REPORT OF LATERAL LOAD TESTS ON ELLIOT CRANES

WELD CRANE NO. 5441

Submitted to the Crane Specifications Committee
of the A. I. & S. E.

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CRANE NO. 9461

I - SYNOPSIS

This report summarizes the results of some lateral load and vertical load tests on the bridge girders of a 30 ton all-welded steel mill crane of 104 ft. span. The girders were of uniform section for about 92 ft. of the span. The girders rode on four two-wheel equalizer tracks, and were tied together by a box end tie. Lateral load was applied to the top flange by taking up on a turnbuckle fastened to both coverplates. The stresses were measured on both end ties and on fourteen gage lines on each of seven sections on one of the bridge girders. Both the stresses and lateral deflection were measured when the load was applied. It was found that the whole cross-section of the girder resisted the lateral load. End fixity was also present, and the fixed end moment was 67 per cent of that present in a fixed end beam. The test was repeated with gusset plates added between the top flange and end tie and the partial fixity factor increased to 74 per cent.

The girder was also tested under vertical load and found to act according to the ordinary beam theory.

II - INTRODUCTION

The subject of the action of lateral loads on cranes is a disputed topic. The large resistance of box girders to twisting would indicate that the whole section of the girder resists the lateral bending and not only the top flange as is
usually assumed. This is also the conclusion from theory. In order to determine this, tests were made on the bare girders of Crane No. 9441 in the fabricating shop with the end ties bolted to the bridge girders. The whole assembly was supported on the equalizer truck pins. The stresses were measured along the bridge girder, end truck, and knowing the stresses, the section which resists the lateral moment is easily determined. A sketch showing some of the details of the girder is shown in Fig. 1.

The stresses were measured by means of a 10-in. Whit-temore Strain Gage. Gage lines are established at the various sections of the girder where the stresses are desired by drilling two very small holes in the girder 10 in. apart. The distance between the holes is measured with the gage before and after the load is applied. The difference in length between these two readings gives the strain in the member due to the applied load. The results are generally accurate to ± 300 lb per sq in.

The lateral deflection of the girders was also measured by finding the change in distance between the adjacent flanges of the two girders. This was done on both the top and bottom flanges at seven sections along the girder. A Federal dial measuring the distance to 0.001-in. was used to determine this deflection and the difference in readings between the initial and final loads gave the actual movement of the flanges. The
difference between the lateral deflection of the top flange and bottom flange at the same section gave the twist of the girder at that section. Knowing the twist it was easy to determine the lateral deflection of the girder as a whole.

The load was applied by tightening a turnbuckle fastened to the two girders. A calibrated spring was interposed in the system and its closure was measured by 0.001-in. Ames dials. The closure was a measure of the actual load.

The lateral load test was repeated to see if some gusset plates which had been bolted to the end tie and bridge girder would increase the fixity. This was found to be the case.

The girders were also tested under vertical load. This was done by supporting the girders at the center with the ends cantilevered out. The stress readings were then taken. Then both ends were jacked up until the girder was raised off the center support and strain readings were again taken. This manner of loading gives approximately the same bending moment as a load applied at the center of the girder whose magnitude is equal to the weight of the girder.
III - TEST RESULTS: LATERAL LOADS

The girders were first tested with no gusset plates between the end ties and bridge girder, and the lateral deflection curves of the top and bottom flanges are shown in Fig. 2. The measured values were equal to the deflection of both girders, and the plotted values are the measured values divided by two since the deflections of both girders should be equal. The deflections are plotted along the length of the girder and the center line mark denotes the center line of the bridge girder. Similar curves for the test in which gusset plates were used between the end tie and bridge girder are shown in Fig. 3. It is seen that the deflection of the girder in the second test is less than that in the first test. This is due to the fixing effect of the gusset plates. It should be noted that the deflection at the ends is not always zero. This is due to initial slip in the joint. The slip is larger at the bottom than at the top because the girder hangs below the end tie for some distance. Thus, the end tie offers no restraint at this point. The deflection at the bottom flange is therefore greater than for the top flange, and this is a condition somewhat contrary to what one might usually expect.
The difference between the deflection readings of the top and bottom flanges at the same section gave the twist of the girder at that section. This is plotted in angular measure in Fig. 4. The curves do not start at zero due to the slip which occurred at the joint between the bridge girder and end tie. The torsional constants $K$ as computed from the curves are marked on the corresponding curves. The values of $K$ for the 19,800 lb. and 20,000 lb. loads are somewhat higher than that of 61,000 in$^2$ which would be obtained from Bredt's theory. The value of 54,600 in$^2$ for the 8800 lb. load is much too small. However, it is thought that the difference from the theoretical is here a matter of precision. The twist was found by taking the difference of two sets of readings (the lateral deflections of the top and bottom flanges), whose values were very close to each other, so that a very small error in the measured values would give a rather large percentage error in the difference.

The lateral deflections shown in Fig. 2 and 3, corrected for initial slip and the twist shown in Fig. 4, are plotted in Fig. 5.

The stresses were measured on the front girder. Readings were taken on each of seven sections along the girder and also on the center section of the end truck. On the top
coverplate, readings were taken at the edges and 5 in. from the edge. They were also taken at the center line of the web and at distances 20 in. and 40 in. above and below the web center line. Some readings were also taken on the bottom flange plates. Fig. 6 shows the stress distribution at the center section of the girder; and the plotted points show where the gage lines were located. The stress is plotted from the adjacent side of the box section as an origin. The stress distribution in the web should be noted. On the outside or compression side, it is seen that very little stress is taken near the mid portion of the web. This is due to either local bending or buckling of the web. Under lateral loads, the web acts as the compression flange of a beam and the h/t ratio of 240 for the clear depth of the web is so high that the stress is not uniform over the web cross-section. For the lateral load alone, the theoretical critical buckling stress would not be far from 1000 lb per sq in., but the dead load stresses present raise this value. The value of the dead load stresses cannot be figured since they would depend on the method of fabrication.

It is more likely that the reason for the stress variation is a combination of local bending and perhaps incipient buckling. If the webs had buckled, the flanges would be highly
overstressed. The test was repeated a number of times, and similar readings were taken on the center section of the webs of both girders. The results were very inconsistent, and showed that the webs were unstable at this stress. It must be remembered that the stresses measured were taken on the outside of the web, and it is thought that if it had been possible to get inside the girder and take corresponding readings on the inside of the web, that the average of the outside and inside stress would not have been far from the theoretical.

In Fig. 7 is shown the variation of stress along the girder on the inside and outside edges of the top coverplate. Fig. 8 gives the average web stress along the length of the girder. The irregular behavior of the web is clearly shown.

Fig. 9 gives the stress distribution along the top coverplate of the bridge girder for the test in which gusset plates were used. The stresses are seen to be less than those in Fig. 7 which represents the same test without the gusset plates. This shows that the gusset plates reduce the girder stresses.

The stiffening effect of these gusset plates also shows up in the location of zero stress. For Fig. 7, the point of zero stress averages 16-3/4 ft. from the end of the girder. This corresponds to a finiteness factor of 67 per cent.
The statement 67 per cent partial fixity means that the
moment at the end of the bridge girder is 67 per cent of
that which would be present if the girder acted as a
fully fixed beam.

For the girder with the gusset plates, the point
of zero stress was 18-1/2 ft. from the end, and this cor-
responds to a partial fixity of 74 per cent.

In this test, the slip which occurred in the con-
nections between the end tie and bridge girder reduced the
fixity. The gusset plates, of course, reduced the slip
somewhat and in this manner increased the fixity factor.
It is, however, impossible to get 100 per cent fixity since
the elastic deflection in the end ties allows the end of
the bridge girder to give somewhat, and this effect increases,
as the stiffness of the girder with respect to the end tie
increases. For eight-wheel cranes, the end tie is relative-
ly more flexible than for two-wheel cranes, and in the case
of the crane tested, the maximum possible end fixity due to
frame action was 54 per cent. Thus the efficiency of the
joint when the gusset plates were used was \( \frac{0.74}{0.84} \) or 85 per
cent. For the first test, when no gusset plates were used,
the joint efficiency was \( \frac{0.67}{0.88} \) or 76 per cent. It should
be remembered, however, that no attempt is usually made to
to take advantage of the end fixity in design, and the gir- 
ders are designed as simple beams.

The theoretical stresses shown in the diagrams were 
computed using the end fixities found from the points of 
contraflexure, and using a value of 24,300 in$^4$ for the hor- 
izontal moment of inertia. This is the gross moment of the 
whole girder section. The check is seen to be very close. 
The sharp break in the stress lines are due to the change 
in the depth of the girder section close to the ends, which 
causes a variation in the moment of inertia.

As an additional check, the lateral deflection was 
computed using the value of 24,300 in$^4$ for the moment of in- 
ertia and the fixity factors found from the stress curves. 
The theoretical deflection for the first run without the 
gusset plates is 0.572 in. From Fig. 5, the measured de- 
flection is seen to be 0.504 in. and this gives a good check. 
For the test with the gusset plates, the computed deflection 
was 0.516 in. which compares with a measured value of 0.500 
in. Thus the use of the whole girder to resist the lateral 
forces, and the values of the fixity factors is justified by 
the check between the observed and computed deflections.

The stresses at the 8000-lb. load were too small to 
measure with any precision; but the lateral deflections were 
measured. The center deflection of 0.205 in. measured at 
this load corresponds to a fixity factor of 80 per cent.
This is equivalent to a joint efficiency of \( \frac{0.90}{0.88} \) or 91 per cent which is a very high value, and greater than that at the higher loads. The drop in the efficiency at the higher loads is probably due to the fact that initially the joint carries most of the load by friction; and as the load increases, the frictional resistance is overcome, and the fixity value of the joint is somewhat reduced during this range.

It should be remembered that the design lateral load, due to all causes, is about 14,300 lb. The tests were therefore made at about 35 per cent overload and the stresses and deflection should be in line with what will occur when the crane is in use except as these are modified by the walkways, etc.

The end moments in the bridge girders caused corresponding end moments in the end ties. The stresses in the end tie were measured and were found to be 3000 lb per sq in. at the corners of the box section. This corresponds to the theoretical value of 3100 lb per sq in. computed from the fixed end moment. It was found in the test, however, that the large manholes in the sides of the box end tie prevented that portion of the sideplates in line with the manholes from carrying any stress. This was taken into account in the computation mentioned above.
The theoretical deflections were computed by the following formula:

\[ d = \frac{EJ}{4EI} - \frac{MI}{3EI} \]

The \( M \) is the fixed end moment as found from the measured stresses, and the other terms are as usually given in the handbooks.

The twist was computed by the formula \( \theta = \frac{EJ}{3K} \)

where \( \theta \) is the angle of twist in radians, \( E \) is the shear modulus, and \( K \) is computed from Bredt's formula. \( K \) equals \( \frac{4A^2}{\int \frac{da}{b}} \) and for a box section equals \( \frac{2b^2d^2t}{bt_1 + dt} \) where \( b \) and \( d \) are the width and depth respectively, \( t_1 \) equals the web thickness, and \( t \) is the average of the top and bottom flange thicknesses.

**IV - TEST RESULTS - VERTICAL LOAD**

As stated in the introduction, the girders were loaded by a vertical load which was equivalent to the weight of the girder itself. The stresses in the top coverplate along the length of the girder are shown in Fig. 10. They check very well with those of the ordinary beam theory as should be expected. Fig. 11 shows the stress distribution at the center section of the girder under the action of
vertical loads. The local bending in the thin webs is shown by the refusal of the web to carry its full share of stress at the center line of the web while it does carry most of its share close to the flanges of the girder. This causes the curvilinear stress distribution in the webs.

V - CONCLUSIONS

The tests on the welded box crane No. 8411 show that this crane acted in the following manner.

1. The whole section of the box girder resisted the lateral moment.

2. Partial end fixity was present between the bridge girder and end tie. In the tests made with no gusset plates on the end tie, the end fixity varied from 20 to 37 per cent as the load was increased, from 6000 to 19,000 lb.

3. A small gusset plate between the end tie and bridge girder increased the fixity factor from 67 to 74 per cent for a 20,000-lb. load.

4. The joint efficiency varied from 75 to 91 per cent.

5. The twist of this welded girder was checked fairly well by blade theory.

6. Slip occurred in the joint between the end tie and bridge girder. There were both rotational and lateral slips of small magnitude.
7. The bottom deflection was greater than the top deflection due to the overhang of the bridge girder below the end trucks, which allowed it to swing easily.

8. Under vertical load, the box girder behaves as one would predict from the ordinary beam theory.

9. An $\frac{h}{t}$ ratio of 240 for the clear depth of the web seems to be about the upper limit of this value, if not a little too high, if the web is to act as is usually assumed in design.
Sketch of Crane #8441

Fig. 1
Note:
No gusset plates used between girder and end tie in this test.

LATERAL LOAD TEST CRANE #8441
MEASURED DEFLECTION OF GIRDER
March 30, 1940
Measuring Deflection

Note: Gusset Plates used between girder and end tie in this test.

LATERAL LOAD TEST ~ CRANE #8441
MEASURED DEFLECTION OF GIRDER

Fig. 3

April 1, 1940
Twist in Radians x 10^-5

Note: No Gusset Plates used in these tests between girder and end tie except as noted.

LATERAL LOAD TEST - CRANE #8441
MEASURED TWIST OF GIRDER

Fig. 4

March 30, 1940
Note—No gusset plates used between bridge girder and end tie in these tests except as noted.

LATERAL LOAD TEST ~ CRANE #8441
DEFLECTION OF BRIDGE GIRDERS

Mar. 30, 1940

FIG. 5
Note: No Gusset Plates used in this test between girder and end tie.
Note: No Gusset Plates used in this test between girder and end tie.

Lateral Load Test ~ Crane #8441
Top Coverplate Stresses

Fig. 7

Mar. 30, 1940
Note: No Gusset Plates used in this test between girder and end tie.

Lateral Load Test - Crane #8441
Average Web Stresses

Fig. 8

Mar. 30, 1940
Note: Gusset Plates used in this test between girder and end tie.

LATERAL LOAD TEST ~ CRANE #8441

Top COVERPLATE STRESSES

April 1, 1940

F15. 9
Load = 40,000#

Vertical Load Test - Crane #8441
Top Coverplate Stresses

Fig 10

April 1, 1940
Load $= 40,000\#$

Vertical Load Test - Crane #8441

Stress Distribution at % of Girder

Apr. 1, 1940

Fig. 11