OVERLOAD BEHAVIOR OF 1/8th SCALE THREE-SPAN CONTINUOUS PRESTRESSED CONCRETE BRIDGE GIRDER

by

W. L. GAMBLE

Issued as Documentation of Progress on the Field Investigation of Prestressed Reinforced Concrete Highway Bridges Project IHR-93 Phase 2b, c, d Illinois Cooperative Highway and Transportation Research Program

Conducted by
THE STRUCTURAL RESEARCH LABORATORY DEPARTMENT OF CIVIL ENGINEERING ENGINEERING EXPERIMENT STATION UNIVERSITY OF ILLINOIS AT URBANA-CHAMPAIGN

in cooperation with the STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION and the U.S. DEPARTMENT OF TRANSPORTATION FEDERAL HIGHWAY ADMINISTRATION

UNIVERSITY OF ILLINOIS AT URBANA-CHAMPAIGN URBANA, ILLINOIS MARCH 1980
This report describes the results of overload tests on two 1/8th scale prestressed concrete bridge girders. The three span continuous girders were made with three pretensioned I-section girders, and were made continuous for live loads by means of reinforcement in the composite cast-in-place deck. The structures were originally built as part of a study of the long-term behavior of prestressed concrete bridges.

The structures were loaded with a model HS-type vehicle loading which was positioned to produce either high shears or approximately maximum moments in the various spans. The final tests produced very large deformations, and the maximum loads reached were in the range of 93 to 99 percent of the theoretical collapse loads.

The behavior of the structures is described in terms of measured load-deflection and load-reaction curves, together with descriptions of the cracking that occurred as a result of the several loadings. Comparisons of measured and computed deflections, reactions, and cracking moments are made. The service-load deflections were generally smaller than expected, and the cracking moments larger than expected.
# Table of Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Introduction</td>
<td>1</td>
</tr>
<tr>
<td>1.1 General Remarks</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Acknowledgements</td>
<td>2</td>
</tr>
<tr>
<td>2. Description of Test Specimens and Tests</td>
<td>3</td>
</tr>
<tr>
<td>2.1 Properties of Test Girders</td>
<td>3</td>
</tr>
<tr>
<td>2.2 Overload Test Conditions</td>
<td>13</td>
</tr>
<tr>
<td>3. Description of Results of Tests</td>
<td>19</td>
</tr>
<tr>
<td>3.1 Test on Model 1</td>
<td>19</td>
</tr>
<tr>
<td>3.2 Behavior of Model 2</td>
<td>43</td>
</tr>
<tr>
<td>4. Discussion of Results of Tests</td>
<td>68</td>
</tr>
<tr>
<td>4.1 Introduction</td>
<td>68</td>
</tr>
<tr>
<td>4.2 Strength of Model Beam MB-2</td>
<td>68</td>
</tr>
<tr>
<td>4.3 Strengths of Models 1 and 2</td>
<td>70</td>
</tr>
<tr>
<td>4.4 Flexural Cracking Moments</td>
<td>75</td>
</tr>
<tr>
<td>4.5 Discussion of Measured and Theoretical Reactions and Deflections</td>
<td>80</td>
</tr>
<tr>
<td>5. Summary and Conclusions</td>
<td>85</td>
</tr>
<tr>
<td>6. References</td>
<td>87</td>
</tr>
</tbody>
</table>
List of Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Model Girder Section Properties</td>
<td>8</td>
</tr>
<tr>
<td>2.2</td>
<td>Properties of Wire Reinforcement</td>
<td>14</td>
</tr>
</tbody>
</table>
# List of Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Elevation of Test Structures and Locations of Beams</td>
<td>4</td>
</tr>
<tr>
<td>2.2</td>
<td>Reinforcement in Pretensioned Girders</td>
<td>5</td>
</tr>
<tr>
<td>2.3</td>
<td>Cross Section of Model Girder</td>
<td>6</td>
</tr>
<tr>
<td>2.4</td>
<td>Cross Sections Showing Bar Reinforcement in Girders</td>
<td>7</td>
</tr>
<tr>
<td>2.5</td>
<td>Cast-in-Place Deck Dimensions and Reinforcement</td>
<td>9</td>
</tr>
<tr>
<td>2.6</td>
<td>Arrangement of Bearing Devices and Load Cells</td>
<td>10</td>
</tr>
<tr>
<td>2.7</td>
<td>Stress-Strain Curve for Pretensioned Reinforcement</td>
<td>12</td>
</tr>
<tr>
<td>2.8</td>
<td>Loading Equipment for Applied Loading Tests</td>
<td>15</td>
</tr>
<tr>
<td>2.9</td>
<td>Loading Positions and Magnitudes on Model 1</td>
<td>17</td>
</tr>
<tr>
<td>2.10</td>
<td>Loading Positions and Magnitudes on Model 2</td>
<td>18</td>
</tr>
<tr>
<td>3.1</td>
<td>Load-Deflection Curves for Test 1, Model 1, MB-3 Loaded</td>
<td>20</td>
</tr>
<tr>
<td>3.2</td>
<td>Load-Reaction Curves for Test 1, Model 1, MB-3 Loaded</td>
<td>21</td>
</tr>
<tr>
<td>3.3</td>
<td>Load-Deflection Curve for Test 2, Model 1, MB-4 Loaded</td>
<td>23</td>
</tr>
<tr>
<td>3.4</td>
<td>Load-Deflection Curves for Test 2, Model 1, MB-4 Loaded</td>
<td>24</td>
</tr>
<tr>
<td>3.5</td>
<td>Load-Reaction Curves for Test 2, Model 1, MB-4 Loaded</td>
<td>25</td>
</tr>
<tr>
<td>3.6</td>
<td>Load-Deflection Curves for Test 3, Model 1, MB-4 Loaded</td>
<td>26</td>
</tr>
<tr>
<td>3.7</td>
<td>Load-Reaction Curves for Test 3, Model 1, MB-4 Loaded</td>
<td>27</td>
</tr>
<tr>
<td>3.8</td>
<td>Load-Reaction Curve for Test 3, Model 1, MB-4 Loaded</td>
<td>28</td>
</tr>
<tr>
<td>3.9</td>
<td>Load-Deflection Curves for Test 4, Model 1, MB-5 Loaded</td>
<td>30</td>
</tr>
<tr>
<td>3.10</td>
<td>Load-Reaction Curves for Test 4, Model 1, MB-5 Loaded</td>
<td>31</td>
</tr>
<tr>
<td>3.11</td>
<td>Load-Deflection Curves for Test 5, Model 1, MB-5 Loaded</td>
<td>32</td>
</tr>
<tr>
<td>3.12</td>
<td>Load-Reaction Curves for Test 5, Model 1, MB-5 Loaded</td>
<td>33</td>
</tr>
<tr>
<td>3.13</td>
<td>Load-Deflection Curves for Test 6, Model 1, MB-3 Loaded</td>
<td>34</td>
</tr>
<tr>
<td>3.14</td>
<td>Load-Reaction Curves for Test 6, Model 1, MB-3 Loaded</td>
<td>35</td>
</tr>
<tr>
<td>3.15</td>
<td>Photograph Showing Damage to Diaphragm at North Pier, Model 1</td>
<td>37</td>
</tr>
<tr>
<td>3.16</td>
<td>Load-Deflection Curves for Test 7, Model 1, MB-5 Loaded</td>
<td>38</td>
</tr>
<tr>
<td>3.17</td>
<td>Load-Reaction Curve for South Pier, Test 7, Model 1, MB-5 Loaded</td>
<td>39</td>
</tr>
<tr>
<td>Figure</td>
<td>Page</td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>------</td>
<td></td>
</tr>
<tr>
<td>3.18</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>3.19</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>3.20</td>
<td>44</td>
<td></td>
</tr>
<tr>
<td>3.21</td>
<td>45</td>
<td></td>
</tr>
<tr>
<td>3.22</td>
<td>46</td>
<td></td>
</tr>
<tr>
<td>3.23</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>3.24</td>
<td>49</td>
<td></td>
</tr>
<tr>
<td>3.25</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>3.26</td>
<td>51</td>
<td></td>
</tr>
<tr>
<td>3.27</td>
<td>53</td>
<td></td>
</tr>
<tr>
<td>3.28</td>
<td>54</td>
<td></td>
</tr>
<tr>
<td>3.29</td>
<td>55</td>
<td></td>
</tr>
<tr>
<td>3.30</td>
<td>56</td>
<td></td>
</tr>
<tr>
<td>3.31</td>
<td>58</td>
<td></td>
</tr>
<tr>
<td>3.32</td>
<td>59</td>
<td></td>
</tr>
<tr>
<td>3.33</td>
<td>60</td>
<td></td>
</tr>
<tr>
<td>3.34</td>
<td>62</td>
<td></td>
</tr>
<tr>
<td>3.35</td>
<td>63</td>
<td></td>
</tr>
<tr>
<td>3.36</td>
<td>64</td>
<td></td>
</tr>
<tr>
<td>3.37</td>
<td>65</td>
<td></td>
</tr>
<tr>
<td>3.38</td>
<td>66</td>
<td></td>
</tr>
<tr>
<td>3.39</td>
<td>67</td>
<td></td>
</tr>
<tr>
<td>4.1</td>
<td>71</td>
<td></td>
</tr>
<tr>
<td>4.2</td>
<td>73</td>
<td></td>
</tr>
<tr>
<td>4.3</td>
<td>76</td>
<td></td>
</tr>
<tr>
<td>4.4</td>
<td>78</td>
<td></td>
</tr>
<tr>
<td>4.5</td>
<td>82</td>
<td></td>
</tr>
</tbody>
</table>
1. Introduction

1.1 General Remarks

This report describes the results of a series of tests in which large overloads were applied to two 1/8-th scale model three-span beams. The structures are models of lines of beams from a highway bridge, and include cast-in-place composite decks on precast, pretensioned girders. The construction and long-term behavior of the models is described in Ref. 1. This report is essentially a follow-up to the work previously reported, and the presentation and discussion of the results of tests in which large loads were applied and large deformations induced occupy most of this report.

The models had originally been built primarily to provide information about the long-term behavior of prestressed concrete members under rather extreme conditions. The use of 1/8-th scale structures led to sections less than 1 in. (25 mm) thick, which caused very high initial rates of creep and shrinkage. As a result of these high rates, a very large differential shrinkage strain between the deck and girder concretes could be obtained in a short period of time. The measured strains and cambers in the model structures were then used in part of the validation process for a computer program to calculate deformations and stresses caused by creep, shrinkage, and relaxation in prestressed bridges, as was first reported in Ref. 2. That work was extended in Ref. 3 and 4.

The model structures were 1/8-th scale models of beams and deck used in a prestressed concrete bridge carrying I-57 in Douglas County, Illinois. The prototype structure had been instrumented for long-term strain and camber measurements, and the bridge and the measured deformations have been reported in Ref. 5.

The models described here were built in 1970, and strains and camber were observed for about two years. The overload tests were conducted in the fall of 1975.

Chapter 2 of this report describes the model structures which were tested, and Ref. 1 contains more details on their construction. The overload test conditions are also described, along with descriptions of the
loading equipment. Chapter 3 is a presentation of the test data, including measured deflections, changes in reaction, and crack patterns. Chapter 4 contains a discussion of the test results, and compares the maximum applied loads with the theoretical collapse loads, and the observed cracking moments with the predicted cracking moments. Chapter 5 is a brief summary and conclusions section.

1.2 Acknowledgements

This work was conducted as part of the Illinois Cooperative Highway and Transportation Research Program, Project IHR-93, "Field Investigation of Prestressed Reinforced Concrete Highway Bridges," by the Department of Civil Engineering, in the Engineering Experiment Station, University of Illinois at Urbana-Champaign, in cooperation with the Illinois Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views of or policies of the Illinois Department of Transportation or of the Federal Highway Administration. This report does not constitute a standard, specification or regulation.
2. Description of Test Specimens and Tests

2.1 Properties of Test Girders

The two test specimens are described in detail in Ref. 1, and some of that information is repeated here. Each three-span specimen was 27 ft 4 3/8 in. (8341 mm) long, as shown in Fig. 2.1. Precast girders were used in all three spans. All girders had the same length, and the differences in the span lengths shown in Fig. 2.1 result from the positions of the end bearings and the dimensions of diaphragms at the interior piers. Details of the girders are shown in Figs. 2.2 to 2.4, and the girder cross section properties are summarized in Table 2.1.

The girders were precast pretensioned concrete and all were originally simply supported. The cast-in-place deck contained reinforcement for negative continuity moments near the interior piers, and diaphragms at the interior piers supplied the compression concrete near the bottoms of the girders. Figure 2.5 shows the deck reinforcement. Once the deck is in place and cured, the structure responds to additional loads as a three-span continuous structure having a composite cross section consisting of the girder plus deck.

The effectiveness of this continuity connection at low load levels was demonstrated in Ref. 1, where measured and computed influence lines for deflection and reaction are compared, with nearly perfect agreement being found. It was recognized that the load levels were quite small during those test, and part of the objective of the tests reported here was to investigate the effectiveness of the same connections at higher load levels.

The arrangements of the supporting reactions are shown in Fig. 2.6. Three of the supports included small load cells so that changes in reactions could be measured during the course of the tests. The fourth support was a hinged support rather than a roller, and did not contain a load cell. The support was the full width of the deck and diaphragm. This was done in order to insure the lateral and torsional stability of the test specimens.

Two additional girders and decks were fabricated and tested to failure. Both were simply supported single span structures, and both are described in Ref. 1. Some of the results of the tests of the second girder, MB-2, will be included in this report, as its construction was identical to that of the
Fig. 2.1 Elevation of Test Structures and Locations of Beams
Fig. 2.2 Reinforcement in Pretensioned Girders
Fig. 2.3 Cross Section of Model Girder
Fig. 2.4 Cross Sections Showing Bar Reinforcement in Girders
Table 2.1 Model Girder Section Properties

Non-Composite I-Section

\[
\begin{align*}
A &= 8.90 \text{ in.}^2 \\
I &= 35.19 \text{ in.}^4 \\
c_b &= 2.64 \text{ in.} \\
c_t &= 3.36 \text{ in.} \\
w_t &= 9.30 \text{ lb/ft}
\end{align*}
\]

Composite Section with Deck, \(E_g = E_d\)

\[
\begin{align*}
A &= 18.36 \text{ in.}^2 \\
I &= 102.0 \text{ in.}^4 \\
c_{bg} &= 4.60 \text{ in.} \\
c_{tg} &= 1.40 \text{ in.} \\
c_{td} &= 2.28 \text{ in.} \\
w_t &= 19.1 \text{ lb/ft}
\end{align*}
\]

1 in. = 25.4 mm

1 lb/ft = 1.488 kg/m (mass)

1 lb/ft = 14.59 N/m (force)
Fig. 2.5 Cast-in-Place Deck Dimensions and Reinforcement

Note: The 8-#G11 Bars Occur Only Over The Supports

#11 Gage Wire = 0.1205 in. (3.06 mm) Diam. Length of #G11 Bars = 32 or 54 in. (813 or 1372 mm)
(a) Typical Roller Support

(b) Reaction at Hinged Support, Showing Diaphragm

Fig. 2.6 Arrangement of Bearing Devices and Load Cells
girders described here. The first had quite different pretensioned rein­forcement, and the results of that test are not relevant to this discussion.

The models were cast with fine-aggregate concrete, because of the small dimensions of the cross sections and because of the congestion of the rein­forcement. The 28 day compressive strengths were between 6.0 and 6.4 k/in.\(^2\) (41.4 and 44.1 N/mm\(^2\)) in most cases, and Young's modulus ranged from 3.1 to 3.7 x 10\(^6\) lb/in.\(^2\) (21.4 to 25.5 kN/mm\(^2\)), at the same age. The values of Young's modulus are rather low, considering the compressive strength, probably because of the absence of aggregate larger than that passing a #4 (4.75 mm) sieve in the girders and a 3/8 in. (9.5 mm) sieve in the decks.

A few concrete specimens were tested at the time of the overload tests. There was no apparent strength gain between tests at 28 days and at about 5 years, probably because the 4 by 8 in. (100 by 200 mm) cylinders were able to dry out very completely in a short time, effectively ending the hydration process. The value of \(f'_C\) ranged from only 6.0 to 6.3 k/in.\(^2\) (41.4 to 43.4 N/mm\(^2\)) at 5 years for specimens of deck and girder concrete from the model designated as Model 2. The tensile strength, as determined by split-cylinder tests, was 405 lb/in.\(^2\) (2.8 N/mm\(^2\)) for girder concrete and 389 lb/in.\(^2\) (2.7 N/mm\(^2\)) for deck concrete, also at 5 years. A compressive strength of 6,000 lb/in.\(^2\) (41.4 N/mm\(^2\)) will be used in all calculations in this report. Measured values of \(E_C\) ranged from 2.9 to 3.35 x 10\(^6\) lb/in.\(^2\) (20 to 23.1 kN/mm\(^2\)) and averaged 3.13 x 10\(^6\) lb/in.\(^2\) (21.6 x kN/mm\(^2\)).

The prestressed reinforcement had the stress-strain curve shown in Fig. 2.7. The material for each strand was made by splitting a 1/4 in. (6.35 mm) galvanized Tiger Brand 3 x 19 Amgal Oceanographic Rope* into its three component strands and using each strand as an individual pretension­ing strand. Each of the three strands was made of 19 wires. The stress­strain curve is similar to that for 7-wire prestressing strand in many respects, although the Young's modulus and proportional limits were lower than are normally found. The rope had an advertised breaking stress of 246 k/in.\(^2\) (1696 N/mm\(^2\)), and had a measured average breaking stress of one strand (rather than the complete 3-strand rope) of 276.6 k/in.\(^2\) (1907 N/mm\(^2\)).

* Trade mark of United States Steel Co.
Fig. 2.7 Stress-Strain Curve for Pretensioned Reinforcement

E = 22,500 kips/in.²
(in air)
(155.1 kN/mm²)
Each strand had an area of 0.00913 in.$^2$ (5.89 mm$^2$).

The non-prestressed reinforcement was annealed wire. It was purchased as black annealed wire which had been straightened, and it was then reannealed at the University of Illinois in order to produce steel with a sharply defined yield point and a long yield plateau. The strength properties of the several sizes of wire used in the different locations are summarized in Table 2.2.

Since the use of a reduced-scale model distorts the normal relationships between dead load moments and span, considerable added mass was required in order to make the dead load stresses the same in the model as they were in the prototype. Two rows of concrete blocks, with masses of about 90 lb (41 kg) and 118 lb (54 kg), respectively, for the upper and lower row, were suspended below the beam. There were six sets of blocks per span so that they simulated a uniformly distributed load reasonably well. The upper row of blocks was added to the girder section as soon as it was in place on the piers, so that the dead load stresses would correspond to those in the bare girder of the prototype, and the lower row was added when the deck was cast. The total mass of each span including all of the blocks was 1.44 kips (653 kg). The modeling relationships that led to the selection of these particular added masses are discussed in Ref. 1.

2.2 Overload Test Conditions

The overloads were applied as modeled AASHTO HS-type vehicle loadings representing a 3-axle truck. This is shown schematically in Fig. 2.8, where the load applied with the hydraulic ram can be traced down to the structure. The relative axle loads are 1:4:4, starting from the front axle. The 21 in. (533 mm) spacing is scaled directly from the AASHTO minimum spacing of 14 ft (4.27 m).

The loads were positioned to produce either maximum positive moment or relatively high shear forces plus large negative moments in each of the spans of the two models. The positioning of the load was constrained to particular positions because the ram was supported from a steel frame which was bolted to the floor. There were bolt holes available in the floor only at 18 in.
Table 2.2 Properties of Wire Reinforcement

<table>
<thead>
<tr>
<th>Gage No.</th>
<th>Diam</th>
<th>Area</th>
<th>$f_y$ (Ave.)</th>
<th>$f_u$ (Ave.)</th>
<th>Location Used</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in.</td>
<td>mm</td>
<td>in.²</td>
<td>k/in.²</td>
<td>k/in.²</td>
</tr>
<tr>
<td></td>
<td>mm²</td>
<td>k/in.²</td>
<td>N/mm²</td>
<td>k/in.²</td>
<td>N/mm²</td>
</tr>
<tr>
<td>13</td>
<td>0.0915</td>
<td>2.32</td>
<td>0.00658</td>
<td>4.25</td>
<td>39.6</td>
</tr>
<tr>
<td>15</td>
<td>0.0720</td>
<td>1.83</td>
<td>0.00407</td>
<td>2.63</td>
<td>45.8</td>
</tr>
<tr>
<td>13</td>
<td>0.0915</td>
<td>2.32</td>
<td>0.00658</td>
<td>4.25</td>
<td>37.8</td>
</tr>
<tr>
<td>13</td>
<td>0.0915</td>
<td>2.32</td>
<td>0.00658</td>
<td>4.25</td>
<td>37.4</td>
</tr>
<tr>
<td>11</td>
<td>0.1205</td>
<td>3.06</td>
<td>0.0114</td>
<td>7.35</td>
<td>44.8</td>
</tr>
</tbody>
</table>
Fig. 2.8 Loading Equipment for Applied Loading Tests
(457 mm) spacings, and the ram had to be positioned over the bolt holes.

Seven tests were done on Model 1 and six on Model 2. The loading positions are shown in Figs. 2.9 and 2.10. Some of these tests were conducted in two or three phases, with load applied to some level less than the maximum and then released, followed by reloading one or more days later. The first series of loadings were generally to 5 kips (22.2 kN) total applied load, with larger loads applied in the final tests on each model. The peak loads are also shown in Figs. 2.9 and 2.10.

In all cases where the loads were applied to produce relatively high shears, the loading position could have been 18 in. (457 mm) closer to the support. This would have placed the first load at about the girder depth or less from the center of the reaction, resulting in very high shear forces. However, because of the very small aggregate used in these structures it was not felt that meaningful information about the shear capacity would be obtained even if shear failures were induced. Consequently, the loading positions were chosen so that the predominate response would be in flexure with some influence of shear, rather than primarily in shear.

None of the tests ended with the total destruction of the loaded span because the loading system, and especially the "truck", was not able to accommodate the very large deflections and changes in slope that occurred late in the final tests.
Fig. 2.9 Loading Positions and Magnitudes on Model 1
Fig. 2.10 Loading Positions and Magnitudes on Model 2
3. Description of Results of Tests

3.1 Test on Model 1

As was noted in the previous section, seven separate tests were conducted on Model 1, with loads positioned to produce relatively high shear forces at each end of each span, plus a final test where the loads were positioned to produce approximately the maximum positive moment in the interior span. The loading positions are identified in Fig. 2.9.

The majority of the test data presented here will consist of graphs of load versus deflection and load versus reaction. These two different kinds of information, plus appropriate comments about the initiation and growth of cracking, can give a relatively complete understanding of the behavior of a structure which is subjected to multiple tests to very high overload levels.

Each of the load-deflection and load-reaction graphs also includes a broken straight line which represents the relationship predicted using an elastic analysis. The measured and predicted deflections and reactions will be discussed and compared in Chapter 4 of this report.

Fig. 3.1 contains load-deflection curves for the first test on Model 1, in which the load was positioned to produce a relatively high shear force near the north abutment of the three span structure. The curves are typical of those observed when the particular member was loaded for the first time. The initial response in the loaded span was quite linear up to loads of about 2.0 kips (8.9 kN), followed by relatively great reductions in stiffness. The first positive moment cracks, in the pretensioned I-section girder, were found when the load was 3.0 kips (13.3 kN), and many more were found at the later loading stages.

Fig. 3.2 shows the loads versus measured reactions. Only three of the four reactions were measured, so there are always some limitations to this information, but a number of useful observations can still be made.

The north abutment reaction was nearly a linear function of the applied load. The south pier and south abutment load-reaction curves were distinctly non-linear. Both of these reactions are direct functions of the negative moment developed at the north pier. The fact that the south pier reaction did not change after the load reached 4.0 kips (17.8 kN) probably indicates that the north pier section had yielded, and could transmit no additional force.
Fig. 3.1 Load-Deflection Curves for Test 1, Model 1, MB-3 Loaded
Fig. 3.2 Load-Reaction Curves for Test 1, Model 1, MB-3 Loaded
The second test on Model 1 was a loading on the south span which was about the same as that in the north span in the first test. The comparisons between these first two tests may be important, as the south span beam had a defect when it was cast, and the test was partially intended to investigate the efficiency of the repair. As noted in Ref. 1, there was a hole in the web of beam MB-4, in a long space below the draped pretensioned strands at the abutment end of the beam. This hole had been filled with concrete, and the beam given some additional curing time, before the prestressing force was transferred to the beam.

Figs. 3.3 to 3.5 show the load-deflection and load-reaction curves for Test 2. The results of the first two test were generally similar, although there were differences in details. The south span beam deflected more than the north span beam had, and this may be the result of either some significant difference in the properties of the beams, or a result of damage done to the structure during the first test, or both. In both tests, the initial positive moment flexural cracks were found at the same load, 3.0 kips, (13.3 kN), and in both tests the reaction at the end of the bridge away from the loaded span decreased during the last phases of the tests.

The crack patterns near the outer ends of beams MB-3 and MB-4 seem to indicate that the repaired end of MB-4 was as strong as that of the beam which did not have the defects. Neither positions of the cracks nor the path of the cracks appear to have been influenced by the patching.

Figs. 3.20 to 3.22 show the cracks which were caused in the beams of Model 1 by the various loadings. The figures are not marked to indicate which loading caused which crack but in many cases that will be fairly obvious. All cracks in the girders were caused by loads applied in that span. Deck cracks in adjacent spans often occurred. The deck cracks are marked on the edge of the deck. Many deck cracks extended only part of the way across the deck, and these drawings are not accurate representations of cracking on the surface of the deck. They do represent the average spacings and the extent of the cracking.

Figs. 3.6 to 3.8 show load-deflection and load-reaction curves for the third test on Model 1, in which the south span beam, MB-4, was loaded the second time. The load-deflection curve for the loaded span is quite different from that for the second test, in that there was a major reduction in stiffness at a load of 1.0 kips (4.45 kN), while the major reduction during the first loading occurred above 2 kips (8.9 kN).
Fig. 3.3 Load-Deflection Curve for Test 2, Model 1, MB-4 Loaded
Fig. 3.4 Load-Deflection Curves for Test 2, Model 1, MB-4 Loaded
Fig. 3.5 Load-Reaction Curves for Test 2, Model 1, MB-4 Loaded
Fig. 3.6 Load-Deflection Curves for Test 3, Model 1, MB-4 Loaded
Fig. 3.7 Load-Reaction Curves for Test 3, Model 1, MB-4 Loaded
Fig. 3.8 Load-Reaction Curve for Test 3, Model 1, MB-4 Loaded
A reasonable interpretation of this is that the first test on a given span causes positive moment cracking only after the bottom fiber stresses overcome both the precompression and the tensile capacity of the concrete, while in the second loading of the same beam only the precompression exists since the tensile capacity has been destroyed by the cracking in the first test.

This pattern of having a major reduction in stiffness at a considerably lower loading during a second test on a span is repeated in all cases.

Tests 4 and 5 were loadings to 5 kips (22.2 kN) on the interior span, MB-5, with the loads positioned to produce relatively high shear forces. The load-deflection and load-reaction curves are shown in Figs. 3.9 to 3.12. The deflections in the loaded span were much smaller than occurred in the end spans when they were loaded, as is to be expected.

There is again the pattern of having the major reduction in stiffness occur at a considerably lower load in the second of the two tests on the interior span. In spite of this difference the maximum deflections imposed by the two tests were comparable, if the residual deflections resulting from the first test are not considered.

The residual deflections were not large in these tests and some of the residual was recovered in a few hours or days after the end of the test. In the tests conducted in two or three phases, the residuals plotted at the beginning of the later phases included the recovery.

In the sixth test the loads were positioned to produce relatively high shears at the interior end of the north span, MB-3. The maximum load reached was 7.03 kips (31.3 kN) at a deflection of 1.54 in. (39.1 mm). Load-deflection and load-reaction curves are shown in Figs. 3.13 and 3.14. By the time this test was conducted, there was enough accumulated damage to the structure to cause the structure to respond quite differently than it did in the first test, when the same beam was loaded.

The deflection at midspan of the loaded span at 5 kips (22.2 kN) was smaller than in the first test at the same load, as is reasonable considering the different positions of the loads in the two tests. However, the deflections in the south span, the second span from the loaded span, remained so small in the later test that they could not be plotted, with a maximum movement of 0.002 in. (0.05 mm) downward. In the first test, the south span deflection was small, but at 0.014 in. (0.36 mm) was measurable. The elastic deflections for the two loading cases are nearly the same.
Fig. 3.9 Load-Deflection Curves for Test 4, Model 1, MB-5 Loaded
Fig. 3.10 Load-Reaction Curves for Test 4, Model 1, MB-5 Loaded
Fig. 3.11 Load-Deflection Curves for Test 5, Model 1, MB-5 Loaded
Fig. 3.12 Load-Reaction Curves for Test 5, Model 1, MB-5 Loaded
Fig. 3.13 Load-Deflection Curves for Test 6, Model 1, MB-3 Loaded
Fig. 3.14 Load-Reaction Curves for Test 6, Model 1, MB-3 Loaded
The measured reaction at the south pier changed at a much lower rate with applied load in the later test, although again the theoretical elastic decreases in reaction for the two loading positions are comparable. In addition to the change in reaction being smaller for a particular load, the reaction at the south pier no longer changed when the load in north span exceeded 5 kips (22.2 kN), and in addition the interior span deflection changed very little after the load exceeded this value. The maximum change in the south reaction was about 0.05 kips (2.2 kN), and undoubtedly corresponds to the reaction change induced by the negative yield moment acting at the north pier. Negative moment cracks became very large at the north pier at this loading, and considerable damage was done to the diaphragm over the north pier. Some of the damage can be seen in the photograph in Fig. 3.15.

Figs. 3.16 to 3.18 show the load-deflection and load-reaction curves for the final test on Model 1, in which the interior span was loaded, with the load positioned to produce a high positive moment. The maximum load reached was 7.44 kips (33.1 kN), with a midspan deflection of 2.19 in. (55.6 mm), without producing failure. The test ended with a failure of the loading equipment, which was not able to undergo the very large changes in slope in the beam which accompanied the maximum deflection. At the end of the test, the "truck" slid and rolled off if its steel bearing plates with a crash, which displaced the load cell and damaged the end of the hydraulic ram. The test was not repeated, as the added benefits appeared small, and the beam had already been severely damaged.

The release of the load which accompanied the failure of the loading equipment was violent. The beam recovered much of the deflection that had been imposed, and the vertical accelerations were so great that the wires supporting the concrete blocks suspended below the beam broke in four of the six locations in the interior span. This disrupted the deflection measuring system, and in addition the bridge then had a considerably smaller dead load moment at the end of the test than at the beginning. Some tensile cracks then developed in the deck near midspan.

It is clear from the form of the load-deflection curve in Fig. 3.16 that the flexural strength of the span had not been reached, as the load was still continuing to increase with increasing deflection rather than being in a range where small increases in load caused unduly large increases in deflection.
Fig. 3.15  Photograph Showing Damage to Diaphragm at North Pier, Model 1
Fig. 3.16 Load-Deflection Curves for Test 7, Model 1, MB-5 Loaded
Fig. 3.17 Load-Reaction Curve for South Pier, Test 7, Model 1, MB-5 Loaded
Fig. 3.18 Load-Reaction Curves for Test 7, Model 1, MB-5 Loaded
As an additional indication that the beam was not about to fail when the test ended, the load-deflection curve for a single span beam, MB-2, is shown in Fig. 3.19. The deflection at peak load was about 3.0 in. (76 mm) when the first of the pretensioned strands broke at the section of maximum moment. The deflections at failure would be expected to be about the same in the simply supported and continuous cases since the negative moment capacities were not high, and the negative moment sections were quite ductile. Additional details of the tests of MB-2 are given in Ref. 1.

The load-deflection curves for the unloaded spans give useful information about the behavior of the structure as the peak load was approached. The south span upward deflections were very small for the first load increment, and then increased fairly rapidly with load until 5.0 kips (22.2 kN) was reached. For greater loads, the additional deflections were much smaller, which probably indicates that the negative moment section at the south pier had yielded.

The north span deflections developed quite differently as the load was applied, although the theoretical elastic deflections are comparable for the two spans. In the north span, the deflection remained quite small until a load of 5.0 kips (22.2 kN) had been exceeded, and there was then a substantial increase in the deflection. It is believed that this was a result of damage inflicted during the previous test, when the north span was subjected to a large overload. At the end of the sixth test there was a large crack in the deck (no measurement can be found, but the crack was in excess of 1/8 in. (3 mm) after the structure was unloaded). When the structure was unloaded at the end of the sixth test, the deck crack remained open, and when the end slope returned to approximately zero, the girder was partially pulled out of its encasement in the pier diaphragm, leaving a gap between the ends of the beams. Upon reloading, there was no compression zone for the negative moments at the pier, and consequently the moment, deflection, and reaction remained small until enough deformation had occurred to close up the gap and re-establish the compression zone.

The load-reaction curves, which are direct measures of the negative moments at the interior piers for this loading, are consistent with load-deflection curves, and with the explanation of the behavior which is contained in the previous paragraph.

The negative moment regions suffered serious distress during the final
Fig. 3.19 Load-Deflection Curve for Simply Supported Beam MB-2

D. L. = 1.44 kips Force (6.41 kN)
Max \( \Sigma P \) = 5.65 kips (25.1 kN)

Midspan Deflection, inches

Midspan Deflection, mm

14" (356) 7" (178) 21" (533 mm)
test, and after unloading, the major cracks immediately over the interior piers were 4 to 6 mm (0.16 to 0.24 in.) wide. The bottom faces of the girders appeared to have slipped out of the diaphragms by similar amounts, considering the total movement on the two sides of a diaphragm.

These great deformations are the result of very large strains in the reinforcement. There is certainly some bond slip along the bars, away from the cracks, but it does not appear that the slip propagated to the ends of the deck bars. There is no indication that any bars broke.

The crack patterns shown in Figs. 3.20 to 3.22 illustrate several kinds of cracks. There are vertical cracks, which are initiated by flexural stresses, and propagated by flexural stresses. There are many cracks which start at the bottom surface of the girder, but become inclined as they travel up into the beam web. These cracks are initiated by flexural stresses, but their paths are greatly influenced by shear stresses, and these cracks are often referred to as "flexure-shear" cracks. There are inclined cracks near the supports which do not reach either surface of the beam, but rather are restricted to the beam web. Some of these cracks appear to be associated with negative moment cracking in the deck, although none actually reach to the deck. Others of these do not appear to be associated with any flexural cracking, and these are initiated when the inclined principal tension in the beam web reaches the direct tensile strength of the concrete. These cracks are often referred to as "web-shear" cracks.

The shear cracks remained small throughout the tests. The shear reinforcement, which was scaled directly from that provided in the prototype girders, was adequate to prevent shear distress beyond initiation of cracking.

Some of the cracking in the interior span, MB-5, deserves additional comment. There was a long sloping crack at the south quarter-point in the span at the end of the tests, as can be seen in Fig. 3.21. This crack, which is inclined only slightly above the axis of the beam and has the wrong slope for a shear crack, is parallel to and along the path of the two draped strands. It appears that a sliding plane was being developed, but the deformations across this plane were still very small when the test ended. A similar crack pattern appeared to be starting to develop near the north quarter-point.

3.2 Behavior of Model 2

Six separate tests were conducted on Model 2, as noted previously. The locations of the loads and the maximum loads reached are summarized in Fig. 2.10.
Fig. 3.20  Cracking in Beam MB-4
Load-deflection and load-reaction curves are shown in Figs. 3.23 to 3.35, and Figs. 3.37 to 3.39 are drawings of the patterns of cracks in the three spans of the structure.

The tests of Model 2 were often in two or three stages. This was partially a result of it actually being tested before Model 1, with new, inexperienced student employees who simply took longer to apply a given load and record all of the data at the beginning of the test period than at the end of tests. The use of two or three stages of loading does not change the final result, but in some instances makes the interpretation of the results of the test more difficult. This may be especially true when residual reactions are considered.

The first loading produced relatively high shear stresses at the interior end of the north span, beam MB-7. Negative moment cracks were found at a load of 3 kips (13.3 kN), and positive moment cracks at 3.5 kips (15.6 kN). There was a major reduction in stiffness when the load exceeded 3.0 kips (13.3 kN), as illustrated by the load-deflection curve shown in Fig. 3.23. A maximum load of 4 kips (17.8 kN) was reached in the first phase of loading, after which the span was unloaded. It was later reloaded to a maximum load of 4.94 kips (22.0 kN). The reloading curve followed the unloading curve most of the way to the previous maximum load, and for larger loads the two curves simply blend together. The residual deflections were relatively small in both cases, with residuals of about 1/6 the maximum deflection in each of the two parts of the tests.

The load-reaction curves plotted in Fig. 3.24 indicate that the negative moment section at the north pier must have yielded and suffered additional damage leading to small decrease in moment as the peak load was approached, as the changes in reaction at the south pier and abutment decreased slightly at the last load increment.

The second test was also a loading of the north span girder, MB-7, with the load positioned to produce high shear near the north abutment. The loading also caused much higher positive moments in the central part of the span than had occurred in the previous test. The load-deflection curves are shown in Fig. 3.25 and the load-reaction curves in Fig. 3.26. It is seen that there was a marked reduction in stiffness in the loaded span at a much smaller load than in the first test, with a significant change occurring between 0.5 and 1.0 kips (2.2 and 4.4 kN). The first loading had produced many cracks, and these apparently started reopening at a load less than 1.0 kips (4.4 kN). The peak
Fig. 3.23 Load-Deflection Curves for Test 1, Model 2, MB-7 Loaded
Fig. 3.24 Load-Reaction Curves for Test 1, Model 2, MB-7 Loaded
Fig. 3.25 Load-Deflection Curves for Test 2, Model 2; MB-7 Loaded
Fig. 3.26 Load-Reaction Curves for Test 2, Model 2, MB-7 Loaded
deflection was not determined, as the dial gage went off-scale shortly before the end of the test.

The unloaded spans behaved about the same as in the previous test, in that the deflections and reactions stopped changing before the maximum load was reached, again indicating that the negative moment section at the north pier had yielded. The residual deflections were quite small. The residual reactions were not consistently measurable, and were just at the limits of the sensitivity of the load cells.

The third test was a loading of the interior span for high shear near the north end of the span, and a maximum load of 5.50 kips (24.4 kN) was reached in a test conducted in three stages. Load-deflection and load-reaction curves are shown in Figs. 3.27 and 3.28, respectively. Positive moment cracks were found when the load was 3.0 kips (13.3 kN), although they undoubtedly started forming when the load was only slightly larger than 2 kips (8.9 kN), considering the form of the load-deflection curve for the interior span.

There is no indication in either the deflections or in the reactions measured at the abutments that the negative moment sections at the interior piers yielded in this test, as both indicators increased rather uniformly throughout the test.

The fourth test was also a loading on the interior span, MB-8, with the load positioned to provide relatively high shear at the south end of the span. The maximum load reached was 6.0 kips (26.7 kN). The test was conducted in two phases.

The initial part of the test followed patterns already pointed out in earlier tests. There was a major reduction in stiffness, as shown in Fig. 3.29, at a load of about 1.0 kip (4.4 kN), while the reduction had occurred at twice that load in the previous test on the same span. In this case the deflections increased fairly uniformly with load until 5.5 kips (24.5 kN) was exceeded, and there was fairly abrupt decrease in stiffness at that load level. The end span deflections did not change much in this range, although one of the dial gages was off-scale at the end of the test.

The reactions, which are plotted versus load in Fig. 3.30, indicate some major changes in behavior as the final one or two steps of load were applied. These do not seem to be consistent with the measured deformations, except possibly at the south abutment, and some of the abrupt change may have been due to some unknown instrumentation problem. The residual reactions at the
Fig. 3.27 Load-Deflection Curves for Test 3, Model 2, MB-8 Loaded
Fig. 3.28 Load-Reaction Curves for Test 3, Model 2, MB-8 Loaded
Fig. 3.29 Load-Deflection Curves for Test 4, Model 2, MB-8 Loaded
Fig. 3.30 Load-Reaction Curves for Test 4, Model 2, MB-8 Loaded
end of the test, which have not been plotted, were larger than usual at the south pier and abutment and may be an indication of some electrical measurement problem.

Test 5 was a loading in the south span, MB-6, to produce a high shear near the north end of the span. The maximum load reached was 6.0 kips (26.7 kN), in a single stage. The load-deflection curves are given in Figs. 3.31 and 3.32. The curve for the loaded span is formed of three separate approximately linear parts extending from 0 to 2.0 kips (8.9 kN), a second portion to 4.5 kips (20.0 kN), and final linear portion at a slightly lower slope. The deflection at peak load was 1.065 in. (27.05 mm), which was much greater than occurred in the interior span under the same load, as is reasonable considering the different restraint conditions. At the end of the test, there was no indication that the span was reaching a collapse mechanism condition, as the load-deflection curve still had a significant slope. The residual deflections after unloading were not recorded.

The deflections of the unloaded spans give some definite indications of the behavior of the structure during the test, and its condition at the beginning of the test. The north span deflections remained almost zero throughout the test, which indicates that the north pier negative moment section was not transmitting negative moments. This is quite different from the situation in the first test, Fig. 3.23, in which the south span developed small but quite measureable deflections when the north span was loaded.

The load-deflection curve for the interior span has three separate phases. At low loads, the deflection developed very slowly. For loads above 2.0 kips (8.9 kN), the rate of deflection with increasing load was much greater. Then for loads above 5.0 kips (22.2 kN), there was a substantial decrease in the rate of deflection versus load. A reasonable interpretation of this sequence may be that initially there was an open crack at the bottom of the girder over the south pier (as a residual deformation from the previous tests), and consequently the section was not able to transmit much moment. As the load and deformation increased, this crack eventually closed, and the section then developed a significant bending moment which reached the yield capacity before the end of the test. The reaction at the north pier would have been a sensitive indicator of this behavior, but there was no reaction measurement there. The north abutment reaction, Fig. 3.33, also indicated that practically no moment was transmitted by the section at the north pier.
Fig. 3.31 Load-Deflection Curve for Loaded Span in Test 5, Model 2, MB-6 Loaded
Fig. 3.32 Load-Deflection Curves for Test 5, Model 2, MB-6 Loaded
Fig. 3.33 Load-Reaction Curves for Test 5, Model 2, MB-6 Loaded
The sixth and final test on Model 2 was a loading for maximum positive moment in the interior span. A peak load of 7.39 kips (32.9 kN) was reached in three loading stages without producing failure of the structure. At the end of the test, the loading equipment again failed to cope with the large deflections and slopes, and the "truck" slipped off of the loading pads. By the end of the test the interior span deflection was 2.92 in. (74.2 mm), which is nearly as great as that of the simply supported beam MB-2 at the time the first strands fractured. The residual deflections were not measured because the gages were knocked out of place when the loading system failed.

The load-deflection curve, Fig. 3.34, for the interior span indicates that the load was very near the maximum that the structure could sustain when the loading equipment problems occurred. The last several increments of load each produced substantial increases in deflection with very small increases in the applied load. At low load levels, the deflection stiffness was about 5.0 kips of load per 1.0 in. of deflection (21.9 kN per 25 mm), while at the end of the test the stiffness was about 0.35 kips of load per 1.0 in. of deflection (1.53 kN per 25 mm). The relative slopes are comparable to those in the simply supported beam MB-2 whose load-deflection curve is shown in Fig. 3.19.

Load-reaction curves are given in Fig. 3.35. Fig. 3.36 is a photo of Model 2 taken late in the sixth test. The large deflections are very evident. The deflections in the end spans continued to increase throughout the test, but the reactions at the ends of the structure either stopped changing or changed at much smaller rates than previously when the applied load exceeded 6 kips (26.7 kN).

The cracks observed in the three spans of the structure are shown in Figs. 3.37 to 3.39. The same general comments about the nature and distribution of cracks that were made for Model 1 apply in this case, except that slip plane parallel to the draped reinforcement was not seen in the interior span of this structure. There was extensive deck cracking, especially at the north end of the south and interior span. None of the deck cracks ever became large, except for a single crack at each interior pier and these particular cracks became very large in both models.
Fig. 3.34 Load-Deflection Curves for Test 6, Model 2, MB-8 Loaded
Fig. 3.35 Load-Reaction Curves for Test 6, Model 2, MB-8 Loaded
Fig. 3.36  Photograph of Model 2 Near End of Final Test
Fig. 3.37 Cracking in Beam MB-6
4. Discussion of Results of Tests

4.1 Introduction

The discussion of the results of the tests will be concentrated on the behavior in flexure. There will be relatively little discussion of the behavior in shear for two reasons. First, there was little shear distress, as the web reinforcement was adequate for the forces imposed on the structure. Second, the girders were made with fine aggregate concrete and the writer is quite skeptical about the existence of consistently valid relationships between the shear strengths of members made with fine aggregates and with normal coarse aggregates.

The capacity and behavior of the simply supported girder MB-2 are discussed in Sec. 4.2, as the strength of the simply supported girder is a key to understanding the strengths of the continuous structures. Sec. 4.3 contains comparisons of the measured maximum applied loads and the theoretically limiting loads as determined using the limit analysis technique. Flexural cracking is discussed in Sec. 4.4, and observed and computed cracking moments are compared. Sec. 4.5 contains comparisons and discussion of the measured reactions and deflections, using results of elastic analyses as a base.

4.2 Strength of Model Beam MB-2

The test of MB-2 ended when the tension reinforcement fractured. The predicted strain in the steel at the time of a flexural failure involving crushing of the deck concrete was in excess of 0.04, so that the fracture of the strand should not be unexpected, considering the stress-strain curve for the strand which is shown in Fig. 2.7.

The computed failure moment for MB-2, at the time of strand fracture, was 123 k-in. (13.90 kN-m). This was computed considering the failure force in each of the eight strands to be 2,525 kips (11.23 kN), the average of the breaking forces in the strand tests. The concrete strength was taken as 6.0 k/lin.² (41.4 N/mm²). The effective depth from the top of the deck to the centroid of the tension reinforcement was 6.26 in. (159.0 mm).

The measured failure moment was 137 k-in. (15.48 kN-m), considering both the applied loads and the uniformly distributed load on the structure.

There does not appear to be any adequate explanation for the difference between the computed and measured moment capacity. The difference of 11.4
percent is outside the normally expected range. The difference cannot be attributed to an error in the computation of the internal lever arm of the force, as the computed lever arm was 97 percent of the effective depth, because of the large area and high strength of the deck concrete.

It does not appear reasonable that the stress in the steel could have been sufficiently larger than that measured in the coupon tests to explain the difference. The stress-strain curve shown in Fig. 2.7 includes a region in which the strain was increasing with virtually no change in stress, and this cannot have been altered much by embedding the steel in concrete. The average reinforcement stress would have had to reach 308 k/in.² (2124 N/mm²), and this seems very unlikely.

If the steel samples in the testing machine had all failed at low strains while the stress was still increasing appreciably with increasing strain, one could perhaps argue that a higher stress could be achieved in a beam, where the steel is gripped by bond, than in a testing machine with its comparatively sharp-toothed grips. However, this does not appear to be possible in this case, considering the large strains developed in the testing machine.

There may have been some variation in the strength of the strand along the length of the spool that was obtained. This would not have been detected, as the test coupons were all cut from approximately the same location in the long length of strand.

There must have been some restraint of the elongations of the lower surface of the beam by the supporting system, but that does not appear to be a major factor, either. In the last test of MB-2, the beam was supported on 1.0 in. (25.4 mm) diam rollers, and the maximum reaction at the ends of the span was 3.55 kips (15.8 kN). Assuming that the coefficient of rolling friction was 0.2, which is preposterously high, and that the horizontal restraint to elongation was applied 1.5 in. (38 mm) below the lower surface of the beam (because of the nature of the bearing devices), the maximum additional bending moment that can be anticipated is about 5 k-in. (0.56 kN-m).

It must be concluded that there is probably no single explanation for the rather high moment capacity observed for beam MB-2. However, it was an important test in that it provides a definite comparison point for the results of tests on the three-span structures. The observed moment of 137
k-in. (15.48 kN-m) will be used as the comparison value in the following sections.

4.3 Strengths of Models 1 and 2

The theoretical flexural strengths were evaluated using limit analysis procedures for reinforced and prestressed concrete. The concepts are practically identical with the concepts of plastic analysis which are sometimes used in connection with the design of steel structures, with the exception that one cannot automatically assume that reinforced concrete sections will provide adequate ductility. In this particular pair of structures, the sections have a great deal of ductility, partially as a result of the relatively high concrete strength, and ductility requirements will not be major concern.

The positive moment capacity at collapse will be taken as that found for beam MB-2. The negative moment capacity at the interior piers has been computed, using the areas of the various longitudinal reinforcement wires in the deck, as shown in Fig. 2.5, and using the yield stresses shown in Table 2.2.

Ignoring strain-hardening, the ultimate negative moment was found to be 48 k-in. (5.42 kN-m). The computed steel strain corresponding to the failure moment was 0.025. However, strain hardening would not be significant because the steel stress-strain curve had a long plateau at the yield stress, and the failure stresses exceed the yield stresses by margins that are much smaller than expected in normal hot-rolled reinforcing bars, as can be seen in Table 2.2.

The theoretical collapse load was evaluated for the end span of the structure using a fully plastic limit analysis. The results are summarized in Fig. 4.1. Fig. 4.1 is a graph showing theoretical collapse load as a function of the position of the load. The calculations were done for the two separate cases of the equivalent three-axle vehicle turned with the front, or light, axle nearest the simply supported end of the span and then nearest the fixed end of the span.

The information in the figures shows that the minimum collapse load occurs if the vehicle is placed with the central axle about 60 in. (1520 mm) from the fixed end of the span, and with the light axle nearest the simply supported end of the span. The computed minimum load is 6.50 kips (28.9 kN),
Fig. 4.1 Collapse Load versus Position of Load in End Span
and any other position of the load requires a higher total force to cause flexural collapse. If the vehicle is turned around, so that the light axle is nearest the interior fixed end, the minimum collapse load is 6.70 kips (29.8 kN), when the central axle is 53 in. (1350 mm) from the fixed end.

These loads were computed by assuming a fully-plastic collapse mechanism such as is shown in the upper part of Fig. 4.1. Two plastic hinges are assumed, one at the fixed end of the span and the other under the central load, and the structure is then given a small virtual deflection. The internal work done by the rotation of the plastic hinges is set equal to the external work done by the deflection of the loads, including the self-weight of the beam. The applied load is then found. This process was repeated for different locations of the three-axle vehicle until the curves shown could be plotted, and the minimum possible collapse loads determined. It must be confirmed that the positive moment hinge is actually at the central load point, and this is true for the entire range of values of x which are of interest in this case.

None of the loadings applied to the end spans of the two test structures ever approached the theoretical flexural collapse loads. The maximum load reached was in Test 6 on Model 1, in which the north span was loaded to 7.03 kips (31.3 kN). For that particular loading, x = 40.5 in. (1029 mm), and the theoretical collapse load is slightly over 8.0 kips (35.6 kN). The loading positions had been selected primarily so that they would produce large shear forces, and no end span loadings were done with the vehicle positioned at or near the critical positions for flexure.

Fig. 4.2 summarizes the results of limit analyses of the flexural capacity of the interior span. The collapse mechanism of the interior span includes three plastic hinges, one at each support and one under the central load of the vehicle in the region of maximum positive moment. Fig. 4.2 shows the theoretical collapse load as a function of the position of the vehicle. The lowest collapse load occurs when the central load of the vehicle is about 57 in. (1450 mm) from the end of the span, and the corresponding load is about 7.63 kips (33.9 kN).
Fig. 4.2 Collapse Load versus Position of Load in Interior Span
The final loadings on both structures were conducted with the central load 63 in. (1600 mm) from the end of the span, and the theoretical collapse load for this position is 7.72 kips (34.3 kN). Models 1 and 2 reached maximum loads of 7.44 and 7.39 kips (33.1 and 32.9 kN), respectively, without reaching total collapse. The experimental loads were thus about 96 percent of theoretical failure load when the tests ended.

The loadings in which large shear forces were imposed in the interior spans were positioned so that the central loads were 43 to 45 in. (1090 to 1140 mm) from the end of the span, and the theoretical flexural failure loads for this position are about 8.4 kips (37.4 kN), while the largest load applied was 7.03 kips (31.3 kN).

The limit analyses were done assuming that the negative moment hinges were at the center of the interior supports. The hinges, as evidenced by the very wide cracking that occurred, were actually at the faces of the diaphragms, 1.5 in. (38 mm) from the center of the support. In three of the four cases the hinge was on the interior span side of the support, while it was in the end span at the north pier of Model 2.

If the support hinges are both in the interior span, 1.5 in. (38 mm) from the center of the support, the computed collapse load for the case of the central load 63 in. (1600 mm) from the center of the support is 8.02 kips (35.7 kN) rather than 7.72 kips (34.3 kN). The failure load is higher as a result of the shorter effective span.

Both hinges were in the interior span in Model 1, and the 7.44 kip (33.1 kN) maximum load was about 93 percent of the collapse load computed taking into account the actual hinge locations.

The theoretical collapse load for test 6 on Model 2, considering the negative moment hinges to be at the north faces of the diaphragms of both interior piers, becomes 7.93 kips (35.3 kN), so the applied load was also 93 percent of the theoretical load computed using the more refined mechanism.

On the basis of the load-deflection curves shown in Figs. 3.16 and 3.34, Model 2 was considerably closer to its actual flexural collapse than was Model 1 when the loadings ended, but both were short of the theoretically limiting loads by about the same percentages.
4.4 Flexural Cracking Moments

Flexural cracking is probably of more interest to the designer than the collapse load, since the allowable service load stresses normally govern the proportioning of a composite I-girder highway bridge. The computed collapse loads are usually much higher than required for structures such as the prototype on which these models were based.

Consequently, comparisons of expected and observed moments at first flexural cracking are made in this section. This requires two specific stress values, neither of which can be known with great precision. The first is the pre-stressing force remaining in the structure at the time of the test, since this determines the precompression at the tension face. The second is the modulus of rupture, or the effective tensile stress capacity that can be mobilized at the same tension face. In addition, assumptions must be made about which parts of a composite section resist which forces during the interval between casting of the deck and the test which produces the flexural cracks.

Information on the prestressing force is contained in Refs. 1 and 2. It appears that the force remaining in the various members was probably in the range of 6.5 to 7.5 kips (28.9 to 33.4 kN), on the basis of strain measurements and interpretations of the measurements which are given in Ref. 2. The range is a result of small variations in the concrete properties, of variation in the initial pretensioning force, and of moments at the interior piers which develop with time as a result of differential shrinkage between the girder and deck.

Stress distributions along the lower surface of the girders were computed using the two limiting forces, in combination with a number of assumptions. It was assumed that the pretensioning force was resisted by the girder section, with none of the force being transferred to the deck with time. Moments which may develop with time at the interior piers were neglected, which is equivalent to saying that all spans were simply supported and therefore identical in several respects.

The stresses found are plotted versus position in the span in Fig. 4.3, and the tensile stresses at the bottom fiber caused by the dead load moment are also plotted. The distance between the curves represents the effective precompression in excess of dead load that exists in every span. Two curves are
Fig. 4.3 Girder Stresses due to Prestressing and Dead Load Forces
plotted for the total stress, one corresponding to each of the two limiting steel force values of 6.5 and 7.5 kips (28.9 and 33.4 kN). These curves have discontinuities in slope at the drape points in the span.

Cracking moment values were then computed, considering both of the limits on the precompression stress, as shown in Fig. 4.3, and considering two different values of the modulus of rupture. The modulus of rupture values were 6 $\sqrt{f_c}$ and 7.5 $\sqrt{f_c}$ lb/in.$^2$ (0.5 $\sqrt{f_c}$ and 0.62 $\sqrt{f_c}$ N/mm$^2$), which represents a range which will contain a reasonably large percentage of test results.

At a particular section along a girder, the net precompression stress, that is the precompression remaining after the stress from dead load has been accounted for, is found and added to the modulus of rupture. This total stress is then multiplied by the section modulus for the composite section for stresses at the lower face of the member. The resulting moment is the live load moment theoretically required to cause cracking. The theoretical live load moment at cracking is then added to the dead load moment to obtain the total, or gross, cracking moment.

The gross cracking moments are plotted in Fig. 4.4, for the two limiting cases of the larger prestressing force acting in conjunction with the larger modulus of rupture, and for the smaller prestressing force acting in conjunction with smaller modulus of rupture. The dead load moment diagram is also plotted in the same graph, in order to provide some perspective about how much of the capacity is used in resisting live load and dead load forces. The distance from the dead load moment diagram to a gross cracking moment curve represents the live load cracking moment at the particular section considered.

A number of points representing observed cracking moments are also plotted in Fig. 4.4. The numbers beside the data points indicate which particular girder that point came from. There are two different sets of data represented. The numbers with the MB-prefix are for the first flexural crack observed in that particular beam. These moments were computed assuming that the live load moment distribution could be determined using an elastic analysis. In each particular case, the load used in the moment computation
Fig. 4.4 Comparison of Computed and Measured Cracking Moments
was the best estimate of the load at the time of initiation of cracking. This load was determined by examination of the load-deflection curves, noting the load at which there was a significant departure from the initial steep part of the curve. This load was ordinarily lower than the load existing when cracks were found visually, because the loads were applied in relatively large steps and the girders were examined only at the ends of the steps, while the cracks could have formed at any load greater than the previous step.

The data points which simply have numbers represent moments acting when particular later cracks were found. Care was taken to identify the last crack away from the loading point, and hence at the lowest moment point, for each of these determinations.

It can be seen that the cracking moments were in general higher than could reasonably be expected following the analysis method used. The lowest data points lie at the upper bound of expected values. There may be several reasons for this discrepancy. One possibility is that the tensile strength of the small-aggregate concrete used in the girder was higher than expected. This has been observed by Vanderbilt, Sozen, and Siess (6), for example, in similar concrete mixes. Modulus of rupture values in small beams, 1.75 in. thick (44.5 mm), reached about $14 \sqrt{\frac{f_{c}}{C}}$ lb/in.$^2$ (1.2 $\sqrt{\frac{f_{c}}{C}}$ N/mm$^2$). It does not appear likely that concrete in the test girders was that strong, but this could have been a contributing factor.

Work reported in Ref. 3 makes it clear that there can be a significant transfer of force between the girder and slab with time as a result of creep and shrinkage of the two concretes. This transfer tends to lead to slightly higher precompression stresses at the lower face of the girder than occur if all of the steel force is assumed to be resisted by the girder section.

As an off-setting factor to the two just quoted, there is a tendency to develop positive moments at the interior supports after the deck and interior diaphragms are cast. However, this probably cannot adequately be evaluated, as the tendency is for this moment to dissipate, or relax, out of the structure over a long period of time, and the moment at the time of these tests must have been small.

Even though the predicted cracking moments were in general lower, and sometimes considerably lower, than the observed values, the agreement is still reasonably good and may be taken as a confirmation of the general design and analysis procedure.
4.5 Discussion of Measured and Theoretical Reactions and Deflections

This section will be used to compare some aspects of the behavior of the test structures with the theoretically expected behavior. At low loads, the structures should have responded as if they were elastic, and some comparisons can be made.

All of the graphs in Chapter 3 which show reactions versus load also contain broken lines giving the theoretical elastic reactions. Fig. 3.2, for the first test on Model 1, is a good example. The north span was loaded in that test, and it can be seen that the north abutment reaction was very nearly equal to the theoretical value in the first half of the test, and never departed far from the elastic value. Reactions at the south abutment and south pier departed significantly from elastic values when the load exceeded 2.0 kips (8.9 kN).

Both of the south reactions were a result only of negative moments at the north interior pier, and it appears that this moment became larger than the theoretical elastic moment after the north span developed positive moment cracks and hence lost considerable stiffness. Then later in the test the north pier section apparently yielded, and the south pier reaction no longer changed, as is evident in the graph. The theoretical change (reduction) in the south pier reaction which accompanies the development of the yield moment of 48 k-in. (5.42 kN-m) at the north pier is 0.66 kips (2.94 kN), while the measured value was 0.71 kips (3.16 kips). The theoretical increase in the south abutment reaction is 0.11 kips (0.49 kN), while the measured change was 0.15 kips (0.67 kN). The measured changes are in reasonable agreement with the computed values. The reduction in the south abutment reaction during the last stage of the test may be a result of slip in the joint at the south pier. The section at the south pier is subjected to positive moments during this loading, and there is no positive moment reinforcement.

Similar comparisons can be made for the other tests. It becomes quite clear that the negative moment connections at the piers became less efficient as the tests progressed and as damage accumulated. In the later tests the reaction changes away from the loaded span were almost always smaller than the elastic values.
When an interior span was loaded, the reduction in the abutment reaction was limited to a force equal to the negative moment capacity of an interior pier divided by the end span length. The resultant force is 0.44 kips (1.96 kN). The measured reactions were generally close to this value. In the final test on Model 1, Fig. 3.18, the north and south abutment reaction changes were 0.45 and 0.49 kips (2.00 and 2.18 kN), respectively, which are in excellent agreement with the theoretical values.

The large deviations from the theoretical elastic reaction at the south pier that occurred in the final test on Model 1, Fig. 3.17, seem strange, but they can be explained qualitatively in terms of the response of a structure which has already sustained considerable damage and permanent residual deformation. Elastic reactions for a number of three-span structures containing various hinges are shown in Fig. 4.5, to aid in the following discussion. The measured abutment reactions, Fig. 3.18, are also important to understanding this response.

Under the initial load, the structure responded as if the interior span were simply supported, with only very small reaction changes at the abutments and very small negative moments developing at the interior supports, presumably because of open cracks in the compression zone. As the load was increased, the cracks at the south pier closed and that section was then able to resist significant moments, and the structure responded as a two-span beam, or as a three-span beam with a hinge at the north pier. The reaction changes at the south pier and abutment are considerably larger in the two-span structure than in the three-span structure. Later, the section at the south pier yielded and responded to additional load as if it were a hinge, but at the same time the north pier section began resisting moment for the first time, and the structure responded to additional load approximately as a different two-span structure, or as a three-span structure with a hinge at the south pier. The changes in slope of the plot of south pier reaction versus applied load are consistent with this sequence of events, as are the abutment reactions.

Theoretical and measured reactions can be compared in a relatively straightforward manner. Young's modulus of the concrete does not enter the analysis, as long as it is constant along the length of the structure, and it will be found that the results are not sensitive to small variations in the concrete properties.
Fig. 4.5 Elastic Reactions for Three-Span Beams Containing Various Hinges
Theoretical and measured deflections are more difficult to compare, since the calculation of a deflection requires knowledge of both the value of Young's modulus and of the moment of inertia of the section. In the case of a concrete structure, there is always some uncertainty about the value of $E_c$, even when there are stress-strain measurements on concrete specimens from the structure. In the case of a composite concrete structure there is some uncertainty about the effective moment of inertia since two different concrete materials have been used in the girder and deck parts of the final cross section, and the relative values of $E_c$ in the two parts enter the calculation of the moment of inertia.

Deflections were computed assuming that $E_c = 3.1 \times 10^6$ lb/in.$^2$ (21.4 kN/mm$^2$) and that $I = 102.0$ in.$^4$ (42.46 x $10^6$ mm$^4$). The moment of inertia value was obtained assuming that the values of $E_c$ for the deck and girder were the same, as the cylinder test data indicates.

Each of the load-deflection curves which was presented in Chapter 3 contains a broken line indicating the theoretical elastic response in addition to the measured values, which have been plotted as solid lines.

Several features of these comparisons stand out and should be commented upon. In the initial tests on both structures, the deflections at low loads were smaller than expected. This may be a result of Young's modulus being higher than anticipated, or may be a result of other factors such as restraint due to support bearing friction, or may be due to the presence of the large diaphragms at the interior supports.

The major cause of the discrepancy is probably in the value of Young's modulus. Reference 1 contains an influence line for midspan deflection for one of these structures, in which measured and computed values are compared. In that case, $E_c = 4 \times 10^6$ lb/in.$^2$ (27.6 kN/mm$^2$) was necessary to make the computed and measured values agree, and that higher value would improve the agreement in the earlier tests described here.

As the tests progressed, there was clearly a deterioration in stiffness of the test structures. This can be shown by looking at the load-deflection response of both the loaded spans and the adjacent unloaded spans. Considering first the loaded spans, the response to the first load increment in the later
tests was approximately equal to that predicted by the elastic analysis. In the first tests on each structure, the initial was always smaller than predicted.

Considering the unloaded spans, it was observed that the initial response in the first few tests was always deflections slightly smaller than were predicted elastically. In the later tests on each structure, the deflections in the unloaded spans were generally much smaller than predicted using the elastic analysis results.

Both of these changes are related to deterioration of the stiffness of the negative moment connections at the interior piers, and the trends seen in the deflections are consistent with the trends noted earlier in connection with the discussion of the reactions.
5. Summary and Conclusions

This report describes the behavior of two three-span prestressed concrete bridge girder models which were subjected to large overloads. The girders were 1/8th scale models of a line of precast, pretensioned girders plus cast-in-place composite deck from a particular structure which was built in Douglas County Illinois, and which is described in Ref. 5. The girders were I-sections, and the three separate girders in each structure were made continuous for live load forces by means of negative moment reinforcement contained in the cast-in-place deck, plus concrete in diaphragms at the interior piers.

The model structures were 27.36 ft (8341 mm) in total length. The structures are described in Chapter 2 of this report, and many other details of their construction are described in Ref. 1. The models were originally built as part of a study of the long-term behavior of prestressed, composite, continuous highway bridges, and that phase of the investigation is reported in Ref. 1 and 2. The current study is concerned with the response of these structures to large overloads, and the loading equipment and positions of the load in each of the 13 separate tests are given in Chapter 2.

Chapter 3 describes the results of the tests, and presents graphs of load versus deflection and load versus reactions. Drawings of crack patterns and a few photographs are also presented, with appropriate comments about the development of various forms of damage as the tests progressed.

Chapter 4 is a discussion of the results of the tests. Sec. 4.3 compares the maximum applied loads with theoretically computed collapse loads. Neither of the structures collapsed under the applied loads, but it is shown that the maximum loads in the two final tests were in the range of 93 to 96 percent of the failure load, and the condition of the structures was consistent with these values.

Sec. 4.4 compares the observed moments at first cracking with those which can be predicted using straightforward stress calculations procedures. The observed cracking moments were in general larger than expected, but the difference was not so large as to undermine confidence in the calculation method. Some possible causes for the discrepancy are discussed.
Sec. 4.5 discusses the relationships between the theoretical elastic and measured load-deflection and load-reaction curves. Reasonable agreement is found for the load-reaction curves. The load-deflection curves present some difficulties, as the cylinder test specimens apparently did not adequately represent the value of Young's modulus of the concrete in the structure. In the first tests on both structures, $E_c$ was apparently about $1/3$ greater than indicated by the cylinder tests. It was noted that there was a continuous degradation of stiffness in the successive tests, as damage accumulated.

It was earlier concluded that these small specimens were suitable for the study of some of the aspects of the effects of creep and shrinkage on the behavior of prestressed concrete structures. It can further be concluded that these small specimens are also suitable for the study of the behavior of structures subjected to high overloads which approach the flexural collapse loads.
6. References


