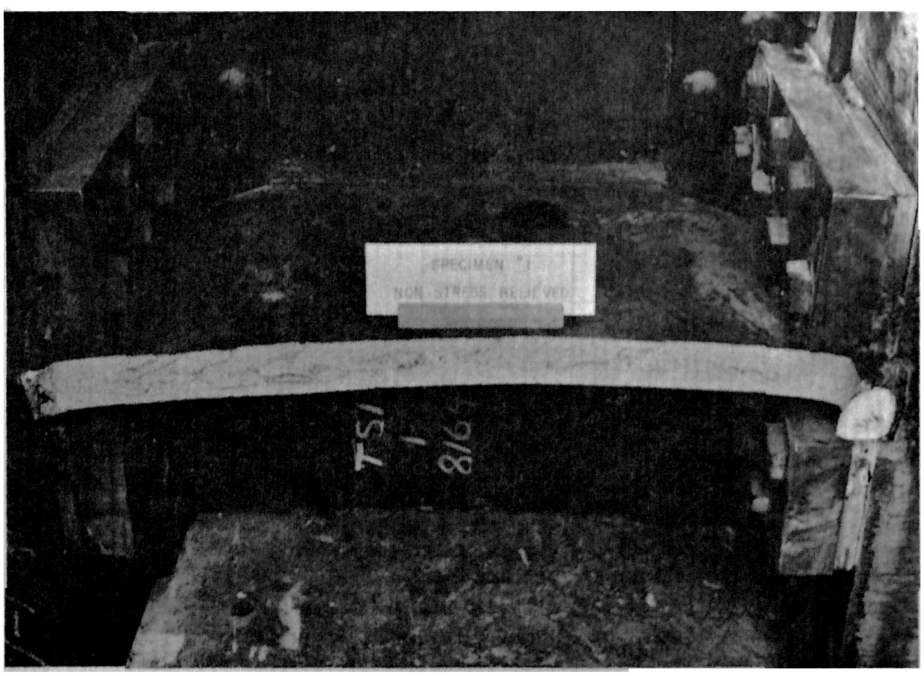

FIG. 15 TYPICAL FAILURE

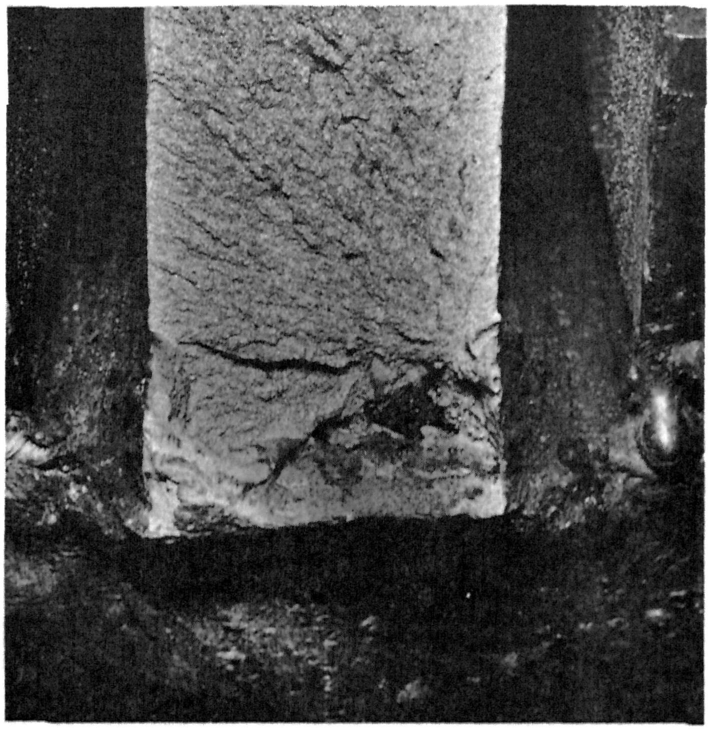
FIG. 16 SLAG INCLUSION

INVESTIGATION FOR THE EARTHQUAKE RESISTANT DESIGN OF LARGE SIZE WELDED AND BOLTED GIRDER TO COLUMN CONNECTIONS

BY J.G. BOUWKAMP

QUESTION BY: C.A. POWER - NEW ZEALAND

Fig. 4 shows that the test tensions of the flange plates are opposing - under seismic action these forces would be in the same direction. Fig. 13 shows a large percentage of load taken by end welds across DD under test. Have welds along EE alone sufficient strength to develop full strength of the flange plates?

AUTHOR'S REPLY: The directionsof the forces on the welded plates as shown in figure 4 are indeed erroneous when compared to the direction of forces produced by an actual earthquake. However, the test arrangement permitted only the application of opposing loads.

In order to determine the strength of the side welds E-E (see figure 13) the final test on specimen 4 was performed with the weld along D-D cut in one of the two welded plates. Under the application of opposing loads failure occurred at a load of 1342k, which is 60% of the ultimate load of 2200k which was recorded for specimen 3. One can expect that, due to the improved stiffness condition in a completely welded plate as compared to the plate tested in specimen 4, the ultimate load under actual seismic conditions would be larger than the now recorded failure load. One can expect that failure would probably occur close to the failure load of specimen 3. But even in case the lower failure load would be true, the joint would undoubtedly still withstand a design load of approximately 150k (according to the Uniform Building Code). Even a load of approximately 450k for this floor level under El Centro conditions would be taken by the joint without failure; particularly when we notice from figure 10 that the yield load for the joint tested was approximately 875k.

QUESTION BY: E. Del VALLE - MEXICO

1. You have the columns formed by three plates joined by rivets. Have you joined the plates in that way in order to increase damping of the structure or just to reduce secondary stresses due to welding?

2. On figs. 12 and 13 you show a nearly parabolic distribution of stresses in the fillet welds. I would expect a distribution with stresses larger

at the ends of the weld than at the center. Have you measured the stresses at a lower stage of load, and if so, how has the abovementioned type of distribution been prevented?

AUTHOR'S REPLY:

1. I believe that the engineers specified riveted rather than welded built-up columns primarily to increase damping and not with the intent to reduce residual stresses due to welding.
2. May I note first that the welds were full-penetration butt-welds. As far as the nearly parabolic stress distribution is concerned, I must admit that we were also surprised initially to find this type of a distribution.

It should be noted that the observed sections were located about $1\frac{1}{2}$in. from the inside face of the column flanges. Therefore it might very well be possible that high shearing stresses at the ends of the welds did occur without being noticed.

The observed strains near the ends of these sections were proportional to the applied load until P_y was reached. These strains represented a shear stress distribution as shown in figures 12 and 13. The principal reason for this form of distribution in our opinion is the presence of high axial and transverse stresses (parallel and normal to the sections E-E and F-F respectively). The axial stress of course is due directly to the applied load. The transverse stress of practically the same magnitude as the axial stress, is a result of the high transverse restraining action in the welded plate due to the column flanges and adjacent bolted T-sections. The simultaneous occurrence of these two high normal - almost principal - stresses prevented the development of high shearing stresses along the particular sections.

An additional reason for the low shearing stresses in sections E-E and F-F near the face of the column-flange edge but of secondary nature is probably the manner in which the bolted splice plates transferred the applied load into the welded plates. Referring to figures 2, 3, and 5, we note that the bolt holes in the welded plate were close to the center portion of the plate and close to the line connecting the edges of opposite column flanges. The load transfer through these holes resulted undoubtedly in a concentrated force flow to the end weld as the test results indicated. This resulted consequently in a less concentrated load transfer to the side welds and might have contributed to the absence of high shearing stresses near the end of these particular sections.

The shearing stresses in the far end of the section I believe are less because of the relatively small force which is transferred through the side welds.