Experiments on Box Type Girders*

By Motoharu Taneda**

In order to study the strength and the rigidity etc. of box type girders for overhead travelling cranes, full-sized model experiments were conducted. Two model girders were used. The upper deck and the two web plates of one model (Girder 2) were reinforced by longitudinal stiffeners, while those of the other (Girder 1) were not. Load was applied on top of the inner web plate at the center of the span of each girder, which was supported at its ends.

Results are as follows:

(1) The stress distributions on the plate surfaces were generally irregular, while deflections, buckling loads, failure loads of girders and shapes of plates after failures were coincident with their calculated values.

(2) The effects of longitudinal stiffeners on the strength of Girder 2 appeared clearly.

(3) The torsion produced by the eccentric load has not a remarkably bad effect on a box type girder of such dimension ratio as used in the present experiments.

1. Introduction

Among the so-called box girder type overhead travelling cranes, which make use of a box type girder for the main girder and have no auxiliary girder, there are two types, A and B, as shown in Fig. 1. In the type A, the traversing rail is located in the center of the upper deck plate, while in the type B, it is located on top of the inner web plate. In the type B, there is the defect that the load is applied on the girder eccentrically, thereby causing torsion in the girder. On the other hand, however, the type B has the following advantages: Unlike the type A, there is no need to insert reinforcements in the bottom part of the rails and it is easy to arrange the hoisting equipment. Practical use of the type B, however, is much less common than the type A.

Full-sized model experiments were carried out to study the load carrying capacity, rigidity and failure state etc. of the type B, and the results
Table 1 Specifications of test girders

<table>
<thead>
<tr>
<th>Span L</th>
<th>Depth of girder D</th>
<th>Width of deck plate B₁</th>
<th>Space of web plates B₂</th>
<th>Space of diaphragms S</th>
<th>Thickness of plates t</th>
<th>Dimensions of horizontal stiffener</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder 1</td>
<td>10000</td>
<td>600</td>
<td>400</td>
<td>200</td>
<td>1000</td>
<td>4.5</td>
</tr>
<tr>
<td>Girder 2</td>
<td>10000</td>
<td>600</td>
<td>400</td>
<td>300</td>
<td>1000</td>
<td>4.5</td>
</tr>
</tbody>
</table>

Table 2 Results of material tests

<table>
<thead>
<tr>
<th>Elastic limit</th>
<th>Limit of proportionality</th>
<th>Yield point</th>
<th>Tensile strength</th>
<th>Young's modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>21.7</td>
<td>19.8</td>
<td>31.5</td>
<td>41.9</td>
<td>20800</td>
</tr>
</tbody>
</table>

The elastic limit is the stress of test piece at which its gauge length has the permanent elongation 0.005 per cent

![Fig. 1 Types of box girder crane](image)

Fig. 1 Types of box girder crane

![Fig. 3 Loading apparatus](image)

Fig. 3 Loading apparatus

![Fig. 4 Section in the center of the span of girder](image)

Fig. 4 Section in the center of the span of girder

These girders were made by welding. The number of diaphragms of each girder is 11. Fig. 2 Principal construction of test girders are given below.

2. Method of research

2.1 Test girders

Two girders, Girder 1 and Girder 2, were used in the experiments. The principal construction and specifications of these girders are shown in Fig. 2 and Table 1, respectively. As shown in Fig. 2, Girder 2 differs from Girder 1 only in that longitudinal stiffeners have been inserted in the center of the upper deck plate and web plates. Other construction features of Girder 2 are exactly the same as those of Girder 1. Table 2 gives the mean values obtained from material tests on the component materials of the girders.

2.2 Loading apparatus

An outline of the loading apparatus is shown in Fig. 3. As shown in this figure, the girder was supported by saddles at both ends and a concentrated load was applied on top of the inner web plate at the center of the span. The jacks shown in the figure were raised or lowered in order to continuously change the load.

2.3 Method of measurement

The stress on the outer surface of the girder was measured mostly at the center of the span by means of an electric resistance wire strain meter. Piano wires were extended over the whole length of the span, in order to measure the deflection of the girder when a load is applied. Also, a measurement was made on the unevenness of the
plate surface around the center of the span, using a depth micrometer and straight edges.

2-4 Assumptions in theoretical calculations

The various theoretically calculated values listed below are all based on the assumption that the girders are simply supported at both ends. The effect of the shear lag on the stress distribution of the girder was not taken into consideration. Also, the following was considered: As shown in Fig. 4, the concentrated load $P$ which was applied on top of the inner web plate was divided into the force $P'$ (magnitude same as $P$) which was applied to the line passing through the centroid of the girder section and the eccentric moment $P''B_0/2$ produced by $P''$ (magnitude same as $P$) and $P$. Then, $P'$ and $P''B_0/2$ produced bending and torsion in the girder, respectively.

3. Results of experiments and discussions

3-1 Stress distribution over plate surface

Because of the constructions of the test girders, it was possible to measure the stress distribution only on the outer surface of the plate. Several examples of the distribution of longitudinal normal stress $\sigma_x$ are shown in Fig. 5. As seen in this figure, the experimental value $\sigma_x$ generally shows irregular distribution. Also, the trend of this distribution does not show too much change with the increase in load. From the facts that there is a tendency for the experimental value to be closer to the calculated value in the parts closer to the boundary of the plate and that the irregularities in the web plates are great in the compression side as compared with the tension side, it is presumed that the irregularity of stress distribution is a manifestation on the surface of the plate of the secondary stress due to local unevenness of the plate.

3-2 Failure of girders

3-2-1 Phenomena

Fig. 6 shows the relation between the deflection $\delta$ of the upper deck plate at the center of the span and the load $P$. As one example, this figure shows the deflection at the outermost end of the deck plate. The dotted lines in the figure show the paths of unloadings in the experiment.

The notations $\delta_0$, $P_0$, $P_a$, and $P_b$ in Fig. 6 all represent calculated values and are as follows:

$\delta_0$: Theoretical value of elastic deflection

$P_0$: Load when the longitudinal normal stress of the upper deck plate at the center of the span reaches the yield point

$P_a$: Load when the longitudinal normal stresses

Computed: $\delta_0$ -- $P_0 = 11\%$

Computed: $P_0 = 13\%$

Computed: $P_a = 13\%$

Fig. 5-1 Distribution of normal stress $\sigma_x$ (Girder 1)

$\delta_0$ $\sigma_x$ kg/cm²

Upper

2000 1000 0 -1000 -2000 -3000

Lower

2000 1000 0 -1000 -2000 -3000

Computed: $\delta_0$ -- $P_0 = 11\%$

Computed: $P_0 = 13\%$

Computed: $P_a = 13\%$

Fig. 5-1 Distribution of normal stress $\sigma_x$ (Girder 1)

$\delta_0$ $\sigma_x$ kg/cm²

Upper

2000 1000 0 -1000 -2000 -3000

Lower

2000 1000 0 -1000 -2000 -3000

Computed: $\delta_0$ -- $P_0 = 11\%$

Computed: $P_0 = 13\%$

Computed: $P_a = 13\%$

Fig. 5-2 Distribution of normal stress $\sigma_x$ (Girder 2)
of the various parts of the center section of the span are considered to reach the yield point uniformly

In other words:

$$
\delta_v = \frac{PL^2}{48EI} + \frac{PB_1L(B_2 + D)}{32B_2DHG} (\text{by the membrane analogy})
$$

- : At innermost end of deck plate
$$P_a = \frac{4Z}{L} \sigma_y \frac{W}{2}, \quad P_b = \frac{4H}{L} \sigma_y \frac{W}{2}$$

where

- $E$: Modulus of longitudinal elasticity
- $G$: Modulus of transverse elasticity (taken as 810 000 kg/cm²)
- $\sigma_y$: Yield point
- $W$: Own weight of girder (Girder 1: 902 kg, Girder 2: 993 kg)
- $I$: Geometrical moment of inertia of girder
- $Z$: Modulus of section of girder
- $H$: Plastic modulus of section of girder

$P_a$ is the estimated buckling load of the girder and was calculated on the basis of the following thinking: If the parts of the girder which are separated by the diaphragms, horizontal stiffener, and web plate (or deck plate) of the girder are considered as separate rectangular plates with simply supported edges and if it is considered that these plates will not yield under loading, according to calculations some of these plates will buckle first when the concentrated load $P$ is gradually increased. The concentrated load $P$ at this time was taken as the estimated buckling load $P_b$.

According to calculations, the place which will buckle at first is the upper deck plate (buckling due to simple compression) in the center of the span of Girder 1, while in Girder 2, it is the compression side of the web plates (buckling due to bending) in the center of the span.

In calculating the buckling stresses of the various plates, the shearing stresses were ignored because it was much smaller than the normal stresses.

In both Girder 1 and Girder 2, the estimated buckling loads $P_a$ were smaller than the calculated values $P_b$ of the yield loads.

As shown in Fig. 6 (a), in Girder 2 the deflection $\delta$ increased coinciding very well with the theoretical value of the elastic deflection $\delta_e$ up to a load of 15 tons, but from a load of about 18 tons which was less than $P_a$, the permanent deformation began to increase. The maximum load was 22 tons ($A_2$ point), and at this point an unevenness which could be seen with the naked eye appeared for the first time on the upper deck plate and the compression sides of inner and outer web plates in the center part of the girder. Even when the load was returned to zero from this point, the unevenness remained nearly unchanged. After $A_2$ point, the load which could be applied tended to decrease as the deflection increased and the unevenness on the various surfaces of the compression side of the central part showed outstanding increases.

The positions on the surfaces where the unevenness appeared under the $A_2$ point and $B_2$ point are shown in Table 3. The deformation of the inner web plate was slightly larger than the deformation of the outer web plate, but in

![Fig. 6 Deflection of the upper deck plate at the span center (at outermost end of the deck plate)](image)

![Fig. 7 Deformation in the vertical section of girder](image)
Table 3: Positions where the visible unevenness appeared on the plates in Girder 2

<table>
<thead>
<tr>
<th>Web plate</th>
<th>In direction of span</th>
<th>In direction perpendicular to span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inner</td>
<td>Span 3.0 m~5.5 m</td>
<td>Compression side (Half above stiffener)</td>
</tr>
<tr>
<td>Outer</td>
<td>Span 3.5 m~5.5 m</td>
<td>Same described above</td>
</tr>
<tr>
<td>Deck plate</td>
<td>Upper</td>
<td>Span 4.0 m~5.5 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flanges</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lower</td>
</tr>
</tbody>
</table>

Table 4: Positions where the visible unevenness appeared on the plates in Girder 1

<table>
<thead>
<tr>
<th>Web plate</th>
<th>In direction of span</th>
<th>In direction perpendicular to span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inner</td>
<td>Span 0.5 m~9.0 m</td>
<td>Compression side and a part of tension side</td>
</tr>
<tr>
<td>Outer</td>
<td>Span 0.5 m~9.5 m</td>
<td>Same described above</td>
</tr>
<tr>
<td>Deck plate</td>
<td>Upper</td>
<td>Span 1.0 m~8.5 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Whole</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lower</td>
</tr>
</tbody>
</table>

view of a vertical section, as shown in Fig. 7, the tendency for unevenness to appear was approximately symmetric to the y-y axis.

As shown in Fig. 6 (b), Girder 1 followed approximately the same process as Girder 2, but Girder 1 differed particularly from Girder 2 on the following points:

1. Prior to reaching the maximum load of 16 tons (A1 point), a large unevenness suddenly appeared in the center of the upper deck plate at a load of only 14 tons.

2. As shown in Table 4, at A1 point the unevenness spread through the whole length of the span (after the load was removed, however, the unevenness nearly disappeared with the exception of the center part of the span).

3. The unevenness of the web plates advanced to the tension side.

3.2.2 Discussions As can be seen from Fig. 6, the load carrying capacity 22 tons of Girder 2 coincides well with the estimated buckling load \( P_b \) (21.4 tons), while the load carrying capacity 16 tons of Girder 1 is much higher than its \( P_b \) (11.6 tons).

It was not possible to get exactly the load when the buckling on the plate surface of Girder 1 occurred. However, because the load was 14 tons when large unevenness which could be seen with the naked eye appeared for the first time on the upper deck plate of Girder 1 as stated above and, also, because the load was 11~12 tons when the relation between the strain around the center part of the upper deck plate and the load was no longer linear, the \( P_b \) is close to the actual value if these times are considered as the first stages of buckling in Girder 1. Also, when the center part of the upper deck plate of Girder 1 is considered as a plate with simply supported edges which is subjected to simple compression, and if Kármán’s effective width supposition is used to calculate the failure load \( P_{at.} \) of the plate:

\[
P_{at.} = \frac{4AZ}{L} \left( \frac{\pi^2 E t}{12(1-\nu^2)} \right) B \frac{W}{L/2} \frac{L/2-L/20}{L/2-L/20}
\]

where \( \nu \): Poisson’s ratio (taken as 0.3)

This value coincided very well with the load carrying capacity of 16 tons of Girder 1. Consequently, although in Girder 1 the upper deck plate buckled as a plate compressed simply, it can be considered that strength reached up to 16 tons.

On this point, the buckling of the web plates which was the cause of failure in Girder 2 was buckling by bending and the buckling stress was quite close to the yielding stress so that it is believed that there was hardly any difference between the buckling load and strength.

3.3 Local unevenness on the plate surface

Several examples of unevenness on the plate surface before and after failure are shown in Fig. 8. This figure shows the unevenness by use of contour lines when the plane containing the three points \( A, B, \) and \( C \) is taken as the standard plane. The displacement in the outer direction of the box is taken as positive.

A look at the figure shows that the unevenness before and after failure generally follows the same trend, but the position of the high points in the unevenness have shifted considerably according to the location. It is considered that this is because the center parts which are farthest removed from the diaphragm etc. are most prone to deformation.
When (a) and (b) in Fig. 8 are compared, in Girder 1 the web plate is deformed as one half-wave from the upper part to the lower part, while in Girder 2 the upper half above the stiffener is deformed as one half-wave and the effect of the stiffener is clearly shown. The numbers of half-waves when the unevenness is viewed in the longitudinal direction between both diaphragms are two and three for Girder 1 and Girder 2, respectively. As for these numbers of half-waves, it was found that the actual number of half-waves coincided well with the theoretical number of half-waves at the buckling, when in Girder 1 ABDC was considered as a plate with simply supported edges subjected to both simple bending and simple compression.

From this point also, in carrying out the buckling calculation of girders, it is believed appropriate to carry out the calculations on the supposition that each part separated by diaphragms and stiffener etc. is a plate.

When we look at Fig. 8 (c), the upper deck plate of Girder 1 has deformed in the form of two half-waves in the longitudinal direction of the girder, but theoretically it should buckle easiest in the form of three half-waves. As for the causes of this difference, there are the large deformation of the web plates in the form of two half-waves and the welding of a rail for load transmission on part of the upper deck plate.
As for the theoretical buckling load of the upper deck plate, the difference is not too large when the half-wave number is two or three⁹.

When the state after failure is viewed in Fig. 8, it can be seen that all the nodal lines of the waves are approximately perpendicular to the longitudinal direction of the girder. Consequently, it can be said definitely that the various plates of the center part of the girder have not been buckled by shearing.

3.4 Effect of torsion due to eccentric load

The shearing stress due to torsion is only several per cent (calculated value) of the maximum value of the normal stress due to bending. Also, the difference in deflections on the inner and outer ends of the upper deck plate which occur because of torsion is only several per cent (calculated value) of the total deflection at the center of the span. As described above, the deformations which actually occurred in the web plates and upper deck plate are approximately the same in shape as the buckling resulting from normal stress. The outer web plate is greatly deformed as well as the inner web plate. Then, it can be considered, from the above, that the torsion produced by the eccentric load has not a remarkably bad effect on box type girders of such dimension ratio as used in the present experiments.

Since the experimental values of deflections in the elastic ranges or buckling loads of the girders generally coincide very well with their calculated values as stated above and, also, since bending stresses of about the same amount are produced in both the inner and the outer web plates even near the center of the girder, it will be generally appropriate to use the assumptions described in section 2.4 in making designing calculations for girders.

4. Conclusion

When the results of present research are summarized, they are as follows:

(1) The stress distributions over the girder plate surfaces were generally irregular due to the local unevenness of the plates.
(2) The deflections of the girders in the elastic ranges coincided well with the calculated values.
(3) The buckling loads and failure loads of the girders generally coincided with their calculated values.
(4) The forms of the plates of the girders after failures coincided well with the theoretical wave forms in the case of bucklings.
(5) The effect of horizontal stiffeners in Girder 2 was clearly shown. In other words, the load carrying capacity of Girder 2 was almost 40 per cent higher than that of Girder 1. Also, at time of failure, the range in which the unevenness appeared on Girder 2 plate surfaces was much smaller than that of Girder 1.
(6) It is believed that the torsion produced by the eccentric load has not a remarkably bad effect on box type girders of such dimension ratio as used in the present experiments.
(7) In calculating the box type girders which will be subjected to eccentric load, it is considered generally appropriate to use the assumptions described in section 2.4.

Acknowledgement

The author would like to express heartfelt thanks to the following persons who gave him such kind guidance, aid and cooperation in carrying out the present research: Dr. T. Kobori, Chief of Research Section, Kameari Works, Hitachi, Ltd.; Dr. H. Miyamoto and Mr. S. Yagi of the same Research Section; Mr. T. Egawa, Chief of The 2nd Inspection Section; Mr. Y. Hiraguri, Manager of The 1st Engineering Department; Mr. N. Onish, Chief of Crane and Conveyor Design Section; Mr. T. Murata of the same Crane and Conveyor Design Section.

References

(3) ibid. p. 330.