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Load Test and Evaluation of the 18th Avenue Bridge

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Abstract: This report describes the structural evaluation and load test of the 18th Avenue Bridge located between Warden Avenue and Kennedy Road in Markham. The structure is a one-lane bridge, 9.14 m long. It is of a concrete beam and slab construction. Design drawings showing reinforcement details are not available, so an analytical evaluation of bridge strength was not possible. The bridge is in generally poor condition with extensive spalling of concrete and corrosion of exposed reinforcement in the exterior girders. It had been posted for 8 tonnes. Load testing revealed that the structure is much stiffer than in theory with some restraint at the supports. Testing was stopped at a load level at which measured responses in the most deteriorated exterior girder became non-linear. Because of the advanced deterioration of the exterior girders, nonlinearity was observed under eccentric loading prior to central loading. Based on the maximum eccentric test load applied to the bridge, posting loads of 16, 20 and 29 tonnes at evaluation levels 1, 2 and 3 respectively, are recommended. If traffic could be restricted somehow to the central portion of the bridge, posting loads of 20, 26 and 35 tonnes could be adopted.

Comments:

Key Words: bridge strength, load testing, 18th Avenue Bridge, concrete bridges

Distribution: MTO Head Office and Regions, selected structural and geotechnical engineers

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1/ Test Summary

Bridge:	18th Avenue Bridge in the Town of Markham, MTO Site No. 37-104
Type of Bridge:	One-lane, single-span concrete slab-and-girder bridge
Posted Load:	8 tonnes
Span/Width:	9.14 m/5.59 m
Angle of Skew:	0°
Date of Test:	October 23, 1986
Test Engineers:	A. Agarwal, J. Maheu
Job Captain:	J. Klubal
Supervisor, Instrumentation:	B. Kwok
Test Load Applied:	Central Loading: 2 axles: 41 tonnes; 4 axles: 56 tonnes Eccentric Loading: 2 axles: 36 tonnes; 4 axles: 50 tonnes
Responses Measured:	Longitudinal strains in the beams and deck at midspan and near the abutments, transverse strains at the bottom of the deck at various locations between the beams, strains across diagonal cracks in the beams at the west abutment, vertical displacements at midspan, and horizontal movement at the abutments.
Conclusions:	Posting loads may be increased to 16, 20, and 29 tonnes for evaluation levels 1, 2 and 3, respectively. If traffic were to be restricted to the central portion of the bridge, these could be raised to 20, 26 and 35 tonnes. These posting loads are based on the assumption of re-evaluation within 5 years.

2/ Bridge Description

2.1/ General Description

The 18th Avenue Bridge is a one-lane concrete structure crossing a small stream in the Town of Markham on 18th Avenue between Warden Avenue and Kennedy Road. A view of the bridge is shown in Figure 1. It consists of four beams 457 mm wide and 572 mm deep spaced at 1676 mm on centres. They are integral with a deck slab approximately 260 mm deep. There are copings over the exterior beams. The overall length of the bridge is 10.21 m with a clear span of 9.14 m. There is no evidence of free bearings at either abutment. Geometric details of the bridge are shown in Figure 2.

The date of construction is not known and no plans are available. The amount and location of reinforcing steel is, therefore, unknown.

2.2/ Condition Survey

The bridge is in generally poor condition. There has been extensive spalling at the underside of both exterior girders at various locations, exposing reinforcing steel in advanced stages of corrosion. The south girder is in worse condition than the north, with the entire bottom layer of steel exposed at midspan. The deterioration is so extensive that it is not possible to determine the exact size and number of bars originally present, although they appear to have been bundled together. Some have completely disintegrated leaving nothing but a trace of iron oxide on the concrete. Figure 3(a) shows a view from the underside of this girder.

The interior girders are in much better condition as seen in Figure 3(b). The northerly interior girder (girder C) has a small amount of concrete spalled off at each abutment. All four girders have diagonal cracks at the west abutment. They start at the abutment wall near the girder bottoms, rising at an angle of approximately 30° from the horizontal

The deck is in good condition with a limited amount of spalling on the underside in the south-east corner. The coping is in poor condition with extensive cracking and large portions broken away. Continuous longitudinal cracking on the outside faces of the exterior girders just below the copings suggest that they do not act fully integral with the rest of the superstructure.

The original railings consist of concrete posts and rails. Several elements are missing and have been replaced by steel rails. What remains of the original is in very poor condition and would not be expected to contribute to the structural integrity of the bridge.

3/ Test Procedure

3.1/ Details of Instrumentation

The bridge was instrumented to measure longitudinal strains, transverse strains, crack openings, vertical deflections and horizontal displacements at various locations. Figure 4(a) identifies those sections at which transducers were mounted. Section B, the midspan section was heavily instrumented to monitor the longitudinal flexural strains at various levels in the girders and deck slab at the locations shown in Figure 4(b). Transducers at the top of the slab were limited to those locations near the curb which would not interfere with the movement of the test vehicle. A strain transducer could not be mounted at the bottom of girder A due to the poor condition of the concrete and steel. Transverse strains in the bottom of the slab were measured midway between the girders. Vertical deflections of each girder were also measured at this midspan section.

Strains were also measured in the interior girders at Sections A and C near the abutments. These were intended to measure the degree of fixity or bearing restraint, as were the horizontal movements of the exterior girders at the abutments. The locations of these transducers are shown in Figures 4(c) and (d). In addition, strain transducers were mounted at right angles across the diagonal cracks near the west abutment at the locations shown in Figure 4(d).

3.2/ Details of Loading

The geometry of the test vehicle and its transverse and longitudinal positions on the bridge are shown in Figure 5. Because of the short span, flexural effects in the bridge were governed by the rear dual axle loads and loading by concrete blocks was concentrated at the rear of the truck. Maximum shear occurred under the combined effects of both the front and rear tandems. Table 1 lists the axle spacings and weights at the different load levels. Load level 7 represents the largest load normally applied to the rear tandem of the test truck during a bridge test.

4/ Test Results

The bridge was tested under load levels 1 to 6 along both loading lines 1 and 2, although for load level 6 strain readings were not taken for all truck positions along the eccentric line 2. Due to some non-linearities in the strains and deflections in exterior girder A under eccentric loading at load level 6, only the central line 1 loading was included for load level 7.

4.1/ Longitudinal Strains

An indication of the transverse distribution of load is given by the plots of tensile strains in Figure 6. These suggest a highly irregular distribution, with girder B registering negligible strains under all loading configurations. Measurements of tensile strains in concrete may be misleading since cracks may or may not be included in the distance covered by the strain transducers, and tensile strains would be larger at crack openings. However, the only midspan location where significant compressive strains were measured was on the top of the deck near the south exterior girder as seen in Figure 7.

Even under the central line 1 loading the flexural strains in girder A were larger than in girder D, the north exterior girder. However, these may still translate into smaller moments actually being resisted because the advanced deterioration of girder A results in a reduced stiffness.

In contrast, the largest flexural strains in the interior girders were measured in girder C, the northerly girder. However, these were tensile strains only and a direct comparison may not be valid. Longitudinal strains were also measured in the interior girders near the abutments. These are plotted with the midspan strains in Figure 8. Both girders resisted negative bending moments at the supports with larger strains measured in girder C than in girder B, and in both cases, larger strains at the east abutment than at the west. These indicate some rigidity against rotation. The cracking observed at the west abutment would provide a partial release from this rigidity, explaining the smaller strains developed at that end. Measurements of horizontal movement of the exterior girders at the abutments were negligible under all loading conditions, providing further evidence of the support rigidity.

The largest longitudinal strains are plotted against applied load in Figure 9. The maximum strains in interior girder C occurred under central loading, while those in exterior girder A occurred under eccentric loading. In each case the tensile strains become non-linear at higher loads, but were mostly recoverable. The largest tensile strain measured was 93 $\mu\varepsilon$, and the largest compressive strain -114 $\mu\varepsilon$.

4.2/ Transverse Strains

The transverse strains measured in the bottom of the deck slab at various locations between the girders were generally small and did not follow a consistent pattern with applied load. The largest value recorded was a tensile strain of 30 $\mu\varepsilon$.

4.3/ Strains Across Diagonal Cracks

The strains measured across the diagonal cracks at the west abutment give an indication of the opening or closing of those cracks. As seen in Figure 10, these strains were generally small and they were fully recoverable. The largest strains were measured at the inside face of girder A. If the maximum measured tensile strain of 42 $\mu\varepsilon$ was largely concentrated at the crack, it would represent a widening of the crack of approximately 0.008 mm based on a 200 mm long strain gauge.

The strains in girder B are evidence of the combined actions of vertical shear and torsion at the supports. Under central line 1 loading there appears to be a clockwise twisting of the girder at the west abutment with the crack on the north face (channel 7) opening and the one on the south face (channel 10) closing. The direction of the twist is reversed under eccentric line 2 loading.

4.4/ Vertical Deflections

Midspan deflections under different loading configurations are plotted in Figure 11. The patterns generally conform to the transverse position of the truck. Although the deflections of girder B are somewhat less than those of girder C under central loading, they are of the same order of magnitude. Under eccentric loading girder B deflects most as would be expected. These deflections contradict the distribution of strains measured at midspan (Figure 6), which implied that girder B resisted little or no load. However, as mentioned in Section 4.1, tensile strains in concrete may not be a reliable indication of the load being resisted due to the random distribution of cracks. Significant strains could occur across a crack while neglible strains are recorded in the concrete adjacent to the crack. Deflections are a more reliable indication of load distribution since they are related to the accumulative strains over the length of the beams.

The maximum deflections are plotted against applied load in Figure 12. The measured deflections were much smaller than the theoretical values calculated from a grillage analysis program assuming a simply supported span and stiffnesses based on the full uncracked concrete section. However, the deflections in exterior girder A under eccentric loading showed signs of becoming non-linear at load level 5. Strains in this girder also appeared non-linear and for these reasons eccentric loading was stopped after load level 6. However, all deflections were largely recoverable.

5/ Calculation of Posting Loads

Posting loads are calculated in accordance with Section 14 of the Ontario Highway Bridge Design Code [1]. Since the reinforcement details are not known, it is not possible to determine the scale-down factors and posting loads from the theoretical capacities of the bridge components. Instead, the resistance of the bridge must be based on the largest test loads it safely supported, which provide a conservative estimate of the actual capacity. The scale-down factors are calculated for both longitudinal shear and flexure. In each case, the live load resistance is set equal to the largest shear or bending moment resisted by the bridge under test loading. The design values are calculated for the appropriate OHBD truck at each evaluation level. The scale-down factor is then calculated as follows:

$$F = \frac{L_t}{\alpha_L \, (1+DLA) \, L_e}$$

where: L_t = the load effects resisted by the bridge under test loads

L_e = the calculated load effects due to the evaluation loads

α_L = live load factor = 1.25, based on 5 year re-evaluation period

DLA = dynamic load allowance = 0.30

Due to the poor condition of the bridge, it is recommended that it be re-evaluated within five years. For this reason, the reduced live load factor of 1.25 was used.

Table 2 summarizes the calculation of the scale-down factors and posting loads for the test vehicle at load level 6, the largest load which crossed the bridge along both the central and eccentric loading lines. Shear and flexure give the same scale-down factors at evaluation level 1, while shear governs at evaluation levels 2 and 3. The posting loads are 16, 20 and 29 tonnes, respectively.

The calculations are repeated in Table 3 for the test vehicle at load level 7, which crossed the bridge along the central line only. Shear governs the scale-down factor at all three evaluation levels. The corresponding posting loads are 20, 26 and 35 tonnes at evaluation levels 1, 2 and 3, respectively.

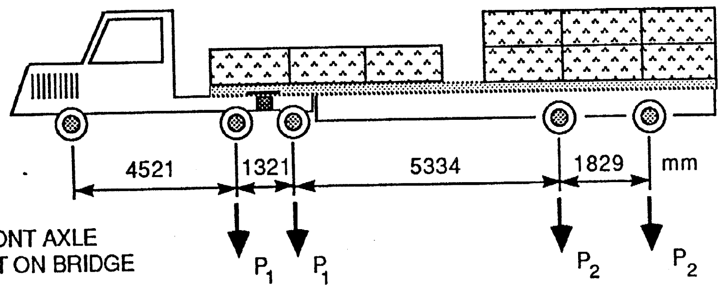
6/ Conclusions and Recommendations

The results of this load test reveal that the bridge does not behave as the idealized structure normally assumed during design or evaluation. For example; this bridge is stiffer than in theory, deflecting much less under load than expected due to the effects of end fixity and bearing restraint in the horizontal direction. However, the response of individual but similar elements under similar loading is highly non-uniform. This makes it difficult to draw generalized conclusions which could be applied to other bridges of like construction and condition.

It can be concluded, however, that the bridge can support loads substantially higher than the 8 tonne limit currently posted. The minimum safe loads can be increased to 16 tonnes for single posting, or 16, 20 and 29 tonnes for triple posting. If traffic were to be restricted somehow to the central portion of the bridge, these could be increased to 20, 26 and 35 tonnes. Consideration should be given to the type of vehicles using the bridge, in particular, farm equipment, before any lane width restrictions are implemented. All the above posting loads are based on the assumption of re-evaluation within 5 years.

References

[1] Ontario Highway Bridge Design Code, Ministry of Transportation and Communications, Downsview, Ontario, 2nd Edition, 1983.



LOAD LEVEL	NUMBER OF BLOCKS	FRONT TANDEM LOAD, $2P_1$		REAR TANDEM LOAD, $2P_2$		FOUR AXLE TOTAL	
		(kN)	(Tonnes)	(kN)	(Tonnes)	(kN)	(Tonnes)
1	0	92.6	9.4	81.9	8.4	174.4	17.8
2	10	130.1	13.3	135.8	13.9	265.9	27.2
3	16	133.5	13.6	188.7	19.2	322.2	32.9
4	22	136.2	13.9	242.1	24.7	378.3	38.6
5	28	138.8	14.2	295.5	30.1	434.3	44.3
6	34	141.5	14.4	348.9	35.6	490.4	50.0
7	40	144.2	14.7	401.4	41.0	545.6	55.7

Table 1/ Test Vehicle Axle Weights and Spacing

FORCE EFFECT	FLEXURE	SHEAR	
EVALUATION LEVELS	1,2,3	1	2,3
GOVERNING TEST VEHICLE CONFIGURATION	349 kN 1.83 m	142 kN 349 kN 1.32 5.33 1.83 m	142 kN 349 kN 1.32 5.33 1.83 m
MAXIMUM TEST B.M. OR SHEAR, L_t	646 kN.m	334 kN	334 kN
GOVERNING O H B D TRUCK CONFIGURATION	280 kN 1.2 m	60 kN 280 kN 3.6 1.2 m	280 kN 200 kN 1.2 6.0 m
EVALUATION B. M. or SHEAR, L_e	559 kN.m	290 kN	304 kN
SCALE DOWN FACTORS	0.71	0.71	0.68
POSTING LOADS			
LEVEL 1	16 Tonnes	16 Tonnes	....
LEVEL 2	....	....	20 Tonnes
LEVEL 3	....	....	29 Tonnes

Table 2/ Calculation of Posting Loads Corresponding to Test Load Level 6

FORCE EFFECT	FLEXURE	SHEAR	
EVALUATION LEVELS	1,2,3	1	2,3
GOVERNING TEST VEHICLE CONFIGURATION	401 kN 1.83 m	144 kN 401 kN 1.32 5.33 1.83 m	144 kN 401 kN 1.32 5.33 1.83 m
MAXIMUM TEST B.M. OR SHEAR, L_t	743 kN.m	382 kN	382 kN
GOVERNING O H B D TRUCK CONFIGURATION	280 kN 1.2 m	60 kN 280 kN 3.6 1.2 m	280 kN 200 kN 1.2 6.0 m
EVALUATION B. M. or SHEAR, L_e	559 kN.m	290 kN	304 kN
SCALE DOWN FACTORS	0.82	0.81	0.77
POSTING LOADS			
LEVEL 1	....	20 Tonnes	....
LEVEL 2	....	....	26 Tonnes
LEVEL 3	....	....	35 Tonnes

Table 3/ Calculation of Posting Loads Corresponding to Test Load Level 7,
Central Loading Only

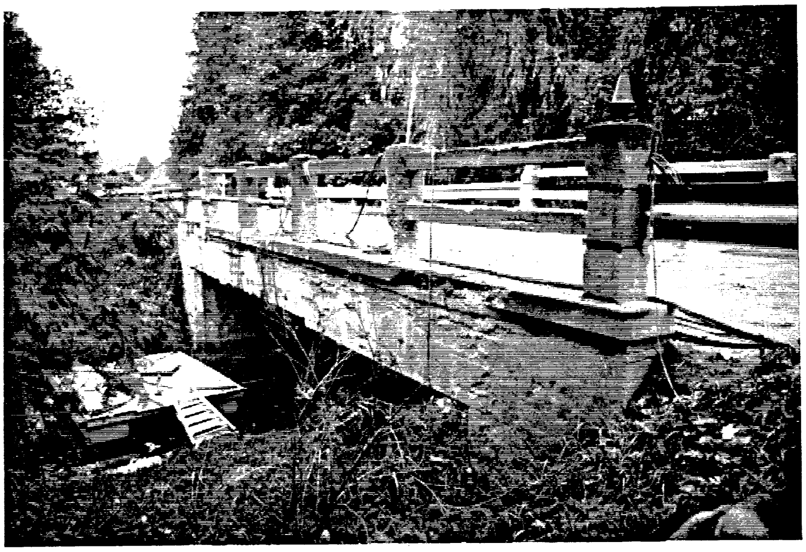
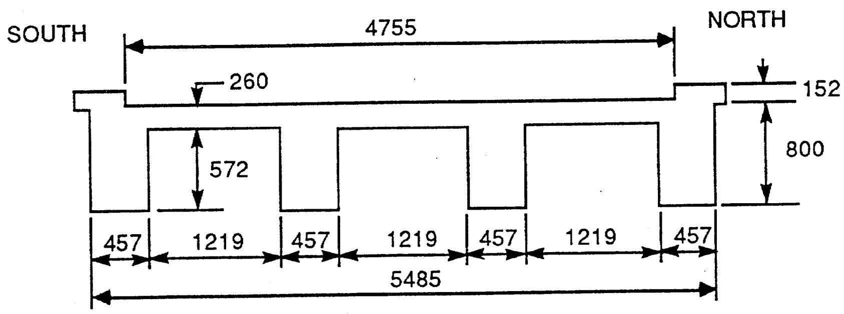
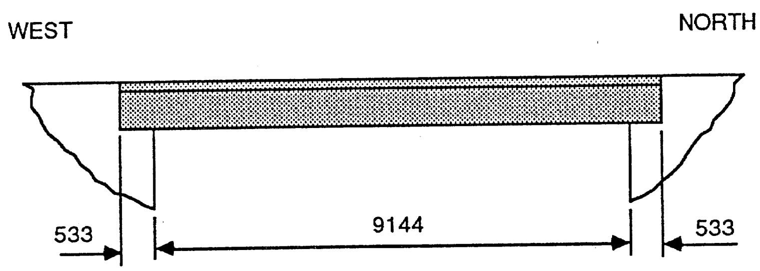


Figure 1/ View of the 18th Avenue Bridge



(a) CROSS-SECTION



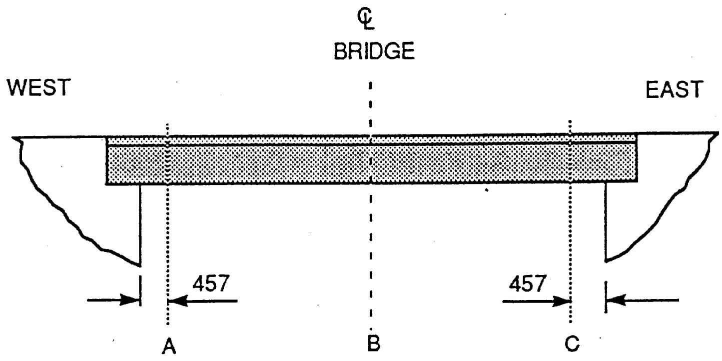
(b) ELEVATION

Figure 2/ Bridge Geometry

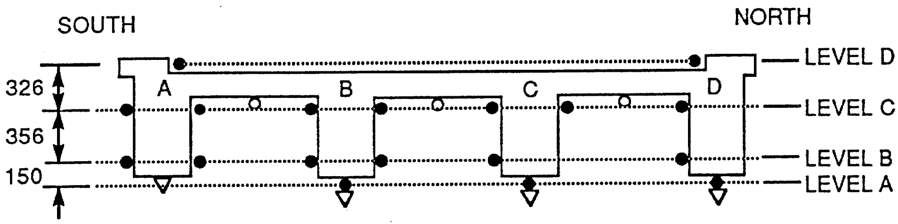
(a) GIRDER A (SOUTH)

(b) GIRDERS B, C and D (NORTH)

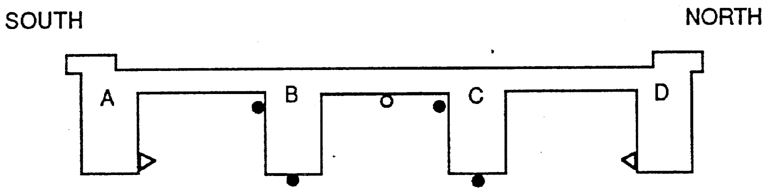
Figure 3/ Views of the underside of the bridge



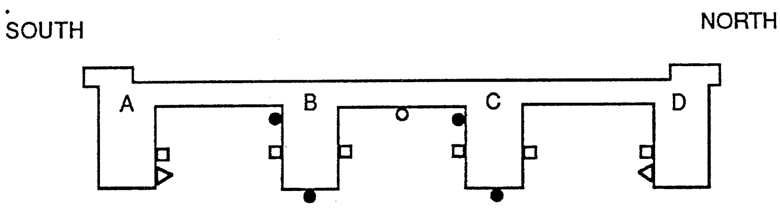
(a) INSTRUMENTED SECTIONS



(b) INSTRUMENTATION AT SECTION B



(c) INSTRUMENTATION AT SECTION C

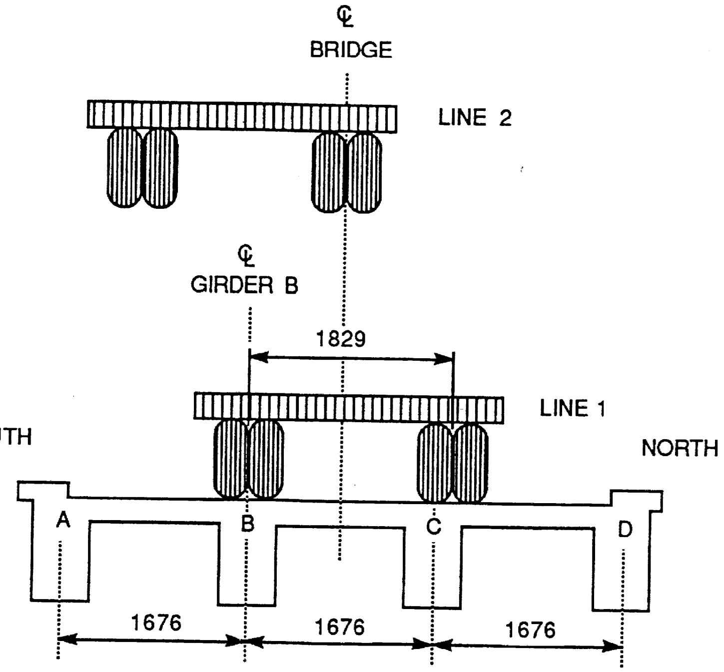


(d) INSTRUMENTATION AT SECTION A

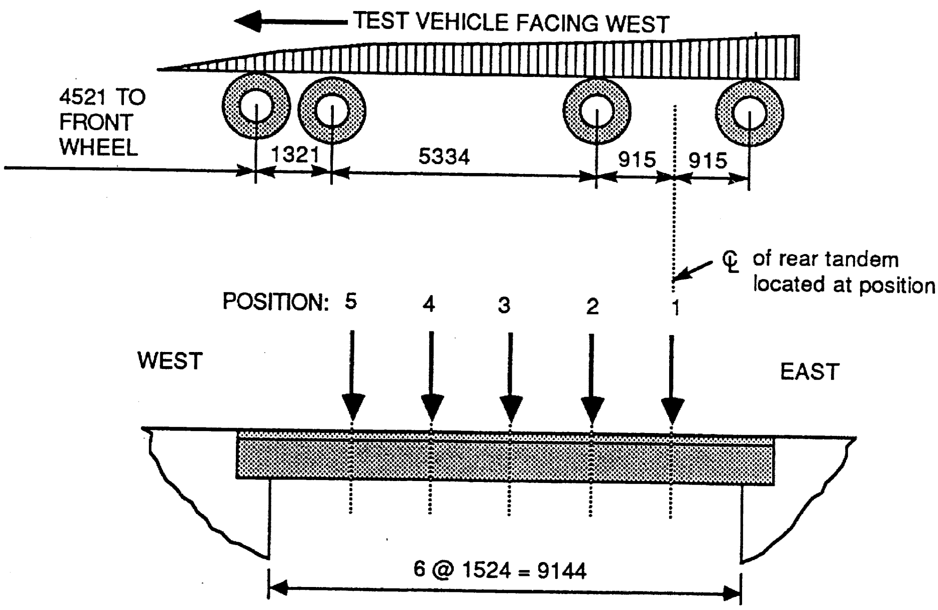
LEGEND

- ● – Longitudinal strain transducers
- o – Transverse strain transducers
- □ – Strain transducers across diagonal cracks
- ▽ – Vertical displacement transducers
- ◁, ▷ – Longitudinal horizontal displacement transducers

Figure 4/ Details of Instrumentation



(a) TRANSVERSE POSITIONS OF TEST VEHICLE



(b) LONGITUDINAL POSITIONS OF TEST VEHICLE

Figure 5/ Details of Loading

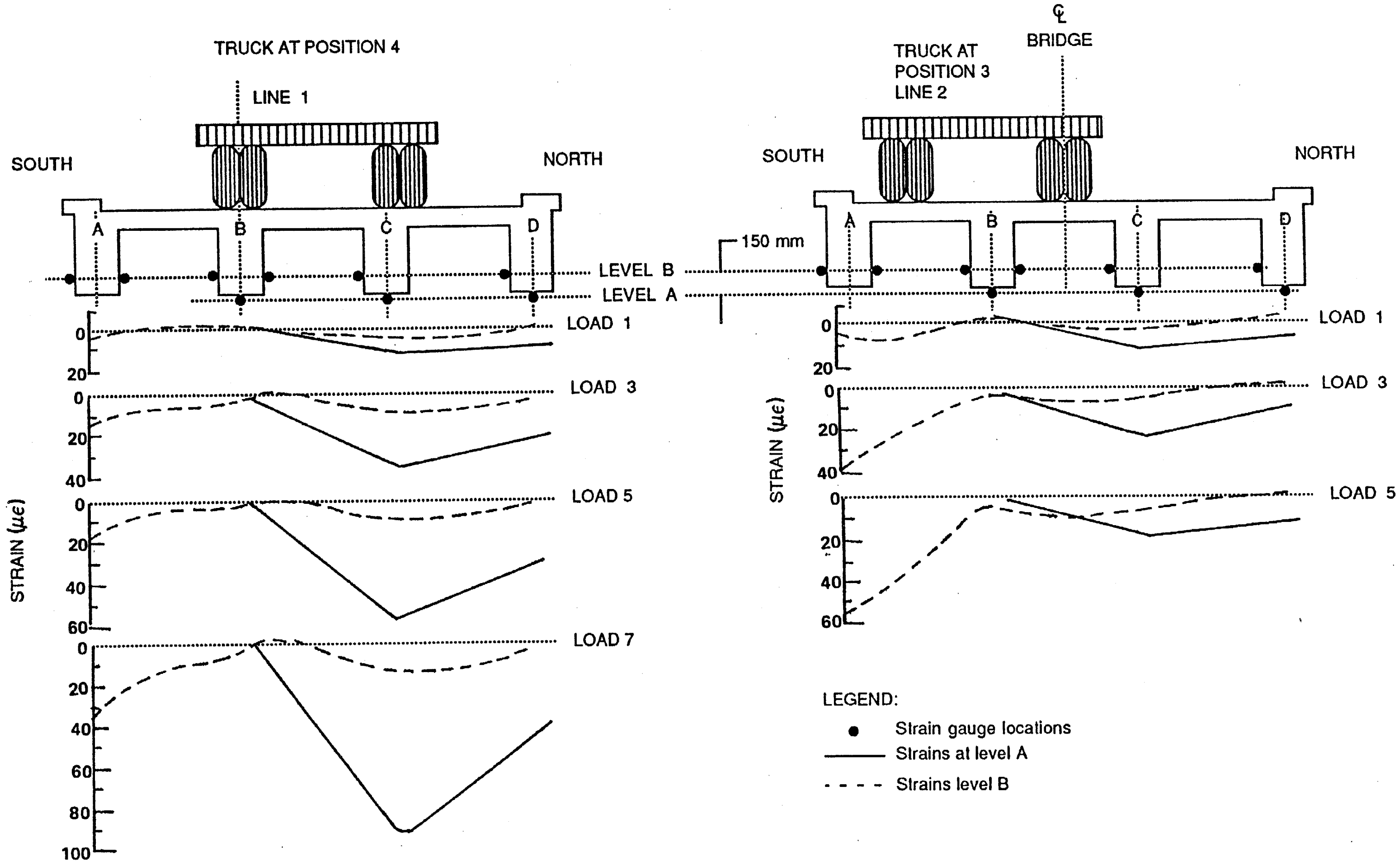


Figure 6/ Transverse Distribution of Tensile Longitudinal Strains at Midspan

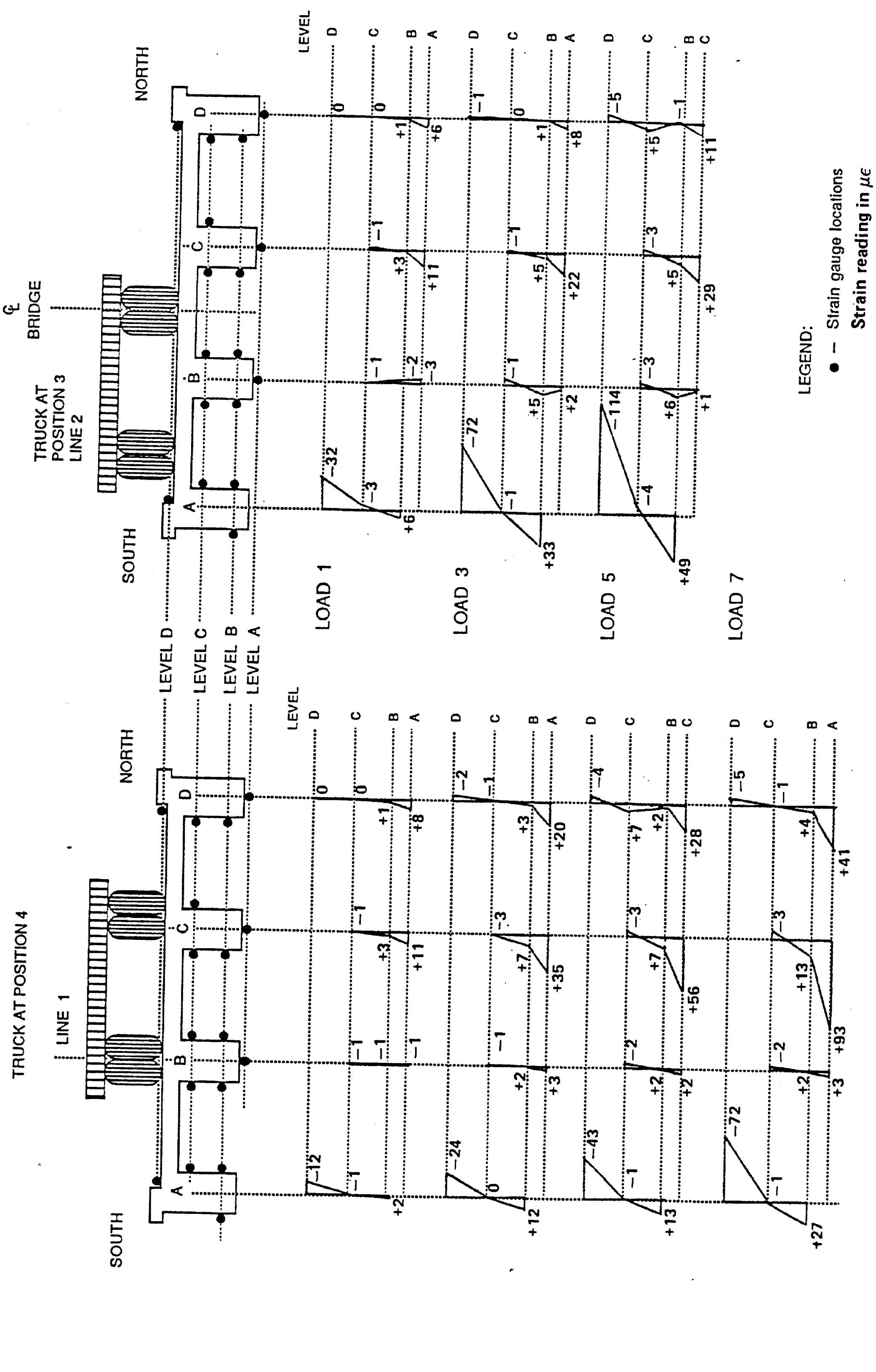


Figure 7/ Longitudinal Midspan Strains

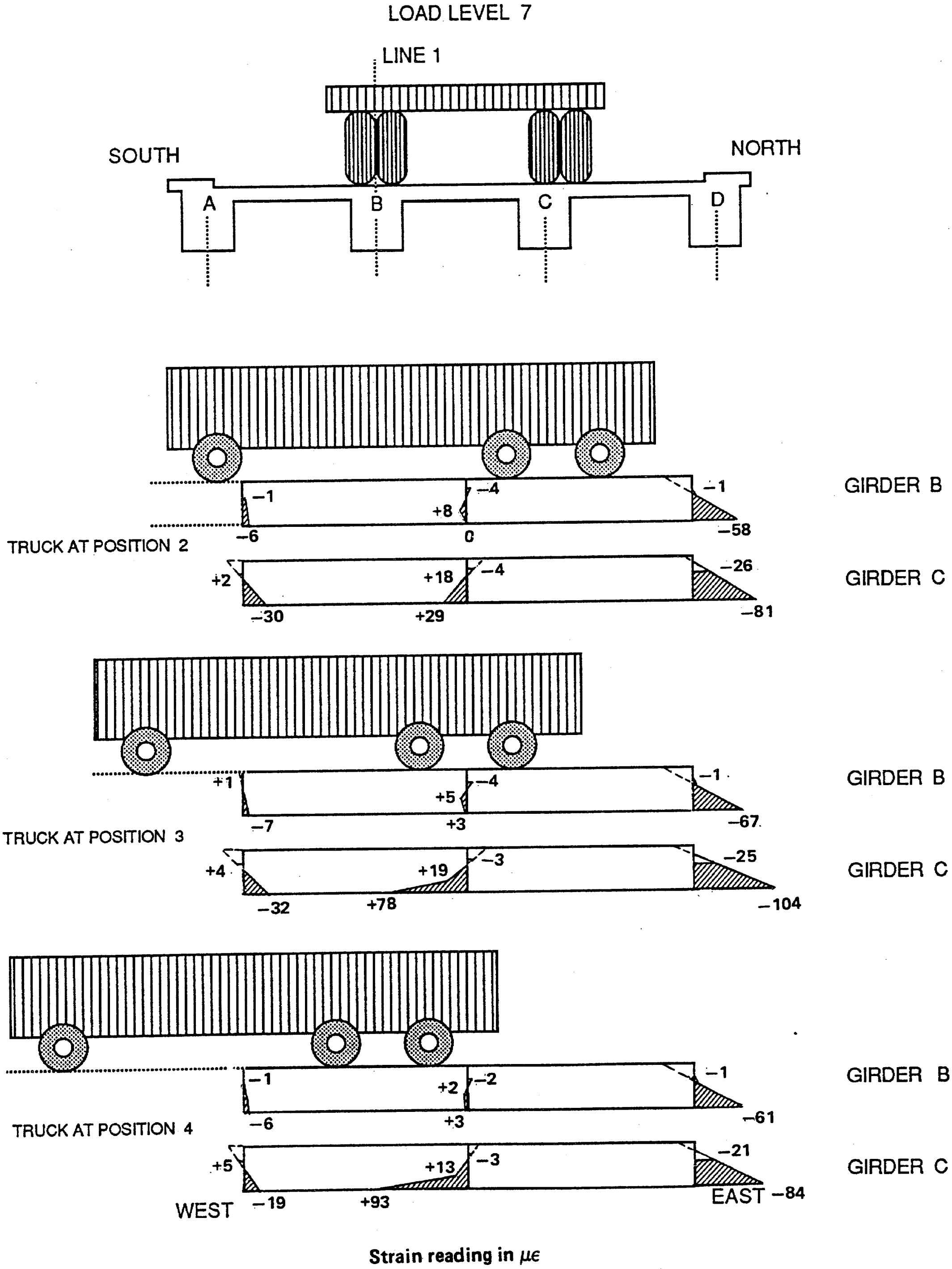


Figure 8/ Longitudinal Strains Along Interior Girders

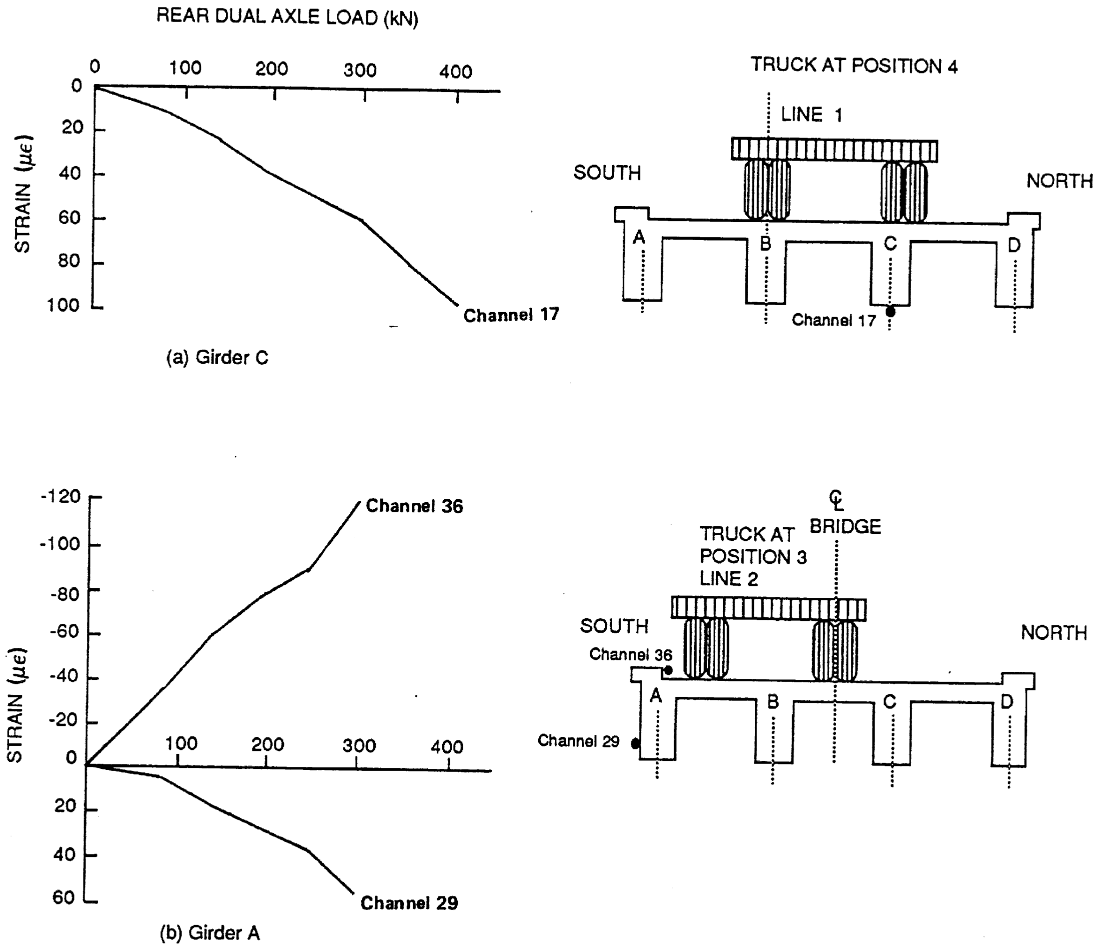
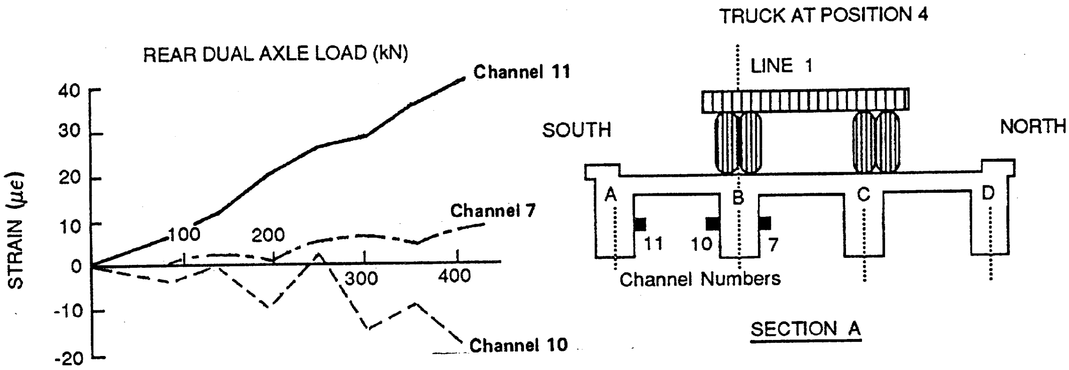
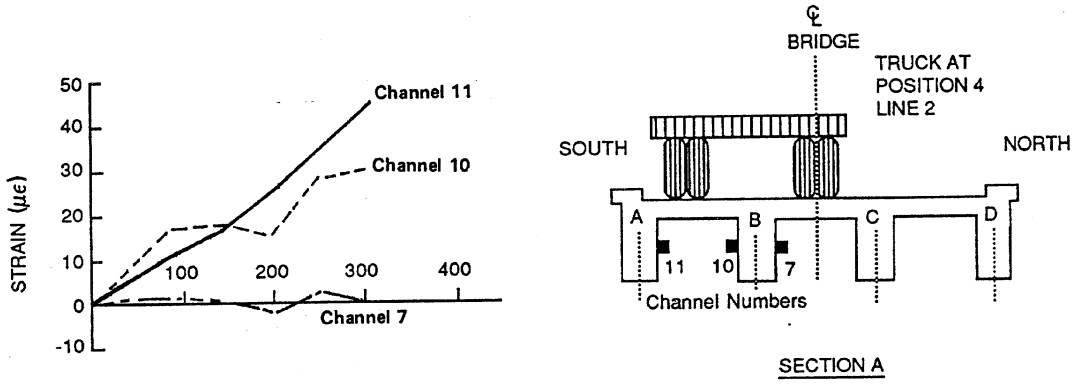


Figure 9/ Longitudinal Strains vs. Applied Load



(a) Line 1 Loading



(b) Line 2 Loading

Figure 10/ Strains Across Diagonal Cracks

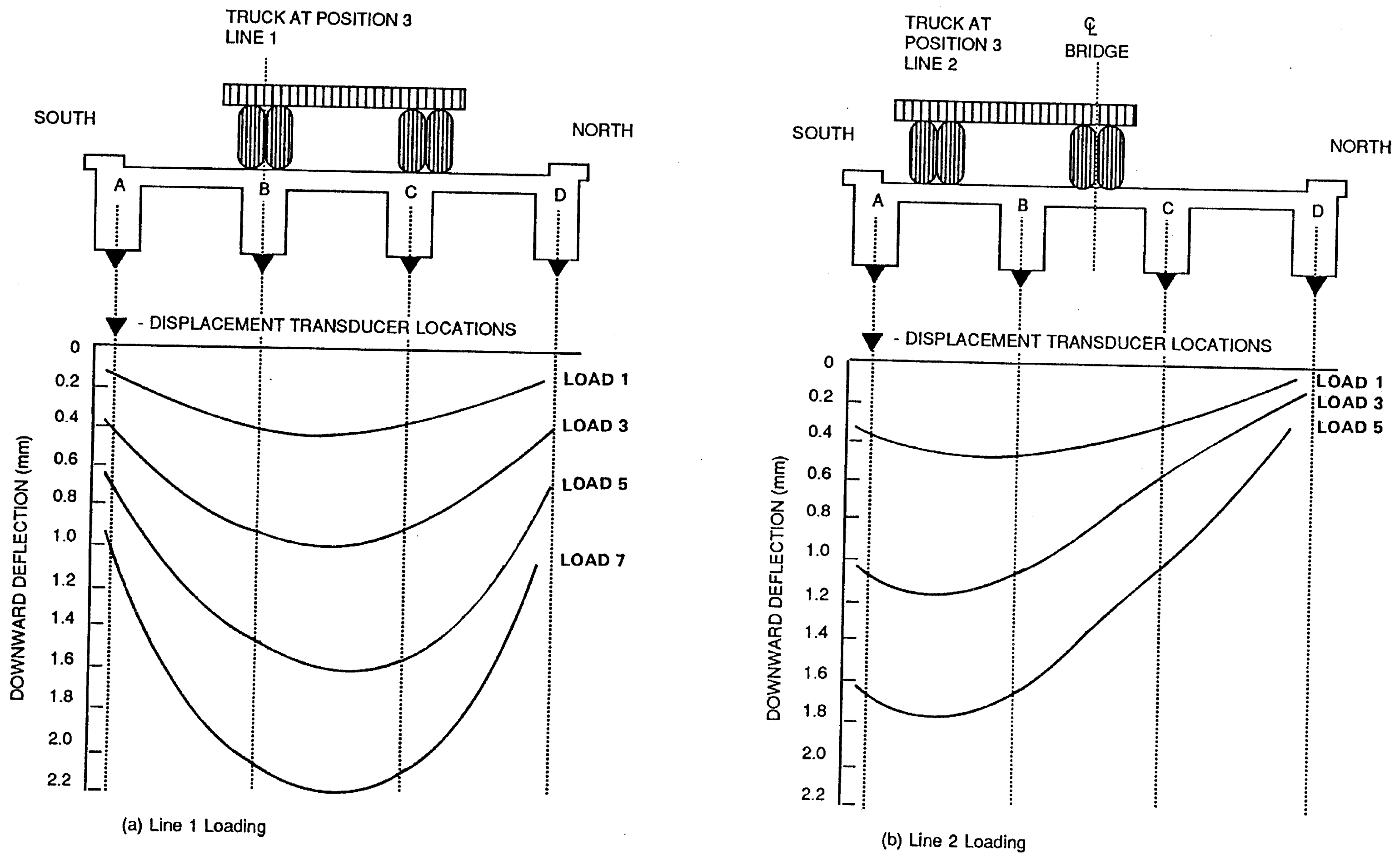


Figure 11/ Vertical Deflections at Midspan

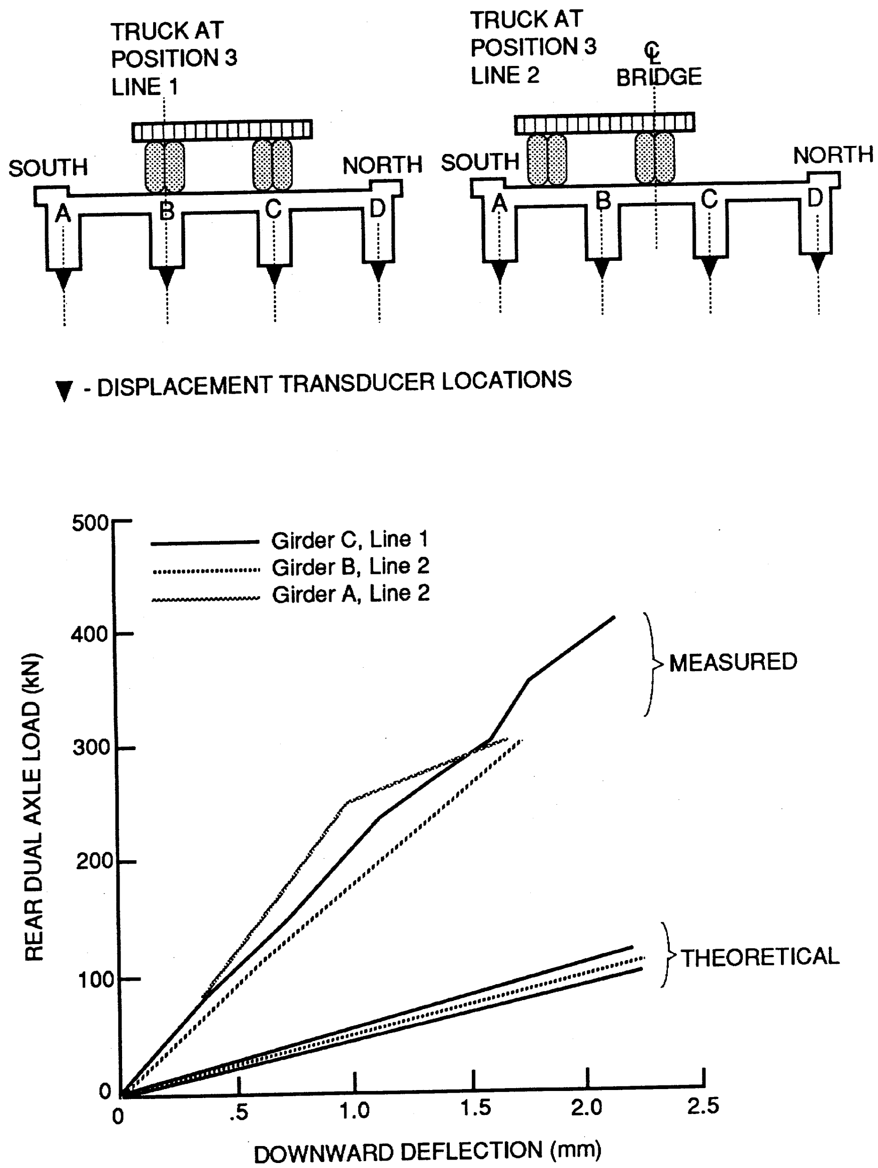


Figure 12/ Load vs. deflection