FRITZ ENGINEERING LABORATORY
LEHIGH UNIVERSITY
BETHLEHEM, PENNSYLVANIA

Bruce 193.15

SUMMARY OF CONCLUSIONS OF CRANE GIRDER TESTS

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This summary presents briefly the conclusions of five research reports. For more detailed information the reader is referred to the reports which are as follows:

1. LATERAL LOAD TESTS ON BOX GIRDER MILL CRANES
April 30, 1941
2. LATERAL LOAD TEST ON A TRUSS I-BEAM CRANE
May 12, 1941
3. DYNAMIC TESTS ON MILL CRANES
June 27, 1941
4. TORSIONAL PROPERTIES OF FABRICATED I-BEAM AND BOX SECTIONS
June 16, 1941
5. BOX GIRDER BUCKLING TESTS July 3, 1941

a. Conclusions from Report 1 - Lateral load tests, in which the lateral load was applied to the top flange, showed that the whole section of a box girder acts to resist the lateral load. This section should be used in computing the stresses.

The amount of elastic twist and the torsional stresses can be computed very closely by the formulae of Bredt's Theory. However, in cranes with riveted end connections, there is an appreciable amount of twist caused by slip in the connection, which is additional to the elastic twist.

Partial end fixity was present in all the cranes tested. This fixity would become rather small as the end connection loosened in service. However, if the end connection were designed to take the lateral moment and were designed so that no slip would take place, the lateral moment could be reduced by a factor somewhere between 60 and 75 per cent.

The end tie should be designed to resist the lateral end moment, otherwise excessive bending stresses may occur in the end tie.

In some cranes the walkway acts with the girder and in other cranes it does not. It can not be assumed that the walkway will act with the girder unless it were specifically designed to do so, and this action should be checked by test.

The stresses in a crane under vertical load can be computed by the ordinary beam theory.

b. Conclusions from Report 2 - The lateral load test on the trussed I-beam crane showed the following results for this crane.

The whole truss resisted the lateral load as a space frame.

The whole I-beam girder acted as a part of the truss.

The top chord of the I-beam must be designed to transfer the applied load in bending to the panel points.

Secondary stresses were present throughout the truss.

In designing such a truss crane, the bottom chord of the truss should not be made too small, since this chord is almost as effective as the top chord in resisting twist and lateral deflection. Both top and bottom chords should be about equally as strong.

A truss crane, if properly designed, is as stiff and strong as a box crane.

Three approximate design methods were given in the report. The solution of the truss as a space frame is the only one which can be used to determine the stresses in all the members.

c. Conclusions from Report 3 - The following results were noted in the dynamic tests.

The maximum lateral force on a crane is due to braking and is approximately twenty per cent of the load on the braked wheels.

Some end fixity was present which reduced the stresses due to the lateral force. However, this fixity should not be counted on in design and the full value of ten per cent for the lateral load should be used.

Jerk impact, as measured, varied from 9 to 33 per cent.

The impact due to running the crane off wedges to simulate bad joints was quite large. For the tests made, impact values were measured which were 56 and 100 per cent of the live load or 31 and 51 per cent of all the loads acting in the impact. These impact tests were made by running both ends of one bridge girder off wedges at the same time. This condition is severe, since it is unlikely that bad rail joints would occur simultaneously at both ends of the bridge girder. Since this seems to be the impact factor which would govern in design, further tests should be made with the wedges under only one end of the bridge girder.

The closer the load is to the hoist drum, the greater will be the impact.

The maximum impact observed on the skullcracker crane was about 25 per cent. However, there was a great deal of vibration in this crane, probably due to the very high speed of the hoist, which caused constant vibratory stresses in the crane.

d. Conclusions from Report 4 - The following results were obtained in the torsion tests.

Fabricated I-beam and box sections do not behave the same as the equivalent solid sections due to the slip which occurs in the seams of such sections under torsional loading. This slip increases the twist and stresses of such sections markedly.

The torsional constant of the riveted built-up I and box sections tested in this program was about one-third that of the equivalent solid section.

If the angle of twist θ is computed using the reduced value of the torsion constant, shear stresses computed from this value will agree fairly well with actual values in I-beam sections.

Direct secondary stresses are large in a built-up I-section. Direct secondary stresses are small in a box section when stressed in the working range. These direct stresses vary with the square of the angle of twist.

Bredt's theory gives good approximate values for the shear stress in riveted box sections. In applying Bredt's theory to a fabricated box section, one must be careful to use the stress path. This will not be a rectangular section when the corners are formed by riveted angles.

Built-up welded I-beams fabricated with flange angles had a torsion constant two-thirds that of the equivalent solid section. Built-up welded box girders have a torsional constand close to that predicted by Bredt's theory.

The outstanding legs and parts, not included in the box, of a fabricated box section have little effect on the torsional properties of the section.

The limiting values for the torsion constant of a fabricated beam will be the value for the solid section and the sum of the values for the component parts. The actual value of the constant will be between the boundary values and will depend on the degree of slip.

e. <u>Conclusions from Report 5</u> - For welded box girders with l/b ratios up to 110, and riveted girders with l/b ratios up to 80, which were the limiting ratios in the test program; there is no tendency for lateral buckling to occur even when the girders were loaded with a lateral load of 10 per cent the vertical load. It is therefore not necessary to use a lower value for the compressive design stress than for the tension design stress. The critical buckling load for a load at the span center will be:

$$P = \frac{16.93\sqrt{B_1C}}{l^2}$$

where the notation is that given in the report.

There was a shift of the neutral axis towards the compression side of welded girders as the load was applied, apparently caused by welding stresses.

Internal weld stresses lowered the buckling strength of the cover plate in the w/t series. These stresses may be small in full size cranes.

The buckling stress of the cover plate of a box girder can be closely computed from the theoretical buckling formula which assumes simply supported edges for the plate. For steel this formula reduces to:

$$\sigma = 108{,}000{,}000 \left(\frac{t}{w}\right)^2$$

If welding stresses are present they should be subtracted from the stress given by the above formula to determine the allowable stress due to the load. If the buckling stress is above the elastic limit of the material, it should be reduced by an appropriate factor. If the w/t ratio does not exceed 39 for 36,000 lb per sq in. carbon steel, buckling will not occur before yielding of the plate.

The rail acts as a stiffener on the cover plate of a crane.

The allowable w/t ratio should be computed using the distance between the stiffeners when stiffeners are used. The stiffener must be rigid enough so that it will not buckle.

The alloy steel girder in the w/t series was somewhat stronger than the carbon steel girder because it did not yield locally as rapidly as the carbon steel girder.

Crane girders must have rails stiff enough to transfer the wheel load to the diaphragms without causing appreciable stresses in the cover plate. Otherwise the strength of the cover plate will be reduced.

The buckling stress of carbon steel webs subjected to pure bending can be determined from the theoretical buckling formula using simply supported edges. This formula is:

$$\sigma = 648{,}000{,}000 \left(\frac{t}{h}\right)^2$$

When the computed buckling stresses are above the proportional limit, they should be reduced by an appropriate factor.

If the h/t ratio does not exceed 134 for 36,000 lb per sq in. carbon steel, buckling will not occur before yielding of the plate. The stress to be used in computations is the maximum stress in the unsupported edge of the web plate.

When compressive stresses due to lateral load are present in the crane, the above h/t ratios for pure bending should be reduced.

Longitudinal stiffeners are very effective in preventing web buckling. They should be used when high h/t ratios are used.

Vertical stiffeners are of little use in preventing web buckling due to bending.

In choosing a factor of safety to be used in computing the allowable plate thickness ratios, consideration should be given to the large reserve strength in box girders after the plates buckle.

The stresses in the diaphragms of box girders under bending loads are very small unless buckling occurs. The diaphragms then act to reduce the buckling.

The stresses in the diaphragms of riveted girders are appreciable under torsion loads. The stresses in the diaphragms of welded girders under torsion loads are negligible.