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I. Lyse
I. E. Madsen

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Structural Behavior of Battle-Deck Floor Systems

BY

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AND

INGVALD E. MADSEN, JUN. AM. SOC. C. E.

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STRUCTURAL BEHAVIOR OF BATTLE-DECK FLOOR SYSTEMS

BY INGE LYSE,1 M. AM. SOC. C. E., AND INGVALD E. MADSEN,2 JUN. AM. SOC. C. E.

SYNOPSIS

Results of two years of investigating the behavior of battle-deck flooring are reported in this paper. Four one-third sized models and one full-sized floor panel, designed on the basis of the results obtained on the preliminary models, were tested under the action of a concentrated wheel load, such as the specifications recommended for H-20 loading. The battle-deck flooring acted as an integral unit distributing the wheel load over various stringers, with the amount of load taken by the several stringers depending on their spacing. The plate acted with the stringer so as to form a T-beam which, if taken into account in the design, might result in an economy of 10 to 15 per cent. The width of plate contributing to the T-beam action was also found to depend on the stringer spacing. When the stringers were coped in on the floor-beams, partial fixation resulted with further economy in design. The models were loaded with dead weights, and the full-sized floor panel was tested by means of a jack and spring device. The test results gave a basis on which to formulate a rational design method for battle-deck floor systems.

INTRODUCTION

Experiments were made on four one-third sized models, based on a prototype bridge floor consisting of two panels 20 ft long and 10 ft wide. The first model represented a floor with a \( \frac{3}{4} \)-in. plate on stringers, spaced at 24 in. The tests showed that such a floor was not capable of supporting an H-20 loading. Consequently, the second model had stringers welded between those of the first model, making the prototype a floor with a \( \frac{3}{4} \)-in. plate on stringers, spaced...
12 in., center to center. This model proved adequate in supporting the load, and the next step was to determine the more economical design. Comparative cost data indicated that although a widening of the stringer spacing might increase the weight of the floor slightly, the decrease in welding cost would more than offset the gain in weight. Thus, for the third floor model, a prototype was selected which consisted of a $\frac{3}{4}$-in. plate laid on stringers spaced at 30 in., and for the fourth model, a $\frac{1}{3}$-in. plate on stringers spaced at 24 in.

The results of the model tests showed that the floors behaved according to certain relationships. Design methods and procedure, as established from these results, served for the construction of the full-sized floor panel. The agreement between the design stresses and the measured stresses in this full-sized floor indicated a check on the design assumptions. The full-sized floor consisted of 12-in. standard I-beams, spaced on 26-in. centers, with a $\frac{1}{8}$-in. plate. The floor was 16 ft 9 in. in span and 9 ft 5 in. wide. This span length was adopted because the full-sized panel was tested as a simple beam with the stringers resting on the floor-beams, representing the distance between the points of contra-flexure of a 20-ft panel length with the stringers in the bridge floor coped into the beams.

**The Problem**

The problem of determining the behavior of battle-deck flooring may be divided broadly into two parts: (1) The stresses in the plate; and (2) the action of the stringer; that is, the amount of load carried by each stringer and the interaction of plate and stringers in resisting flexural stresses. The investigation was limited to the study of these conditions in relation to battle-deck flooring for highway bridges subjected to concentrated loads. For an adequate solution of the problem it was necessary to determine:

(a) The strength and deflection of battle-deck flooring under concentrated loads;
(b) The properties of the floor-plate in distributing the load over various stringers;
(c) The width of plate acting with the stringers as the compression flange of a T-beam;
(d) The length of plate under concentrated load affected by the load;
(e) The effect of changing the distance between the stringers;
(f) The degree of fixation at the ends of the stringers; and,
(g) The degree of fixation at the ends of the plate.

Little work has been done in this field, although some mathematical studies have been made on the width of plate acting in a T-beam under various loading conditions. Even less is known of the distribution of load among the various stringers of the floor. For uniform loads over a floor, simple relationships obtain, but for concentrated wheel loads such as were used in this investigation, the problem becomes very difficult.

What actually happens to a stringer floor under load is readily visualized, but a strictly mathematical analysis is practically impossible because the floor

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is statically indeterminate to a high degree. As the load is applied the stringer beneath it deflects and the plate deflects with it, acting as a beam to transmit shear to the next stringer. This second stringer will also deflect and carry on the distribution. Theoretically, the distribution will go to all the stringers, although, practically, the effect may become so small after being distributed over three or four of them that any further distribution may be neglected. This action is quite different from that usually assumed in design, namely, that the load is spread equally over a certain definite number of stringers. Since the deflection of a beam varies as the cube of the span (that is, the spacing between the stringers) and inversely as the cube of its thickness, a floor with equal spacings between the stringers has a constant proportion of the shear transmitted between each stringer. In other words, the load on a floor is distributed throughout the floor in a geometrical ratio.

The action of the plate under a concentrated load cannot be determined readily. The plate may be regarded as a rectangular plate supported along the edges, but the behavior is complicated by the fact that the deflection of the stringers varies along the plate, giving it an elastic support; and, in addition, the rotation of the stringers makes the degree of fixation in the plate an uncertain quantity.

In this investigation strain readings were taken before and after loading to determine the stresses. Slope readings were also taken along the stringers before and after loading. These readings were plotted, and the resulting curves were differentiated once to obtain the moments, and then a second time to obtain the shears. They were also integrated to obtain the deflections, and the deflections were measured directly for a check. From mechanics, the slope curve, differentiated twice, and multiplied by $EI$, gives the shear. Since the sums of all the shears of the stringers must be equal to the applied load, the value of the moment of inertia, $I$, was selected so that this became true for the experimental results. Knowing the value of $I$, it was easy to determine how much plate was acting with the stringer as a T-beam. The values of the shears on each stringer indicated how much load was carried by the various stringers. Furthermore, when the T-beam action of the stringer was known, the section modulus could be computed, and the stress could be determined from the moment curves which had been computed. These stresses were compared with the measured stresses, and their agreement served as a check on the work. The stresses in the plate were determined by means of Huggenberger tensometers.

**Program**

The four models were tested with the dead loads in various positions. In what is called a typical run, strain readings were taken along, and transversely between, the stringers, and level-bar readings were taken along the stringers both before and after the load was applied. The difference between the initial and final readings was the effect due to the applied load. The strain, multiplied by the modulus of elasticity, gave the stress at any point in the stringer; and the difference in the level-bar readings gave the slope due to the applied load. Such tests were made on the models with the load in
all parts of the floor. However, the load in the middle of the stringer caused the larger stresses and, therefore, governed the design. In the full-sized panel, tests were made only with the load in the middle of the floor.

For these tests, the sum of all the slope readings along a stringer at any point would give the deflection of the stringer at that point. However, this was supplemented by tests in which Ames dials were placed along the stringers to measure the actual deflections. These results agreed quite closely with the integration of the slope readings and served as a check on them.

The plate stresses were obtained by measuring the strains in the plate with the tensometers. The load was placed in various parts of the plate, and tensometers were located at all points where any effect was noticed, thus recording the distribution of the plate stresses. In making some of the models, strain measurements were also taken on the stringers before and after welding to furnish an idea as to the severity and the effect of the welding stresses.

**Testing**

The models were made of standard rolled-steel sections and plate. They were constructed on the basis that the model should be one-third size. However, since the prototype could not be reduced in all proportions without expensive machining, the stringers were machined so as to keep the clear spans of the plate in proportion, and they were designed so that the stresses in the model would be approximately the same as in the prototype. The ideal could not quite be attained since the smallest I-beams rolled gave about a 25% excess over the computed section modulus. The panels in the model were 80 in. by 40 in. The first two models consisted of two panels welded together. The last two models consisted of only one panel each, but they were welded in the same frame and their floor-beams consisted of channels. After the
testing of the last two models was completed, a filler plate was inserted between the two adjacent channels, and weld metal was deposited so as to make an I-beam and cause the panels to be continuous. A view of the first two models is shown in Figs. 1 and 2. All the models had the same type of stringers, but the stringer spacing and the plate thickness varied. The plate in the first two models was steel metal sheeting of a low strength and yield point. This type was selected because the uniformity of thickness which sheeting possesses was an important factor in the ¼-in. plate thickness used in these models. The ¼-in. and ⅛-in. plates in the third and fourth models were of the regular structural grade of steel and passed the specifications.

Great care was necessary in welding the models in order to avoid warping, particularly in the first two where the plate was so thin. They were fabricated by first tacking the stringers on to the plate, after which the weld metal was deposited, alternating from one spot to another on the floor so as to minimize the heat and thus decrease the tendency to warp.

The models were loaded by a dead-weight loading rig which is shown in Fig. 1. The Bureau of Public Roads, United States Department of Agriculture, has approved a loading area of 20 in. by 10 in. for the rear wheel of an H-20 loading. This assumes a tire 20 in. wide, with a pressure of 112 lb per sq in., giving 10 in. of longitudinal bearing.

The loading rig consisted of a cast-iron block to which a frame was fastened to carry the additional dead weight. The initial weight of the rig was 400 lb. The load was applied, in 50-lb increments, through a steel bearing plate which was one-third size, or 6⅔ in. by 3½ in. A piece of soft rubber, 1⅞ in. thick, was placed under the bearing plate to keep the load constantly uniform as

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the plate deflected, and a piece of cellotex was placed under the rubber to keep the area in contact with the plate constant.

A load of 2,489 lb should cause the same stresses in the model as would the rear wheel of an H-20 truck in the full-sized panel. A load of 2,500 lb was used in the model tests.

To show that the loading rig gave a uniform load distribution on the plate, and to compare its action with that of a tire, comparative load tests were made.

![Full-Sized Floor](image)

The floor was tested by means of the loading rig, and strain measurements were taken around the load at a number of critical points. Then the floor was tested by means of a tire, as shown in Fig. 3. The results of these tests were compared and found to be equal within the limits of experimental error, showing that the loading rig gave essentially the same results as an actual wheel load.

No trouble was experienced with warping in the full-sized floor panel, due to welding, so that it was unnecessary to adopt any special welding procedure. The plate was so thick that it dissipated the heat rather quickly. The structural grade of steel was used in the floor.
The floor was set in a frame as shown in Fig. 4. It was tested by jacks such as that shown in Fig. 5, the load being measured by the deflection of calibrated springs. It was possible to ascertain the load on the floor by this method to within 1 per cent. In order that the pressure on the floor be kept uniform as the plate deflected, the deflection of the plank on which the spring set-up was placed, was computed, and the plank was made of such a thickness that its deflection would be approximately equal to that of the plate. The deflection of the plank was computed on the basis of a beam on a yielding foundation under uniform pressure.

The models were held in a frame consisting of vertical posts made of 8-in. channels braced with angles (see Figs. 1 and 2). The floor-beams of the models were welded to the vertical channels to simulate the connections of beams to hangers in an actual bridge construction. The full-sized panel was held in a truss made of beams and angles, the details of which are shown in Fig. 4.

The strains in the stringers were measured by a fulcrum-type Whittemore strain-gage, equipped with a 0.0001-in. Ames dial. It is accurate to about 600 lb per sq in., as a tolerance of 0.0002 was allowed in repeating a reading. Since temperature changes cause strains in a structure that would be measured by the gage, the observations were made when these changes were at a minimum. Usually, therefore, temperature could be neglected, but when variations occurred, corrections were applied to the strains from observations on mild steel standards.

The tensometers which were used in the plate tests had gage lengths of 1 in. and 0.5 in. They were accurate to within about 500 lb per sq in., depending on the gage length.

Two level-bars were used. The one in the model tests had gage lengths varying from 1 in. to 6 in. However, the 5-in. gage was used almost entirely. It was fitted with a very sensitive bubble so that readings could be repeated to 0.0001 in., if the point hit the same spot on the floor. Hitting the exact spot was practically impossible, and, therefore, the accuracy was limited by the irregularities in the plate surface. For the full-sized model, the bubble was mounted so that the level-bar had a 15-in. gage length. This would make the instrument three times more accurate than it was on the 5-in. gage length, but in this case, again, the accuracy was limited by the irregularities in the plate surface. These irregularities were worse in the full-sized plate than in the models. In all cases the spots where the micrometer point of the level-bar rested were ground smooth and polished with an emery cloth.
Test Data and Relationships

Many tests were made on all the models and on the full-sized panel. It is impossible to describe all these tests, however, and only the results of the significant and important runs will be given.

Tests on First Model of 24-Inch Stringer Spacing.—A series of nine runs was made on the first model. Some of the load positions included the quarter-points of the stringers, the center of the span, and the points adjacent to the floor-beams. As far as stringer stresses are concerned, the load in the center of the span caused the critical stresses.

In Series 16 (see Fig. 6), the load was placed between Stringers E and G, in the middle of the first panel. The results of the tests are shown in Tables 1 and 2. The two adjacent stringers, E and G, received 80% of the load, whereas Stringers C and I received about 10 per cent. The stringers that were away from the direct influence of the load carried over about 25% of their load to the adjacent stringer. The beams acted as if they were partly fixed, the average fixation factor on the left being about 15% and that on the right about 39 per cent. Due to this fact, the shear on the left was less than that on the right. These fixation factors were the ratios of the moments at the supports to those of a fixed-end beam.

In order that the shears should equal the applied load, the moment of inertia of a stringer had to be 3.54 in.\(^4\). This required 5.50 in. of plate acting in the compression flange. With these values the computed center moment was found from the slope curves, and the fiber stresses were computed. The measured stresses did not agree with those computed as well as expected, and the reason for this will be discussed subsequently.

The center deflection was computed from the formula:

\[
y = \frac{W L^3}{192 E I} ( - 4 + 2 F_1 + F_r ) \tag{1}
\]
in which: \( W \) = wheel load; \( L \) = span length; \( E \) = modulus of elasticity; \( I \) = moment of inertia; \( F_l \) and \( F_r \) are the fixation factors at the left and right ends of the span, respectively. Equation (1) was derived from moment-area theorems and, although it is approximate, it is reasonably accurate, if the fixation factors do not differ by more than 50 per cent.

**TABLE 1.—Test Results**

<table>
<thead>
<tr>
<th>Stringer*</th>
<th>Shear, in Pounds</th>
<th>Load, in pounds</th>
<th>Percentage of total load</th>
<th>Left, ( F_l )</th>
<th>Right, ( F_r )</th>
<th>Computed center moment, in thousands of inch-pound</th>
<th>Computed</th>
<th>Measured</th>
<th>Computed</th>
<th>Measured</th>
<th>Ratio to next stringer*</th>
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<tr>
<td>C</td>
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<td>138</td>
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<td>10.4</td>
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* Stringers: (a) 17, (b) 18, (c) 21, (d) 22.
TABLE 1.—(Continued)

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<th>Stringer</th>
<th>Left</th>
<th>Right</th>
<th>Load, in pounds</th>
<th>Ratio to next stringer</th>
<th>Percentage of total load</th>
<th>Left, ( f_l )</th>
<th>Right, ( f_r )</th>
<th>Computed center moment, in thousands of inch-pounds</th>
<th>Ratio to next stringer</th>
<th>Computed</th>
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<th>From slope readings</th>
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<td>H..</td>
<td>109</td>
<td>124</td>
<td>233</td>
<td>0.50</td>
<td>9.4</td>
<td>18</td>
<td>43</td>
<td>3.9</td>
<td>0.46</td>
<td>+2.2</td>
<td>+2.5</td>
<td>0.0201</td>
<td>0.0190</td>
<td>... 0.43</td>
</tr>
<tr>
<td>I..</td>
<td>49</td>
<td>49</td>
<td>98</td>
<td>0.42</td>
<td>3.9</td>
<td>0</td>
<td>40</td>
<td>1.8</td>
<td>0.45</td>
<td>+1.0</td>
<td>+0.3</td>
<td>0.0095</td>
<td>0.0090</td>
<td>... 0.47</td>
</tr>
<tr>
<td>( T_f )</td>
<td>1179</td>
<td>1271</td>
<td>2450</td>
<td>... 97.9</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
</tr>
</tbody>
</table>

(c) Series 6, Third Model

<table>
<thead>
<tr>
<th>Stringer</th>
<th>Left</th>
<th>Right</th>
<th>Load, in pounds</th>
<th>Ratio to next stringer</th>
<th>Percentage of total load</th>
<th>Left, ( f_l )</th>
<th>Right, ( f_r )</th>
<th>Computed center moment, in thousands of inch-pounds</th>
<th>Ratio to next stringer</th>
<th>Computed</th>
<th>Measured</th>
<th>From slope readings</th>
<th>Measured</th>
<th>Ratio to next stringer*</th>
</tr>
</thead>
<tbody>
<tr>
<td>A..</td>
<td>17</td>
<td>11</td>
<td>28</td>
<td>0.06</td>
<td>1.1</td>
<td>-10</td>
<td>-5</td>
<td>0.6</td>
<td>0.06</td>
<td>+0.3</td>
<td>+0.5</td>
<td>0.0028</td>
<td>0.0037</td>
<td>0.0045 0.08</td>
</tr>
<tr>
<td>B..</td>
<td>220</td>
<td>234</td>
<td>454</td>
<td>0.31</td>
<td>18.5</td>
<td>-38</td>
<td>-27</td>
<td>10.4</td>
<td>0.39</td>
<td>+5.4</td>
<td>+5.8</td>
<td>0.0458</td>
<td>0.0450</td>
<td>0.0435 0.48</td>
</tr>
<tr>
<td>C..</td>
<td>735</td>
<td>722</td>
<td>1457</td>
<td>1.00</td>
<td>59.4</td>
<td>+27</td>
<td>+24</td>
<td>25.4</td>
<td>1.00</td>
<td>+13.1</td>
<td>+14.2</td>
<td>0.0944</td>
<td>0.0938</td>
<td>0.0935 1.00</td>
</tr>
<tr>
<td>D..</td>
<td>234</td>
<td>240</td>
<td>474</td>
<td>0.32</td>
<td>19.3</td>
<td>-47</td>
<td>-36</td>
<td>11.4</td>
<td>0.45</td>
<td>+5.9</td>
<td>+6.6</td>
<td>0.0495</td>
<td>0.0497</td>
<td>0.0490 0.53</td>
</tr>
<tr>
<td>E..</td>
<td>28</td>
<td>15</td>
<td>43</td>
<td>0.09</td>
<td>1.7</td>
<td>+5</td>
<td>-23</td>
<td>1.2</td>
<td>0.10</td>
<td>+0.6</td>
<td>+1.0</td>
<td>0.0037</td>
<td>0.0038</td>
<td>0.0045 0.07</td>
</tr>
<tr>
<td>( T_f )</td>
<td>1234</td>
<td>1222</td>
<td>2456</td>
<td>... 100.0</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
</tr>
</tbody>
</table>

The deflection was obtained by adding the areas under the slope curve. The agreement between the observed and measured deflections served as a check on the accuracy of the differentiation and the computations.

In Table 1(b) are shown the results of Series 17A on the first model. In this run, the load was placed directly on Stringer \( E \) (see Fig. 6), at its center. However, the loading area was so wide that the bearing block overlapped the plate and some of the load was transferred directly through the plate to Stringers \( C \) and \( G \). The ratio between the shear carried from one stringer to another...
TABLE 1.—(Continued)

<table>
<thead>
<tr>
<th>Stringer*</th>
<th>Left</th>
<th>Right</th>
<th>Load in pounds</th>
<th>Ratio to next stringer</th>
<th>Percentage of total load</th>
<th>Left, ( F_t )</th>
<th>Right, ( F_r )</th>
<th>Computed center moment, in thousands of pounds per square inch</th>
<th>Ratio to next stringer</th>
<th>Center Stress, in pounds per square inch</th>
<th>Center Deflection, in thousands of pounds per square inch</th>
<th>Ratio to next stringer*</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>92</td>
<td>82</td>
<td>174 0.30</td>
<td>6.7 +16 +6 3.3 0.26</td>
<td>+1.8 -1.2 -2.0</td>
<td>0.016</td>
<td>0.017</td>
<td>0.012</td>
<td>0.30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>288</td>
<td>301</td>
<td>589 0.47</td>
<td>22.8 -16 -10 12.6 0.57</td>
<td>+6.7 +6.0 -3.3 -2.8</td>
<td>0.058</td>
<td>0.057</td>
<td>0.054</td>
<td>0.63</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>625</td>
<td>625</td>
<td>1250 1.00</td>
<td>48.4 +23 +24 22.1 1.00</td>
<td>+11.9 +12.8 -5.7 -2.8</td>
<td>0.091</td>
<td>0.091</td>
<td>0.091</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>235</td>
<td>214</td>
<td>449 0.38</td>
<td>17.4 -4 -13 9.3 0.42</td>
<td>+5.0 +4.4 -2.4 -1.8</td>
<td>0.042</td>
<td>0.042</td>
<td>0.043</td>
<td>0.46</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>63</td>
<td>63</td>
<td>126 0.28</td>
<td>4.9 +7 +7 2.4 0.26</td>
<td>+1.3 -0.6 -0.7</td>
<td>0.011</td>
<td>0.011</td>
<td>0.010</td>
<td>0.26</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>1303</td>
<td>1258</td>
<td>2588 100.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\( \text{SERIES 19, FOURTH MODEL} \)

\( \text{SERIES 31, FULL-SIZED FLOOR} \)

* See Fig. 6. † \( T = \text{total} \). Values actually computed to nearest 10 lb; recorded as shown to conserve space.

in the direction away from the load is 0.24. The average value of \( F_t \) was 23% and that of \( F_r \), 35 per cent. The value required for the moment of inertia of the stringers to make the shear equal to the load was 3.35 in.\(^4\) for the interior stringers, and 2.84 in.\(^4\) for the exterior stringer, \( A \) (see Table 2). These values for \( I \) were developed as follows: First, the shears were computed using a value of \( I \) corresponding to full T-beam action; that is, using 8 in. of plate for the top part of the \( T \) in the interior stringers, and 4 in. for the exterior stringers. This made the value of the shears too large. Next, the values for the moment of inertia were multiplied by a factor so that the shears would equal the load,
TABLE 2.—Properties of Sections Used in Computations for Table 1

<table>
<thead>
<tr>
<th>Description</th>
<th>Series 17A, First Model</th>
<th>Series 21, First Model</th>
<th>Series 6, Third Model</th>
<th>Series 19, Fourth Model</th>
<th>Series 31, Full-Sized Floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) T-beam action*</td>
<td>11.00</td>
<td>10.00</td>
<td>12.00</td>
<td>13.00</td>
<td>14.00</td>
</tr>
<tr>
<td>(2) Moment of inertia = 1 in.(^4)</td>
<td>1.56</td>
<td>1.56</td>
<td>1.56</td>
<td>1.56</td>
<td>1.56</td>
</tr>
<tr>
<td>(3) Section modulus = S in.(^3)</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

* Inches of plate in T-beam action. \(^{\dagger}\) Moment of inertia = 1 in.\(^4\). \(^{\ddagger}\) Section modulus = S in.\(^3\)

and then the width of plate required for this value of the moment of inertia was computed.

The left reactions were again found to be less than the right, as would be expected, since the left fixation factor, \(F_l\), was the smaller. It is seen that Stringer \(E\), directly beneath the load, received fully 50% of the load.

In Series 19, the load was placed on Stringer \(C\), which was next to the exterior stringer. Consequently, the load could not be distributed so widely. In this case, the loaded Stringer \(C\) carried 55% of the load, or 5% more than in the former series.

The worst loading condition for the floor occurred in Series 21, Table 1(c), when the load was placed directly over Stringer \(A\), which thus received 65% of the load. However, this loading would not be attained in an actual bridge, since, in order to place the wheel directly over an exterior stringer, a considerable part of the wheel would project beyond the floor. When the load was placed as close to the exterior stringer as is practical, it took about 50% of the load, and its design would be about the same as that of an interior stringer.

Referring to the stringer stresses of Series 21, the measured stress gave values that would require much more plate in T-beam action than the total plate width between stringers. This shows that there were other stresses in the stringer besides those due to bending. The observed slopes of Stringer \(A\) during this test are shown in Fig. 7(a), and the corresponding stresses in Fig. 7(b).

The results of quarter-point loading on the floor were quite interesting. The length of plate in T-beam action was the same as that in the corresponding center of span loadings, but the shear transferred from one stringer to another was only 0.08 of the shear in the stringer, instead of the 0.25 which occurred in the former cases. This agrees with the fact that the shear transferred from one stringer to another depends on their relative deflection; and since the stringers deflected less at the quarter-point, the difference in deflection between two adjacent stringers was less. Carried to the extreme, when the load is at the end of the stringer, one stringer will take all the load and must be designed for it.
Supplementary tests were also made on this model. In Fig. 8 the deflection of Stringer $E$ is plotted against the load, which was placed in a position corresponding to Series 16. It is seen that the load-deflection relation is a straight line (as would be expected), showing that the width of plate acting as a T-beam was constant and did not vary with the load. The center deflection diagram for the plate is also given in Fig. 8. The curve slopes upward to the left showing that the load required for equal increments of deflection increased with the increase in the load on the plate. In other words, catenary action helped to support the plate.
In Fig. 9, the deflections of the plate and Stringer E are plotted against the longitudinal axis of the stringer for a 2,500-lb load. The deflection curves for quarter-point loading are also given. In the stringers, there is a slight initial reverse curvature of the deflection curve at the ends of the span. This shows a slight degree of fixation. The curve for the plate, in sharp contrast to that of the stringers, slopes gradually and then changes quite sharply beneath the load.

![Graph showing load-deflection relation for center stringer and plate](image)

In all these tests the stringers rotated considerably. However, the top of the stringer showed little or no rotation, which indicated that the stringers must have rotated about the plate. This rotation is at least partly due to the deflection of the floor plate. Evidently, the stringers were subjected to torsional forces and thus contributed to the load distribution.

Tests on Second Model of 12-Inch Stringer Spacing.—The second model was the same as the first, except that the stringers were spaced 4 in., center to center, instead of the former 8 in. The results were similar to those of the first test, but due to the decreased stringer spacing, the floor was stiffer so that the load was distributed over more stringers.

The results of Series 32 (see Fig. 6), in which the load was placed directly on top of Stringer F, are shown in Table 1(d). The width of the loaded area was so great, relatively, to the stringer spacing that when Stringer F was loaded, the area overlapped on the two other stringers, E and G. Full T-beam action was present in this series. Stringer F took the largest proportion of the load—about 30 per cent. The ratio of shear carried from one stringer to another, in the direction away from the load, was about 0.43, instead of the 0.25 found in the first model. The average fixation factor was about 12% at the left ($P_l$) and about 35% at the right ($P_r$).

The center moment, stress, and deflection were computed for the second model in the same manner as for the first. The slope readings checked the deflection. However, the ratios between the various stringers did not stay quite as constant for the shear, moment, and deflections, as they did in the first model. This shows that the slope curves were not exact second-degree.
curves, although they were so assumed, and, consequently, a slight error occurred in the differentiation and integration. The load on the other sections of the floor gave results similar in nature to those shown for Series 32.

After the regular runs had been made on the second model an attempt was made to test it to destruction. It was loaded by means of a 20-in. I-beam extended out as a cantilever from an 800 000-lb testing machine. It was first loaded to 15 710 lb, which is more than six times the design load, at which time the deflection under the load was 0.531 in. The load was released to 4 700 lb and the deflection was 0.240 in., of which 0.118 in. was permanent set.

The model was loaded again to 28 510 lb, when the testing had to be discontinued due to the incipient yielding of the loading beam and the bowing of the vertical legs of the frame holding the floor. At this load, the total deflection was about 1 in. The stringer under the load had yielded and thrown much of the load on to the two adjacent stringers, which were also beginning to yield. Upon removal of the load, a permanent set of about 0.5 in. was observed.

There was no sign of yielding in the plate, and the only sign of failure was a scaling of the whitewash in one of the welds holding the floor-beam in the supporting frame.

Tests on Third Model of 30-Inch Stringer Spacing.—The third model was based on a prototype with a ½-in. plate welded to stringers spaced on 30-in. centers. This model, as well as the fourth, consisted of only one span, and since the stringers were coped into the floor-beams there was a partial degree of fixation. In these models, the plate was extended to the center of the exterior stringer, A, whereas it overlapped 2 in. from the center of the exterior
stringer, $E$. This was done to see whether the overlapping plate was efficient in T-beam action.

The results of the tests on the third model were very similar to those of the first two. The results of the case in which the load was placed on top of Stringer $C$ are given in Table 1(c). Stringer $C$ took 59.4% of the load, which is a greater proportion than in the first two models. This was due to the greater stringer spacing. For a similar reason the width of plate in T-beam action was not as large as in the first two models. The fixation factor for the loaded stringer was small, as would be expected. For the other stringers the value of $F$ is negative, denoting an applied moment at the end of the stringer. The reason it is negative is that, due to the partial fixation of the loaded stringer, the floor-beam rotates and imposes applied moments on the adjacent stringers.

A number of tests were made in a study of the plate stresses in the third and fourth models. Similar tests were made on the full-sized floor and since these were more complete and gave essentially the same results as those on the models, only the results on the full-sized floor plate are given in this paper. Full T-beam action was not present in these models. Accordingly, the compression in the top part of the T-beam would be expected to decrease away from the stringer. This decrease was found to be linear, as shown in Fig. 10, for the variation of the compression in the plate between Stringers $C$ and $D$ when the load was placed between them.

Cross-bracing was welded between the stringers in one of the bays of this model. It consisted of $\frac{1}{4}$-in. by 2-in. plates welded to the stringers and the plate, at 12-in. intervals. Although the secondary bracing was spaced this closely, the plate stresses were reduced very little.

**Tests on Fourth Model of 24-Inch Stringer Spacing.**—The fourth model was based on a prototype with a $\frac{3}{16}$-in. plate welded to stringers at 24-in. centers. In this model, an attempt was made to evaluate the welding stresses. Strain readings were taken on the stringers and on the part of the plate over the stringers, before and after the floor was welded together. The welding stresses were greatest in the plate because most of the welding was done there. The longitudinal welding stresses in the plate varied from about 4000 lb per sq in. over the exterior, to 9000 lb per sq in. compression over the interior, stringers. The stress along the bottom of the stringers was about 2000 lb per sq in. in tension. The transverse welding stress in the plate between the stringers varied from 15000 lb per sq in. to 20000 lb per sq in. in compression. These stresses did not seem to affect the test results.
The results of the various tests of the fourth model were similar to those of the previous models. Table 1 gives the results for the case in which Stringer C was loaded. This stringer is seen to take 48% of the load.

After the regular runs had been made on the third and fourth models, they were welded together to form a two-panel floor. A run was taken similar to that in the third model for which the results are given in Table 1(e), and the results for the two-panel model were very similar to those for the third model except that the fixation factor at the intermediate floor-beam was increased to 50 per cent. The stresses and deflections in the floor-beams were found to be smaller in accordance with the increased fixation factor.

![Deflection and Permanent Set of Plate and Stringers B and C. Breaking Test](image)

Both the third and fourth models were tested to destruction. They were loaded in a manner similar to that used for the second model; that is, a cantilever load from the big testing machine. The third model was loaded up to 26,744 lb and the fourth, to 27,911 lb. In each case, the testing had to be discontinued due to incipient yielding of the loading beam. At these maximum loads, which were about eleven times the design load, large deflections were present although nothing broke. The large reserve strength is due to the fact that as one stringer yields, the increase in load is taken by the adjacent stringers while the original stringer still holds its yield-point load. This process continues until all stringers have yielded.

In the third model the load was placed on the plate between Stringers B and C (Fig. 6). Fig. 11 shows a diagram of the load-deflection and permanent-set curves of the stringers and the plate. At the maximum load, the deflection was 2 in. and the permanent set was 1 in.
The fourth model was tested with the load directly over Stringer C. The results were about the same as for the third model. A diagram of the deflection of the centers of the stringers under various load increments is shown in Fig. 12.

![Deflection Curves for Center of Stringers](image)

**Fig. 12.—Deflection Curves for Center of Stringers. Breaking Test, Model No. 4**

**Tests of Full-Sized Floor Panel.**—The full-sized floor panel was built according to a design procedure, determined by a study of the results of the model tests. The panel was 16.75 ft long, center to center bearing, and 9.42 ft wide. This floor was built so that it would act as a test on the design procedure, and serve as a check on the model tests. How well it did this is illustrated in the comparison of the design values with the measured values of stress and deflection for the critical sections of the floor, shown in Table 3. The check is very good considering that the properties of structural sections may vary as much as 5 per cent. The tests on this full-sized floor were similar to those of the models. Table 3 gives the results when the load was on top of Stringer C.

**TABLE 3.—Test of Full-Sized Floor**

<table>
<thead>
<tr>
<th>Description</th>
<th>Maximum stress in plate, in pounds per square inch</th>
<th>Maximum deflection of plate, in inches</th>
<th>Maximum stress in loaded stringer, in pounds per square inch</th>
<th>Maximum deflection of loaded stringer, in inches</th>
<th>Percentage of wheel load taken by loaded stringer</th>
<th>Width of plate in T-beam action, in inches</th>
<th>Weight of floor (stringers and plate), in pounds per square foot</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design value</td>
<td>27 900</td>
<td>0.113</td>
<td>13 300</td>
<td>0.169</td>
<td>53</td>
<td>17.5</td>
<td>44.0</td>
</tr>
<tr>
<td>Measured value</td>
<td>26 600</td>
<td>0.111</td>
<td>13 900</td>
<td>0.170</td>
<td>56</td>
<td>17.5</td>
<td>...</td>
</tr>
</tbody>
</table>

Because of the large dimensions of the full-sized floor, it was possible to obtain more data concerning the stresses in the plate than had been possible in the models. A large number of tensometer readings were taken in order to obtain the stress distribution in the plate. Fig. 18(a) shows the distribution of the transverse stress in the plate along the length of the floor. This stress is the greatest in the plate, since it lies along the short span between the
stringers. It is seen to have a peak of 26,000 lb per sq in. at the center of the load, rapidly decreasing asymptotically along the plate. The compression in the top of the plate is seen to be similar to the tension in the bottom. Complete readings could not be obtained for the compression side since the load was in the way. The distribution of the transverse stress in the bottom of the plate, across the width of the floor, is shown in Fig. 13(b). In the loaded span, the stress in the plate was zero at the edges of the stringer flanges. The stress changed to compression along the flange, and rapidly dropped to zero at the next stringer.

An over-load test was made on the full-sized floor. It was loaded up to 56,000 lb, or two and one-half times the design load. The deflection of the floor increased linearly with the increase in load. At a 50% over-load, there was an over-all permanent set under the plate of 0.005 in. At the maximum load of two and one-half times the design load the permanent set was 0.025 in. No signs of failure or yielding could be determined with the exception of a scaling of the whitewash near one of the welds, between the plate and the stringer. After the load was removed, the floor appeared just as perfect as ever, and one could never have detected by eye that it had been overloaded. The maximum deflection of the plate had been 0.269 in.

**Discussion of Results**

The results of these tests show that the battle-deck floors acted as an integral unit. The load was distributed from one stringer to another by means of the plate, which acted as a cantilever beam, in proportion to the relative deflection of the stringers. This distribution factor was a constant for a definite stringer spacing and plate thickness. In all cases, as shown by the tables, the carry-over factor was larger for the stringers close to the load than for those away from the load. Considering the case in which the load was directly over one stringer, the adjacent stringers took, not only the load carried over by their relative deflections, but also some of the load itself, as the wheel load was so wide that it overlapped the loaded stringer.
In some cases, the stresses in the stringer did not check the computed stresses, particularly for the smaller stringer spacings, because the computed stresses only included bending stresses. Since the stringers were welded to the floor-beams, direct tension could occur in addition to the bending moments. Four types of stresses are probably added to the simple bending stresses along the tension flange: First, due to the tension in the lower flange, its length is increased, imposing compression on the ends of the beam, and thus tending to reduce the flange tension; second, due to the deflection of the beam, the longitudinal axis shortens and causes tension along that axis; and, third, due to the rotation of the bottom flange, shortening takes place which causes tension along the gage line. Finally, these effects in any one stringer have a reaction on the floor-beam which, in turn, applies a couple and a tensional stress to the other stringers.

The effect of these stresses is greatest on the loaded stringer as it reduces the compression and increases the tension. This stringer has a great effect on the rotation of the floor-beam which, in turn, tends to offset the secondary stresses in the adjacent stringers. In the stringers a distance away from the load, the tension is reduced in some cases to zero, and the compression is increased. Table 1(b) illustrates this phenomenon.

The width of plate in T-beam action was found to increase as the stringer spacing decreased, whereas the load taken by a stringer increased with the increase in spacing. Both these results are logical and Fig. 14 presents the relationships obtained. These relationships are useful for the design of battle-deck floors.

A stringer needs to be designed only for the effect of one rear wheel load since the usual axle spacing on trucks is so large that the effect of one wheel is not carried over to the other. There seems to be no need of making the exterior stringer larger than the interior stringers since it is practically impossible for the center of the wheel to come over the center of the exterior stringers. Usually, the exterior stringer will not be stressed higher than the interior stringers if it is the same size. The stringer next to the exterior will take about 5% more of the load than any of the other interior stringers, because a full wheel load can rest on it and the exterior will not help support it as much as the interior stringers. Thus, it will be overstressed about 5%, if it is the same size as the other interior stringers.

When coped into the floor-beams of a single span, the stringers had a fixation factor of about 25 per cent. When the spans were continuous, the
factor was as high as 50 per cent. A substantial saving in material can be
affected if this partial fixation is taken into account in design. The foregoing
statements apply to web-plate connections; the fixation factors for web-angle
connections were slightly smaller.

In designing battle-deck floors the smaller the stringer spacing is made,
the lighter will be the resultant floor. However, the increased welding cost of
the lighter floors will probably make them uneconomical unless the importance
of light weight is particularly great, such as in lift spans.

The over-load tests showed that the battle-deck floor had a large reserve
strength and was practically impossible to break. The tests on the full-sized
floor panel would indicate that, although the measured stress was fairly high,
the plate thickness could be reduced to $\frac{5}{8}$ in., or $\frac{3}{16}$ in., and still be amply
strong to take an H-20 load.

Most plates in battle-deck flooring have been designed on the assumption
that the plate under the load acts as a fixed-end beam. No account has been
taken of the longitudinal distribution of the load. Fig. 13 shows the dis­
tribution of the plate stress. The longitudinal distribution is seen to extend
over about four times the clear span of the plate, or about 84 in.

The point of contra-flexure of the plate fell close to the edge of the stringer
flange. One of two assumptions may explain this: The first is that the plate
acts as a simple span between the flange of stringers; and, the second is that
the plate acts as a fixed beam with the point of contra-flexure at the edge of
the stringer flange. In either case the result is the same, and the first assump­
tion is the easier to use in computation. The second assumption is probably
closer to what actually takes place since the plate forms a fixed beam of varying
cross-section, the depth between the flanges being the depth of the plate itself,
and the depth over the stringer flanges being the thickness of the plate plus
the flange. The increased depth of beam at the stringers decreased the stress
over the stringer far below what would be expected. The tension stress was
larger than the compression stress, probably because a catenary stress of
about 1,000 lb per sq in. was present.

The curve in Fig. 13(b) shows that the longitudinal distribution of stress
varied from a peak at the center of the load, decreasing asymptotically. Neg­
lecting the small curve at the peak, this variation may be assumed to be para­
bolic. Thus, the computation of the stresses in the plate becomes very simple:
First, the total moment which the plate must support is computed on the
assumption that the plate is a simple beam between the edges of the stringer
flanges. Next, the average stress in the plate is computed over the length of
four times the clear span of the plate, since the load is distributed over that
distance. For the average stress this gives:

$$s_{ave} = \frac{M}{4SL} \hfill (2)$$

in which $M$ is the moment; $L$, the clear span; and $S$ is the section modulus
of the plate per inch of plate. The maximum stress in the plate will be three
times the average since the distribution of stress is parabolic. This semi-
empirical method of determining the plate stress was used in computing the
stresses in the full-sized floor and was checked by the observed stresses. The values of the stresses in the one-third sized models checked even more closely than those for the full-sized floor. This method of determining plate stresses may be expected to give quite accurate results for one-way slabs under concentrated wheel loads.

**Recommended Method of Design**

The results of the four models and the full-sized floor panel indicated that battle-deck floors may be designed by the following procedure:

1. When the stringer spacing is determined, obtain from Fig. 14 the load taken by T-beam action of one stringer and its contributing plate width. With this information, the moment on the stringer is easily found and a trial stringer is selected. The properties of the T-beam section can be determined as soon as the plate thickness is found.

2. When the trial stringer section has been selected for the T-beam, the clear span between the stringers is known. The trial section can be determined quite closely in the first step since great changes in the top of the T-beam change the value of the section modulus only slightly.

3. The required plate thickness, \( t \), is determined by the formula:

\[ s_{\text{max}} = \frac{3M}{4SL} \]  

and is found directly by changing Equation (3) to,

\[ t = 3 \sqrt[2]{\frac{M}{2s_{\text{max}}L}} \]  

4. Knowing the plate thickness, the section of the T-beam is determined, and the stresses in the stringer are computed. If the stresses are not satisfactory another section is selected.

5. Design the stringer connections for the full load in shear.

6. Partial fixation may be taken into account and resulting economies effected by using a fixation factor of 25% when the span is simple and 50% when it is continuous.

Wide-flanged beams, in general, will be economical since they reduce the clear span of the plate. However, the lightest wide-flanged beam may not meet the specification that the web must be at least \( \frac{3}{8} \) in. in thickness. In general, lateral bracing should not be necessary for the floor when the stringers are coped in on the floor-beams.

When the plate and stringers are selected, the remainder of the floor is designed according to the usual methods. The welds between the plate and the stringers are designed for longitudinal shear. The resulting welds will also be strong enough to take care of the horizontal and catenary stresses in the plate.
Summary

The results of the tests indicated that:

1.-Battle-deck flooring acts as an integral unit.
2.-Inherent welding stresses may be found in the plate but they do little harm under static load. Care in welding will minimize these stresses.
3.-A tire imposes an essentially uniform load over the area upon which it rests.
4.-The plate acts with the stringers to form a T-beam, reducing the stringer stress about 15 per cent.
5.-The load taken by a stringer and the width of plate acting with the stringer vary with the stringer spacing.
6.-The stringers distribute the load in proportion to their relative deflections; thus, the distribution is greatest when the load is in the center of the panel and decreases as the load approaches the floor-beams. The distribution factor varies with the thickness of the plate and the distance between the stringers.
7.-The plate acts as a simple beam between the edges of the stringers. The load is distributed longitudinally over a distance equal to four times the clear span. The distribution is parabolic, with the maximum stress three times the average stress.

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