DEPARTMENT OF DESIGN

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COLUMN-TEST COOPERATIVE PROJECT

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SYNOPSIS

The purpose of the Louisiana Column-Test Cooperative Project was to investigate some structural problems concerning the behavior of columns and compression members. These arose in connection with the design and construction of the Calcasieu River Bridge at Lake Charles, Louisiana, and were considered to be of sufficient importance to warrant a comprehensive program of tests. The project was undertaken under the auspices of the Louisiana Department of Highways and the Bureau of Public Roads.

● The bridge, with an over-all length of 6,895 ft., furnishes a direct connection from the City of Lake Charles to US 90. It also provides a navigation channel 380 ft. wide, over 200 ft. of which the vertical clearance above ordinary high water is 135 ft. The structure has two roadways of 26 ft. each, separated by a 4-ft median strip, and is designed for an H 20 (44) live load.

The river crossing consists of an 840-ft. cantilever structure, with anchor arms of 210 ft., cantilever arms of 120 ft. and a suspended span of 180 ft All panel lengths are 30 ft and the trusses are spaced 67½ ft. center-to-center. The cantilever structure is flanked on both sides by two simple deck spans, each of 180-ft. length. The west approach consists of a series of 52 deck girder spans, with a total length of 2,868 ft. Similarly on the east, there are 52 deck girder spans of 2,457-ft. total length. In both approaches the span lengths vary, the maximum being 90 ft. In general, the piers are of the two-column type, with every fourth span having a four-column braced pier. On the west, the maximum approach grade is 3 8 percent, while on the east, the maximum is 5.0 percent. The entire structure was designed by the Louisiana Department of Highways. Since the Federal Government participated in the cost, the Bureau of Public Roads reviewed and approved the design plans and specifications. Construction was begun early in 1948, and the bridge was opened to traffic on September 28, 1951.

The trusses of the 840-ft. cantilever structure are fairly heavy, in spite of the fact that silicon steel is used in the main members. The chords are box sections, made up of two web plates, four flange angles, and two perforated cover plates. The maximum depth is 28½ in. back-to-back of angles. The cover plates on the heaviest members are each 18 in. by ⅝ in. with 10- by 20-in. perforations at 3-ft. 4-in. centers. These cover plates replace the lacing bars and batten plates which would formerly have been used, and furthermore, the areas of these plates outside the perforations are assumed to be fully effective in carrying stress. In reviewing the design of the structure, in 1944, the Bureau of Public Roads raised a question concerning the ability of relatively light perforated cover plates in box sections to properly transmit shear and to distribute stresses throughout the compression members themselves, when the areas of the web plates and the flange angles form large percentages of the total areas of the members. The ability of the net cover plate areas to carry full, direct stress was also questioned.

Another question arose in regard to the effectiveness of the column sections used in the approaches. The columns of some of the highest single piers are made up of eight angles, one web plate and two cover plates. In appearance, each one resembles an I-beam with a channel attached to each flange. The web plates extend in a longitudinal direction. Transversely, they are braced at midheight, but there is no bracing in the longitudinal direction.

In the four-column braced piers, the individual columns are braced both longitudinally

and transversely. The section used for all of the four-column braced piers, and for all except a few of the highest single piers, consists of a wide flange beam with a channel riveted to each flange.

Since in both types of columns just described the flanges are wide and heavy, it was felt that there might be a tendency for the flanges to rotate. In order to overcome this tendency, it might be necessary to connect them by batten plates, either with or without lacing. It would, however, be highly desirable to omit these batten plates, if there were assurance that rotation and distortion of the flanges would not occur.

There were therefore two questions on which further information was desired. One was concerned with the use of perforated cover plates on compression members of the trusses and the other had to do with the effectiveness of the column sections in the approach spans. Accordingly, the Louisiana Department of Highways and the Bureau of Public Roads entered into an agreement to plan and carry out a program of tests. Since it was desirable, if possible, to test full-sized specimens, it was necessary to consult with testing laboratories in order to determine the capacities of testing machines available and the lengths of the specimens which they could accommodate. The result was that the shorter, but heavier, columns were tested at the National Bureau of Standards while the lighter, but longer, ones were sent to the University of Illinois for testing.

COLUMN TC-1

The bottom chord member L_4L_5 of the cantilever structure was selected as typical of the heavily stressed compression members in trusses. It is made up as follows:

- 4 angles 4 in. by 4 in. by $\frac{1}{2}$ in. (2 ft. $4\frac{1}{4}$ in. b. to b.)
- 2 web pls. 28 in. by $\frac{5}{8}$ in.
- 2 web pls. 28 in. by $\frac{3}{4}$ in.
- 2 cov. pls 18 in. by $\frac{3}{8}$ in. (with 10-in. by 20-in holes at 3-ft.-4-in. centers).

Made of silicon steel, with a length of 30 ft. 1 in. between panel points, this member has a slenderness ratio of 38.2 about the X axis and 38.4 about the Y axis. Its estimated ultimate strength is about 4,750,000 lb. The 10,000,000-lb. testing machine at the National Bureau of Standards was available for use, but the extreme distance between its heads was only 23 ft. 6 in. For the test specimen, therefore, it was decided to use the full cross-sectional area, but to substitute A-7 carbon steel for the silicon steel and tô make the over-all length 22 ft. 10 in. With this length, the slenderness ratio was 29.2 about the X axis and 29.3 about the Y axis. It was planned to test a series of three members of this type, the exact proportions of the second and third specimens to be determined after the results of the first test were known. The columns of this series are shown in Figure 1. It should be noted that at one end these columns have full bearing on the base plates, while at the other, the sections are stopped 2 in. short of the base plates. The loads are transferred by rivets from the columns into plates and delivered by them to the base plates. It was intended that these plates would act somewhat as gusset plates in joints where the contact surfaces have no bearing.

Prior to the testing of Column TC-1, a total of 90 single SR-4 gages and 28 rosettes were installed. Since each rosette is actually three gages in one, this arrangement required the tabulation of 174 gage readings. In particular, 36 single gages were placed at close intervals along the peripheries of the two perforated holes on Reference Line A of Figure 1. In general, gages were placed in identical locations on the insides and the outsides of plates so that differences in readings could be compared. Compressometers having gage lengths equal to the length of the column were attached to each of the four corners. Lateral deflections at midheight were measured by means of wires stretched along the center line of each face. The column had a gross area of 105.5 sq. in. and a net area through the 10-in. holes of 98.0 sq. in. Tensile tests made from coupons removed from the various plates and angles indicated that the yield points ranged from 39,200 psi. to 43,700 psi., with an average of 40,900 psi. Ultimate strengths ranged from 63,400 psi to 68,200 psi., the average being 66,000 psi. The modulus of elasticity averaged 29,600,000 psi. and the average elongation in 8 in. was 27.2 percent.

During the test, loads were applied in increments of 400,000 lb. in the elastic range and in 200,000-lb. increments beyond the elastic range. Readings of gages, compressometers,

and deflectometers were made for each increment of load and at zero load after each increment. The stress-strain relationship remained fairly linear up to a load of 2,800,000 lb. Thereafter, the strains increased at a more rapid rate. At this load, the gages indicated that yielding was taking place in the metal. The column failed at a load of 3,300,000 lb. Failure was characterized by local buckling at the center, as shown in Figure 2. In the cover plates, the maximum strains measured along the periphery of the perforations occurred at points on the curved portions close to the points of tangency. At a load of 3,200,000 lb., the strain was 0.0046 in. per in. For the 2,800,000-lb. load, at which yielding of the metal began, the average stress on the cross section was 28,600 psi., while the maximum must have been the yield point of the metal, which was about 40,900 psi. The average stress was therefore only 70 percent of the maximum.

The allowable load on this column, based on the allowable unit stress, is 1,450,000 lb. The load on the column at which yielding began was 1.93 times, and the load just before failure was 2.21 times the allowable design load.

COLUMN TC-2

In view of the results of the test of Column TC-1, it was decided to use heavier cover plates on Column TC-2. No changes were made in the web plates, nor flange angles, but the thickness of the cover plates was increased from $\frac{3}{8}$ in. to $\frac{3}{4}$ in. with 10- by 20-in. perforated holes, as before. The arrangement of gages was similar to that used in Column TC-1, except that some rosettes were replaced by single gages in cases where the stress had been found to be practically parallel to the column axes. On Column TC-2, gages were added on the cover plates and the flange angles along Reference Line A (see Fig. 1), opposite the perforations. Of single gages, 154 were used on Column TC-2 and there were also 8 rosettes, making a total of 178 gages to be read. The physical properties of the materials, determined from tensile tests of coupons, indicated that the yield points ranged from 35,400 psi. to 39,900 psi., with an average of 37,900 psi. The ultimate strengths averaged 67,400 psi., the range being from 64,600 psi. to 71,400 psi. The modulus of elasticity averaged 30,100,000 psi., and the average elongation in 8 in. was 26.8 percent. The column had a gross area of 119.0 sq. in. and a net area through the perforations of 104.0 sq. in.

Column TC-2 was tested at the National Bureau of Standards, and the procedure followed that used on Column TC-1. Loads were applied in increments of 400,000 lb. in the elastic range and in 200,000-lb. increments beyond the elastic range. The gages, compressometers, and deflectometers were read after each increment of load and at zero load after each increment. The relationship between stress and strain remained linear up to a load of 2,800,000 lb. After that, the strains increased at a more rapid rate. At this load, the gages indicated that the metal was yielding. The column sustained a maximum load of 3,490,000 lb., and the failure was characterized by local buckling at the center, as shown in Figure 3. Along the periphery of the perforations in the cover plates, the greatest strains again occurred on the curved portions very near the points of tangency. At these points, the strain was 0.0029 in. per in. for a load of 3,200,000 lb. Under the load of 2,800,000 lb., at which yielding began, the average stress was 26,900 psi., while the maximum must have been close to 37,900 psi., the yield point stress of the material. The average stress on the section was therefore only 71 percent of the maximum.

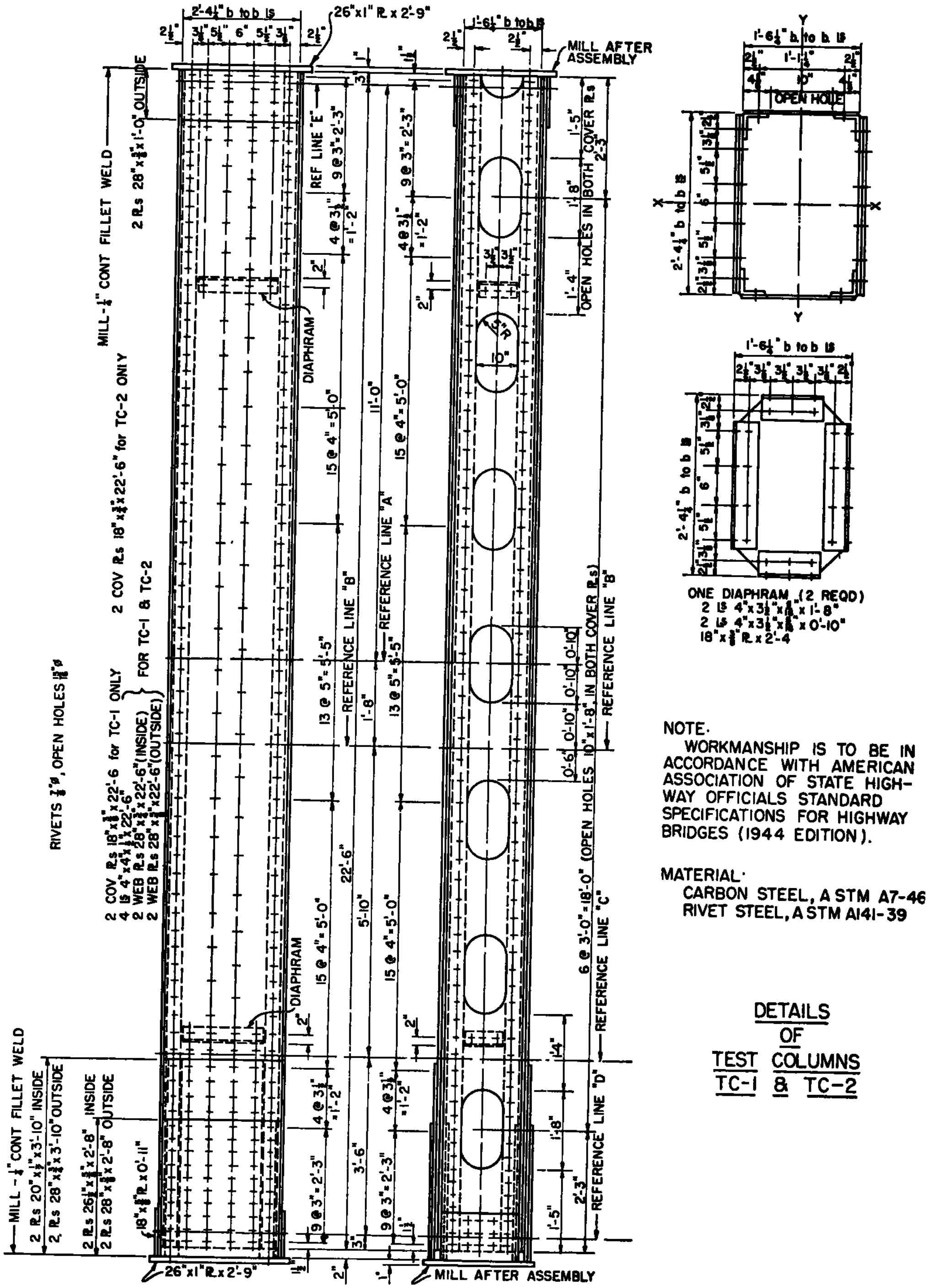
The allowable load on this column, based on the basic allowable unit stress, is 1,540,000 lb. The load on the column at which yielding began was 1.82 times, and the load just before failure was 2.21 times the allowable design load.

The tests of Columns TC-1 and TC-2 settled the questions concerning perforated cover plates and the testing of a third column of the series was deemed not necessary.

COLUMN 52-C-1

The columns of the approach bents are long and it was not possible to use full-size speciments in testing. In the highest of the two-column bents (Bent No. 52), the columns are 102 ft. long and they have the following cross section:

8 angles 4 by 4 by $\frac{1}{2}$ in. ($26\frac{1}{2}$ in. b. to b.)
1 web pl. 26 in. by $\frac{9}{16}$ in.
2 cover pls. 20 in. by $\frac{1}{2}$ in.



NOTE.
WORKMANSHIP IS TO BE IN ACCORDANCE WITH AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES (1944 EDITION).
MATERIAL.
CARBON STEEL, ASTM A7-46
RIVET STEEL, ASTM A141-39
DETAILS OF TEST COLUMNS TC-1 & TC-2
ONE DIAPHRAM (2 REQD)
MILL AFTER ASSEMBLY
26"x1" PL x 2'-9"
REFERENCE LINE "A"
REFERENCE LINE "B"
REFERENCE LINE "C"
REFERENCE LINE "D"
REF LINE "E"
DIAPHRAM

Figure 1

In the direction normal to the web plate, these columns are braced at midheight, but parallel to the web plate there is no bracing. The slenderness ratio about the axis normal to the web is 104.4 and about the axis parallel to the web 112.5. Since the 3,000,000-lb. testing machine at the University

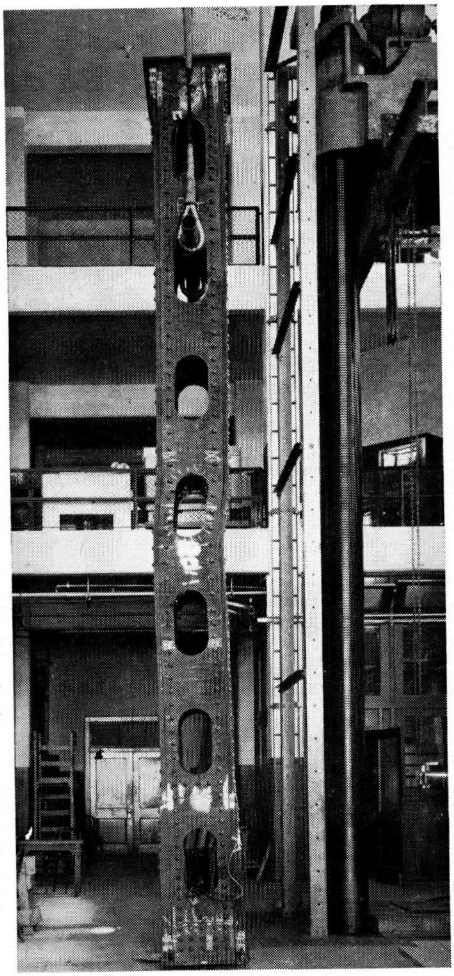


Figure 2. Column TC-1 after failure.

of Illinois could accommodate specimens up to 35 ft. in length, it was decided to use test specimens of $\frac{3}{4}$ size, with a length of 34 ft. 8 in. over-all. Column 52-C-1 had the following cross-section:

8 angles 3 by 3 by $\frac{3}{8}$ in. ($20\frac{1}{2}$ in. b. to b.)
1 web pl. 20 by $\frac{7}{16}$ in.
2 cover pls. 15 by $\frac{3}{8}$ in.

The slenderness ratio about the axis normal to the web was 46.5 and that about the axis parallel to the web, 103.7. In both the structure and the test specimen, the axis of bending was the axis parallel to the web and the slenderness ratios about this axis were approximately equal. The details of Column 52-C-1 are shown in Figure 4.

Five sections, near the ends, the quarter points and the center of the column were laid off, and at each one of these sections twelve

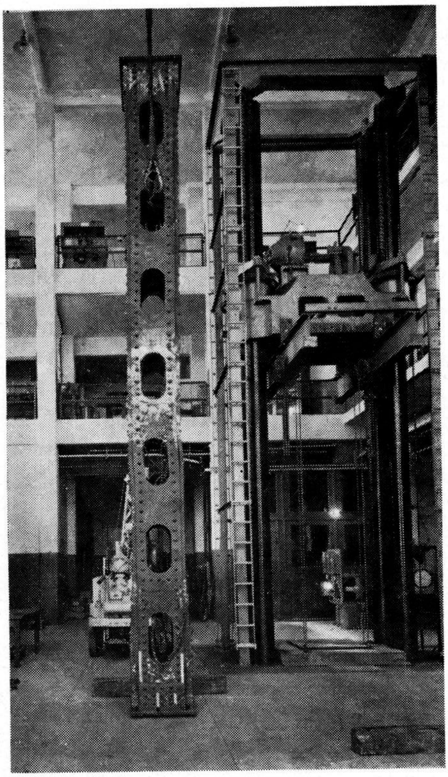


Figure 3. Column TC-2 after failure.

single SR-4 gages were installed. Mechanical gages of 24-in. length were also provided, gage holes being punched 12 in. above and below each of the sections. Four piano wires, stretched from the top to the bottom of the specimen, offered a means of measuring deflections. The physical properties of the materials, determined from tensile tests of coupons removed from each piece, indicated that the yield points ranged from 38,800 to 44,700 psi., the average being 41,700 psi. The ultimate

strengths ranged from 62,700 to 64,800 psi., with an average of 64,000. The elongation in 8 in. averaged 26.2 percent.

Column 52-C-1 was installed in the testing machine with the web plate extending in a applied in increments of 100,000 lb., except that no readings were taken at 300,000- and 500,000-lb. loads. The measured strains on all the five sections were fairly linear for loads up to and including 800,000 lb. After that, the

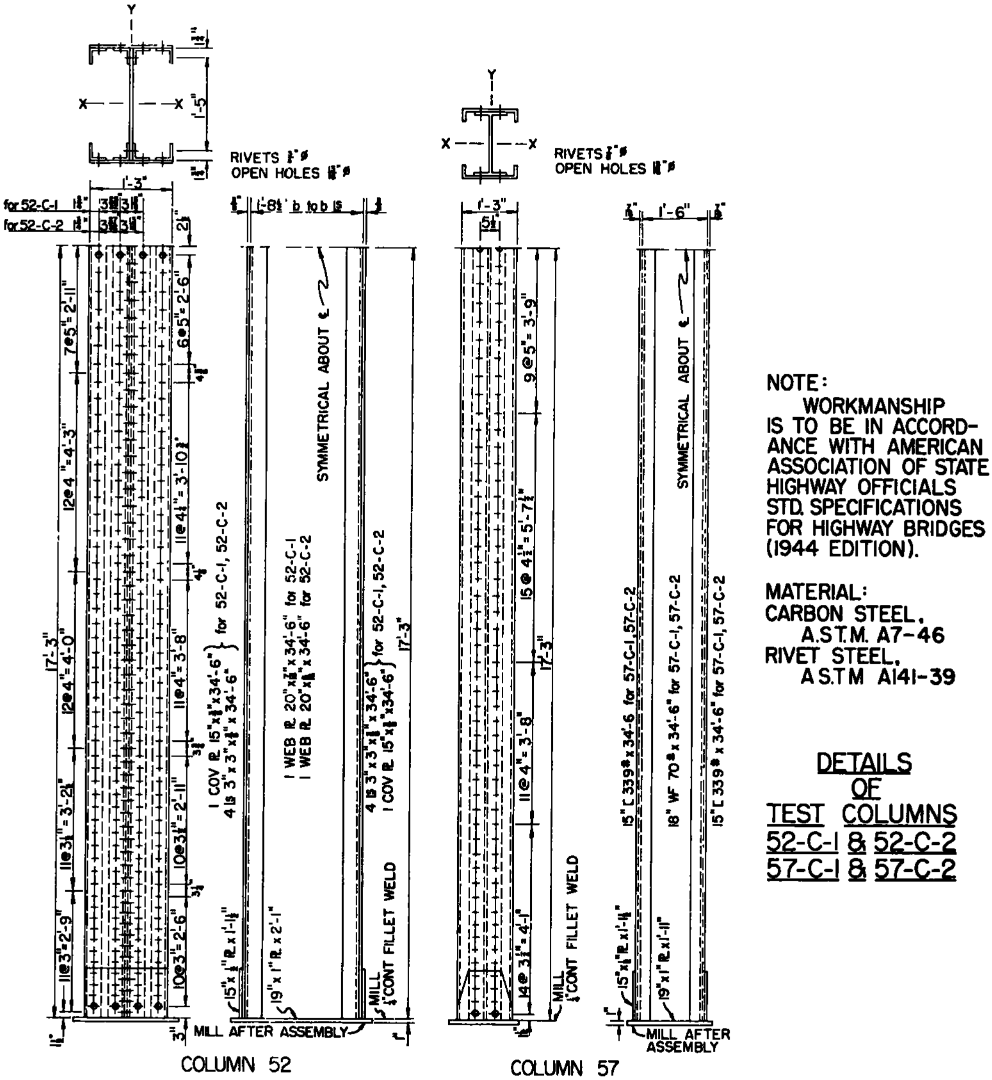


Figure 4.

north-south direction. The open sides were therefore to the east and the west. Initial deflections were about 0.33 in. to the east at midheight, and about 0.24 in. to the north at the lower quarter point.

In the testing of Column 52-C-1, loads were strains increased at a more rapid rate. The gages indicated that yielding began at a load of 1,100,000 lb. Since the cross-sectional area of the column was 36.88 sq. in., the average stress at this load was 29,800 psi. The corresponding maximum stress was the yield

point stress of 41,700 psi. The average stress was therefore about 71.5 percent of the maximum stress on the section. The column sustained a maximum load of 1,190,000 lb. At the load of 1,100,000 lb., the deflection to the east (normal to the web) was 0.78 in., and that to the north was 0.31 in. Failure appeared to be due to flexure of the column as a whole, accompanied by rotation of the flanges. Local buckling of the cover plates also occurred at several points. Near the center of the column, the distance between the toes of the flanges increased in the east face and decreased in the west face. At the 1,100,000-lb. load, the east face opened about 0.06 in., while the west face closed about 0.04 in. Figure 5 is a view of the column after failure.

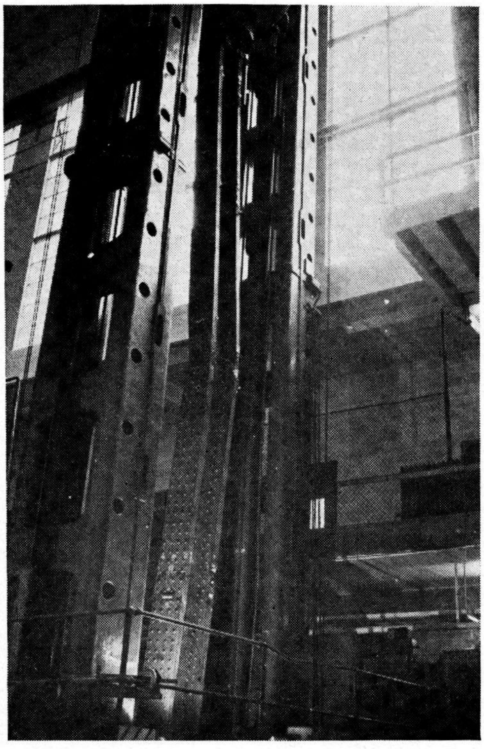


Figure 5. Column 52-C-1 after failure.

The allowable load on this column, based on the basic allowable unit stress, is 454,000 lb. The load on the column at which yielding began was 2.42 times the allowable design load.

COLUMN 52-C-2

Following the test of Column 52-C-1, it was decided to fabricate another, similar to 52-C-1, but with the web-plate thickness reduced from $\frac{7}{16}$ in. to $\frac{5}{16}$ in. This column was designated 52-C-2. Its gross area was 34.38 sq. in. The physical properties of the materials, determined from the tensile tests of coupons showed that the yield points ranged from 37,500 psi. to 39,800 psi., with an average of 38,400 psi. The ultimate strengths varied from 59,800 psi. to 65,700 psi., the average being 62,400 psi. The elongation in 8 in. averaged 28.4 percent.

While the purpose of the reduction in web-plate thickness was to observe the effect on the general behavior of the column, the plate was now so thin that there was a possibility that local buckling would occur at relatively low unit stresses. Therefore, all the gages, both electrical and mechanical, of Column 52-C-1 were duplicated on 52-C-2. In addition, four SR-4 gages of 6-in. length were placed at the four corners at the midheight, and in order to observe the behavior of the web, eighteen SR-4 gages were placed on each face of the web along its center line, from the lower quarter point to the center. Web deflection stations were installed from the center of the column to the upper quarter point.

Column 52-C-2 was installed in the testing machine with the web extending in a north-south direction. It had initial deflections of about 0.09 in. to the north, near its midheight and about 0.12 in. to the west, about 5 ft. above midheight. During the test, loads were applied in increments of 100,000 lb. within the elastic range. When the strains began to increase at a more rapid rate, the load increments were reduced to 25,000 lb. The stress-strain relationship was fairly constant for loads up to and including 800,000 lb. Thereafter, the strains increased at a more rapid rate. At a load of 950,000 lb., the gages indicated that yielding of the metal was taking place. The column finally sustained a maximum load of 1,075,000 lb. As had been anticipated, local buckling of the web plate did occur. The tendency was evident in the early stages of the test, and it became quite pronounced before the maximum load was reached. The corrugations were not symmetrical, as would have been expected from theoretical considerations. The effects of the lateral

deflections and of the flange rotation may have been sufficient to produce the variations. As loads were applied, the initial lateral deflection to the west decreased and at a load of 950,000 lb , the deflection was 0.40 in to the east. The flange rotations were rather small up to a load of 800,000 lb., after which they increased rapidly. At the 950,000-lb. load, the west face between the toes of flanges closed about 0.070 in., while the east face opened 0.095 in. The failure of Column 52-C-2 appears to have been due to a combination of flexure of the column as a whole, buckling of the web plate and rotation of the flanges.

From the gage readings, it was evident that yielding of the metal began to occur at a load of about 950,000 lb , indicating that the maximum stress must have been about 38,400 psi. Over its area of 34.38 sq. in. the corresponding average stress was 27,600 psi. The average stress then was about 72 percent of the maximum.

The allowable load on this column, based on the basic allowable unit stress, was 423,000 lb. The load on the column at which yielding began was 2.25 times the allowable design load.

At the conclusion of the tests of 52-C-1 and 52-C-2, the questions which had been raised concerning the behavior of columns of this type had been settled, and it was considered not necessary to test a third column of the series.

COLUMN 57-C-1

In the four-column braced piers, the greatest unsupported lengths are found in Bent 57, where they equal 46.7 ft. in each direction. These columns are made up of a 21-in. WF 73-lb. beam, with an 18- by 42.7-lb. channel riveted to each flange. The slenderness ratios are 59.0 about the axis normal to the web and 112.0 about the axis parallel to the web. A reduction in size of test specimen was again necessary in order to come within the length limitation of the testing machine. The first test specimen of this series, designated Column 57-C-1, had the following cross section:

1—18-in. WF 70-lb. beam
2—15-in. by 33.9-lb. channels

With an over-all length of 34 ft. 8 in., this specimen had a slenderness ratio of 52.0 about the axis normal to the web, and of 100.6 about the axis parallel to the web. These ratios agree fairly well with those of the columns in the structure. When Column 57-C-1 was received at the University of Illinois, a large number of strain marks, probably caused by the punching operations, were visible around each of the rivets in the flanges of the beam. Tensile tests made on coupons removed from the beam and from the channels after the column had been tested, indicated that the yield point varied from 40,700 psi to 43,500 psi., with an average of 41,900 psi. The ultimate strengths ranged from 63,000 psi. to 67,900 psi., with an average of 65,600 psi. The average elongation in 8 in. was 26.1 percent. Gages were installed on Column 57-C-1 in exactly the same manner as on Column 52-C-1. The column was installed in the testing machine with the web of the beam extending in a north-south direction. Initial deflections were about 0.14 in. to the south and about 0.09 in. to the west.

In the test of Column 57-C-1, loads were applied in increments of 200,000 lb. At a load of 600,000 lb., horizontal strain lines began to appear across the web of the beam, indicating that yielding in shear was taking place. This yielding was accompanied by a spalling off of the mill scale along lines or bands where the yield point of the material in shear had been exceeded. These bands extended through the web at an angle of 45 deg. The reason for this action is not known and no satisfactory explanation has been offered. Because of it, the strain measurements in the web for loads of 600,000 lb. and over were excluded.

The relation between compressive stress and strain in Column 57-C-1 remained fairly linear up to and including the 1,000,000-lb. load. After that, the strains increased at a more rapid rate. The gages indicated that yielding in compression occurred at a load of 1,200,000 lb., accompanied by maximum stresses equal to the yield point of the metal. The column area was 40.36 sq. in., and for the 1,200,000-lb. load, the average stress was 29,730 psi., about 71 percent of the corresponding maximum stress. The column sustained a maximum load of 1,330,000 lb. before failure.

At a load of 1,280,000 lb., which is 96 percent of the maximum load carried, Column 57-C-1 deflected 0.60 in. to the west. Under this same load, the distance between the toes of the flanges had increased about 0.036 in. in the west face and decreased about 0.054 in.

in the east face. The failure of the column appears to have been due to the flexure as a whole, accompanied by a slight rotation of the flanges. The extent to which the shear yielding in the web influenced the strength is not known. Figure 6 shows the column after failure.

The allowable load on this column, based on the basic allowable unit stress is 500,000 lb. The load on the column at which yielding

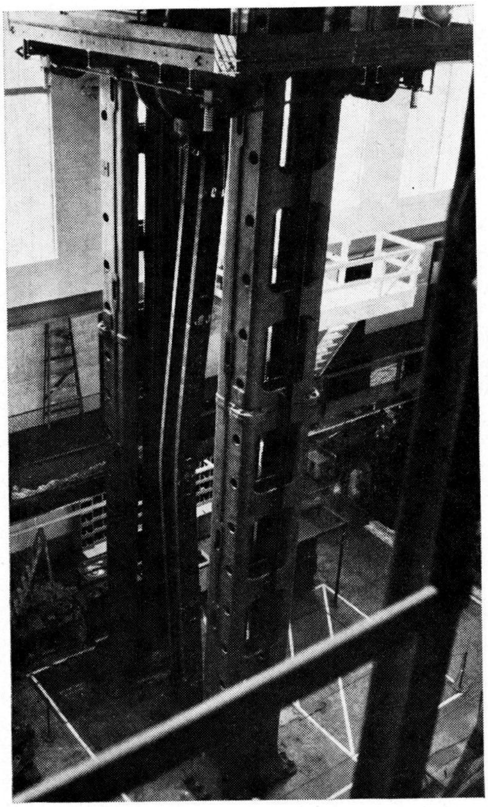


Figure 6. Column 57-C-1 after failure.

began was 2.40 times the allowable design load.

COLUMN 57-C-2

Because of the unusual behavior of Column 57-C-1 and the uncertainty regarding the effect of the yielding of the web in shear, an exact duplicate was fabricated and designated 57-C-2.

When received at the University of Illinois, strain lines were again visible around each of the rivets in the flanges of the beam. As before, they may have been caused by the punching of the rivet holes. Tensile tests made from coupons removed from the webs of the beam and channels showed that the yield points ranged from 37,400 to 39,600 psi., the average being 38,500 psi. Ultimate strengths varied from 63,800 to 64,300 psi., with an average of 64,000. The elongation in 8 in. averaged 30.0 percent. A slight modification was made in the arrangement of the SR-4 gages. The two gages on the web of wide flange beam were omitted in order to eliminate the effect of the excessive web yielding in the early stages of the test. In their place, four additional gages were mounted on the toes of the beam flanges in each of the five sections described earlier. Further, four SR-4 gages of 6-in. in length were placed in the four corners of the column at the midheight section. The 24-in. mechanical gages were also installed, as before. Deflection measurements were read from piano wires stretched between top and bottom of the specimen.

When Column 57-C-2 had been installed in the testing machine, with the beam web in a north-south direction, it had initial deflections of 0.03 in. to the north and of 0.09 in. to the west. During the test, loads were applied in increments of 200,000 lb., up to a load of 800,000 lb. From 800,000 to 1,100,000 lb., the increment was 100,000; and for loads over 1,100,000 lb., the increment was 50,000 lb. The column sustained a maximum load of 1,265,000 lb. The relationship between stress and strain remained fairly linear up to and including an 800,000-lb. load. Thereafter, the strains increased at a more rapid rate. The gages indicated that yielding was taking place at a load of 1,000,000 lb., and this was accompanied by maximum stresses equal to the yield point of 38,500 psi. Since the area was 40.36 sq. in., the average stress for the 1,000,000 lb. load was 24,800 psi., about 64 percent of the maximum.

As loads were successively applied to the column, the initial deflection to the west decreased until at a load of 1,250,000 lb., the upper half of the column deflected 0.42 in. to the east. Under this same load, the distance between the toes of the flanges in the west face decreased 0.03 in. and that in the east face had increased by an equal amount.

The allowable load on this column, based on the allowable unit stress, was 500,000 lb. The load on the column at which yielding began was 2.00 times the allowable design load.

With the testing of Columns 57-C-1 and 57-C-2, the points in question had been clarified, and it was considered not necessary to test a third column of the series.

SUMMARY

The tests clarified the questions which had been raised in regard to certain features incorporated in the designs of three types of compression members, although the over-all strengths of some of the test specimens fell somewhat below expectations.

In relatively rigid compression members, the $\frac{3}{8}$-in. perforated cover plates performed satisfactorily until yielding had taken place in the web plates. Within the elastic range, the $\frac{3}{4}$-in. cover plates showed little advantage. In the plastic range, however, the deformations in the thicker cover plates were considerably less than in the thinner plates.

The columns of the 52-C series were rather flexible. The initial deflections and also those under load were of some magnitude. In the elastic range, the flange rotations were not excessive, but in the plastic range, they increased rapidly. One test-column web plate with an excessive ratio of width to thickness buckled at a relatively low stress.

In the 57-C series, the columns were more rigid and both the initial deflections and those under load were small. The flanges rotated very little, even after yielding was taking place.

For all the columns tested, the loads at which yielding should have occurred were estimated by means of the secant formula. The columns of the TC series were very rigid and they had practically no deflection in the elastic range. The average stresses should have gone up almost to the yield point before yielding began. However, they reached only about 70 percent of the expected values. The columns of the 57-C series likewise had rather small deflections, and it was expected that their average stresses at yielding would be about 80 percent of the yield point stresses. Actually, they reached only about 68 percent. On the other hand, the 52-C series columns were more flexible. Their deflections, although not excessive, were considerably greater than those of the 57-C series. It was expected that the average stresses at yielding would be only about 63 percent of the yield point stresses, but actually they reached 72 percent. Thus, they carried about 14 percent more load than was expected, account being taken of all the measured deflections.

Columns TC-1, 52-C-1, and 57-C-1 had been tested before the members of the Lake Charles Bridge were fabricated. No changes in the design of these members were recommended or made. The bridge has since been opened to traffic and the members are performing satisfactorily.

The research project is being continued at the University of Illinois. A complete analysis of all the test data will be made. At the present time, the causes of the discrepancies in the load carrying capacities are being studied, but no conclusions have been drawn.

ACKNOWLEDGEMENT

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A. H. Stang was in charge of the tests at the National Bureau of Standards until his retirement, when he was succeeded by W. R. Campbell.

At the University of Illinois, W. M. Wilson was in charge of the testing, and upon his retirement, he was succeeded by N. M. Newmark. W. H. Munse was in immediate charge of the tests.