Measured and Computed Stresses in Three Castellated Beams

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THREE CASTELLATED BEAMS, representative of those fabricated for the American Dental Association Building in Chicago, were tested at the Civil Engineering Structural Testing Facility of the University of Missouri at Columbia. The three beams were each approximately 37 ft in length and expanded between 40 and 50 percent from a nominal depth of 18 in. In one case rectangular plates were interposed between the chords in order to gain additional depth. The primary purpose of the testing was to obtain a comparison between the stresses measured with electrical strain gages at various points of interest and the stresses at those same locations as determined by a simplified Vierendeel truss analysis similar to an approach proposed by Toprac and his colleagues at the University of Texas.[1] The difference lies in the assumption of the distribution of primary bending stress in the chords. A uniform distribution was assumed in the Texas approach, but a linearly varying distribution was assumed in the design of these beams. The latter approach is described in detail by Boyer in a previous article in this Journal[2] and will not be reiterated here, except to note that for simplification inflection points are assumed at the center of each "truss member" and vertical shear on the beam is assumed to be shared equally by each of the two "chords".

Two of the beams tested were somewhat unusual in that at several locations the hexagonal web holes which are normally a result of the castellating process were enlarged by removing web "posts". This was done in order to provide access for ductwork larger than would be accommodated by the normal size web holes. The enlarged holes were then reinforced at their edges by bars welded perpendicular to the web. It was in one of these beams having the enlarged holes that additional depth was achieved with web inserts. Interest in this particular beam was also centered on the behavior of the connection of the beam-web-to-column-flange at one end. Both of the beams with the enlarged holes were representative of the floorbeams in the building, and were loaded with a uniform load.

A third beam tested was representative of one of the girders supporting the floorbeams. Because of the heavy concentrated reactions applied to it, and a desire to limit its depth, the designers called for flange cover plates and longitudinal web reinforcing bars running the full length of the beam near the two horizontal edges of the web holes. Vertical web stiffeners were used at the points of concentrated loading. It was partly because of these deviations from the normal that interest was directed toward the applicability of the simplified Vierendeel analysis. The ultimate moment carrying capacity was not a primary goal for study in this series of tests.

BEAM DESCRIPTION AND TESTING PROCEDURE

The smallest of the three beams was 37 ft long and cut from an 18 WF 60 expanded to a 24 in. depth (Fig. 1). A uniform service load was simulated with eight hydraulic jacks spaced along the beam such that none was placed over a web hole. Enlarged web holes occurred at five equally spaced locations along the beam's length. The design required that horizontal stiffening bars be added at the top and bottom of each of these holes in order to increase the section modulus of the chords. However, vertical stiffening in order to increase the modulus of the adjoining posts was required at only one enlarged open-

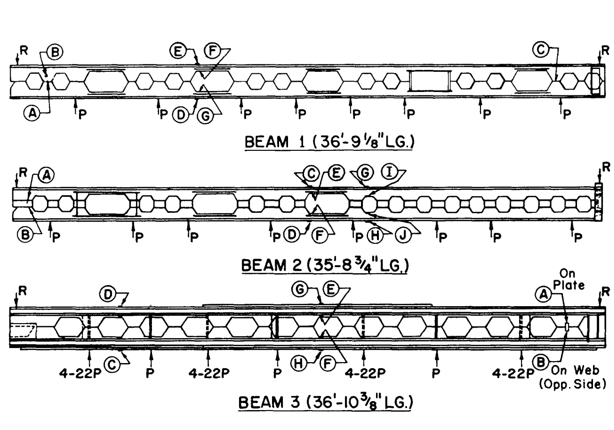

Fig. 1. Location of gages

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ing. Strain gages were placed at the center of the second post from each end so that the shearing stresses could be monitored in these high shear regions. The remaining gages measured flexural strains in regions of high shear and regions of combined shear and moment. Simple support was provided by rollers and plates.

The second beam, a 35 ft-$11\frac{1}{4}$ in. floorbeam, was different in that 5-in. deep increment plates were used in expanding an 18WF 70 to slightly over 27 in. (Fig. 1). In this case three passages for large ducts were provided. The uniform loading was again simulated by forces from eight jacks. One reaction was wall bearing and the other supported by a beam-web-to-column-flange bolted connection. Rosette gages for measuring shearing strains were placed both at the center of the plate in one end post and also near the butt weld joining the plate to the beam web at a point adjacent to the hole. At this latter location a combination of shearing, axial, and flexural stresses would exist, according to the Vierendeel truss approach. At a location of high moment in the beam, gages measured the flexural strains in the bars reinforcing the holes and in the beam flanges. In addition, at this location the flexural stresses in the web near the edges of an unreinforced hole were monitored, as were the flange stresses.

The third beam was an 18 WF 114 by 37 ft-0 in. expanded to a depth of 26.38 in. Although the reactions from the floorbeams which would supply the loading were not all equal, they could, for laboratory convenience, be classified nicely into two groupings in which the larger of these averaged 4.22 times that of the smaller. Two sets of jacks, each connected to a separate hydraulic system, were used to apply the loads in this ratio. The design required that a $2\frac{1}{2}$ x 6 x $\frac{3}{8}$ in. reinforcing doubler plate be welded to the web at the inflection point of one of the end posts in order to reduce shearing stresses. 15-ft long cover plates (8 in. x $\frac{3}{8}$ in.) were used. On the compression flange, 4-in. wide filler bars extended the remaining length of the beam. It was in this beam that horizontal stiffening bars were welded to the web for the full length of the beam. Flexural stresses were measured in these bars and the flanges, both in a region of combined shear and moment and in another location of high moment. Shearing stresses were measured both on the face of the reinforcing doubler plate and at the corresponding position on the opposite face of the web, where there was no plate. The reactions were both wall bearing. Vertical stiffeners were located at concentrated loads.

The beams were tested in an inverted position with the loading jacks reacting against the laboratory floor. The reactions were resisted by an overhead loading frame. A light horizontal truss in the plane of the compression flange provided essentially a continuous lateral support.

The load was applied in increments and the data recorded under static conditions. In the cases of Beams 2 and 3 the load was reduced to almost zero after the design load had been attained, in order to observe any permanent deformations occurring at the working load. The load was then reapplied to the maximum that was planned for the test. In all three cases this was either at or in excess of 160 percent of the working load.

The behavior of the three beams during testing was much the same. Yielding first appeared on the tension side of the beam around the ends of the short bars reinforcing the enlarged web holes. For Beam 1 this happened at less than the working load. Such yielding was undoubtedly due to residual web tension stresses arising from contraction of the stiffening bars after welding. Such yielding did not exhibit any significant progression during the remainder of the test. In the beam having 5-in. deep increment plates it was noted that yielding appeared on the plates near the welds at slightly less than

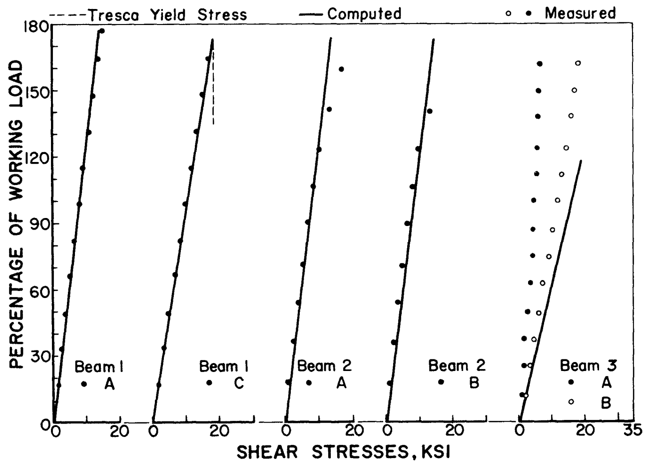

Fig. 2. Horizontal shear stresses

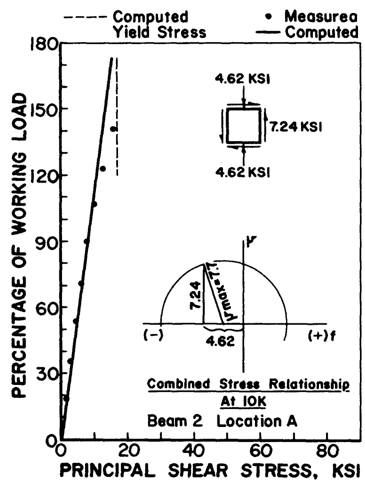

Fig. 3. Principal shear stress

the working load. This occurred in the half dozen or so plates nearest each end. At about 125 per cent of the working load these web inserts exhibited signs of general yielding. It should be noted that when this particular beam was unloaded from the working load plateau it was observed that such plate yielding had not caused any significant permanent deformation of the beam as a whole.

MEASURED AND COMPUTED STRESSES

At locations **A** and **C** of Beam 1 and **A** and **B** of Beams 2 and 3 (Figs. 1 and 2) it is seen that the measured shearing stresses correlate very well with the stresses that would be computed by merely dividing the expected horizontal shear force in the post by the cross-sectional post area. This is somewhat at variance with a theoretical parabolic distribution of shear stress which would exist on a rectangular prismatic cross section where the maximum stress occurs at the central axis of the section and is equal to 1.5 V/A. It indicates that in these tests either the shear at the inflection point of the post was not as high as the Vierendeel approach would indicate, or quite possibly the reentrant corner of the web hole creates a stress concentration which has a levelling effect on the stress distribution. However, gage **A** of Beam 2, which was placed at the center of the plate and thus was less influenced by a stress concentration, also shows that V/A produces a good indication of the shearing stresses to be expected in the elastic range.

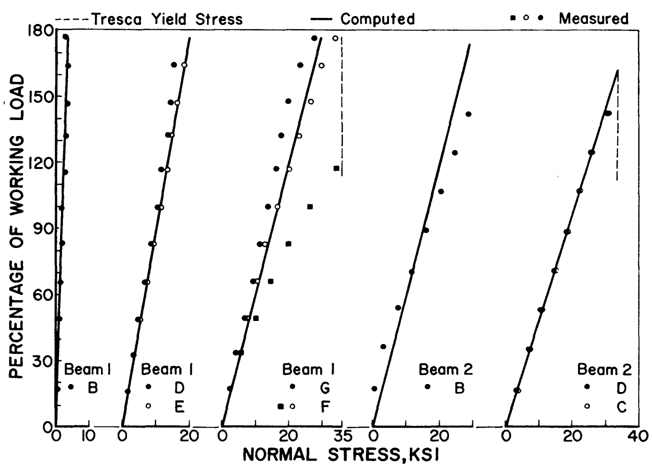

Fig. 4. Normal stresses

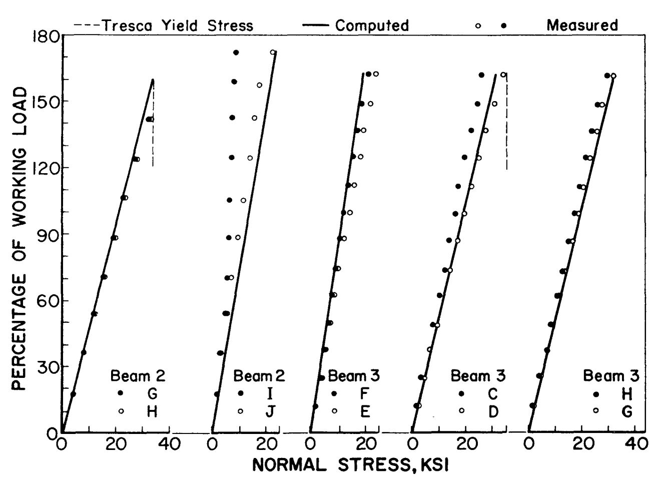

Fig. 5. Normal stresses

The stresses, which can only be deduced from measured strains, are shown only up to a limit which is designated as the computed yield stress. This limit has been arrived at on the basis of the Tresca yield theory. In essence, the theory states that yielding will occur in the material when the maximum shear stress at a point reaches half the tensile yield stress as obtained from the usual tension coupon test. It is one of the easier theories to handle and is known to describe adequately the commencement of yielding in ductile materials such as steel when they are subjected to a biaxial stress condition. This is the situation in the end posts over the reactions. The simplified method of analysis assumes that the total shear on the cross-section at an opening is divided equally between the upper and lower chords; thus, a vertical force equal to one-half the reaction is assumed to exist in the end posts. If the planes on which the stresses are computed are oriented as in Fig. 3, and there are both shearing and normal stresses on these planes (shown in Fig. 3 for a jack load of 10 kips), then there is some other orientation of planes for which the shear stress is even higher. The simplified Vierendeel truss analysis also indicates no axial forces at intermediate posts. Therefore the shear stress on a horizontal plane at locations **A** and **C** of Beam 1 is the maximum, and yielding should commence when this shear stress reaches $F_y/2$.

On Beam 2 the situation for gage **A** is somewhat different from the situation for gage **B**. At **A** a compressive force is combined with a shear force. At **B**, the edge of the hole, there is a computed compression stress from both an axial force and a flexure condition, but no shearing stress. Thus, since no shear stresses would be expected, yielding should commence at location **B** when the sum of the axial and bending stresses reaches F_y.

A Mohr's circle construction for the stress combination at gage **A** on Beam 2, where the axial force is half the beam reaction, produces a maximum shear stress that is only seven percent larger than the horizontal shear stress, and which acts on a plane oriented about nine degrees from the horizontal (Fig. 3).

At gage **B** on Beam 2 it is seen again that the measured horizontal shear stress is predicted very well if the horizontal shearing force is divided by the horizontal area

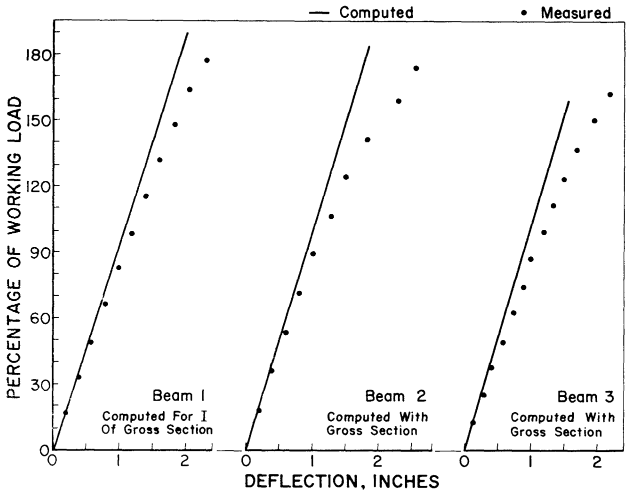

Fig. 6. Beam deflections

of the post. Thus, the measured stress is again not consistent with the expected parabolic shear distribution that should exist on a horizontal section at this point, i.e., no shearing stresses at the edge of the hole. The strain gage, because of its size, was actually centered on a point about $\frac{3}{4}$-in. from the edge of the hole, but even at this point the shearing stresses should be insignificant if the simplified strength of materials approach is accepted. The situation is obviously affected by a concentration of stress around the reentrant corner.

At other gage points, the observed stresses were very close to those computed by the simplified analysis. The only exception was gage **F** of Beam 1, which yielded early; this may be explained by the close proximity of this gage to a fillet weld at the reinforcing bar.

DEFLECTIONS

Figure 6 shows the measured deflections compared with those computed on the basis of the gross moment of inertia, i.e., disregarding the hole but allowing for the increased depth. In all three cases the measured deflections below the working load were not more than ten percent greater than those computed. The difference is probably due to local flexing of the chords and posts acting as flexural members in themselves. In the case of Beam 2 it can be seen that the deflection rate increased when the increment plates exhibited the previously noted yielding.

In two of the cases, Beams 2 and 3, the load was released after the working load had been attained in order to observe any permanent set that may have occurred at that time (not shown in Fig. 6). In both cases the permanent set at working load was only $\frac{1}{10}$ in. The three beams were eventually loaded to 1.60, 1.74, and 1.63 of the working loads respectively, and the permanent sets observed at these levels were 0.335, 0.731, and 0.438 in. respectively.

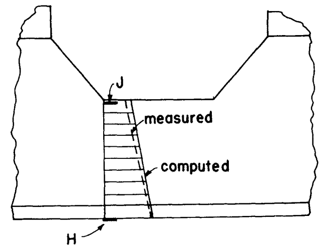

Fig. 7. Working load stress distribution in chord of Beam 2 as determined by gages H and J

SUMMARY AND OBSERVATIONS

It is evident that the simplified Vierendeel truss analysis used in the design of the beams was quite adequate for describing the actual stress behavior (see Fig. 7). Both the measured shear and normal stresses due to the applied load were equal to or less than the computed stresses. With the exception of local yielding around welds, the behavior of each beam was essentially elastic up to the working load. Local yielding due to residual stresses around welds had no significant detrimental effect on beam performance, even though it appears that in Beam 2 these residual stresses were probably about 40 percent of the yield strength of the material, and, because they were measured at mid depth of the plate, apparently existed throughout most of the plate. Redistribution of shear among the posts because of yielding and consequent strain hardening allowed the plates to withstand an eventual 74 percent overload.

Local yielding around welds and reentrant corners was in fact detected at various locations in all three beams at less than the design load. From previous studies conducted in developing the plastic design concepts it is known that such localized yielding normally does not affect the ultimate strength of a beam.

ACKNOWLEDGMENT

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