ANALYSIS OF SPECIFICATIONS FOR THE  
STRUCTURAL DESIGN OF MILL CRANES  

PROGRESS REPORT OF CRANE GIRDER RESEARCH  
AT LEHIGH UNIVERSITY  

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

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March 18, 1940
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ANALYSIS OF SPECIFICATIONS FOR THE
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I. SYNOPSIS

This is a report outlining the results of a study of
a number of crane design specifications in common use. The
study of the specifications has been approached from three
angles: (1) the theoretical basis for the specifications,
(2) the experimental basis for the specifications, and (3)
an outline of additional tests needed to confirm any impor-
tant assumptions.

The study has shown that some of the specifications
differ radically from each other, so that cranes designed on
the basis of one specification would have an altogether dif-
f erent rating on another specification. Some specifications
were found to omit certain forces acting on the structure.
That this was done was recognized in some cases by using a
lower design stress. Such a procedure would normally give a
girder which was amply safe, but perhaps not as economical as
it might have been.
The specifications have been studied with an eye toward economics wherever possible. It appears that in some of the cases studied, advantage of all the economies has not been taken. This has been due to the fact that in a great many cases, specifications have been based on old bridge specifications for I-beam girders; and these are not always applicable to the very common box girder which is so largely used in crane construction.

Several pilot tests were made on small box girders of both the bolted and welded type to compare them with the results of Arendt's theory on such girders. The spacing and manner of holding these diaphragms was varied to see how they affect the torsional constant.

Another test was made on a long, narrow, box girder to see if the usual compressive design stresses apply to the top flange of box girders.

This study has been limited to the design of the structural details of the bridge girders; and no attempt has been made to go into the details of the machinery, etc.

II. DISCUSSION OF SPECIFICATIONS

1. Criteria - Topics covering all important details of girder design will be taken up as they occur in a typical crane specification. The analysis of the specification clauses has been based on a study of their relation to theory, experimental tests, and actual practice.
Many details which are used have no basis for their design, but have been developed in practice. In most cases they do their job, and in many cases refinements in their design would not be warranted by any savings likely to ensue. In some cases, worthwhile economies are probable.

2. **Loads on the Girder** - The maximum concentrated, vertical load on a girder requires no comment. It includes the maximum load to be lifted, plus the weight of the loading gear, plus the weight of the trolley.

The stresses due to the dead load of the girder and its rigging can safely be assumed to be a uniform load along the girder.

The lateral forces assumed to act on the girder vary greatly in the various specifications. Theoretically, no matter how fast the crane is running, the maximum decelerating force occurs when the wheels are on the point of slipping. The accelerating force depends on the crane load and horsepower of the bridge motor. It is usually less than the braking force and can never exceed it since the wheels would slip.

The braking force depends on the friction between the runway rail and the bridge wheels, and upon the number of braked wheels. The coefficient of friction will vary from 0.10 - 0.25 depending on rail conditions. In some tests that were made on the crane at Frits Laboratory, the maximum friction measured was 0.25 but more frequently it was slightly
over 0.15. More lateral load tests on cranes in mills should be made as a check on these results.

A value of 0.25 is probably too high since it is rather unlikely that an operator would slide the wheels at a spot where the friction was greatest while carrying maximum load. The German Specifications call for a lateral load of one-seventh of the load on the braked wheels which corresponds to a friction factor $u$ of 0.145. The British Specifications call for one-twelfth the total load which corresponds to a $u$ of 0.166 if one-half of the wheels are braked as is usually the case. A friction factor of 0.2 should be amply conservative.

The lateral force due to the dead load can be assumed to act through the center of gravity of the section. The lateral force due to the live load and trolley acts in the plane of the top of the bridge rail.

There is very little uniformity in the specifications regarding the portion of the girder which is assumed to resist the lateral forces. Theoretically, if the girder has sufficient diaphragms the whole section can be assumed to act. The present mill tests should provide the answer to this. Specifications which require the top flange to resist the lateral load are thus probably necessary severe for box girders, although they may not be too far off for I-beam girders. The reason for the difference, is that the box girder is so stiff in twisting, that the top flange cannot deflect under lateral load without
forcing the whole box to go with it. The I-beam, on the other hand, twists easily under small forces, since the web is relatively flexible, so that often the top flange has taken practically all the load before any part of it is transferred to the bottom flange.

Another action of the girder under lateral load which is not usually taken into account is the end fixity which may be present. This factor depends on the type of end tie which is used. If fixity is present, it may reduce the lateral load center moment as much as fifty per cent. This is another factor upon which our mill tests may throw some light. The lateral load is one of the topics about which little is known, and which, nevertheless, is very important since the stresses due to the lateral forces are in some cases thirty to forty per cent of the total used in design.

The lateral force applied to the top of rail also causes a large torque on the section. For continuously welded box girders, the twist due to this torque can be accurately predicted by Brødt's theory. Flange angles or outstanding legs of plates have little effect on the behavior of the section. Brødt's theory breaks down for riveted girders, since it does not take into account the slipping which occurs along the seams of riveted girders. Nor have any tests been found in which this effect has been measured.
As a part of the present study a number of pilot tests were made in torsion on small 2 by 3 \(\frac{\pi}{4}\) by \(\frac{1}{8}\)-in. welded and bolted box sections. The bolted girder was made to simulate the action of a riveted girder, by drilling the holes so that the bolts would fit tightly, and by keeping the bolts tight. Tests were made on the bolted girder with the diaphragms fastened on one, two, and three sides with diaphragm spacings from three to forty-eight inches. The diaphragms were found to have two effects, they kept the girder in shape and also prevented relative slipping of the sides of the box. For the case where the diaphragm spacing was equal to the depth of the girder, the bolted girder was one-twentieth as stiff in torsion as the welded girder, and as the diaphragm spacing was increased, it dropped off until it was about one-hundredth of that of the welded girder. The riveted girder should behave better than the bolted, but the pilot test shows definitely the importance of the effect of slipping in the seams of a box girder.

The shearing stresses in the girder were about two and one-half times those in the welded girder, due to the slipping in the seams. The stresses in the welded girder were about forty per cent greater than that predicted by the theory. In the welded girder, diaphragms and flange angles which were welded to some of the boxes had little effect on the stiffness of the box.
3. **Impact** - Nothing either theoretical or experimental could be found on stresses in cranes due to impact. All specifications differed markedly. Some based impact on the size of the crane, some on the speed of hoist, and some merely on service conditions. A few tests have been made which measured the deflection of aluminum cranes due to impact loads, and the largest measured was forty-four per cent of the static load deflection. In a steel crane, where the modulus of elasticity is higher, the impact also is higher.

There are two kinds of impact, one which occurs under normal working conditions, and the other which is due to abnormal conditions such as hitting another crane or running over an excessively low joint. Some information on the first type of impact might be obtained by tests of cranes in mills, using scratch gages, and by observing impact factors on cranes equipped with scales.

4. **Other Loads** - In addition to the torque produced by the lateral forces, a torque is set up by the starting and stopping of the bridge drive motor. Other torsional moments are due to the overhanging loads on the girder such as the footwalk, etc.

For outdoor cranes, a wind load of thirty pounds as usually specified per square foot of projected area should be ample for any winds likely to occur.
5. Load Combinations - It is extremely unlikely that all of the maximum loads will act simultaneously upon the girder and only judgment can decide what combination to use for design. Most specifications require the calculation of stresses to be based on the following conditions.

1. Direct Stress (A or B, whichever is greater).

   A. Sum of stresses due to dead load, trolley, lifted load and impact.
   
   B. Sum of stresses due to dead load, trolley, lifted load and lateral forces.

2. Shear Stresses.

   Sum of stresses due to vertical load, lateral load, and the algebraic sum of the torques acting at any section.


   When secondary stresses are computed and added to the primary stresses, the allowable design stress may be increased by twenty-five per cent.

Both load combinations A and B should be considered in calculating the direct stresses. Some crane designs, where impact is important, will be governed by combination A. However, in most designs, combination B will generally be the governing factor. It is rather unlikely that an operator will handle his crane in such a way that when he is lifting his maximum load, he will have his maximum lateral force. Yet, there is always the possibility that this may occur and it should be considered in the design.
It is very improbable that impact and lateral load will occur at the same time, since the bridge would not be in motion when maximum impact was taking place. One exception to this statement is possible, and impact as well as lateral load would occur when the crane is being braked while going over a bad runway joint. However, this is a condition which could be more economically taken care of by fixing the joint, rather than by making a crane heavy enough to stand these impact stresses, which are of a high order of magnitude.

A higher allowable stress should be used when the secondary stresses are computed, since if the crane is made of a ductile material, any local yielding due to secondary stresses will not reduce the strength of the structure. Such yielding would, of course, produce permanent deformations.

For our purposes, we can define secondary stresses, as those caused by distortions of the members rather than by applied forces. As such, these stresses have little effect upon the load carrying capacity. The most important ones in crane design are the secondary stresses which occur due to torsion at the center of the bridge girder. This is probably not so very important in well-designed box girders. However, in I-beam cranes, the stresses may be quite serious, particularly if the rivets have loosened, and large permanent twists will be left in such a girder.
6. **Reversal of Stress** - The usual method of designing bridges for stress reversal is to increase both the compressive and tensile stress by fifty per cent of the smaller stress and design the member so it will be capable of resisting either increased resultant stress. Connections are proportioned for the sum of the increased stresses.

Tests made at the University of Illinois show that the fatigue limit of mill plate with holes is as low at 26,000 p.s.i.* for carbon steel at 2,000,000 cycles of stress. This is for a stress cycle varying from zero to the maximum tension. The clause seems to be inadequate if these test results are considered. However, the specification is one on which a large number of bridges has been built. No doubt this is because it is rather unlikely that a member will get this many cycles of maximum stress in its lifetime.

Recently, more scientific ways have been presented of computing the fatigue design stress. However, until the designer knows how many cycles of maximum stress his structure will receive, complicated methods of estimating the fatigue stress are not warranted. It is a subject which deserves further study.

* This abbreviation will be used for pounds per square inch
The clause on connections is very conservative in order to insure against exceeding the friction grip of the rivets in a joint. This generally occurs in a riveted joint slightly below usual working stresses, and if exceeded, the play in the joint will cause the rivets to rapidly loosen.

7. Allowable Stresses - The allowable stress to use in design is chiefly a matter of judgment and depends on many factors. If all loads can be closely predicted, and the stresses therefrom accurately calculated, a much higher design stress may be used than if the opposite is the case. Throughout the years the trend has been to raise the allowable design stresses, as knowledge of the behavior of material, and of structural analysis has been increased. Recently the A.I.S.C. raised their design stress in tension to 20,000 p.s.i. with a resultant economy in design, whereas but a few years ago they were using 16,000 p.s.i.

In most structures the ultimate strength of the material has little significance. Failure of a structure is usually considered as the point where excessive deformation occurs. This point usually depends upon the yield point of the material. For compressive stresses, failure occurs when the member buckles. In neither case has an actual breakage occurred, so the ultimate strength has little effect on the allowable load. The same thing is true in laboratory tests where in many cases the specimen cannot be broken although it may be twisted all out of shape.
The buckling point depends on the shape of the member, on the stress, and the modulus of elasticity. For this reason, members made of high tensile steel will hold no more load than one of ordinary carbon steel if buckling occurs in the elastic range. If buckling occurs in the plastic range, the member with the higher yield point will carry the greater load.

Many of the design stress values given in the various crane specifications are similar to those in the A.S.A.A. Bridge Specifications and represent extensive judgment and experience.

However, a few comments on some of the values are appropriate.

First, the value of the allowable compressive stress in a box girder seems much too low. A common clause states that:

\[ f_0 = 15,000 - 0.33 \left( \frac{f}{f'} \right)^2 \]

This reduces the stress to take into account the possibility of the girder buckling sideways. However, test results of box girders in bending show no failures due to lateral buckling of the flange. Consideration of what happens theoretically also bears this out. As the beam tends to twist sideways, the torsional rigidity of the box prevents the rotation, and since a box girder is so efficient in torsion, the beam cannot buckle laterally. Thus, the allowable compression stress for a box girder should not depend to such a high degree on the length-width ratio. If the girder is designed so that local crippling
will not occur, then the allowable stress in compression could be almost the same as that in tension. At the present time, the difference in allowable stress between the tension and compression sides encourages the use of unsymmetrical sections which are more inefficient than symmetrical sections.

In order to check this fact, a test was made on a long narrow box girder of 1/8-in. material; the girder was 18 ft. long, 10 in. deep and the flange was 3 in. wide. For a girder of these proportions the allowable design stress by the preceding formula is equal to zero. The formula, of course, is not meant to apply in this range; but it does indicate that the girder should fail at a relatively low load.

There was some difficulty in welding the small light girder due to distortion of the thin section, and when it was finished it had a twist in it, which caused the center to be about 3/8-in. out of line with the ends. This resulted in some eccentricity when the load was applied.

The load was applied at the center of the girder by a beam fastened to the center diaphragm with a bolt and passing through holes in the web. This manner of loading allowed the girder to deflect sideways and also to rotate. Thus no factors were present which would prevent buckling. The beam failed at a load of 4000 lb. which corresponded to a maximum computed stress of 35,000 p.s.i. The highest stress readings were taken at a load of 4500 lb., and the maximum measured stresses at this
point were about 36,000 p.s.i. This is a little less than the
yield point of the web material which was about 37,000 p.s.i.
The yield point of the flange material was 44,000 p.s.i.

The girder failed initially when the web started to
buckle. When this occurred the flange buckled between two
intermittent welds. There was no lateral buckling whatsoever.

This pilot test is considered significant in that it
indicates that something is radically wrong with the allowable
compressive design stress in box girders. Neglect of torsional
stiffness is the reason for the difference. Proper account
of the torsional action which prevents the lateral buckling of
such girders can undoubtedly result in very appreciable econo-
 mies in design.

The above test results, however, would not affect I-beam
design appreciably since I-beams have comparatively little tor-
sional strength.

The other common way for a girder to fail is by web
buckling. In the usual designs, this is prevented by adding
stiffeners to the web.

There are two types of web buckling, or rather buckling
may be caused by two factors. In regions of heavy shear, shear
buckling may occur; or, as it is sometimes considered, diagonal
compression buckling. Also where large compressive stresses
are present, compression buckling of the web may occur. The
latter is somewhat similar to column buckling except that the stress varies over the web of the girder instead of having a constant value as it has in a column.

When shear buckling occurs, comparatively long waves appear in the web. Vertical stiffeners shorten the length of these waves and increase the buckling load, thereby increasing the allowable shear stress.

As a rule, after a web buckle, it will still hold its load so that web buckling does not necessarily mean immediate failure of the girder. Thus, usually a lower factor of safety can be used.

Compression buckling forms relatively short waves in the web. Hence, vertical stiffeners have little effect in preventing buckling. Horizontal stiffeners, on the other hand, would be very efficient, but are seldom used. Thus specifications usually keep the \( \frac{d}{t} \) ratio down to a point where compression buckling cannot occur.

Most of the theory on elastic stability or buckling was originally presented by Timoshenko, but in many cases it is presented in such a manner that it is inconvenient for the use of the designer. A recent paper by Neissciff and Lionhard in the January 1940 Proceedings of the A.S.C.E., has revised and extended Timoshenko's work so that it may be easily used by the practical designer. It is also very complete.
In a great many specifications, a clause is given which states that stiffeners are required when the unit shear exceeds
\[
\frac{18,000}{1 + \frac{1}{7200} \left(\frac{h}{t}\right)^2}
\]
and also if \( \frac{h}{t} \) is over 60.

The first part of this clause is not necessary, because for values of \( \frac{h}{t} \) less than 60 it gives shearing stress values above the allowable of 12,000 p.s.i. Theoretically, for a shear stress of 12,000 p.s.i., stiffeners should not be required until the \( \frac{h}{t} \) ratio is over 60, and tests have borne this out. However, the theory assumes ideal conditions with no irregularities in the plates, so the allowable value of \( \frac{h}{t} \) has been reduced by specification writers to 60 to allow for the irregularities caused by shop fabrication.

It should be noted, however, that the above minimum \( \frac{h}{t} \) ratio for required stiffeners is based on the supposition that the web is stressed to the allowable shear stress of 12,000 p.s.i. In many girders, and particularly double-web girders, minimum plate thickness requirements result in designs where the shear stress is far below the allowable. In such cases, higher ratios for \( \frac{h}{t} \) should be considered.
Theoretically, stiffeners are required when:

\[
\frac{h}{t} \text{ is greater than } \frac{6600}{\sqrt{v}}
\]

If this is reduced to allow for irregularities in the same ratio as was done before to make \( \frac{h}{t} = 60 \), the following equation for required stiffeners results. Stiffeners are required when:

\[
\frac{h}{t} \text{ is greater than } \frac{6600}{\sqrt{v}}
\]

The above equation will also give stiffener requirements for high tensile steels. If a steel with an allowable shear stress of 18,000 p.s.i. were used it will give a \( \frac{h}{t} \) ratio of 43.8. A common requirement in this case is that stiffeners are required when:

\[
\frac{h}{t} \text{ is greater than } 49.
\]

The common formula for stiffener spacing:

\[
l = 255,000t \cdot \sqrt{\frac{v}{h}}
\]

was derived by Noyney of the American Bridge Company. It was based on Timoshenko's theory and has some minor approximations. It is a little inconvenient to use, and Neissciff and Lienhard suggest for steel the formula:

\[
l = 10,500t \cdot \sqrt{\frac{v}{h}}
\]
This will give a somewhat smaller stiffener spacing than Hovey's formula, but the results agree very well in most cases. Both formulae are based on the same theory, but the approach in working out the values of the constants have been somewhat different.

Since vertical stiffeners are of very little use in preventing compression web buckling unless spaced at small intervals less than $2/3$ the clear depth, it follows that the $h/t$ ratio of the web must be kept below the safe value of an unstiffened girder. The usual value given in most specifications is 170. This value is based on the assumption that the maximum compressive stress in the web at the toe of the flange angle equals 0.65 the maximum allowable design stress, or 11,700 p.s.i. for 18,000 p.s.i. design stress. Thus this factor would not apply to a welded girder since the maximum stress in the clear depth of the web is practically equal to the maximum design stress of 18,000 p.s.i.

It is suggested that this ratio be limited by the expression:

$$\frac{h}{t} \text{ is less than } 170 \sqrt{\frac{18,000}{\text{maximum compression stress in clear portion of web}}}$$

This expression again will allow economies in design by allowing higher $h/t$ ratios when the girder is not fully stressed.
It also prevents the use of too high an \( \frac{h}{t} \) value in sections where the usual requirement would not apply. It is based on a safety factor of 1.83 which is the ratio of the yield point of 35,000 p.s.i. to the design stress of 19,000 p.s.i.

Some girders have been designed with \( \frac{h}{t} \) ratios higher than 170. These girders, however, have usually had their allowable compressive design stresses reduced to values often as low as 12,000 p.s.i. to provide for lateral buckling of the compression flange. When this has been the case, a high \( \frac{h}{t} \) ratio provides as much margin of safety as a lower one with a higher design stress.

The economical design of high tensile girders is handicapped by the limiting values of \( \frac{h}{t} \) given by most specifications. These values can be raised a good deal by proper use of horizontal stiffeners. Only when the \( \frac{h}{t} \) values are thus safely raised can the maximum economy and reduction in weight be attained with the use of high tensile steels.

8. Design Details — The usual clause for maximum rivet spacing is to require sixteen times the thickness of the minimum section. This is based on some old tests and works well in practice. The theoretical value is 26, but this does not take
into account the irregularities which occur in practice. The
design values do not seem too conservative when the irregu-
larities are taken into account.

The ratio of the width of an unstiffened coverplate of
a box girder to its thickness should not exceed 36. This value
should be used to prevent local crippling of the plate. The
theoretical value from the elastic theory is 46. However, this
value is reduced since the proportional limit of carbon steel
is assumed to be 29,600 p.s.i. and this reduces the buckling
load. For the same design stress, the silicon steel would have
the \( \frac{b}{t} \) ratio of 46 since its proportional limit is much higher.
This topic is one on which the proposed tests on box girders
should provide information.

Actually, the trolley rail will act to a large extent
as a stiffener, but it also transfers stress to the plate from
the wheel loads, so this effect may be discounted somewhat. If
a higher \( \frac{b}{t} \) ratio is desired, provision should be made for the
use of horizontal stiffeners.

In girders where the flange is not fully stressed, the
allowable width-thickness ratio may be multiplied by the factor

\[
\sqrt{\frac{18,000}{\text{actual stress in member}}}
\]

This may give a more economical section when the cover
plate is not fully stressed.
The width of the outstanding legs in compression (except where reinforced by plates) shall not exceed twelve times the thickness for girder flanges and sixteen times the thickness for other members. Here again the theoretical value is 12 when the proportional limit is considered. For a higher proportional limit, the value is 10. The value of 12 in girders is raised to 16 for other members due to the greater edge fixity usually present in other members.

A common clause in structural specifications states that web splices should be made equal to the strength of the web. Even though splices are designed to be as strong as the member, tests show that this is not the case. It is recommended that web splices be placed at sections where the stress is not a maximum.

The clause that plate girders should be proportioned by the moment of inertia of their net section is sometimes taken to mean a girder with the rivet holes out of both flanges, and sometimes a girder with the rivet holes out of the tension flange. Recent tests show that the neutral axis remains substantially at the center of the beam for symmetrical beams, and that the average stress on the net section should be used. It is recommended that girders be designed on their net section, deducting holes from both the tension and compression flanges. When computing deflections, however, the gross moment of inertia should be used.
The design of the diaphragms is a subject on which there seems to be no data, and on which considerable experimental work should be done. Where stiffeners are required, diaphragms can be substituted instead, and they will act both as a diaphragm and stiffener.

Diaphragms probably act in three ways.

First, they distribute lateral and vertical load throughout the whole girder section. This is very important in transferring lateral loads from the compression flange to the whole girder. If there were no diaphragms, a large part of the lateral load would have to be taken by the compression flange. For this purpose, diaphragms should be heavy enough to transfer the load without shear buckling.

Second, diaphragms maintain the shape of the section.

Third, diaphragms prevent slipping along the joints when torsion acts on the girder. This is a very important function of the diaphragm, and the more diaphragms present the stiffer will be the girder.

Enough short diaphragms should be installed to prevent the stress in the trolley rail from exceeding the allowable fibre stress.

It is recommended that the girders shall be rigidly attached to the end trucks by adequate end ties. If this is not done, the torsional vibration which is set up by the accelerating and braking forces, will eventually loosen the girder. Even
when the girder is not loaded, this vibration can be serious; particularly in deep fish-belly girders where the center of gravity of the girder is lower than the supports on the end trucks.

Some specifications allow the trolley rail to be offset to compensate for the torque caused by the weight of the overhanging parts such as the footwalk, etc. Care should be used in doing this, since this dead load torque varies from zero at the center, to a maximum at the end. To balance the dead load torque the trolley rail would have to be on line at the center and offset at the end, which would be difficult to accomplish.

III. SUMMARY

The studies made regarding the background of the specifications show that most of the clauses in the specifications are based on good theory and practice.

However, there are some topics which need further investigation, and they are summarized as follows:

1. Impact. Very little is known about either the value of the amount of impact in a crane or what stresses the impact causes. They are doubtless very important, but whether the present values used agree with reality seems to be unknown.

2. Allowable Design Stress of Top Flange of Girders. The present design values for the compression stress seem to be too low. A pilot test which has been made bore out this assumption. Tests should be made to form a basis for selecting design
stresses, and economy should result from more efficient design. Tests should also be made to determine how much the allowable height-thickness and width-thickness ratios are affected by horizontal stiffeners. This would be important when high tensile steel is used.

3. Torsional Stiffness of Riveted Girders. Slipping in the rivets of a girder greatly reduces its torsional stiffness. This might be serious in an I-beam girder. Knowledge of the torsional constant of the box girder is also necessary in any prediction of the buckling stress of the box girder. Tests are necessary to determine how the torsional constant of built-up box and I-beam girders are affected by rivet slip in the joints.

4. Diaphragms. Little seems to be known about the action of diaphragms or how to design them. At present, they are designed by rule of thumb. Tests should be made in which the stresses in diaphragms are measured in beams subjected to bending and torsion. The results of these measurements should give some information on how to approach the design problem.

5. Lateral Load Tests. Additional tests are necessary on cranes in mills to determine the lateral load and the lateral stresses on cranes in service. Also more tests should be made on cranes in the fabrication shops to determine the distribution of the lateral loads, and the stresses caused by these loads.