Field Testing of a Reinforced Concrete Highway Bridge to Collapse

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A three-span reinforced concrete slab bridge was loaded to collapse. The slab was 9.75 m (32 ft) wide and 30.5 cm (12 in) thick, and on each edge was cast a 92 by 25.4-cm (34 by 10-in) curb. Loading was produced by hydraulic rams that were reacted to by overhead steel beams attached to the piers by tension rods through the slabs. The load was increased at intervals, and at each interval deflection and strain on the concrete surface were measured. The strength of the concrete and steel materials was more than the design minimum values; average cylinder strength was 47.33 MPa (6865 lbf/in^2) compared with a design value of 20.68 MPa (3000 lbf/in^2), and average steel coupon yield strength was 365.77 MPa (53 050 lbf/in^2) compared with a design value of 275.79 MPa (40 000 lbf/in^2). The results indicate that (a) the measured concrete stresses were lower than the calculated values; (b) the load causing first permanent set was accurately predicted by calculating the yield moment in the slab; (c) the collapse load was accurately predicted by considering the formation of yield moments along the centerline and over the piers of the bridge for a channel section loaded around its weak axis; and (d) based on the line load for the center span, it would take 8 HS20-44 trucks placed in the center of the bridge to cause any permanent deflection and 20 HS20-44 trucks to cause collapse.

This paper reports on the field testing to failure of a 10-year-old reinforced concrete highway bridge that was taken out of service when the highway was realigned. The test results are to be used to determine the magnitude of overload permitted on similar bridges. Seldom is a full-sized bridge available for testing to destruction, and, when it is, seldom can an economical loading system be devised for the tests. Most of the literature on strength of highway bridges is based on laboratory tests on small-scale beams and slabs, although Burdette and Goodpasture recently reported on testing to destruction of four highway bridges in Tennessee (1).

The testing program had the following objectives:

1. To compare the design stress with stresses measured on the bridge slab,
2. To determine the stresses caused by a four-wheel load pattern placed at the center of the middle span,
3. To find the load at which the first permanent set occurs and to compare this with the computed value, and
4. To compare the measured and computed ultimate strengths of the bridge.

DESCRIPTION OF BRIDGE

The bridge was on ND-18 located 1.6 km (1 mile) south of Casselton. It was a three-span, cast-in-place, reinforced bridge with a two-lane roadway (Figure 1). The slab was 9.75 m (32 ft) wide and 30.5 cm (12 in) thick, and on each side was cast a 92 by 25.4-cm (34 by 10-in) curb. The spans were 6.1, 7.6, and 6.1 m (20, 25, and 20 ft). The direction of the drainage ditch below the bridge required that the piers and abutments be on a 25-deg skew. Also, the bridge was located on an 8-deg horizontal curve that required a superelevation of the roadway surface on a slope of 1 to 16.

The curb on each side of the roadway was not cast monolithically with the slab; however, it was keyed into the slab, and tie bars were extended from the slab to the curb. A reinforced concrete guardrail and posts existed above the curb. The guardrail was removed before the bridge was tested. The bridge was constructed with No North Dakota State Highway Department class AE-1 1/2 concrete requiring a concrete cylinder strength of 20.68 MPa (3000 lbf/in^2). The reinforcing was intermediate grade with a minimum yield of 275.79 MPa (40 000 lbf/in^2).

TESTING PROGRAM

Application of Loads

The wheel load pattern used by a vehicle causing an overload is unknown; hence, two load patterns were approximated. The first loading was a four-wheel pattern placed at the center of the span and used only for stresses within the linear range. The wheels were 1.8 m (6 ft) apart transversely and 1.2 m (4 ft) apart longitudinally. The second loading was placed linearly across the center of the center span and used to destroy the bridge.

The framing used for the test loads is shown in Figure

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Figure 1. Plan and longitudinal section of bridge.

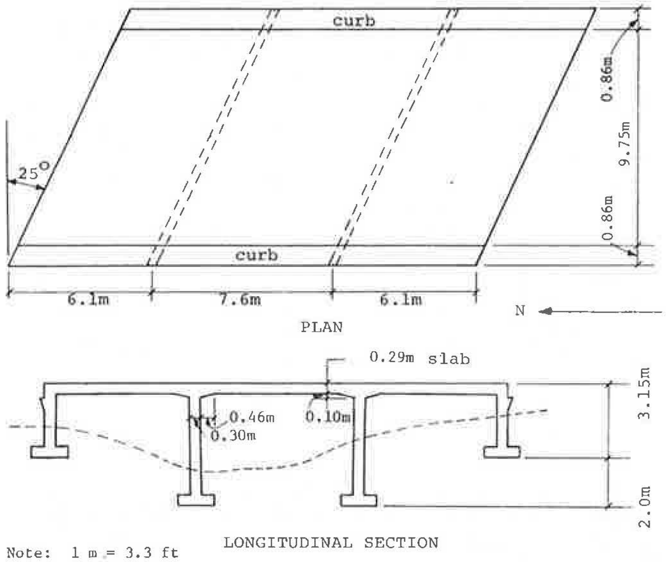
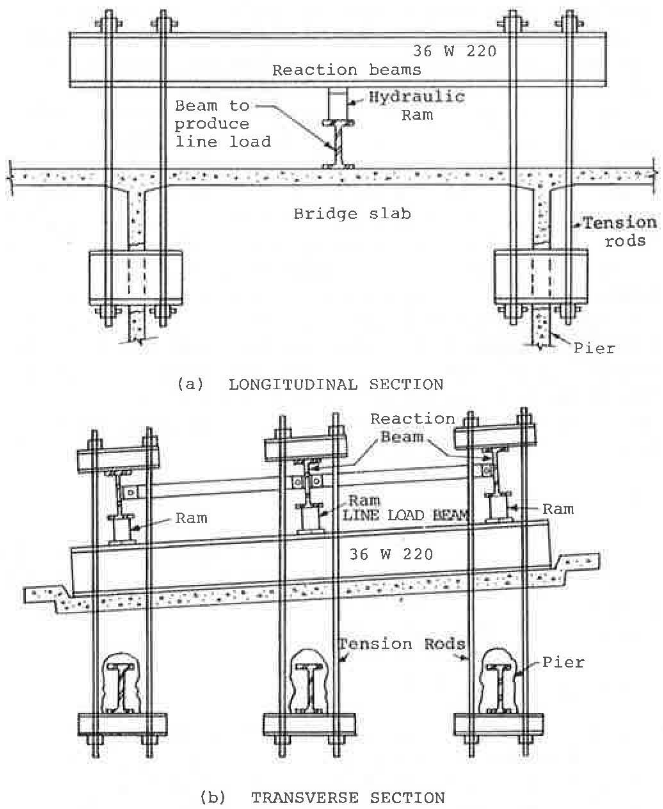


Figure 2. Framing used to provide test loads.



2. The wide flange beam at the center of the span was used to produce the line load on the bridge and wasloaded by three hydraulic rams. The reaction to the rams was provided by three reaction beams, each end anchored to the pier by four tension rods running through 10-cm (4-in) diameter cored holes in the slab. The same framing arrangement was used for the four-wheel load pattern except that the line load beam was replaced with a small frame, which produced the four-point load from the single ram at the center of the bridge.

Bearing plates 20 cm (8 in) square simulated the wheel contact area. The line load beam had a 30-cm (12-in) wide lower flange. Plywood, 1.6 cm (58 in) thick and 46 cm (18 in) wide, was used to provide a more uniform load at the contact surface. When the three-jack arrangement shown in Figure 2b did not produce failure of the bridge, one additional ram was placed at each end of the line load beam.

Loading Procedure

The first loading applied to the bridge was the four-wheel load pattern. The hydraulic rams were activated by a Riehle hydraulic pumping and indicating unit having a 68.95-MPa (10 000-lbf/in^2) pressure capacity. The pumping unit was transported to the site and stored in an enclosed truck. Since the line load beam and jack weight on the bridge was 40 kN (9000 lbf), all ram loadings were increased by this amount. However, this load was not included in deflection or strain measurements inasmuch as all gauges were zeroed when the beam and jack were already in place.

The four-wheel load pattern was loaded to a total of 293 kN (66 000 lbf) in four approximately equal load increments. This load was believed to be well below the load that would cause permanent deflection. After each load increment, when "equilibrium deflection" was reached, deflection dial and strain gauge readings were taken. The equilibrium deflection was arbitrarily established as the deflection at which the deflection rate at midspan was less than or equal to 25.4 μm/min (0.001 in). After the final loading was applied and all strain gauges and deflection measurements were taken, the pressure on the hydraulic ram was released and zero ram load readings were taken.

The line loading pattern was then carried out. The first loading increment was 334 kN (75 000 lbf), and each subsequent load was raised by 222 kN (50 000 lbf). After each loading increment, strain and deflection readings were taken at all locations. To detect the presence of any permanent deflection, the load was removed when a total load of 1001 kN (225 000 lbf) was reached. The same procedure was used at a total load of 2113 kN (475 000 lbf) and again when the load reached 3225 kN (725 000 lbf). The limit of the initial loading system was reached at 3670 kN (825 000 lbf). Although there was a deflection at the center of the span equal to 3.81 cm (1½ in) at this loading, complete failure had not been achieved. The loading was removed and final strain and deflection readings were taken.

Two weeks later, the testing resumed and two additional hydraulic rams were used. For this loading only deflection readings were taken, for many of the strain gauges were broken because of the cracking of the concrete. The loading was applied in 445-kN (100 000-lbf) increments up to a total load of 3114 kN (700 000 lbf), after which 222-kN (50 000-lbf) increments were used until the total load reached 4003 kN (900 000 lbf). At this time the midspan dial reading was reset to zero and read at every 0.6 cm (¼ in), a safe distance away from midspan. After 7.6 cm (3 in) of dial guage had expired, the loading was controlled by transit readings on one of the scales attached to the bottom surface of the bridge slab.

Strain Measurements

Strain measurements were taken at various points on both the upper and lower surfaces of the bridge. These strains were measured with both an electrical resistance strain guage [with a 15-cm (6-in) guage length] and a mechanical strain guage. Because approval from the North Dakota State Highway Department was received in December, the strain guages had to be applied in cold weather.

Figure 3. Deflections due to four-wheel load of 254 kN (57 000 lbf).

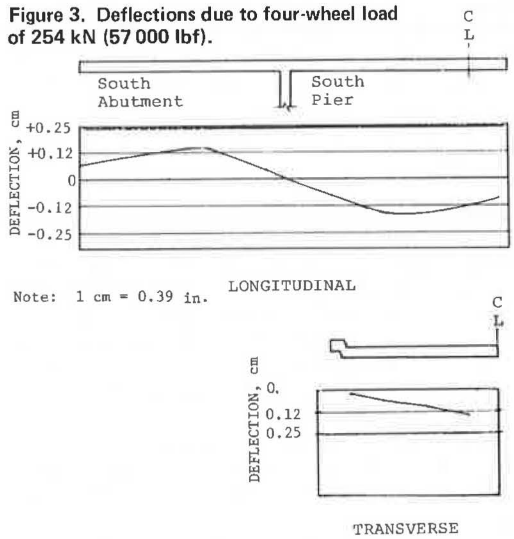
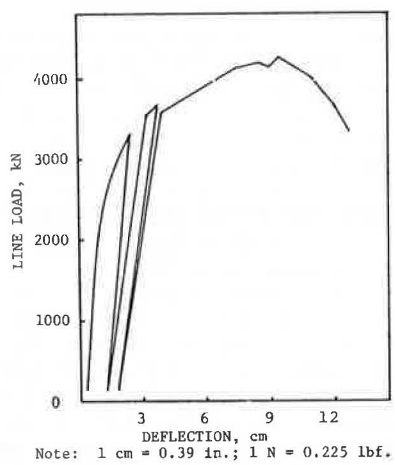


Figure 4. Line load versus deflection.



An unsuccessful attempt was made to locate a glue that would adhere in a temperature of -23°C (-10°F). Finally, regular epoxy glue was applied after the contact surface was heated to about 15.5°C (60°F) for a 10-hour period so that the glue could set properly.

During the heating process, the entire area below the bridge was enclosed with plastic. A propane burner with a blower was placed below the slab and heated an area on the lower surface of about 30.4 cm (12 in) in diameter to about 15.5°C (60°F). Since the top surface of the bridge was not covered, plywood boxes were built and placed over the location where the strain gauges were to be applied. Holes were cut in the boxes so flame throwers could direct heat toward the area. This procedure maintained the temperature required for the concrete surface. The strain gauges were then glued in place and held firmly by pressure plates. The heat was kept on for 10 h while the glue cured.

The gauges were arranged in a 45-deg rosette pattern, with one of the gauge axes parallel to the longitudinal axis of the bridge. Each gauge location contained electrical resistance strain gauges and three sets of mechanical stops for the mechanical strain gauges. For each strain gauge group located on the upper surface, a group was placed on the bottom surface of the slab in the same pattern.

Deflection Measurements

Twenty dial guages [with 25.4-μm (0.001-in) graduation] were used to measure the slab deflections at different locations on the top surface of the bridge. These gauges were mounted on a light-steel framework supported above the center of the two piers, hence essentially independent of the bridge slab being tested. To provide a smooth contact surface for the gauges, small metal plates were attached to the concrete surface.

Another set of deflections were taken on the underside of the slab during the line loading. Scales were attached to blocks of wood that were glued to the bottom surface of the slab. Taut wires were strung between the pier walls and located next to the scale. These readings were taken from a transit telescope located about 15 m (50 ft) away in the bed of the drainage ditch.

TEST RESULTS

Figure 3 shows the deflections due to the four-wheel loading of 254 kN (57 000 lbf). At this loading the deflection at the center of the bridge was 61.6 mm (0.062 in). However, the deflection 66 cm (2.2 in) south of the bridge center was 1.7 mm (0.066 in).

The line load deflection curve for the center of the bridge is shown in Figure 4. The curve indicates the loading cycles reported earlier and shows an ultimate load of 4226 kN (950 000 lbf). Transverse and longitudinal deflections were similar in shape to those reported for the four-wheel load pattern (Figure 3).

A number of 10-cm-diameter (4-in) cores were taken from the bridge slab and tested (AASHO T-148) by the state highway department. The strength results of 12 cores are as follows: average value of 47.33 MPa (6865 lbf/in^2), value range of 43.54 to 51.85 MPa (6315 to 7520 lbf/in^2), and standard deviation of 2.84 MPa (412 lbf/in^2).

At the conclusion of the bridge testing and before the bridge destruction, specimens of steel reinforcing near the supports were removed and tested (AASHO T-68) at the North Dakota State University for yield strength. The results for seven tests are as follows: average value of 365.77 MPa (53 050 lbf/in^2), value range of 307.09 to 406.51 MPa (44 540 to 58 960 lbf/in^2), and standard deviation of 36.04 MPa (5227 lbf/in^2).

CALCULATIONS

Calculations can be used to predict the performance of the bridge. Calculations are used here to indicate the design load stresses, the load causing permanent set, and the ultimate load on the bridge.

Design Load Stresses

The bridge was designed for an HS20-44 load (2). Moments and stresses used in bridge design are given in Table 1. The moments were calculated on the basis of an elastic analysis for a three-span continuous slab. The stresses are based on a working stress theory; the steel concrete modular ratio is equal to nine.

Load Causing First Permanent Set

Except for the cracking of the concrete in tension, a reinforced concrete section will behave elastically until the steel reaches its yield stress. Beyond yield stress, there will be a permanent deflection in the beam. Therefore, it is important to identify the minimum load that will cause a permanent set in the bridge slab.

For the 27.7 cm^2/m (1.31 in^2/ft) of steel in the lower face of the 9.75-m-wide (32-ft) slab, the live load moment necessary to produce yielding is 2210 kN•m (1 630 000 lbf•ft). This moment can be converted to a transverse line load along the center of the bridge by the following equation: Moment = coefficient × load × length. If the length of the exterior span of this three-span, simple supported bridge is used, the coefficient is 0.2106. Using the live load moment given above of 2210 kN•m (1 630 000 lbf•ft) gives a line load of 1721 kN (387 000 lbf) that must be applied along the centerline to cause permanent set.

During testing, the curbs and slabs were observed to be acting together, and the moment was thus approximated in two parts. The moment in the slab was 2074.4 kN•m (1 530 000 lbf•ft), and the moment in each curb for 11.6 cm^2 (1.8 in^2) of steel was 223.7 kN•m (165 000 lbf•ft). The slab moment and the two curb moments give a moment of 2521.8 kN•m (1 860 000 lbf•ft) that will cause yielding. Based on the above coefficient, a line load of 1962 kN (441 000 lbf) would be necessary to cause permanent set.

Load Causing Collapse

The first permanent set load causes a yield stress at one point in the bridge. However, before actual collapse can occur, yielding and hinges must form under the load and over the two piers. The moment in the hinge over the two piers is labeled M_1, and the moment at the center of the midspan is labeled M_2. Two methods of determining M_1 and M_2 are presented. In the first method the 9.75-m-wide (32-ft) slab and the curbs are considered separately and added to the total moment.

M_1 is calculated to be 2664.2 kN•m (1 965 000 lbf•ft) as follows: The slab steel area is 9.55 cm^2/m (1.48 in^2/ft), the live load moment is 2139.5 kN•m (1 578 000 lbf•ft), and the moment in each curb for 14 cm^2 (2.7 in^2) of steel is 262.4 kN•m (193 500 lbf•ft). M_2 is calculated to be 2521.8 kN•m (1 860 000 lbf•ft) as follows: Slab steel area is 27.7 cm^2/m (1.31 in^2/ft), live load moment is 2074.4 kN•m (1 530 000 lbf•ft), and the moment in each curb is 223.7 kN•m (165 000 lbf•ft).

The conservation of energy principle was used to develop a relationship between the applied line load and M_1 and M_2. The result was as follows: Line Load = M_1 + $M_2 \times 4 \div$ center span length. Based on the above values for M_1 and M_2 the line load is 3238.3 kN (728 000 lbf).

Combining the slab and curb gives different values for M_1 and M_2. To find M_1, the compression is all taken in the bottom of the slab and the moment is calculated for the steel in the top of the slab and added to the moment calculated for the steel in the top of each curb. The steel areas are the same as those used in the previous calculation for M_1. The moment is 2662.8 kN•m (1 964 000 lbf•ft). To find M_2, the compression is all taken by the portion of the curbs above the slab. The tension steel of the slab and curb is added, and the moment is 4504 kN•m (3 322 000 lbf•ft). Using the previously derived equation to relate the moment to a loading gives a load of 4470.5 kN (1 005 000 lbf) that is needed to produce a collapse.

Table 1. Design moments and stresses.

Item	Load	At Midspan	Over Piers
Moment, kN•m	Dead + live	23.85	28.63
Concrete stress, MPa	Dead + live	7.72	8.48
Steel stress, MPa	Dead + live	126.2	141.3
Concrete stress, MPa	Live	5.86	5.07
Steel stress, MPa	Live	95.6	86.2

Note: 1 N•m = 0.738 lbf/ft; 1 Pa = 0.000 145 lbf/in^2.

COMPARISON OF TEST RESULTS

Stresses in Slab

The first objective was to compare the design stresses with those obtained by measuring the strains on the bridge slab. At each location, strains were measured for both the upper and lower surfaces. Based on a straight line strain relationship, the strain at the elevation of the steel was calculated. Then, using the modulus of elasticity for concrete and steel, the concrete/steel stresses were calculated. This stress comparison is given in Table 2 for a line load of 814 kN (183 000 lbf).

The first set of calculated stresses was based on a three-span slab over simple supports. The curbs were neglected in these calculations.

The second set of calculated stresses was based on a slab with curbs, and was computed by using the STRUDL program. For this calculation, the bridge was divided into a gridwork consisting of beams running parallel with the piers and parallel with the direction of traffic. The program permitted a separate input of stiffness for each longitudinal and transverse beam. Each longitudinal beam had a constant stiffness except for the edge beams, which were significantly stiffer because of presence of the curbs. The amount of transverse stiffness was adjusted until the deflections agreed with those observed during testing.

The third set of calculated stresses was based on a channel-shaped section. The slab is the web of the channel while the curbs make up the flanges.

The slab model is often used in design, and the channel model is applicable to the collapse load on the bridge. However, the slab with edge beam model is the best representation of actual behavior since it incorporates both slab and curbs while adjusting the relative stiffness to match measured deflections.

The large difference between the measured and calculated stresses cannot be justified. One factor that might account for the difference is the application of strain

Table 2. Stress comparison for a line load of 814 kN (183 000 lbf).

			Calculated Stresses (MPa)					
	Measured Stresses (MPa)		Slab		Slab With Edge Beams		Channel-Shaped Section	
Location	Concrete	Steel	Concrete	Steel	Concrete	Steel	Concrete	Steel
Midspan	1.59	15.9	9.17	148.2	6.27	102.0	0.55	77.2
	2.41	31.0	9.17	148.2	4.55	74.5	0.55	77.2
	0.76	12.4	9.17	148.2	6.72	109.6	0.55	77.2
	2.41	6.89	9.17	148.2	6.79	110.3	0.55	77.2
	1.17	11.7	9.17	148.2	5.45	88.9	0.55	77.2
	1.65	14.5	9.17	148.2	6.20	101.3	0.55	77.2
Piers	0.76	6.89	2.36	39.4	2.57	42.7	1.99	28.6
	1.65	6.89	4.34	72.4	3.10	51.7	3.65	52.4

Note: 1 Pa = 0.000 145 lbf/in^2; 1 N = 0.225 lbf.

gauges in cold weather. The glue may not have fully cured and may have been still partially elastic at the time of testing. This would cause strains to be measured; however, they would be considerably below the actual strains that occurred.

Load Causing First Permanent Set

To determine the load causing first permanent set from test results, it was necessary to plot the load deflection curves for each of the points near the midspan of the slab. Each curve was studied to determine the load at which the curves left the straight line relationship. That load for most of these points was 1668 kN (375 000 lbf), and thus that value was taken to be the line load causing first permanent set.

Two values for permanent set were calculated. The first, 1721 kN (387 000 lbf), was based on using only the moment in the slab, and the second, 1962 kN (441 000 lbf), was based on using the moment in the slab and the curbs. A comparison of the values indicates that the test load was within 3 percent of that predicted from the slab moment and 15 percent below that predicted from the curb moment.

Collapse Load

Figure 4 shows that a collapse load of 4226 kN (950 000 lbf) was obtained. A plastic collapse model that incorporates the slab and curb beams separately predicted a collapse load of 3238 kN (728 000 lbf). To predict a collapse load near the actual failure load, it is necessary to consider slab and curbs as a channel-shaped section loaded about its weak axis so that at the center of the bridge the curbs are in compression and the slab is in tension. A collapse load of 4470.5 kN (1 005 000 lbf) was predicted and is within 5 percent of the actual collapse load.

Bridge Overloads

What magnitude of truck overloads is represented by the reported line loads? The center span truck load design moment of 59.2 kN·m/m (13 000 lbf·ft/ft) on a 9.75-m-wide slab is equivalent to a live load moment produced by a line load of 421 kN (94 000 lbf). For each such line load increase, the bridge will carry an additional HS20-44 truck load in each lane. The following table gives the live load safety factors for various numbers of HS20-44 trucks on the bridge.

Number of Trucks		Safety Factor	
In Each Lane	On Bridge	Permanent Set Load	Collapse Load
1	2	3.96	10.03
2	4	1.98	5.02
3	6	1.32	3.34
4	8	0.99	2.51
5	10		2.00
6	12		1.67
7	14		1.43
8	16		1.25
9	18		1.11
10	20		1.00

The safety factor is given for the permanent set load as well as for the collapse load. Extremely high truck loads are necessary to cause permanent set or collapse load; for example, permanent set will not occur until a load of four times the HS20-44 truck load is placed in each lane.

The 1957 AASHO bridge specifications used in this design did not permit an overload for the HS20-44 loading; however, they did permit overloads up to 100 percent for lighter load when the adjacent lane was not loaded (5).

CONCLUSIONS

As a result of testing and calculations on the reinforced concrete bridge, the following conclusions were made:

1. Strength of concrete and steel materials was more than the design minimum values; therefore, the average cylinder strength for concrete was 47.33 MPa (6865 lbf/in^2); the design minimum strength was 20.68 MPa (3000 lbf/in^2). The average yield strength of the reinforcing steel was 365.77 MPa (53 050 lbf/in^2); the design minimum value was 275.79 MPa (40 000 lbf/in^2).
2. Measured stresses in the concrete and steel for both the line load and the four-wheel load were very close to the calculated stresses. This was partially due to the omission of the curbs in the calculated stresses in the bridge but also may be due to the difficulties of carrying out strain measurements in cold weather.
3. The load causing first permanent set was accurately predicted by calculating the yield moment in the slab and neglecting the moment in the curbs. On this basis the line load causing first permanent set was 168 kN (375 000 lbf); the predicted load was 1721 kN (387 000 lbf).
4. The collapse load for the bridge was accurately predicted by considering the formation of yield moments along the centerline and over the pier of the bridge. These moments were calculated on the basis of a channel-shaped section loaded about its weak axis. On this basis, the line load causing collapse was 4226 kN (950 000 lbf), and the predicted load was 4470.5 kN (1 005 000 lbf).
5. A line load of 421 kN (94 700 lbf) produced the same moment in the center of the bridge as did an HS20-44 truck in each lane. Hence, it would take eight HS20-44 trucks placed in the center of the bridge to cause any permanent deflection in the bridge, and it would take twenty HS20-16 trucks placed in the center of the bridge to cause collapse.

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The opinions, findings, and conclusions in this report are those of the authors and not necessarily those of the North Dakota State Highway Department or the Federal Highway Administration.

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