October 1996

Research Report UMCEE 96-10

LOAD TESTING OF BRIDGES

Report submitted to

the Michigan Department of Transportation

and

the Great Lakes Center for

Truck and Transit Research

by

Andrzej S. Nowak and Vijay K. Saraf

Department of Civil and Environmental Engineering
University of Michigan
Ann Arbor, Michigan 48109-2125

Testing and Research Section
Construction and Technology Division
Research Project No. RC -1347

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the information presented herein. This document is disseminated under the sponsorship of the Michigan Department of Transportation and Great Lakes Center for Truck and Transit Research at the University of Michigan Transportation Research Institute, in the interest of information exchange. The Michigan Department of Transportation assumes no liability for the contents or use thereof.

EXECUTIVE SUMMARY

The objective of the project is to develop an efficient proof load testing procedure for bridges. Knowledge of the actual load carrying capacity is important for a rational management of bridges. The decision concerning bridge repair, rehabilitation and/or replacement has clear economic consequences. The allocation of limited resources must be based on evaluation of existing structures. However, many of such bridges cannot be analytically evaluated because of extensive deterioration and/or lack of documentation. In other cases, analytical methods do not reveal and quantify the actual load carrying capacity. As a result, some bridges, which can still carry the imposed loads (even if for a limited period of time), have to be scheduled for immediate repair or replacement.

Proof load testing can be used as an efficient way to verify the minimum load carrying capacity of the bridge. To be meaningful, the proof load level must considerably exceed the legal load. Michigan allows very heavy trucks and, therefore, the required proof load is also very high. Field testing is an increasingly important topic in the effort to deal with the deteriorating infrastructure, in particular bridges and pavements. There is a need for accurate and inexpensive methods for diagnostics, verification of load distribution, and determination of the actual load carrying capacity. The major requirements for proof load include:

- Magnitude (weight) must be about twice the legal load.
- Possibility for gradual application (proof load must be applied in steps by gradually increasing the load effect).
- Easy to move (self-propelled is the best, use of cranes and/or other lifting equipment can be complicated, time consuming and expensive).
- Easy installation of equipment.
- Short duration (traffic control is often very important, therefore, the test should be limited to a minimum time).

Therefore, in this study, the proof load is applied in the form of military tanks. Each tank weighs over 500 kN (110 kips), distributed over a track length of 4.5m (15 ft). For bridges with a span length of 6m (20 ft), a single tank produces bending moments that are close to the required proof load level. For longer spans up to 15m (50 ft), two tanks are needed. The tanks are brought to the test site by a commercial truck. They are then moved to the required position on the bridge by the tank operator.

The instrumentation includes strain transducers for the measurement of strains and LVDT's (linear variable differential transformer) for the measurement of deflections. The strain transducers are attached to the lower flanges of steel girders. The LVDT's are attached to the bottom of the beams. Tripods are used to anchor the wires to the ground.

The developed procedure is demonstrated on five selected bridges. The selection of bridges and testing procedure was coordinated with the Michigan Department of Transportation (MDOT) staff. Two of these structures are reinforced concrete T-beams, and the remaining three are steel girders with a concrete slab. All bridges are located in lower Michigan and are over 60 years old. The span length of these bridges ranges from 6 m to 15 m (20 to 50 ft). Some of the selected structures showed a considerable degree of deterioration (corrosion). One of the steel bridges was repaired (strengthened) prior to the proof load test and the plans were not available on one of the concrete T-beam bridges.

The M-60 military tanks were furnished by the Michigan National Guard. The moment at mid-span is increased in several steps by gradually moving the tanks towards the center of the span. Tanks are also placed in three different transverse load positions, called upstream, center and downstream.

The smaller than analytically predicted stresses/deflections and a linear behavior indicate the inherent safety reserve of the structure. Other signs of distress, such as cracking, spalling of concrete, etc., were also monitored at all stages of the test.

The measurements were taken by the project team. The equipment used was provided by the University of Michigan. Traffic control was provided by MDOT.

The results clearly indicate that the strains and deflections are smaller than predicted by analysis. The tested bridges showed a considerable safety reserve beyond the legal load level. All of them were found adequate to carry legal truck traffic.

ACKNOWLEDGMENTS

The presented study was sponsored by the Michigan Department of Transportation and Great Lakes Center for Truck and Transit Research (GLCTTR) at the University of Michigan Transportation Research Institute, which is gratefully acknowledged. The authors thank the technical staff of the Michigan DOT, in particular Roger Till, Leo DeFrain, David Juntunen, Richard Gould, Jon Reinke, and Sudakhar Kulkarni for their useful comments, discussions and support, and Thomas Gillespie, of the University of Michigan Transportation Research Institute, for his support.

The Project Team received help from other researchers, students and staff of the University of Michigan. In particular, thanks are due to Andrej F. Sokolik, Sangjin Kim, Chan Hee Park, Jonathan Yaek, and Brian Mohr. They were involved in field instrumentation and measurements.

Thanks are due to the Michigan National Guard, in particular Col. Ed Wilkins, for providing tanks and staff for the experiments. The authors would also like to acknowledge the Michigan State Police for their cooperation. Traffic control was provided by the Michigan DOT.

The realization of the research program would not be possible without in-kind support of the MDOT and the University of Michigan. Measurements were taken using a data acquisition system funded by the University of Michigan. The University of Michigan also provided support for other needed equipment, including a van, and for technician support.

TABLE OF CONTENTS

Executive Summary iii

Acknowledgments vi

1. Introduction 1
2. Selection of Bridges 7
3. Proof Load Level and Load Position 13
4. Monitoring Procedure 29
5. Analytical Modeling 35
6. Bridge No. 1: M-66 over Mud Creek in Woodburry County 37
7. Bridge No. 2: M-50 over Grand River in Jackson County 49
8. Bridge No. 3: M-103 over White Pigeon River in St. Joseph County ..67
9. Bridge No. 4: M-21 over Little Maple River in Clinton County 95
10. Bridge No. 5: M-153 over Fellow Creek in Canton Township 111
11. Summary and Conclusions 133
12. References 135

Appendix A - Additional Results for Bridge No. 1 .... A1

Appendix B - Additional Results for Bridge No. 2 .... A17

Appendix C - Additional Results for Bridge No. 3 .... A57

Appendix D - Additional Results for Bridge No. 4 .... A103

Appendix E - Additional Results for Bridge No. 5 .... A151

Appendix F - Miscellaneous / Hand Calculations A179

Note:

Intentionally left blank

1. INTRODUCTION

Field testing is an increasingly important topic in the effort to deal with deteriorating infrastructure, in particular bridges and pavements. There is a need for accurate and inexpensive methods for diagnostics, verification of load distribution, and determination of the actual load carrying capacity.

Recent studies (FHWA, 1989) indicate that 40 percent of the national bridges are rated structurally deficient or functionally obsolete. More than a fourth of all bridges are over 50 years of age. In addition to the deterioration of bridges due to age, corrosion and poor maintenance, the actual live loads on bridges have also increased considerably. For example, in 1950 the maximum observed gross vehicle weight (GVW) of a truck was only 500 kN (110 kips) in the state of Michigan (Michigan Bridge Analysis Guide, 1983). However, 45 years later during a weigh-in-motion study on several highways, the maximum GVW of 1,110 kN (250 kips) was recorded by Laman (1995).

Some older bridges cannot be evaluated analytically because the documentation is not available. In particular, this applies to reinforced concrete structures with unknown presence and location of reinforcement.

The deficient bridges are posted, repaired or replaced. The disposition of bridges involves clear economical and safety implications. To avoid high costs of replacement or repair, it is necessary to know accurately the present load carrying capacity of the structure, and predict loads and any further changes in the capacity (deterioration) in the applicable time span.

Accuracy of bridge evaluation can be improved by using recent developments in bridge diagnostics, structural tests, material tests, structural analysis and probabilistic methods. Advanced diagnostic procedures can be applied to the evaluation of the current capacity of the structure, monitoring of load history and evaluation of the accumulated damage. Full scale bridge tests provide very useful information about the structural behavior. There is a need for significantly more test data, covering various bridge types. However, extensive test programs are very costly. Therefore, a considerable effort should be directed towards evaluation and improvement of the current analytical methods, on the basis of available test data.

A considerable number of Michigan bridges are more than 40 years old. Some of them show signs of deterioration. In particular, there is severe corrosion on many steel and concrete structures. Therefore, MDOT recently proof-load tested a reinforced concrete T-beam bridge near Grand Rapids to determine the minimum load carrying capacity.

There is a need for more proof-load tests, diagnostic tests, and other tests to verify the analytical evaluations. Therefore, the proposed project is focused on field testing of bridges.

The objective of the proposed project is the development of a proof load testing procedure for the Michigan DOT. The field tests were carried out on selected bridges to determine their minimum capacity, verify the distribution of load, and identify the critical components and sections. Five structures were tested. The selection of bridges and testing procedure was coordinated with the Michigan DOT staff.

The study was based on the available knowledge and data regarding the methodology, structural behavior (material properties, member resistance) and bridge loads. The testing procedures were carried out in accordance with the draft of the Manual for Bridge Rating Through Non-Destructive Load Testing, developed as a result of the NCHRP Project 12-28(13)A (NCHRP, 1993). The project also used the results and experience gained in the prior studies.

The theoretical evaluation of the bridge load capacity requires accurate information about material properties, support behavior, contribution of non-structural members, effect of deterioration, load distribution and slab-girder interaction. For simplicity, conservative assumptions are made to account for these parameters in the analysis. Therefore, it is often observed during load testing of bridges that the actual load carrying capacity is higher than that determined using analytical methods (Bakht 1990). In certain cases, this extra safety reserve in the load capacity can be utilized to prove that the bridge is adequate, thus avoiding replacement or rehabilitation. The objective of this study is to determine the adequacy of selected bridges to carry legal truck traffic without any load posting.

Proof load tests were carried out on selected bridges. The M-60 military tanks, from the Michigan National Guard, are used as the proof load. The moment at mid-span is increased in several steps by gradually moving the tanks towards the center of the span. Tanks are also placed in three different transverse load positions, called upstream, center and downstream. The structure is instrumented, and strains and deflections are measured at selected locations using a portable data acquisition system from National Instruments.

The smaller than analytically predicted strains/deflections and a linear behavior indicate the inherent safety reserve of the structure. Other signs of distress, such as cracking, spalling of concrete, etc., are also monitored at all stages of the test.

In this project, bridges were selected from a list of structures prepared by the Michigan Department of Transportation. Many of these bridges had poor load rating, and moderate to extensive deterioration. The selection criteria included:

- accessibility for the equipment (water level, clearance, etc.)
- low load rating
- extensive deterioration
- missing design documents

A total of five bridges were selected. Two of them are reinforced concrete T-beam structures, and the remaining three are steel girder bridges with reinforced concrete slab. All are located in lower Michigan and are over 60 years old. The span lengths of these bridges range from 6 m to 15 m. (20 to 50 ft)

The measurements were taken by the University of Michigan project team. The equipment used was provided by the University of Michigan. Traffic control was provided by MDOT.

The Report is divided into 11 chapters.

The basic parameters of the selected bridges are presented in Chapter 2. For an easier reference, each bridge is numbered in the order it was tested. The current rating factors for some bridges were provided by MDOT.

Chapter 3 provides the description of the methodology used to calculate the target proof load level. The selection of load, and the load positions are also included. The equipment and its operation are summarized in Chapter 4. A detailed analytical modeling of every structure is required before the test, in order to compare it with experimental results. Two different types of analytical model were prepared using the computer program SECAN. Chapter 5 contains the details of analytical modeling.

The results of the measurements are shown in Chapters 6 through 10. For each tested bridge, the provided data includes a description of geometrical parameters with elevation, cross section, information concerning skewness, and layout of the girders, along with cross-sectional properties of the girders. These chapters contain the response of only selected girders and locations for each bridge. The processed data and the responses of the remaining girders can be found in the appendices. The applied lane moment at mid-span and quarter points are summarized in the tables.

The measured girder deflections and stresses in lower flanges of steel girders are plotted with the applied lane moment. The stresses in steel girders were calculated from the measured strains by multiplying them with the theoretical modulus of elasticity. Experimental results are also compared with analytically predicted values. The lateral distribution of load between girders is shown in terms of stresses and deflections. Longitudinal profiles of the response for selected girders are also shown in the figures. The applied proof load lane moments are compared with 2-unit 11-axle truck moments, and the maximum stresses and deflections are compared with the allowable limits.

The summary and conclusions are presented in Chapter 11.

Note:

Intentionally left blank

2. SELECTION OF BRIDGES

A list of 11 candidate bridges was prepared by MDOT for proof load testing. Major parameters of bridges identified by MDOT are shown in Table 2-1. The selection criteria included:

- low load rating
- signs of deterioration (spalling concrete, cracks, corrosion)
- missing design documents
- accessibility for the equipment (clearance, water level, etc.)

All bridges selected for this study were inspected by the project team, prior to the experimental work. In addition to the inspection of deterioration, careful observations were made regarding the accessibility of the bridge, placement of the LVDT's, strain transducers and data acquisition system, parking of the testing van and the military tanks. Actual dimensions of various bridge components were measured and compared with the design drawings supplied by MDOT. The reduction of flange thickness and the area of corrosion, and concrete spalling were also noted. Details of the field inspection of all bridges are described in the following chapters.

Finally, five bridges were selected for the proof load testing, primarily based on accessibility. These bridges are listed in Table 2-2. The first four bridges in Table 2-2 were tested during the summer and fall of 1995, and the fifth bridge was tested in the spring of 1996.

In the report, each bridge is denoted by a number. The numbers are assigned in the order these bridges were tested.

1. Bridge No. 1 - M-66 over Mud Creek in Woodbury county, Michigan.
   Michigan State Bridge ID: B06-08052

Table 2-1 (a) : List of Bridges Identified by MDOT for Proof Load Testing. (English units)

No.	MDOT ID #	County	Nearest City	Route Carried	Facility Below	Miles to Bridge	Year Built	Structure Length	Bridge Width	Bridge Height	Bridge Clearance	Road Width	Number of Spans	Span Lengths	Structure Type	Additional Comments	ADT	Date Visited	Notes
1	B01-19062 *	Clinton	Ovid	M-21	Little Maple River	77	1929	About 59'	46.7'	About 38"	About 9'	40.0'	1	38'-6"	Steel girder and concrete slab	Good access and shallow water	5100	5/22/95	Deterioration
2	B06-08052 *	Barry	Woodburry	M-66	Mud Creek	96	1918	About 36'	30.5'	About 24"	About 7'	26.0'	1	21'-6"	Reinforced concrete T-beam	Great access and shallow water	2100	5/22/95	Load / no plans
3	B02-70023	Ottawa	Holland	BL I-196 Westbound	Black River	158	1930	About 203'	46.4'	-	About 12'	40.0'	3	each 60'-0"	Steel girder and concrete slab	Poor access and deep water	7400	5/22/95	Deterioration
4	B01-78011 *	St. Joseph	Mottville	M-103	White Pigeon River	140	1931	About 120'	46.4'	About 36"	About 9'	40.0'	2	each 48'-6"	Steel girder and concrete slab	Partly closed with stoplight Poor access	5300	5/22/95	Evaluation of repair / Load
5	B01-78062	St. Joseph	Colon	M-86	Swan Creek	109	1932	About 180'	42.4'	About 34"	About 11'	30.0'	3	each 33'-0"	Steel girder and concrete slab	Piers with steel girders Poor access	6200	5/22/95	Deterioration
6	R01-81063	Wastenaw	Ypsilanti	US-12 Eastbound	Conrail Railroad Tracks	10	1944	About 150'	73.8'	Varies	About 20' midspan	70.6'	3	41'-4" 56'-4" 41'-4"	Reinforced concrete T-beam	Bad condition Poor access	7900	5/13/95	Load
7	R02-81063	Wastenaw	Ypsilanti	US-12 Westbound	Conrail Railroad Tracks	10	1944	About 170'	73.8'	Varies	About 20' midspan	70.6'	3	46'-0" 62'-6" 46'-0"	Reinforced concrete T-beam	Bad condition Cores taken Poor access	11800	5/13/95	Load
8	B01-82081 *	Wayne	Canton Township	M-153	Fellows Creek	12	1920 Widened in 1979	About 45'	80.2'	About 18" and About 32"	About 6'	74.0'	1	25'-6"	T-beams at old span, slab only at new	Great access and little water	32400	5/13/95	Load
9	B04-58053	Monroe	Flat Rock	US-24	Huron River	35	1933	About 230'	78.4'	About 68"	About 11'	60.0'	2	81'-3" 77'-11"	Steel girder and concrete slab	Girder #8 flange gone Poor access	8100	5/13/95	Load / Deterioration
10	B02-38071 *	Jackson	Jackson	M-50 and BL US-127	Grand River	40	1926	About 70'	45.3'	About 34"	About 7'	40.0'	1	48'-0"	Steel girder and concrete slab	Good access and shallow water	11900	5/13/95	Deterioration
11	B01-45013	Leelanau	Leland	M-22	Lake Leelanau Outlet	262	1929	-	49.1'	About 50"	-	36.0'	1	60'-0"	Steel girder and concrete slab	Far away	1600	N/A	Load

* Proof load tested bridges in this study

Table 2-1 (b) : List of Bridges Identified by MDOT for Proof Load Testing. (SI units)

No.	MDOT ID #	County	Nearest City	Route Carried	Facility Below	Kms to Bridge	Year Built	Structure Length	Bridge Width	Bridge Height	Bridge Clearance	Road Width	Number of Spans	Span Lengths	Structure Type	Additional Comments	ADT	Date Visited	Notes
1	B01-19062 *	Clinton	Ovid	M-21	Little Maple River	123	1929	About 18m	14m	About .96m	About 2.74m	12.2m	1	11.7m	Steel girder and concrete slab	Good access and shallow water	5100	5/22/95	Deterioration
2	B06-08052 *	Barry	Woodburry	M-66	Mud Creek	154	1918	About 11m	9.3m	About .61m	About 2.13m	7.9m	1	6.6m	Reinforced concrete T-beam	Great access and shallow water	2100	5/22/95	Load / no plans
3	B02-70023	Ottawa	Holland	BL I-196 Westbound	Black River	253	1930	About 62m	14m	-	About 3.66m	12.2m	3	each 18.3m	Steel girder and concrete slab	Poor access and deep water	7400	5/22/95	Deterioration
4	B01-78011 *	St. Joseph	Mottville	M-103	White Pigeon River	224	1931	About 37m	14m	About .91m	About 2.74m	12.2m	2	each 14.8m	Steel girder and concrete slab	Partly closed with stoplight Poor access	5300	5/22/95	Evaluation of repair / Load
5	B01-78062	St. Joseph	Colon	M-86	Swan Creek	174	1932	About 55m	13m	About .86m	About 3.35m	9.1m	3	each 10.1m	Steel girder and concrete slab	Piers with steel girders Poor access	6200	5/22/95	Deterioration
6	R01-81063	Wastenaw	Ypsilanti	US-12 Eastbound	Conrail Railroad Tracks	16	1944	About 46m	22.5m	Varies	About 6.10m midspan	21.5m	3	12.6m 17.2m 12.6m	Reinforced concrete T-beam	Bad condition Poor access	7900	5/13/95	Load
7	R02-81063	Wastenaw	Ypsilanti	US-12 Westbound	Conrail Railroad Tracks	16	1944	About 52m	22.5m	Varies	About 6.10m midspan	21.5m	3	14.0m 19.1m 14.0m	Reinforced concrete T-beam	Bad condition Cores taken Poor access	11800	5/13/95	Load
8	B01-82081 *	Wayne	Canton Township	M-153	Fellows Creek	19	1920 Widened in 1979	About 14m	24.4m	About .46 and About .81	About 1.83m	22.6m	1	7.8m	T-beams at old span, slab only at new	Great access and little water	32400	5/13/95	Load
9	B04-58053	Monroe	Flat Rock	US-24	Huron River	56	1933	About 70m	23.9m	About 1.73m	About 3.35m	18.3m	2	24.8m 23.8m	Steel girder and concrete slab	Girder #8 flange gone Poor access	8100	5/13/95	Load / Deterioration
10	B02-38071 *	Jackson	Jackson	M-50 and BL US-127	Grand River	64	1926	About 21m	13.8m	About .86m	About 2.13m	12.2m	1	14.6m	Steel girder and concrete slab	Good access and shallow water	11900	5/13/95	Deterioration
11	B01-45013	Leelanau	Leland	M-22	Lake Leelanau Outlet	419	1929	-	15m	About 1.27m	-	11.0m	1	18.3m	Steel girder and concrete slab	Far away	1600	N/A	Load

* Proof load tested bridges in this study

Table 2-2. Summary of Selected Bridges.

Bridge No.	Year Built	Span m (ft)	Type	Reasons for Testing
1	1918	6.5 (21.4)	RC T-Beams	no plans / low load rating
2	1926	14.6 (48)	Steel Girders	deterioration
3	1931	14.8 (48.5)	Steel Girders	evaluation of repair / low load rating
4	1929	11.7 (38.5)	Steel Girders	deterioration
5	1920	7.8 (25.5)	RC T-Beams	low load rating

2. Bridge No. 2 - M-50 over Grand River in Jackson county, Michigan.
   Michigan State Bridge ID: B02-38071

3. Bridge No. 3 - M-103 over White Pigeon River in St. Joseph county, Michigan.
   Michigan State Bridge ID: B01-78011

4. Bridge No. 4 - M-21 over Little Maple River in Clinton county, Michigan.
   Michigan State Bridge ID: B01-19062

5. Bridge No. 5 - M-153 over Fellows Creek in Canton township, Michigan.
   Michigan State Bridge ID: B01-82081

These bridges are more than 60 years old and have moderate to extensive deterioration. For the first bridge, the design details were unavailable and for other bridges the rating factors were low. Bridge No. 3 was under emergency repair at the time of the preliminary inspection. However, the test on this bridge was carried out after the repair was finished.

Material properties for selected bridges are summarized in Table 2-3. The value of yield strength of structural steel for Bridge No. 2 was taken from the coupon tests performed by MDOT. Other values are based on the Michigan Bridge Analysis Guide (1983) according to the year of construction.

Table 2-3. Material Properties of Selected Bridges.

Bridge	Structural Steel		
No.	F_y ,GPa (ksi)	E_s, GPa (ksi)	G_s, GPa (ksi)
1	-	-	-
2	290 (42,000)	200 (29,000)	77 (11,154)
3	207 (30,000)	200 (29,000)	77 (11,154)
4	207 (30,000)	200 (29,000)	77 (11,154)
5	-	-	-
Bridge	Concrete		
No.	f_c', MPa (psi)	E_c, GPa (ksi)	G_c, GPa (ksi)
1	14 (2,000)	18 (2,550)	7 (1,060)
2	17 (2,500)	20 (2,850)	8 (1,190)
3	17 (2,500)	20 (2,850)	8 (1,190)
4	17 (2,500)	20 (2,850)	8 (1,190)
5	14 (2,000)	18 (2,550)	7 (1,060)

$E_c = 57{,}000 \sqrt{f_c'}$ psi, where f_c' is in psi
$\nu = 0.20$ for concrete
$\nu = 0.30$ for steel

Note:

Intentionally left blank

3. PROOF LOAD LEVEL AND LOAD POSITIONS

The rating calculations and a preliminary design check of the selected bridges were carried out using the available design details and the deterioration (section loss) observed during site inspection. The girder moment close to mid-span, due to the 11-axle two unit truck, was found to be the critical limit state for each bridge. Therefore, the proof load testing was designed to verify the moment capacity of steel girders close to mid-span. Before the proof load tests, the target proof load has to be calculated. The type and placement of load, instrumentation and data acquisition setup would depend on the target proof load level.

3.1 Proof Load Level

3.1.1 General

Proof load testing can be used either to find the yield capacity of the structure, or to check its ability to carry a specified live load. Usually, the yield capacity of a bridge is very high and requires exceptionally heavy loads, which make the tests uneconomical and slow. In this study, proof load tests were carried out to verify if the bridge can safely carry the maximum allowable legal load. In Michigan, the maximum mid-span moment in medium span bridges is caused by 11-axle two unit trucks with the wheel configuration shown in Figure 3-1. For such an 11-axle truck, the gross vehicle weight (GVW) can be up to 685 kN (154 kip), which is almost twice the allowable legal load in other states. Most states allow a maximum GVW of 356 kN (80 kip) only. It is more than five times the H15 design load or more than twice the HS20 design load (AASHTO, 1992).

The proof load level should be sufficiently higher than that from a two-unit 11-axle truck, to ensure the desired safety level. Until recently,

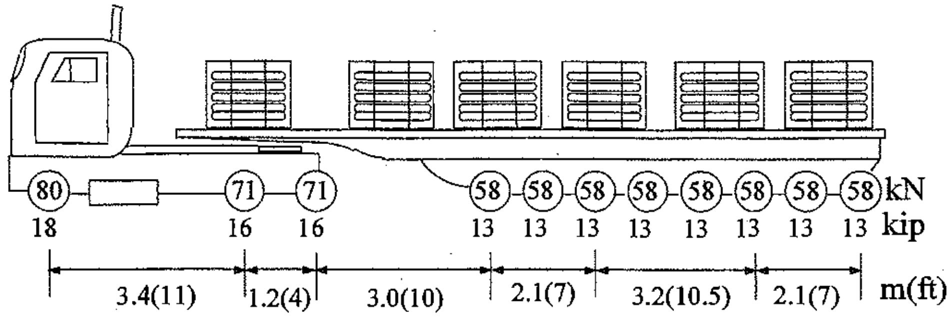


Figure 3-1 : 11-Axle Two-Unit Truck.

the calculation of the appropriate proof load level, was left to the judgment of researchers conducting the test. The final draft NCHRP report titled "Bridge Rating Through Load Testing" by A. G. Lichtenstein (NCHRP, 1993) provides guidelines for calculating the target proof load level. It suggests that the maximum allowable legal load should be multiplied by a factor X_p, which represents the live load factor needed to bring the bridge to an operating rating factor of 1.0. The guide recommends that X_p should be 1.4 before any adjustments are made. It also recommends the following adjustments to X_p, which should be considered in selecting a target live load magnitude.

- Increase X_p by 15 percent for one lane structures or for other spans in which the single lane loading augmented by an additional 15 percent would govern.
- Increase X_p by 10 percent for spans with fracture critical details. A similar increase in X_p shall be considered for structures without redundant load paths.
- Increase X_p by 10 percent if inspections are to be performed less often than 2-year frequency.

- Reduce X_p by 5 percent if the structure is ratable and there are no hidden details, and if the calculated rating factor exceeds 1.0.
- Additional factors including traffic intensity and bridge condition may also be incorporated in the selection of the live load factor X_p.

Application of the recommended adjustment factors, leads to the target live load factor X_{pa}. The net percent increase (Σ) in X_p, is found by summing the appropriate adjustments given above. Then

$$X_{pa} = X_p [1 + (\Sigma/100)] \tag{3-1}$$

The target proof load (L_t) is then:

$$L_t = X_{pa} (1 + I) L_r \tag{3-2}$$

$$1.3 \leq X_{pa} \leq 2.2 \tag{3-3}$$

where,

Σ = net percent increase in X_p, i.e. summation of the appropriate adjustments.

L_r = the live load due to the rating vehicle for the loaded lanes.

I = impact factor.

X_{pa} = the target live load factor.

3.1.2 Impact Factor

Based on the span length, the AASHTO Specifications (AASHTO, 1992) specifies the impact factors of less than 1.3. However, previous studies by several researchers have indicated that the dynamic amplification is much smaller for heavy loads. Chan and O'Connor (1990) showed that the dynamic amplification of load, decreases as the

GVW increases. Later, Chan and O'Connor (1990) reported that the dynamic factor decreases due to the presence of multiple axles in a vehicle. Nowak et. al (1994) also confirmed that the dynamic load, as a fraction of live load, decreases for heavier trucks. In 11-axle trucks, eight rear axles are spaced very close to each other and carry most of the load, which decreases the dynamic amplification. Nassif and Nowak (1995) carried out dynamic tests on several steel girder bridges located in Michigan with spans ranging from 9 to 24 m (30 to 80 ft).

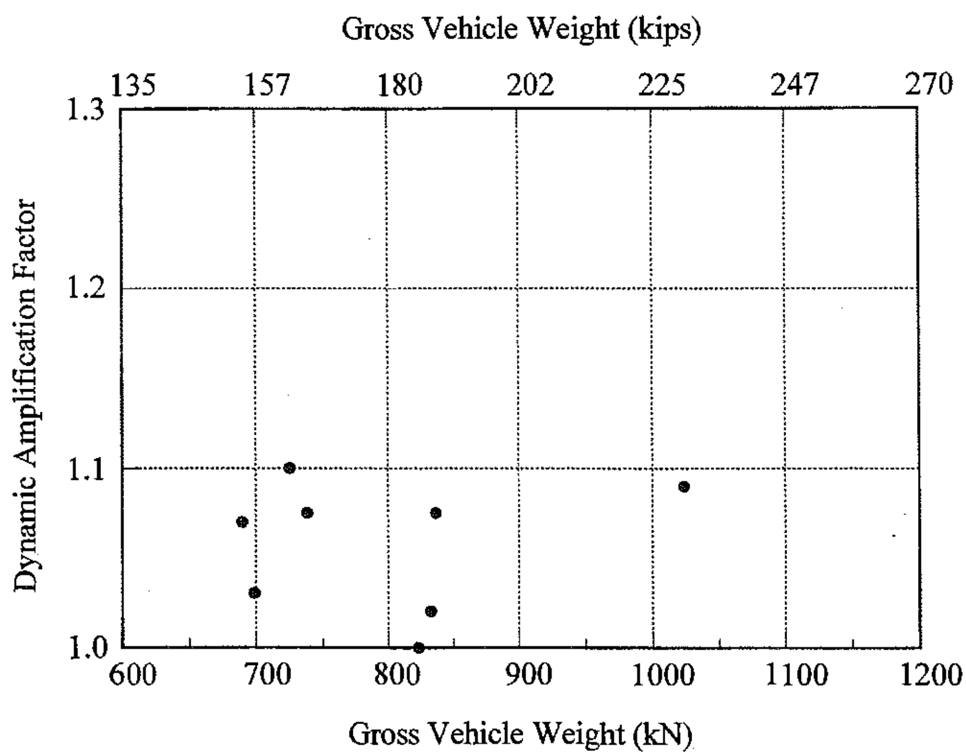


Figure 3-2 : Impact Factors for Heavy Vehicles (Nassif and Nowak, 1995).

As shown in Figure 3-2, the dynamic amplification factors for heavy 11-axle trucks were found to be less than 10 percent. Since allowable loads in Michigan are considerably different than other states, the experimental impact factors would be much more reliable than AASHTO (1992) specified impact factors (see Appendix B of NCHRP Report, 1993). In addition, both the top of the deck and the approach of all five bridges were found to be in good condition, ensuring a low impact factor. Therefore, for this study, an impact factor of 1.10 was selected.

Reduced impact factor resulted in a lower value of the required proof load. For Bridges No. 1 and 3, the provided proof load would satisfy also a higher value of impact factor (1.30).

3.1.3. Calculations

For the first four bridges, It was decided to use only a single lane loading for testing. Hence, it was necessary to supplement the load to account for the live load in the adjacent lanes. Therefore, the target proof load was increased by 15 percent to account for multiple lane loading. All bridges, except Bridge No. 1, were ratable and had no hidden details, therefore, the target proof load was reduced by 5 percent. No other adjustment was applied. The required proof load level was determined as follows:

$X_p = 1.4$

$\Sigma = 15\ \%$ for Bridge No. 1

$\Sigma = 10\ \%$ for Bridge Nos. 2 to 4

$\Sigma = -5\ \%$ for Bridge No. 5

$I = 0.10$ (see section 3.1.2)

$$X_{pa} = 1.4\,[1 + (\Sigma/100)] \quad = 1.61, \text{ for Bridge No. 1}$$
$$= 1.54, \text{ for Bridge Nos. 2 to 4}$$
$$= 1.33, \text{ for Bridge No. 5}$$

All satisfy Equation 3-3

$$\left.\begin{aligned} L_t &= 1.61 \times 1.10\ L_r = 1.77\ L_r\text{, for Bridge No. 1} \\ &= 1.54 \times 1.10\ L_r = 1.69\ L_r\text{, for Bridge Nos. 2 to 4} \\ &= 1.33 \times 1.10\ L_r = 1.46\ L_r\text{, for Bridge No. 5} \end{aligned}\right\} \quad (3\text{-}4)$$

where, L_t is the target proof load or its load effect, and L_r is the maximum allowable legal load or its load effect. Load effects include moment, shear, and axial force, etc. If the proof load test is to be carried out using a vehicle identical to 11-axle truck, then the GVW can be chosen as L_r. If the objective of the proof load is only to create similar load effects as those from a two-unit 11-axle truck, then the critical load effect should be used as L_r. After selecting the appropriate L_r, the target proof load or the load effect (L_t) is determined using Equation 3-4. <u>If the test load safely reaches the target proof load level, then the operating rating factor for a two-unit 11-axle truck would be 1.0.</u>

3.2 Load Selection

If a two-unit 11 axle truck is used as proof load, then its GVW would have to exceed 1,200 kN (273 kip) for Bridge No. 1. In some previous tests by other researchers (Juntunen and Isola, 1995), concrete barrier blocks, each weighing about 22 kN (5 kip), were used as load. However, for this study the required number of concrete blocks would be so large (5 or 6 layers) that it would not be feasible to fit them on one truck.

Other types of loads, such as steel coils, sand and gravel etc., loaded on 11-axle truck, were considered, but it would require considerable effort to place or move the load. In addition, heavy equipments, such as cranes would also be required. Some other options, such as building a water tank on top of the bridge and using water as the proof load, were also investigated, but the size of the tank would have to be very large (e.g. 2.85 m (9.34 ft) high over both lanes for Bridge No. 1, see Appendix F). All these options would require considerable resources and a complete traffic closure for a long time before, during and after the test. In addition, if the test procedure is carried out over a long period of time then the temperature effects can be considerably large and may cause undesirable non-linearity in bridge response.

Finally, instead of GVW (load) the mid-span moment (critical load effect) was selected as L_r. The required proof load mid-span moments for selected bridges were calculated using equation 3-4, and are listed in Table 3-1.

Table 3-1 : Required Proof Load Moments.

Bridge No.	Span in m (ft)	Maximum Moment due to Legal Load in kN-m (k-ft)	Required Proof Load Moment in kN-m (k-ft)
1	6.5 (21.5)	293.0 (216)	519.5 (383)
2	14.6 (48.0)	1,246.5 (919)	2,106.5 (1,553)
3	14.8 (48.5)	1,269.6 (936)	2,145.8 (1,582)
4	11.7 (38.5)	866.7 (639)	1,464.9 (1,080)
5	7.8 (25.5)	417.7 (308)	610.4 (450)*

* two lane loading

The M-60 military tanks were selected as load. Each tank weighs over 55 tons and the load is distributed over a track of 4.5 m (14.5 ft) only. Hence, these tanks cause very high moments at mid-span (also see Section 3.4). For the first bridge, just one tank was enough to generate the required proof load moment, while for the other bridges two tanks were required. For Bridge No. 5, two lanes were loaded at a time. Therefore, for Bridge No. 5, the 15 percent increase for single lane loading was not required. The tanks were provided by the Michigan National Guard. The front and side views of the M-60 tanks are shown in Figures 3-3 and 3-4. For Bridge Nos. 1 and 2, the tanks were placed on flat bed trailers to avoid causing damage to the pavement by tracks of the tanks. Figures 3-5 and 3-6 show the two different trailers used during the proof load test. Only four rear axles of these trailers were used to load the bridge. However, during the testing of the first two bridges, maneuvering the trailers was found to be difficult. For testing of Bridge No. 3, the military trailer was unavailable. Therefore, tanks were placed directly on the pavement. They did not cause any noticeable damage to the pavement and were found much easier to maneuver as compared to the tank and trailer combinations. Therefore, for Bridge Nos. 4 and 5 also, the tanks were placed directly on the pavement. Although, the tanks are wider compared to the 11-axle truck, the girder distribution factor for both vehicles were found to be the same for both the composite as well as the noncomposite structure (see Section 5 and Appendix F). A photograph of M-60 tank on Bridge No. 4 is also shown in Figure 3-7.

Prior to the testing of Bridge No. 1, the axle weights of the four rear axles of trailer and tank combination shown in Figure 3-5, were measured on a weigh station. However, both (left and right) wheels of the rear axles could not fit on the weighing scale at the same time. Therefore, each set of wheels was measured separately, and the total weight of the four rear axles was assumed to be equally distributed among the wheels, which resulted in a axle weight of 147.3 kN (33.1 kip) for each rear axle. During the testing of the Bridge No. 2, a portable weighing scale was used on site to improve the accuracy of the axle weights. Each wheel of both trailers was measured separately. The new axle weights are shown in Figures 3-5 and 3-6. For Bridge No. 3 to 5, tanks were placed directly on the pavement, and the weight of the tank was taken from the documents provided by the Michigan National Guard (Appendix F).

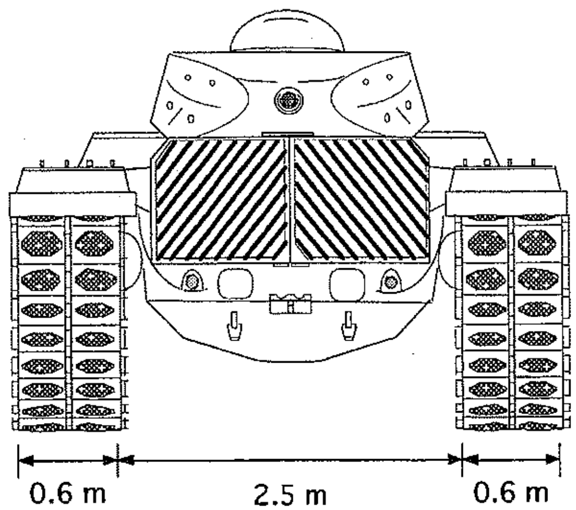


Figure 3-3 : Cross-Section of M-60 Tank.

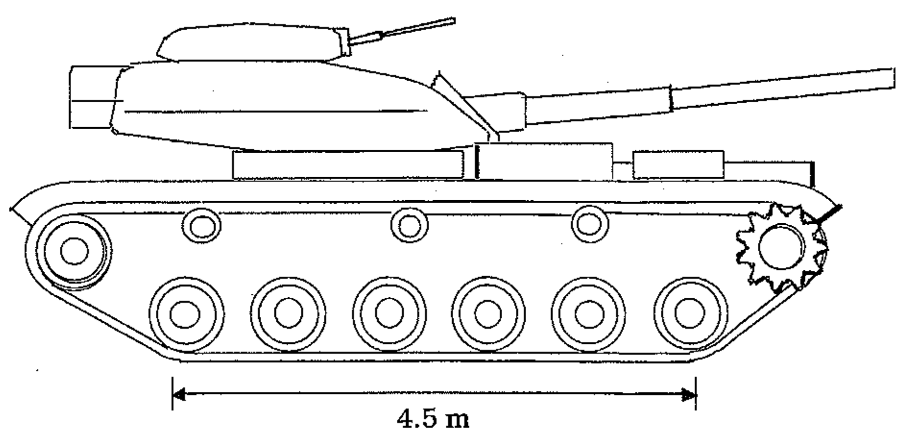


Figure 3-4 : Side Elevation of M-60 Tank.

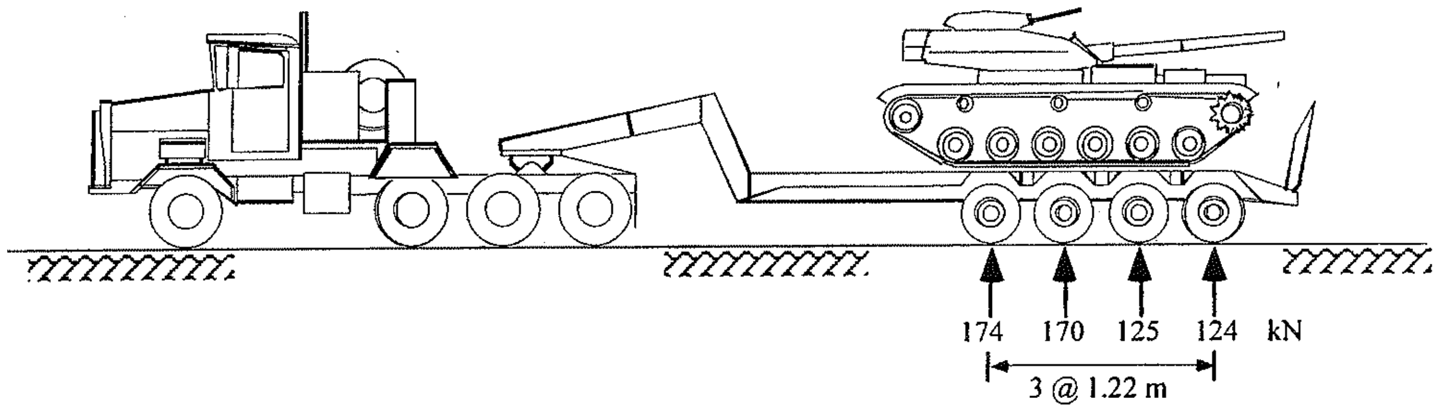


Figure 3-5 : Tank on Military Trailer.

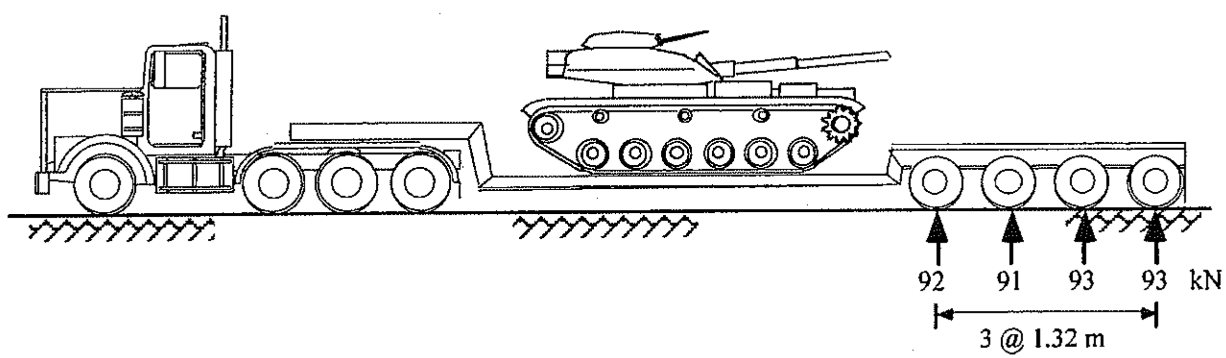


Figure 3-6 : Tank on Commercial Trailer.

Figure 3-7 : M-60 Tank on Bridge No. 4.

3.3 Load Positions

The proof load was applied by gradually increasing the load until the target proof load level was reached. Tanks (trailers for Bridges No. 1 and 2) were moved from supports to the mid-span in several steps to gradually increase the mid-span moment. Each step was referred to as a load-position. For Bridge No. 3, two tanks were not enough to generate the required proof load moment. Therefore, several concrete barrier blocks were placed close to the curbs on each side, as the first load case. A total of twenty eight concrete blocks were available. The tests started on the north span. Twelve concrete barriers were placed along the length on both sides. Before starting the test on the south span, it was decided to increase additional load by placing sixteen concrete barriers.

Tanks were also placed in three different locations in the transverse direction, as shown in Figure 3-8:

- upstream (tanks closer to the upstream railing)
- center and (tanks in center of the bridge)
- downstream (tanks closer to the downstream railing)

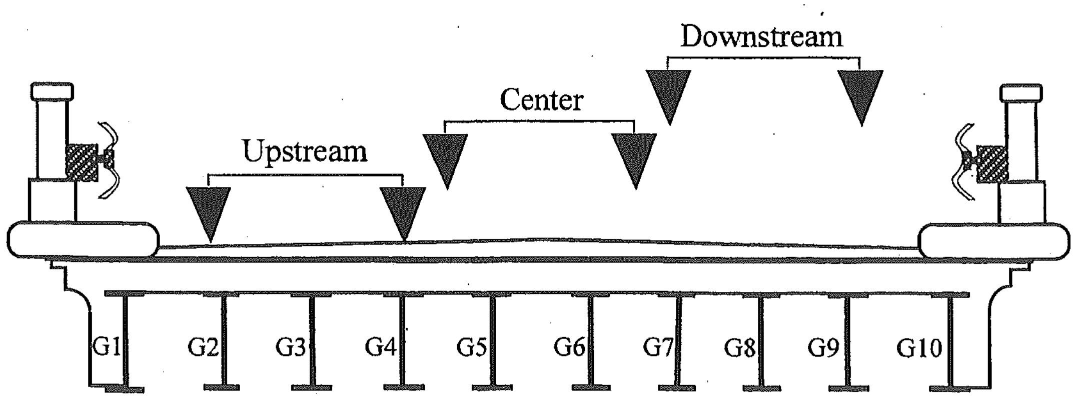


Figure 3-8 : Transverse Load Positions for Bridge No. 2 (others were similar).

Strains and displacements were recorded for each load position (strains could not be measured for concrete girder bridges; also see chapter 4). At all stages of field testing, the bridge response was closely monitored and compared to analytically predicted values. Figure 3-9 shows two tanks on Bridge No. 4 during the maximum load position. Actual lane moments applied during the proof load testing are listed below in Table 3-2.

Table 3-2. Applied Proof Load Lane Moments at Mid-Span.◊

Bridge No.	Applied Lane Moment kN-m (k-ft)			
	Load Position 1	Load Position 2	Load Position 3	Load Position 4
1	225 (167)	605 (445)	630 (466)	-
2	935 (690)	1,565 (1,155)	1,940 (1,430)	2,120 (1,560)
3 - North	710 (524)	1,820 (1,342)	2,015 (1,486)	2,765 (2,039)
3 - South	950 (700)	2,055 (1,515)	2,250 (1,659)	3,000 (2,212)
4	935 (690)	1,200 (885)	1,330 (980)	-
5	525 (385)	685 (500)*	685 (500)**	-

◊ see chapter 6 t0 10 for load positions
* one lane loading ; ** two lane loading.

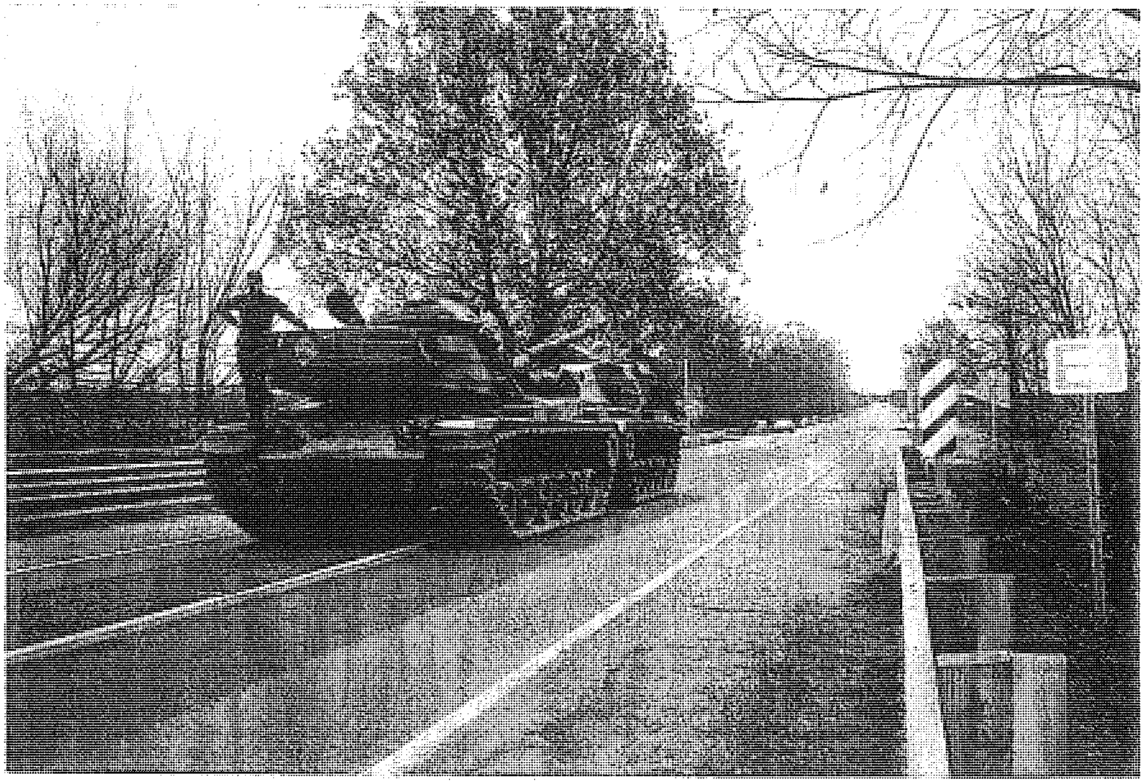


Figure 3-9: Two Tanks on Bridge No. 4.

3.4 Advantages of Using Tanks

Since the mid-span moment is increased by moving the tanks, the load steps could be as small as desired, which lowers the risk of collapse. It is easier to maneuver the tanks in comparison to the concrete blocks, which considerably speeds up the whole process, resulting in less traffic disruption. On average, one bridge can be tested within three hours. It also allows maintaining the traffic over partial width. The full closure is required only at critical time, i.e. maximum load. Use of tanks is also very economical. It does not require any personnel to be present on the bridge, except the tank driver.

3.5 Post-Test Rating

According to the NCHRP report no. 12-28 (13) A (1993), the operating rating factor at the conclusion of proof load test should be calculated as follows:

$$OP = \frac{K_0 L_P}{X_{PA}} \qquad (3\text{-}5)$$

where,

OP = operating level capacity.

L_p = actual maximum proof load applied to the bridge.

X_{pa} = the target live load factor (see Section 3.1).

K_0 = 1.00 if target load is reached.

= 0.88 if a distress level is reached prior to reaching the target load.

Therefore, the operating rating factor (ORF) would be

$$ORF = OP/\ L_r\ (1+I) \qquad (3\text{-}6)$$

where,

L_r = maximum allowable legal load (e.g. maximum moment caused by two-unit 11-axle truck).

Note:

Intentionally left blank

4. MONITORING PROCEDURE

Preliminary analysis showed that the moment capacity of steel girders at mid-span is critical for all bridges. Therefore, all interior girders were instrumented at mid-span. The exterior girders were instrumented for Bridge No. 1 only. However, the deflection and strains were very small. Similar behavior of exterior girders was expected in case of other bridges also, due to the presence of bolted concrete facade, concrete curb and the parapets. Therefore, exterior girders of Bridge No. 2 to 5 were not instrumented. Stresses in the concrete slab were not considered critical, therefore, the slab was not instrumented. Strains were measured using strain transducers at critical locations, such as the highly corroded regions of lower flange close to the mid-span, to monitor the local behavior of primary load carrying members. Deflections were measured using LVDT's to monitor the global response of the structure. Each LVDT was placed on a tripod and connected to the bottom of the girder by a wire. Figure 4-1 shows the setup of LVDT's for measuring girder deflections. Figure 4-2 shows the setup of strain transducers for Bridge No. 1 and Figure 4-3 shows the setup of strain transducers for Bridge Nos. 2, 3 and 4.

The data from LVDT's and strain transducers was collected by a portable SCXI-1200 data acquisition system from the National Instruments. The system consists of a four slot SCXI-1000 chassis, one SCXI-1200 data acquisition card and two SCXI-1100 multiplexers. Each multiplexer can handle up to 32 channels of input data. The current system is capable of handling 64 channels of strain or deflection inputs. Up to 32 additional channels can be added if required. A portable field computer is used to store, process and display the data on site. A typical data acquisition setup is shown in Figure 4-4. The data from all instruments is collected after placing the tanks in desired positions. In addition, the real time responses of all transducers are closely monitored during all stages of testing. For Bridge Nos. 1 and 5, which are reinforced concrete T-beam structures, the strains in the girders could not be measured because of extensive cracking of concrete prior to the test. In such situations, the displacement based strain transducers can be used to measure the concrete strains, however, these transducers were not available at the time of testing. Moreover, the strains in extensively cracked concrete girders are not necessarily expected to increase linearly with increasing load, thus prohibiting their use to establish a linearity criteria. For Bridge No. 3, the LVDT's could not be used, because the water in the river underneath the bridge was too deep.

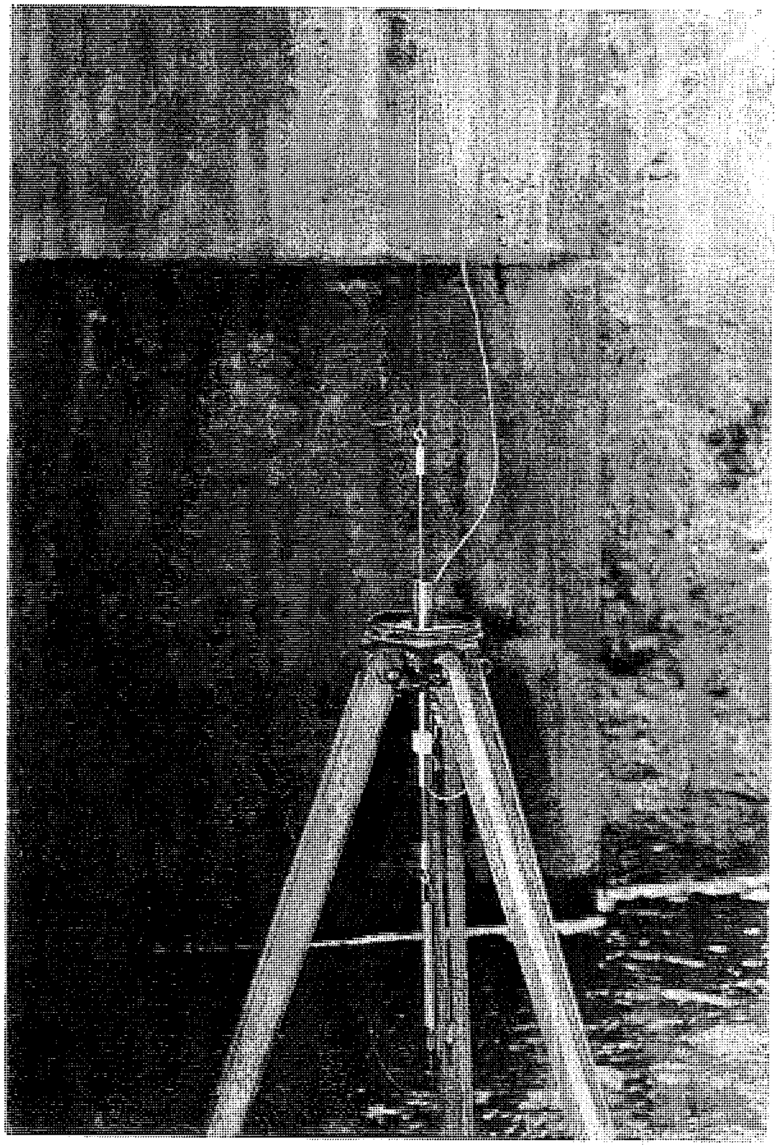


Figure 4-1 : Typical LVDT Setup.

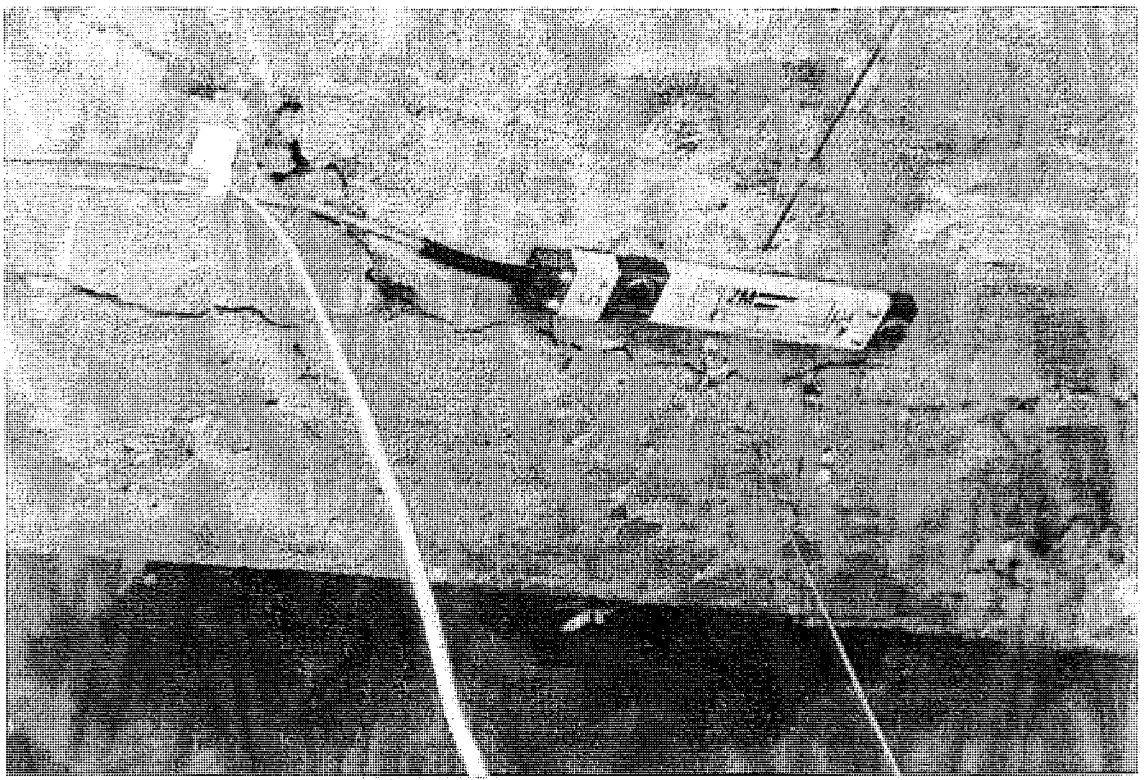


Figure 4-2 : Typical Strain Transducer Setup for Bridge No. 1.

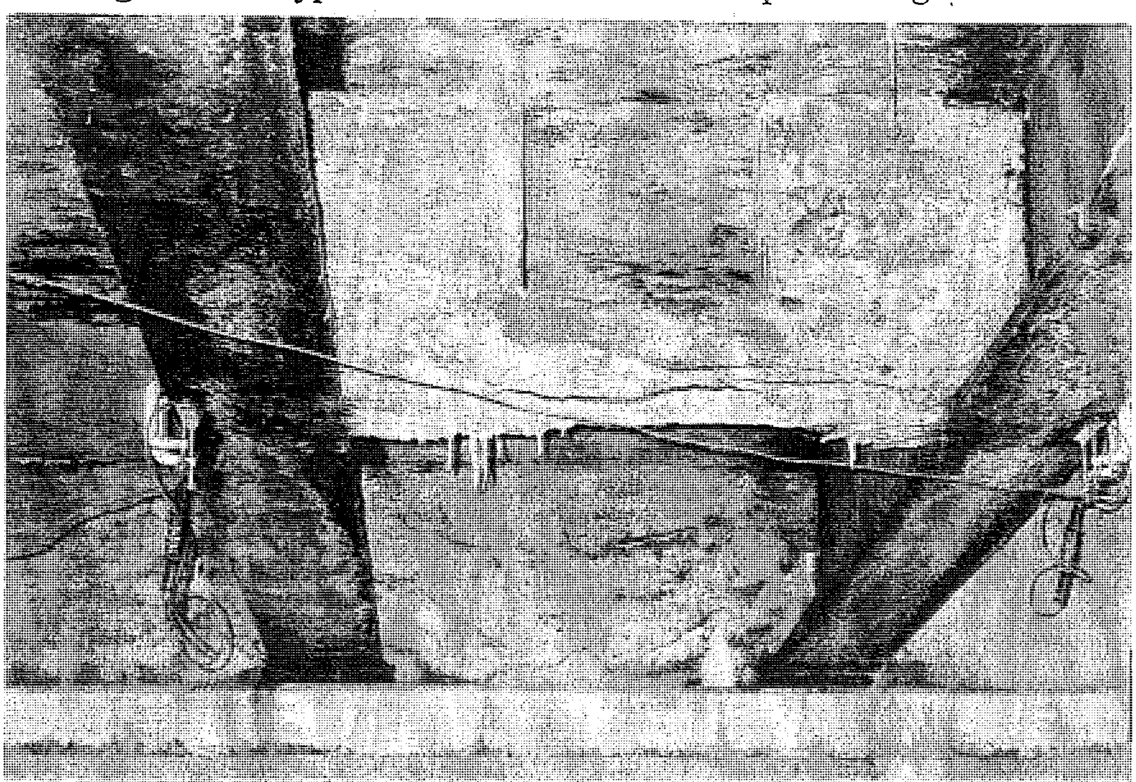


Figure 4-3 : Typical Strain Transducer Setup for Steel Girder Bridges.

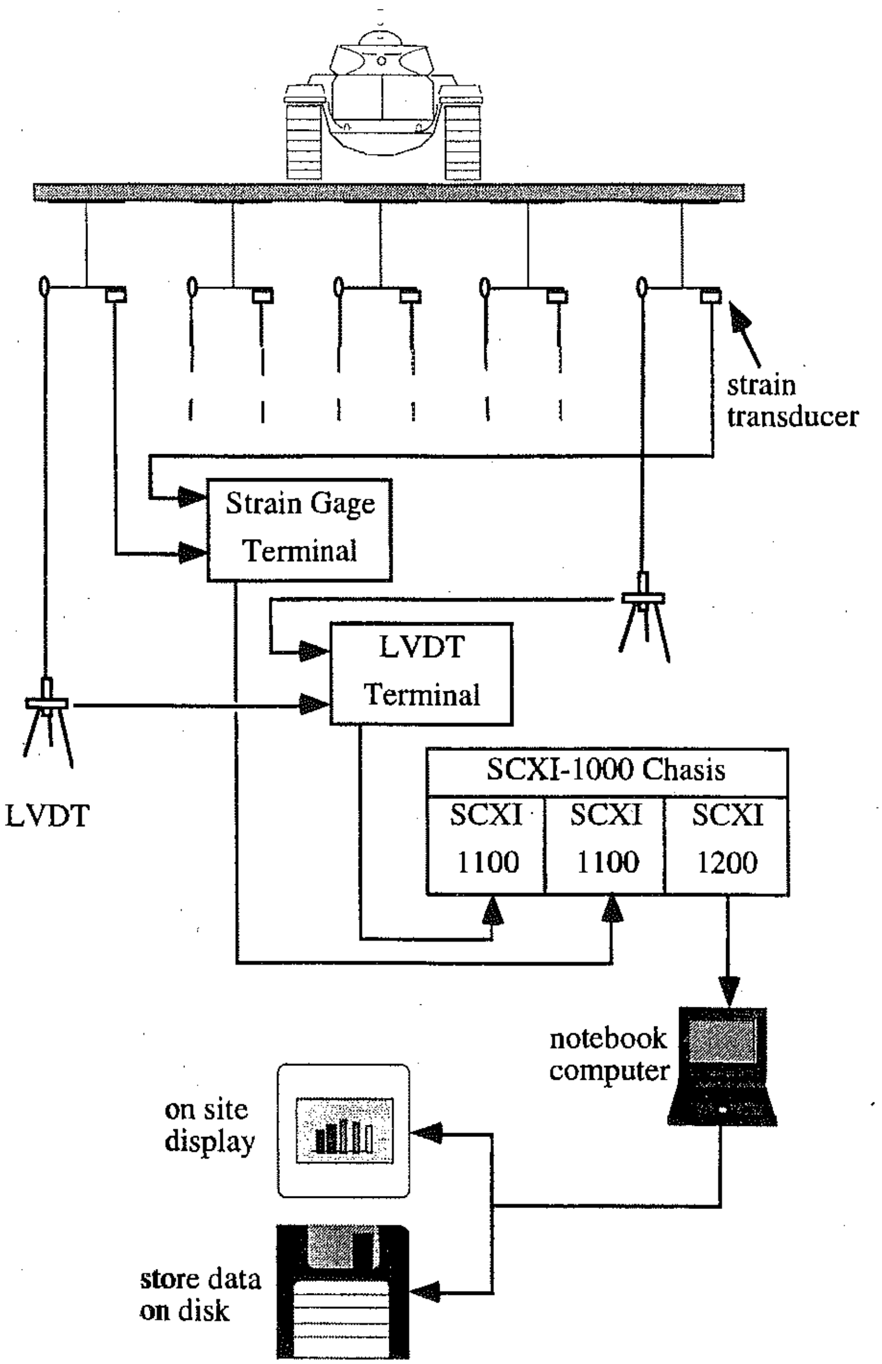


Figure 4-4 : Typical Data Acquisition Setup.

Note:

Intentionally left blank

5. ANALYTICAL MODELING

During the proof load test the response of the structure should be repeatedly compared with the results from analytical models to check if the structural performance is within acceptable limits, to avoid accidental overload or excessive deformations and to determine the suitable termination point for the test. Therefore, before proof load testing, analytical models were prepared for all bridges. The computer program SECAN developed by Mufti et. al (1992) was used for that purpose. It is based on the semi-continuum algorithm developed by Jaeger and Bakht (1989). Semi-continuum algorithm is an analytical procedure (not a numerical procedure such as FEM. etc.) for the analysis of slab-on-girder bridges. The program analyzes only the simply supported and slab-on-girder bridges.

For steel girder bridges, two different types of models were prepared. In first model, no composite action between the slab and steel girder was considered because no shear connectors were provided, and the effect of non-structural members was also not incorporated. In second model, the bond between the concrete slab and steel girders was assumed to be fully composite, and the added stiffness due to participation of secondary members was also included. The available capacity for proof load was determined using the material properties listed in Section 2. Results of analytical modeling are shown in form of graphs in Appendices A to E. For Bridge Nos. 2, 3 and 4, the experimental results were also compared with a composite model with fully fixed support, to estimate the moment restraint offered by end supports. For Bridge Nos. 1 and 5, the exact amount of restraint provided by supports could not be measured since the strain data for these bridges was unavailable.

Note:

Intentionally left blank

6. BRIDGE NO. 1 (B06 of 08052, M-66 over Mud Creek)

6.1 Description

This is a simply supported reinforced concrete bridge over Mud Creek located just southwest of Woodbury, Michigan. The total span length and width are 6.5 m and 9.3m (21.5 ft and 30.5 ft), respectively. It carries one lane in each direction with average daily traffic (ADT) of 2,100. The design drawings and details, such as the amount of steel reinforcement were not available. It was estimated to be built in 1918, and widened on both sides in 1940. The cross-section of the bridge is shown in Figure 6-1. There are six reinforced concrete T-beams spaced at about 1.79 m (5.9 ft), and a 190 mm (7.5 in) thick reinforced concrete slab with 75 mm (3 in) thick bituminous overlay. The side elevation of the bridge is shown in Figure 6-2.

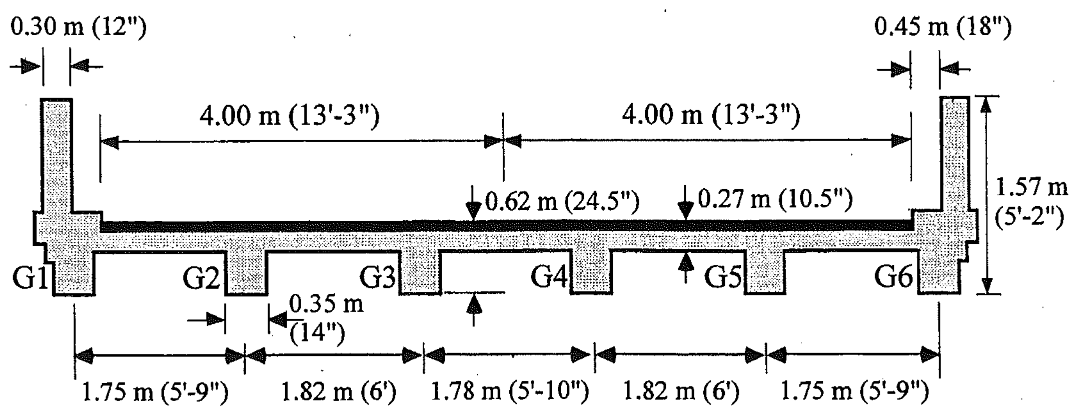


Figure 6-1. Cross-Section of Bridge No. 1.

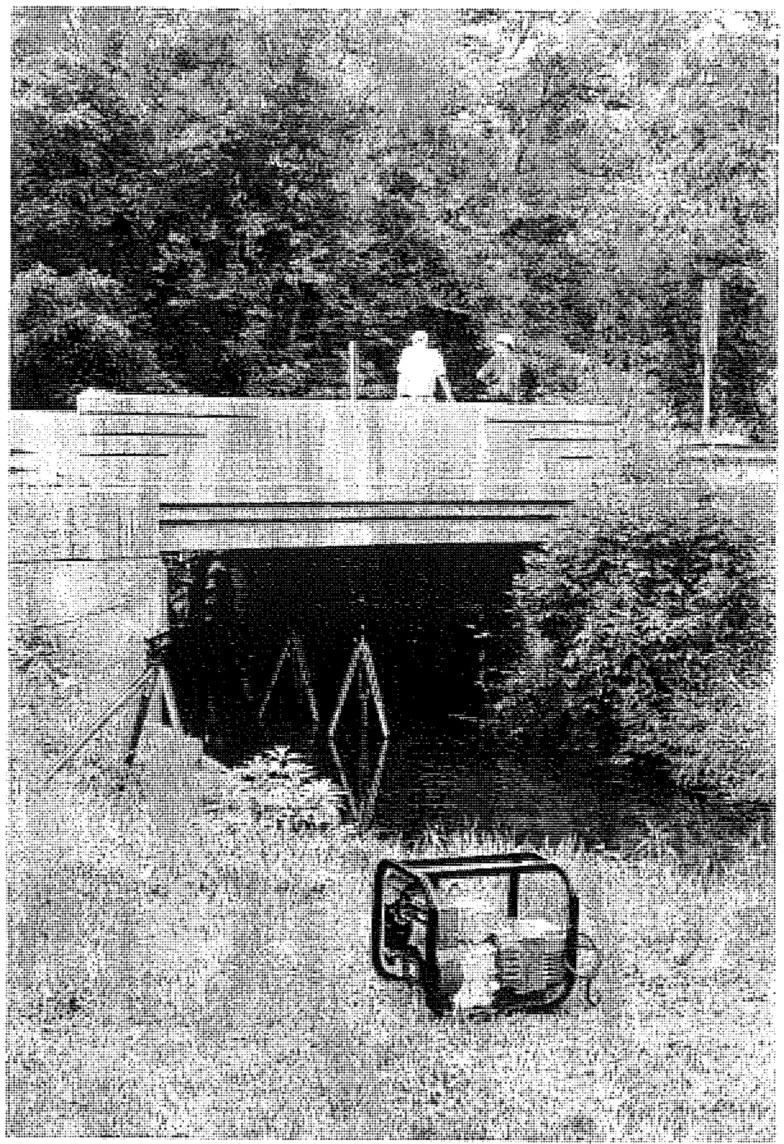


Figure 6-2 : Side Elevation of Bridge No. 1.

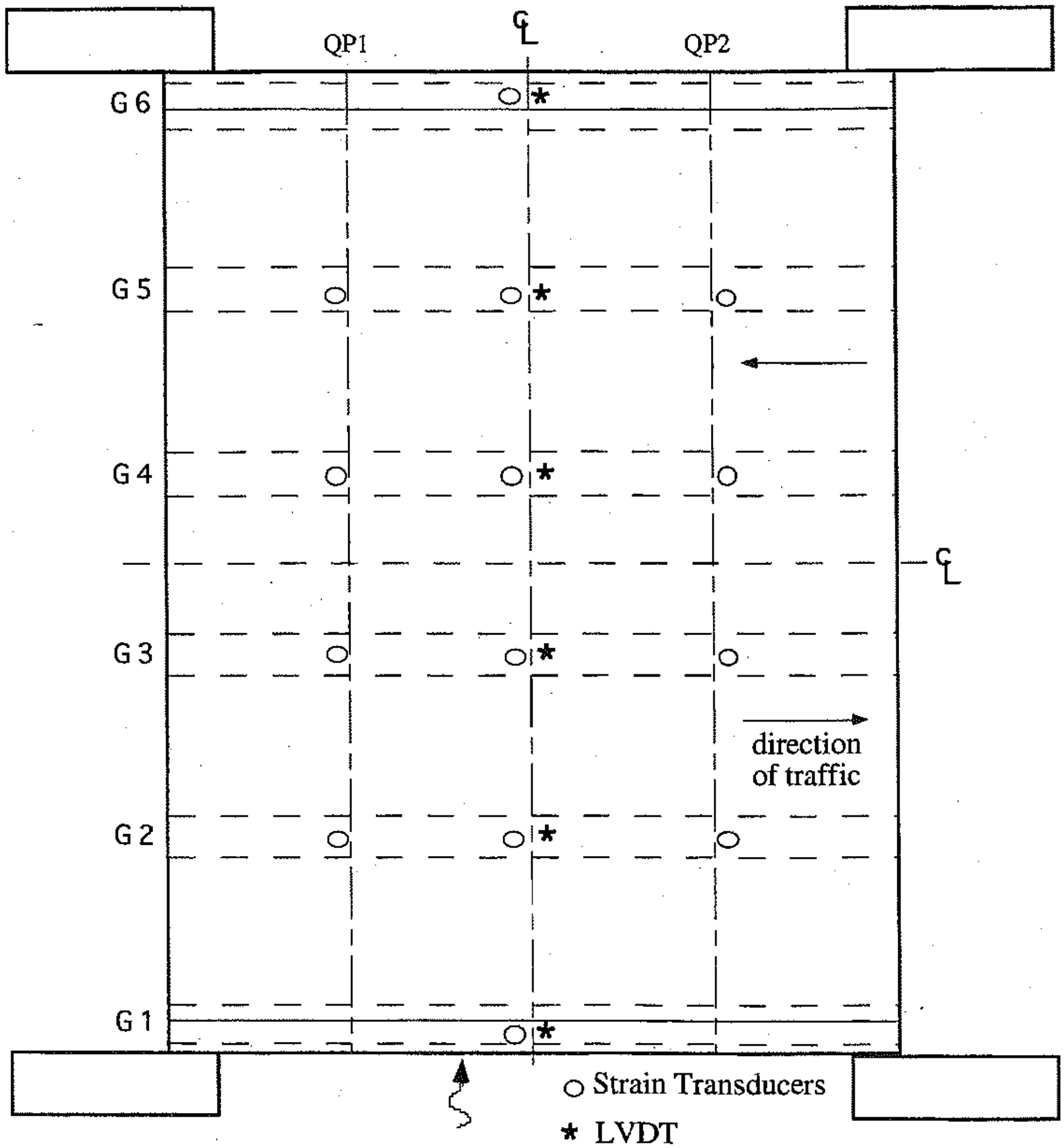
6.2 Pre-Test Inspection

It was observed during the site inspection that the bituminous overlay had several random cracks. The concrete had spalled on the bottom of the deck close to the west drain. Spalling of concrete was also observed in the area close to the east curb. However, all girders were in relatively good condition.

For preliminary rating calculation by MDOT, the amount of steel reinforcement was assumed to be the same as in similar bridges built between 1915-1920. The design compressive strength of concrete was taken to be 13.8 MPa (2.0 ksi). For operating rating, the remaining load capacity to carry the live load and impact was calculated to be 341.8 kN-m (252 k-ft) per lane. Moment capacity at mid-span was considered critical. The inventory rating factor calculated by MDOT for H15 truck was 0.78 and operating rating factor for two-unit 11-axle truck was 0.87. The pre-test analytical results from non-composite and composite models are shown in the form of graphs and compared to the experimental results (Appendix A).

6.3 Instrumentation

Deflections at mid-span of each girder were measured using six LVDT's. Sixteen strain transducers were also placed at the mid-span of all girders and quarter points of selected girders. However, due to extensive cracking of the concrete, it was not possible to measure the strains (see Section 4). Figure 6-3, shows the instrumentation layout for all transducers.



MP : Mid Point.

QP1 : Quarter Point 1.

QP2 : Quarter Point 2.

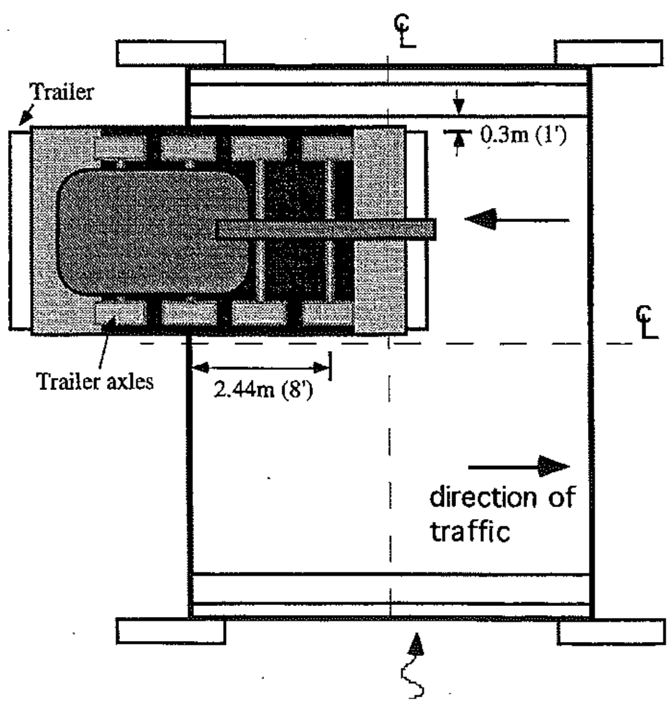
Figure 6-3 : Instrumentation Layout for Bridge No. 1.

6.4 Proof Load Positions

One M60 military tank placed on a military trailer (Figure 3-4) was used as proof load , as shown in Figure 3-4. The load was moved from the support towards the mid-span, in three steps to achieve the target proof load moment (Table 3-1). The tank was placed in three different transverse positions, as shown in Figure 3-7. All load positions are shown in Figures 6-4 to 6-6. The traffic was detoured during the test. The lane moments resulting from these load positions are listed in Table 6-1. The target proof load moment was 519.5 kN-m (383 k-ft).

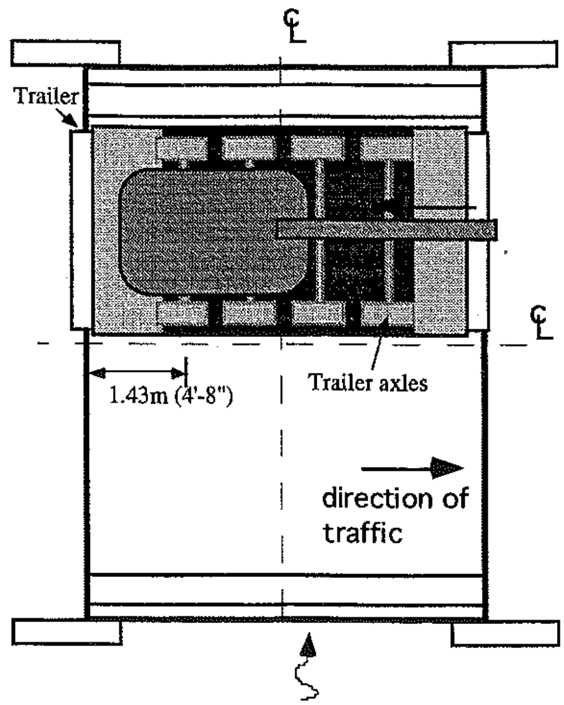
Table 6-1 : Applied Proof Load Lane Moments.

Load Position No.	Applied Lane Moment kN-m (k-ft)		
	Quarter Point 1	Mid Point	Quarter Point 2
1	240 (178)	225 (167)	115 (84)
2	480 (353)	605 (445)	430 (316)
3	420 (311)	630 (466)	445 (330)

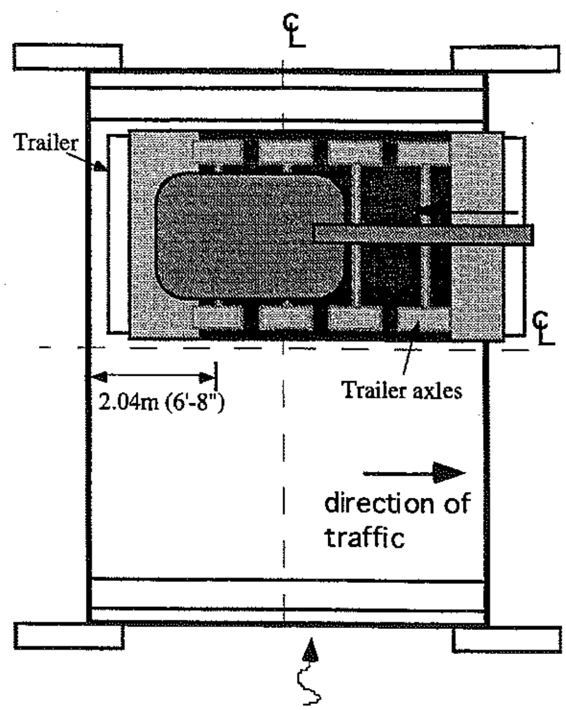


(a) Load Position 1 - Downstream.

Figure 6-4 : Longitudinal Load Positions for Downstream Case.

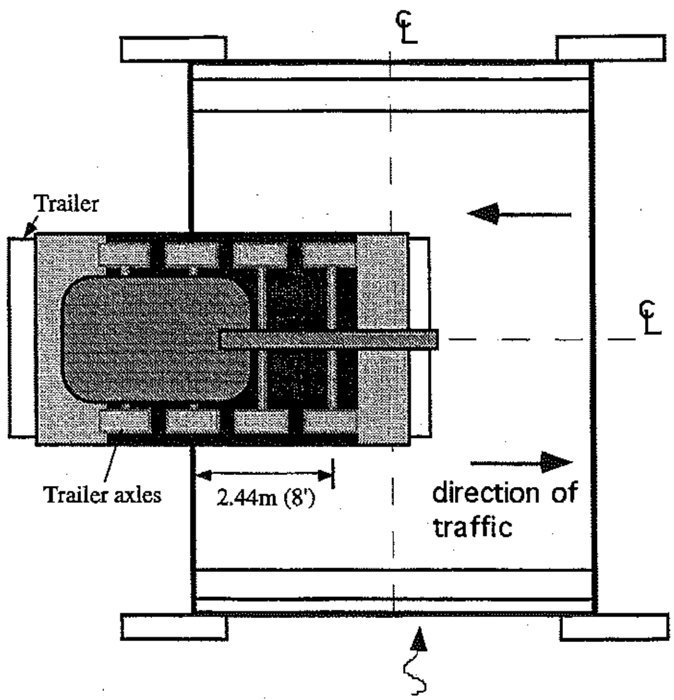


(b) Load Position 2 - Downstream.

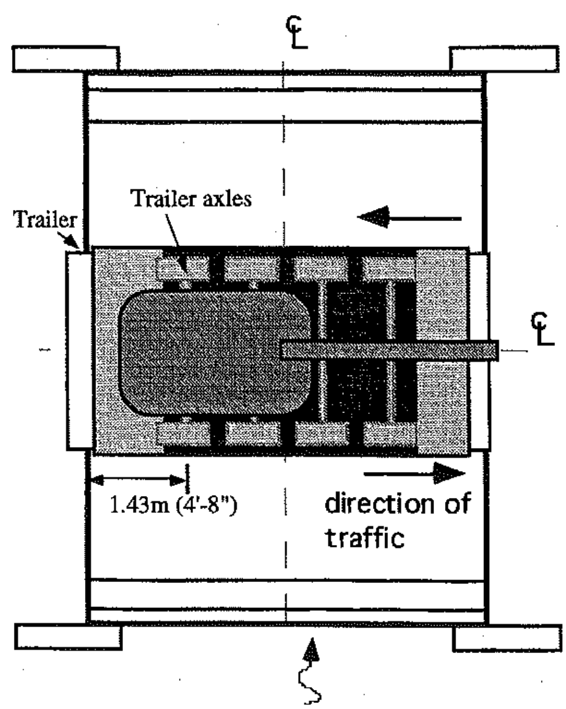


(c) Load Position 3 - Downstream.

Figure 6-4 : Longitudinal Load Positions for Downstream Case. (cont'd)

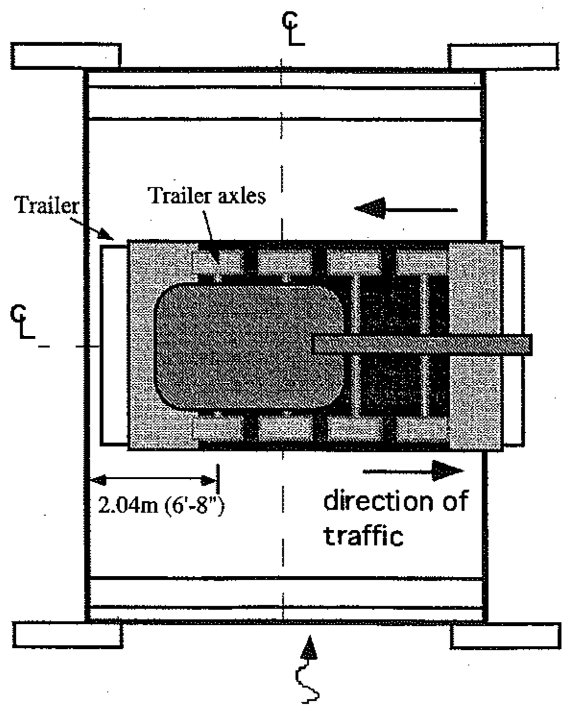


(a) Load Position 1 - Center.



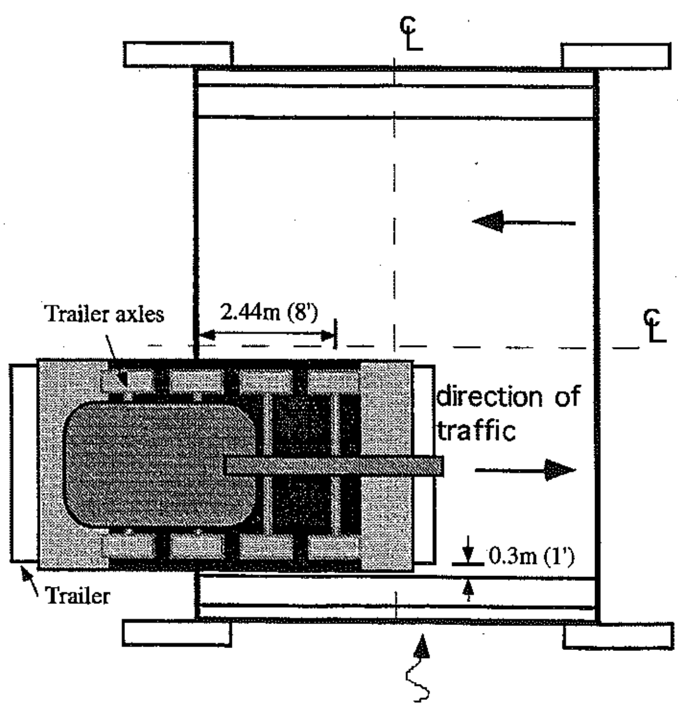
(b) Load Position 2 - Center.

Figure 6-5 : Longitudinal Load Positions for Center Case.



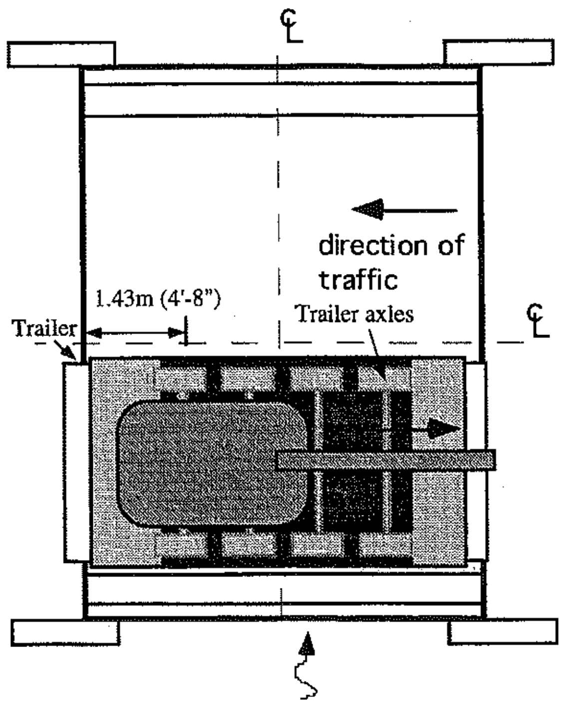
(c) Load Position 3 - Center.

Figure 6-5 : Longitudinal Load Positions for Center Case. (cont'd)

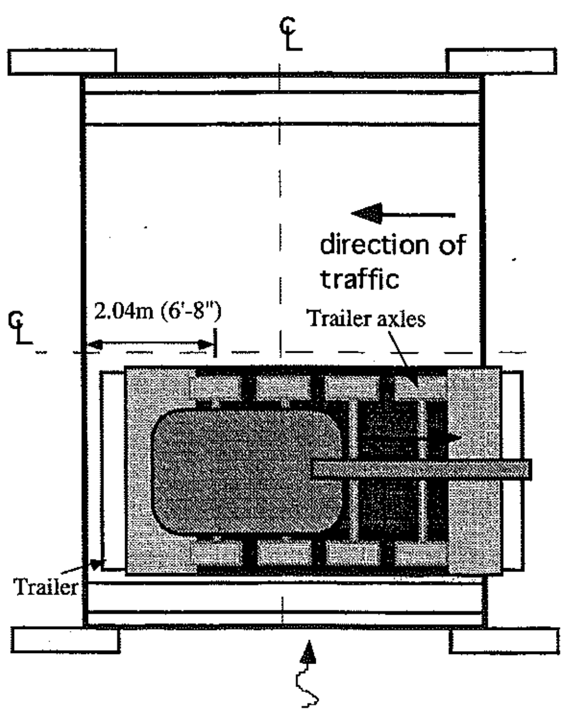


(a) Load Position 1 - Upstream.

Figure 6-6 : Longitudinal Load Positions for Upstream Case.



(b) Load Position 2 - Upstream.



(c) Load Position 3 - Upstream.

Figure 6-6 : Longitudinal Load Positions for Upstream Case. (cont'd)

6.5 Proof Load Test Results

The maximum lane moment achieved during the proof load test of this bridge was 630 kN-m (466 k-ft). According to Equation 3-5, the operating load carrying capacity to carry live load and impact was found to be 393 kN-m (290 k-ft), which is higher than the analytically predicted value by MDOT i.e. 342 kN-m (252 k-ft). The applied proof load moment

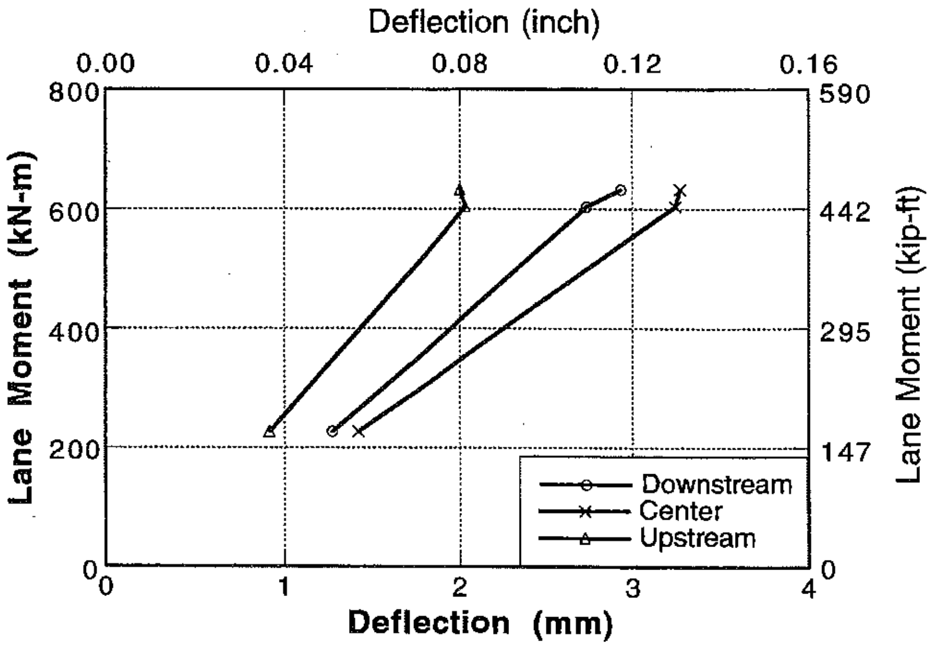


Figure 6-7 : Deflection at Mid-Span of Girder 4 of Bridge No. 1.

was more than 2.7 times the HS20 moment and 2.2 times the two-unit 11-axle moment. After the load testing, the bridge was considered safe to carry the legal truck traffic. The maximum deflection of 3.3 mm (0.13 in) was observed at the mid-span of girder no. 4 under the maximum load level for center loading. Deflections measured at mid-span for girder no. 4 are shown in Figure 6-7.

Deflections for each transverse load position increased linearly with increasing lane moment. Strains in most girders could not be measured due to severe cracking of concrete (Section 4). Both

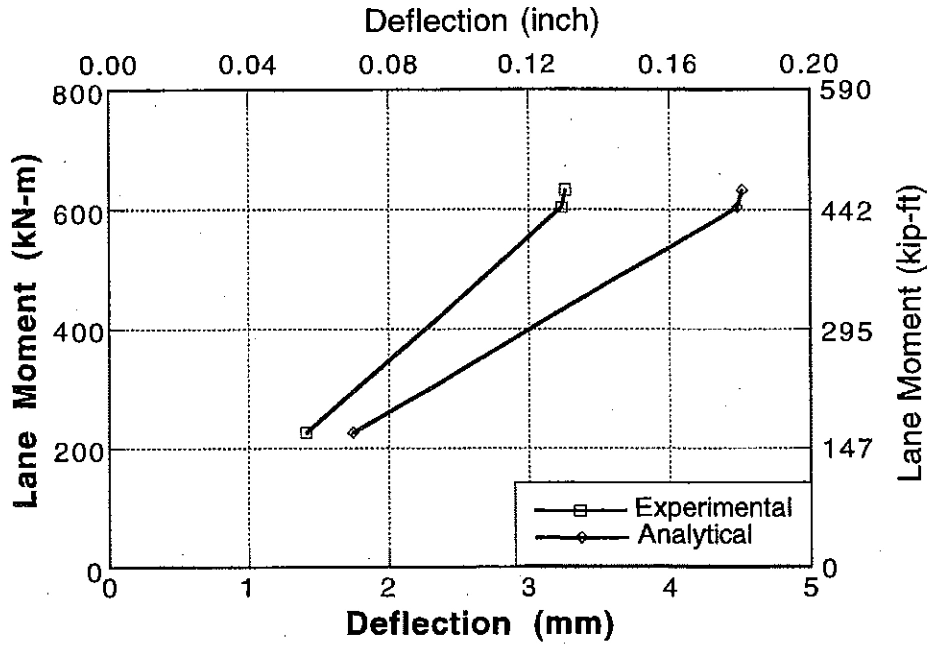


Figure 6-8 : Deflections at Mid-Span of Girder 4 for center loading on Bridge No. 1.

experimental and analytical deflections of girder 4 for center loading are shown in Figure 6-8. This analytical model also included an added stiffness due to the contribution of non-structural members such as sidewalk and railings. The small nonlinearity between load position 2 and 3 shows small increase in stiffness.

For most load cases, the experimental deflections were smaller than the analytically predicted values, which means that the structure is stiffer than the analytical model. The additional unaccounted reduction in deflection at mid-span is caused by partial restraint provided by end supports. The lateral distribution of deflections is also shown in Figure 6-9. Additional graphs and the processed data are listed in Appendix-A. The target proof load was reached without any noticeable non-linearity in bridge response. The new operating rating factor for a two-unit 11-axle truck is 1.21, using Equation 3-6. Therefore, the bridge was found to be safe to carry legal truck traffic.

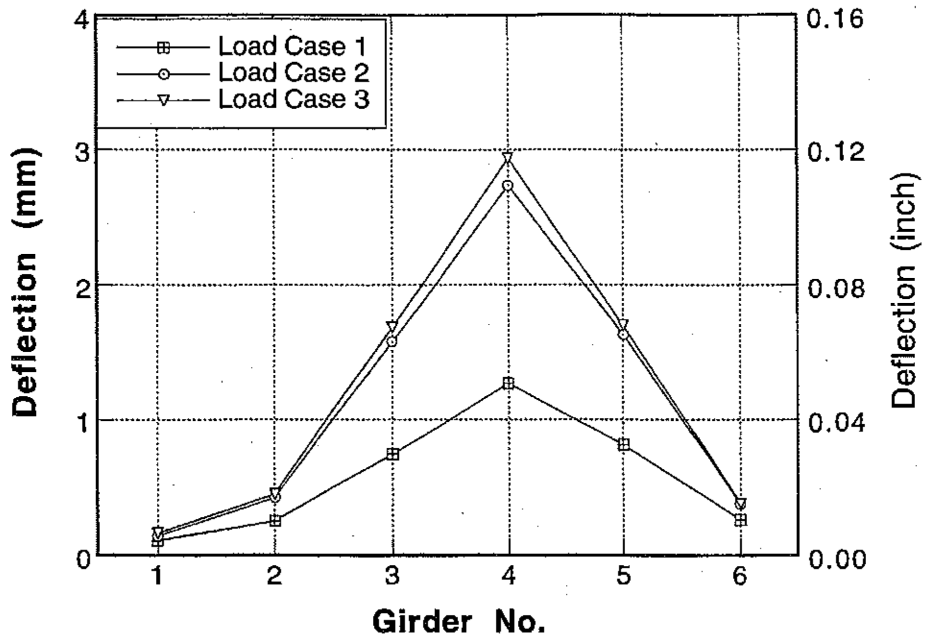


Figure 6-9 : Lateral Load Distribution for Downstream Loading on Bridge No. 1.

7. BRIDGE NO. 2 (B02 of 38071, M-50 over Grand River)

7.1 Description

This bridge was built in 1926 and is located over Grand River in Jackson county, Michigan. The side elevation of the bridge is shown in Figure 7-1. It has one lane in each direction carrying state highway M-50 and the business loop of US-127 with a total ADT of 11,900. As shown in Figure 7-2, it has ten steel girders and a 165 mm (6.5 in) thick reinforced concrete slab with 150 mm (6 in) thick cast-in-place concrete wearing surface and 170 mm (6.7 in) thick bituminous overlay. It is a simply supported single span structure, which was designed to behave as a noncomposite section. The total span length is 14.6 m (48 ft) and the width is 13.8 m (45 ft).

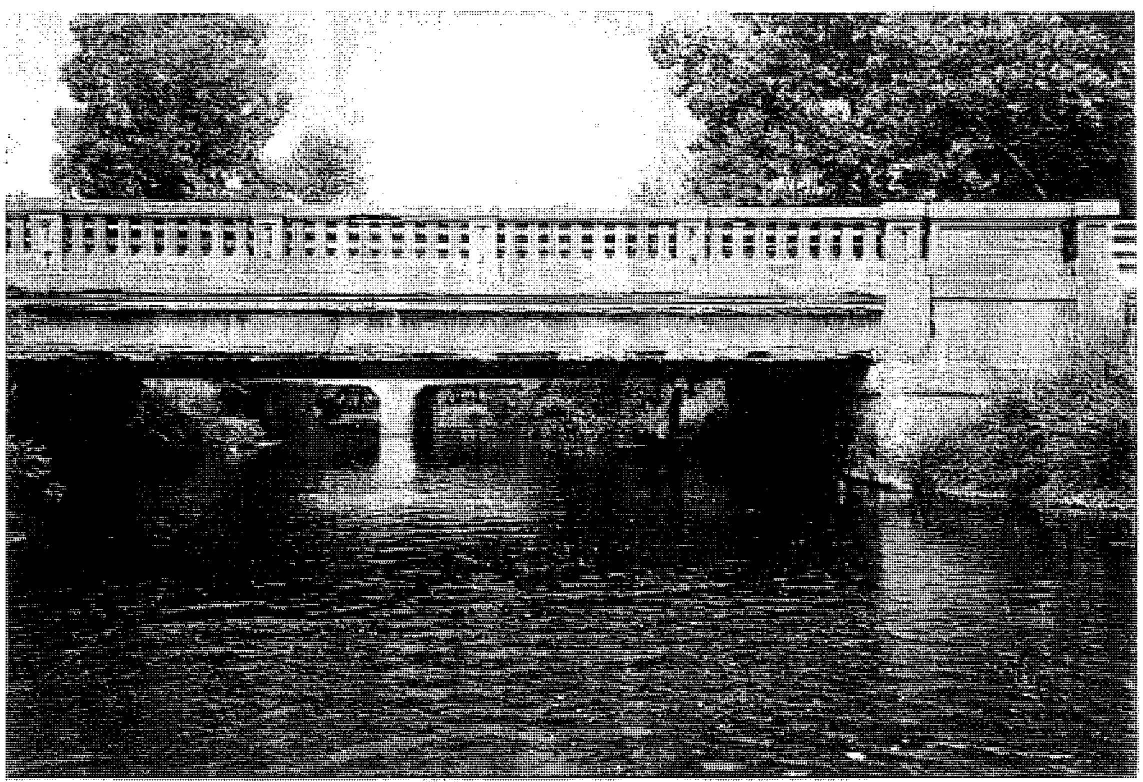
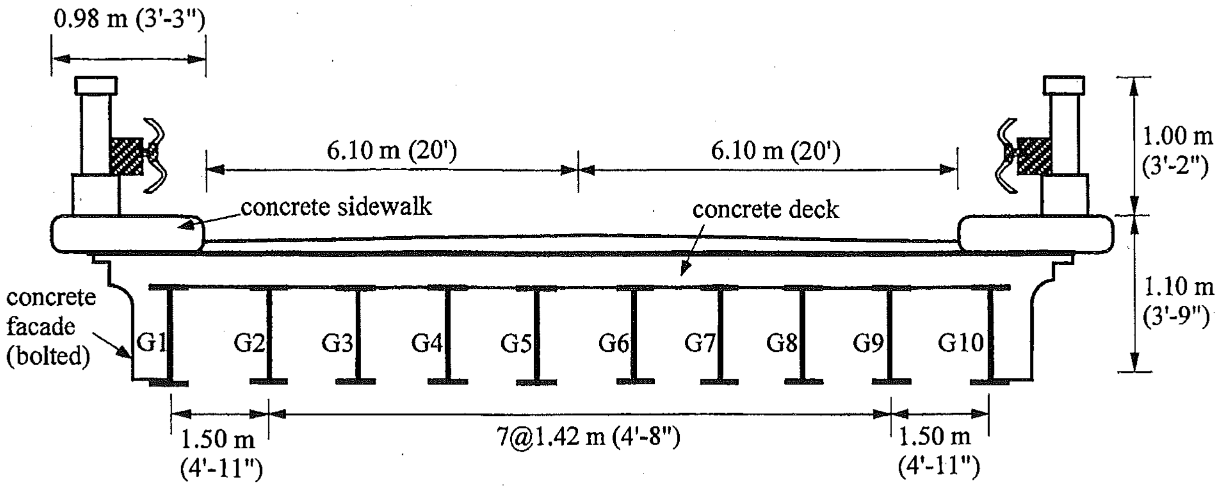
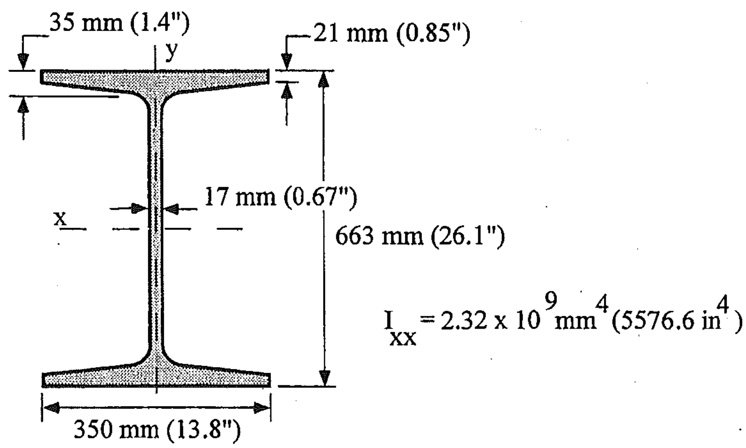


Figure 7-1 : Side Elevation of Bridge No. 2.





Exterior and Interior Girders

Figure 7-2 : Cross-Section of Bridge No. 2.

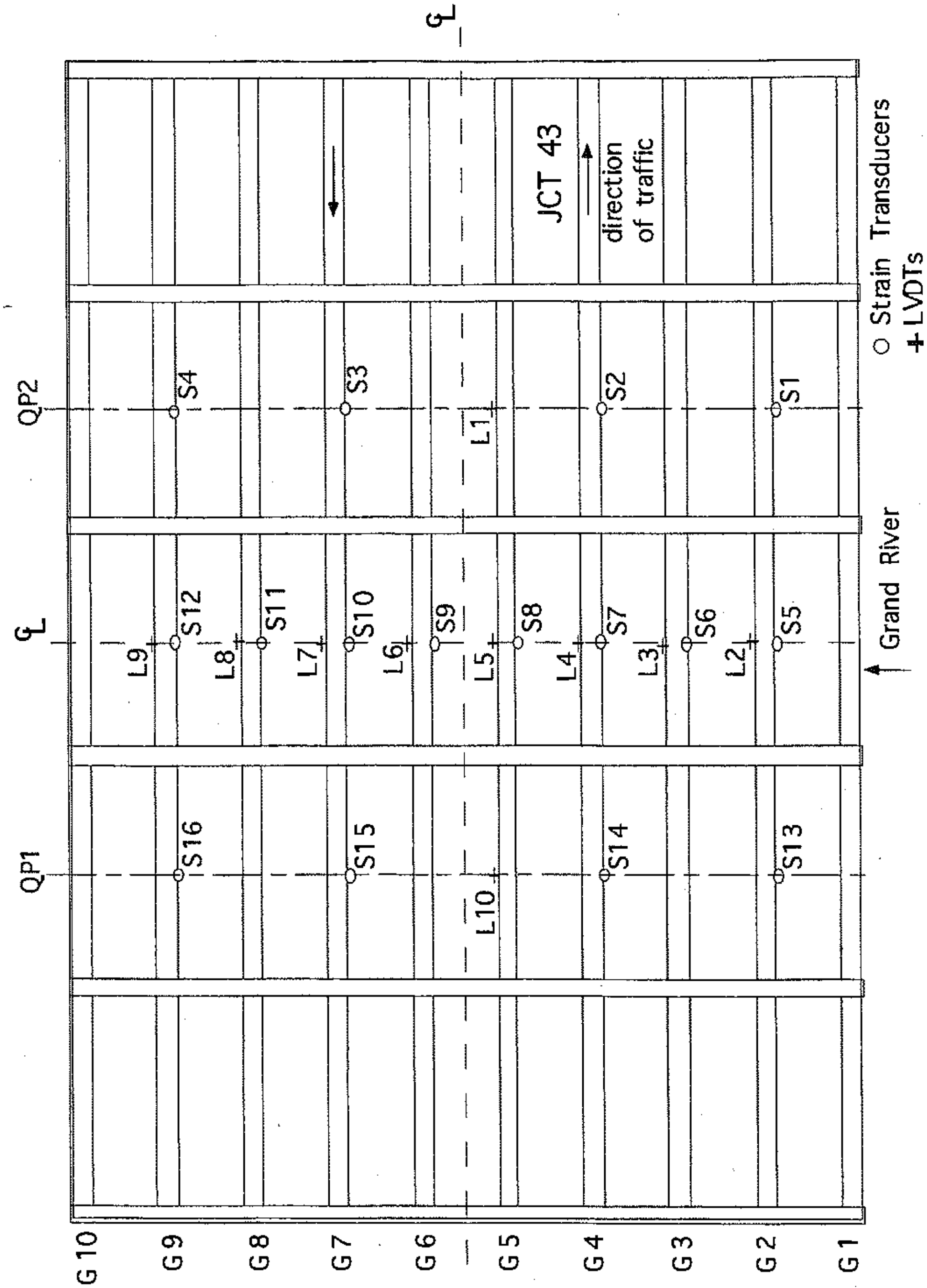
7.2 Pre-Test Inspection

Severe corrosion in the lower flanges of the steel girders was observed during initial inspection. As much as 60 percent reduction in the flange thickness could be seen at some locations close to mid-span. This reduces the moment capacity of the steel girder by about 30 percent. Webs were also corroded at some places but the deterioration was considered to be insignificant. There was not much corrosion in the steel girders near the supports, and the reinforced concrete slab was in fair condition. Therefore, the critical limit state for this bridge is the mid-span moment.

The design compressive strength of concrete was taken to be 17.3 MPa (2.5 ksi), and based on coupon tests performed by MDOT, 290 MPa (42 ksi) was used as the yield stress of structural steel. The remaining operating rating capacity was 1,549 kN-m (1,142 k-ft), prior to the test. Preliminary rating factors were calculated by MDOT using the Michigan Bridge Analysis Guide (1983). The section modulus was reduced only by 5 percent. The inventory rating factor for H15 truck was 0.98 and the operating rating factor for 2-unit 11-axle truck was 0.95. If the section modulus is reduced by 30 percent, then the H15 inventory rating factor would be 0.11 and operating rating factor for 2-unit 11-axle truck would be 0.43. The pre-test analytical results from non-composite and composite models are shown in the form of graphs and compared to the experimental results (Appendix B).

7.3 Instrumentation

Ten LVDT's and sixteen strain transducers were used to instrument the bridge. LVDT's were placed at the mid-span and quarter points of selected girders. Strain transducers were placed at the lower flanges of the steel girders close to the mid-span and quarter points of selected girders. The exact location of the strain transducers was decided based on the amount of corrosion in those areas. The location with the smallest remaining flange thickness was chosen for strain monitoring. Actual instrumentation layout is shown in Figure 7-3. Strain transducers attached to the girder nos 4 and 7 did not work due to seepage of water into the connectors.



MP : Mid Point.

QP1 : Quarter Point 1.

QP2 : Quarter Point 2.

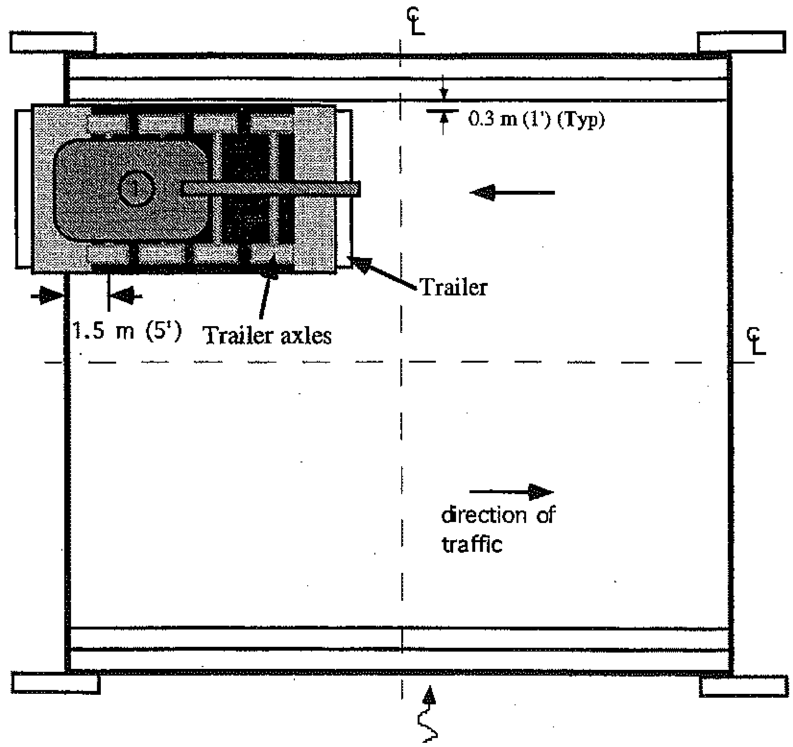
Figure 7-3 : Instrumentation Layout for Bridge No. 2.

7.4 Proof Load Positions

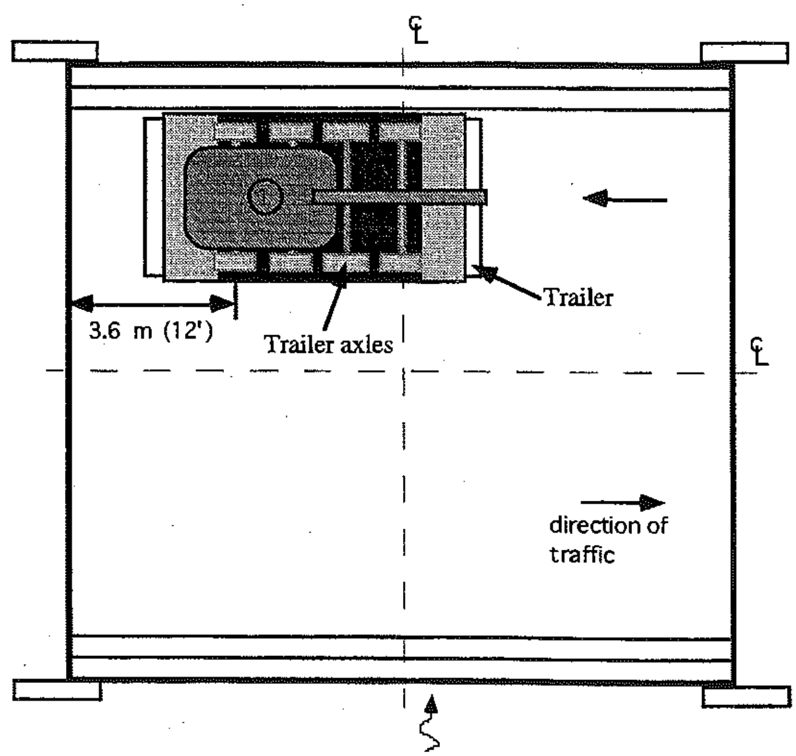
Proof load moment was applied in four increments by using two military tanks; one tank on a military trailer and the other tank on a commercial trailer (Figures 3-4 and 3-5). Only four rear axles of each trailer were used for loading the bridge. For the first two load positions, only one trailer was used, while for the remaining two load cases, the second trailer was added. In addition, the trailers were also placed in three different transverse load positions (Figure 3-7). All load positions used during the test are shown in Figures 7-4 to 7-6. The proof load lane moments from these load positions are listed in Table 7-1. The target proof load moment was 2,106.5 kN-m (1,553 k-ft). The traffic was allowed over partial width during the test, and it was fully stopped only at critical times during maximum load placement, i.e. Load Position 4.

Table 7-1 : Applied Proof Load Lane Moments.

Load Position No.	Applied Lane Moment in kN-m (kip-ft)		
	Quarter Point 1	Mid Point	Quarter Point 2
1	1,170 (865)	935 (690)	470 (345)
2	1,385 (1,020)	1,565 (1,155)	785 (575)
3	1,570 (1,160)	1,940 (1,430)	1,315 (970)
4	1,660 (1,225)	2,120 (1,560)	1,490 (1,100)

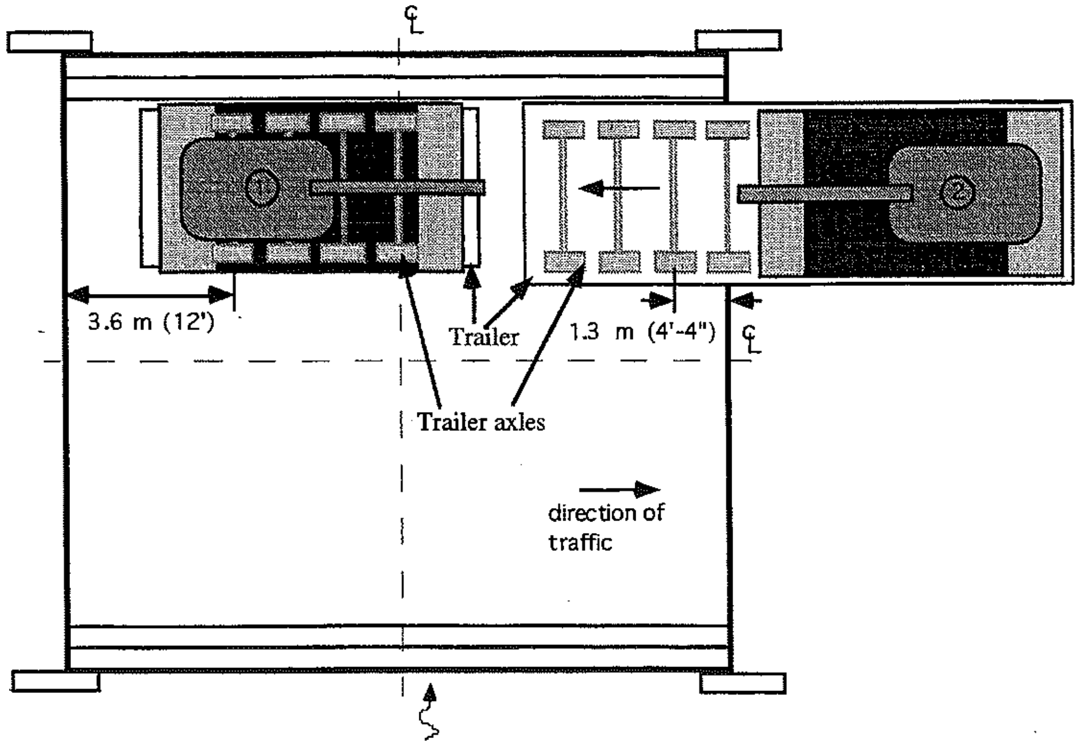


(a) Load Position 1 - Downstream Case.

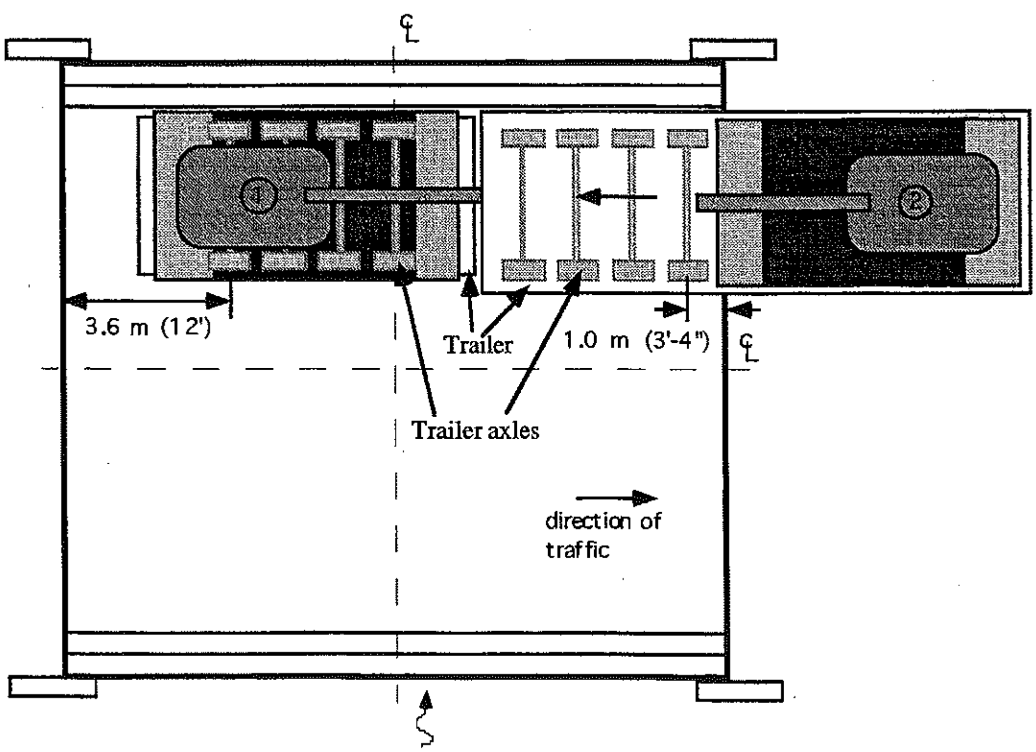


(b) Load Position 2 - Downstream Case.

Figure 7-4 : Longitudinal Load Positions for Downstream Case.

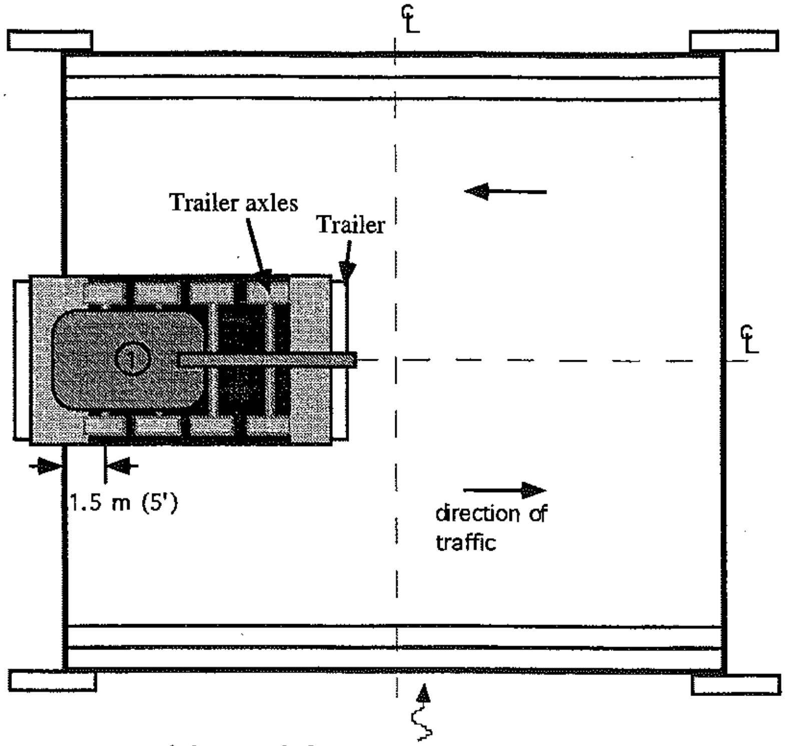


(c) Load Position 3 - Downstream Case.

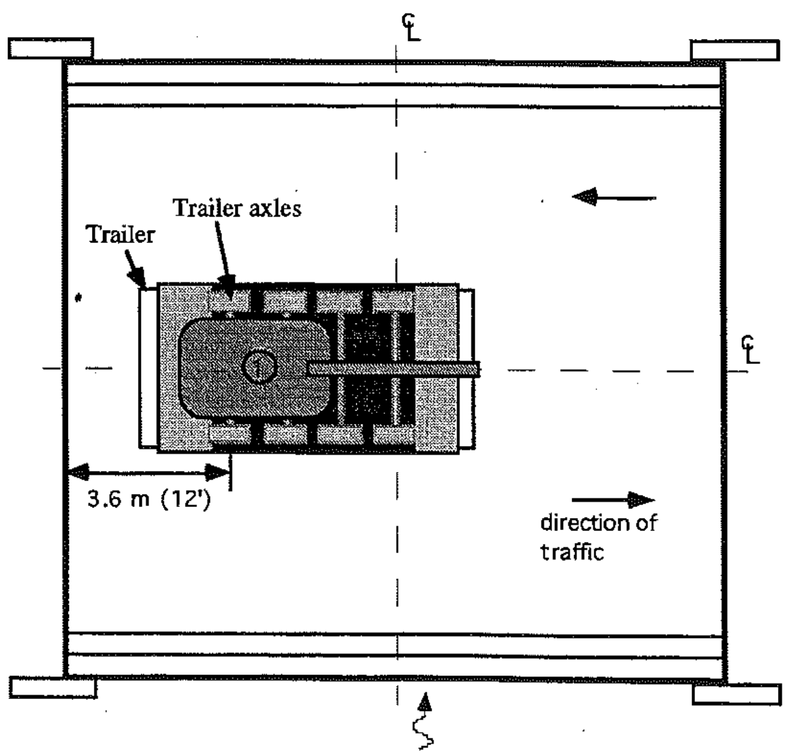


(d) Load Position 4 - Downstream Case.

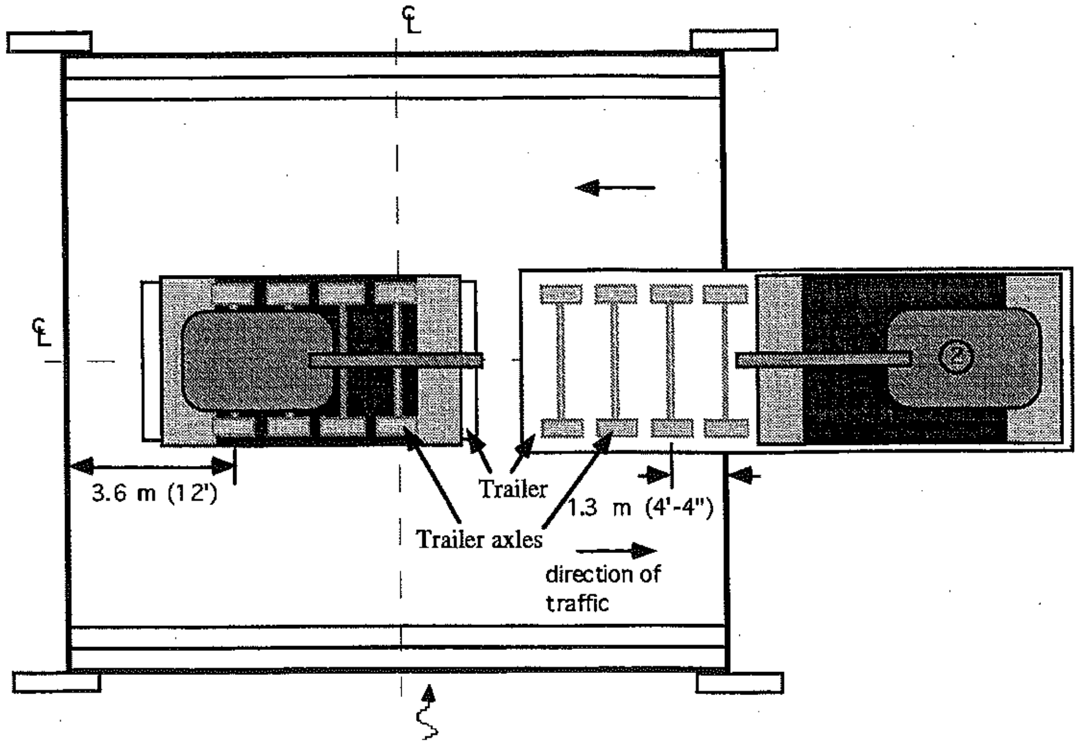
Figure 7-4 : Longitudinal Load Positions for Downstream Case. (cont'd)



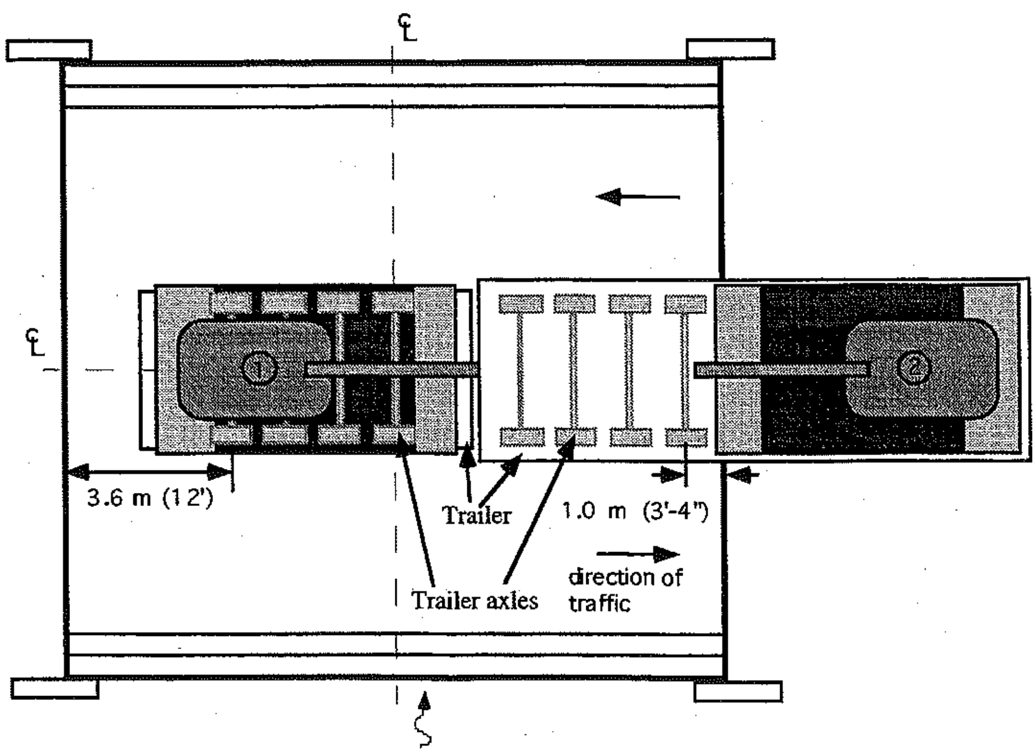
(a) Load Case 1 - Center Case.



(b) Load Position 2 - Center Case.
Figure 7-5 : Longitudinal Load Positions for Center Case.

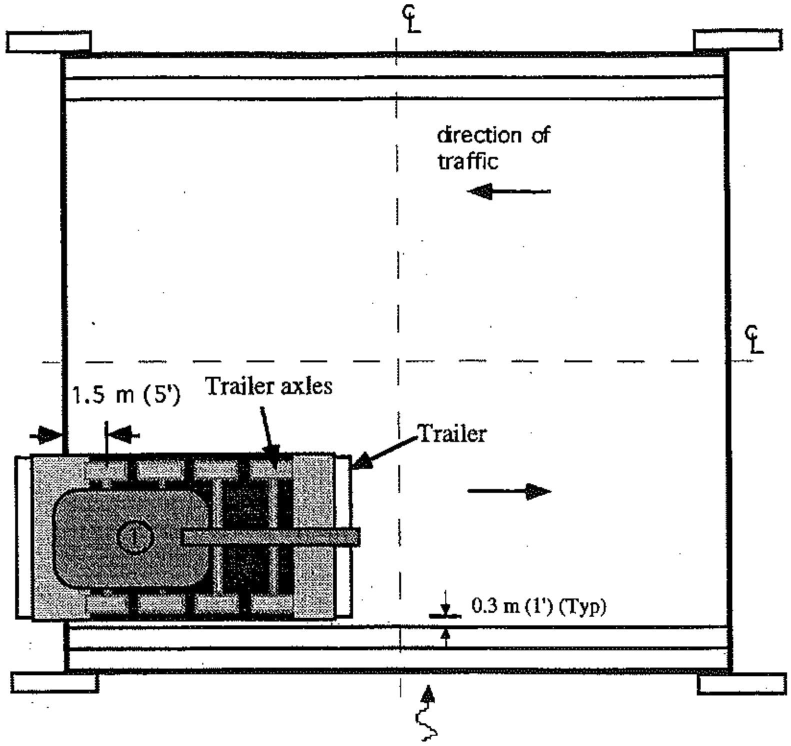


(c) Load Position 3 - Center Case.

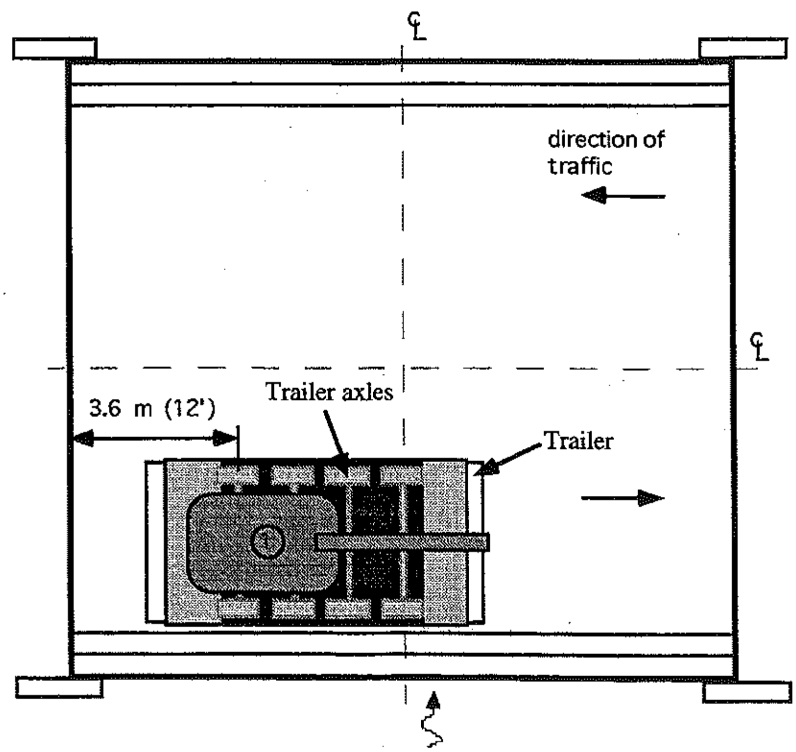


(d) Load Position 4 - Center Case.

Figure 7-5 : Longitudinal Load Positions for Center Case. (cont'd)

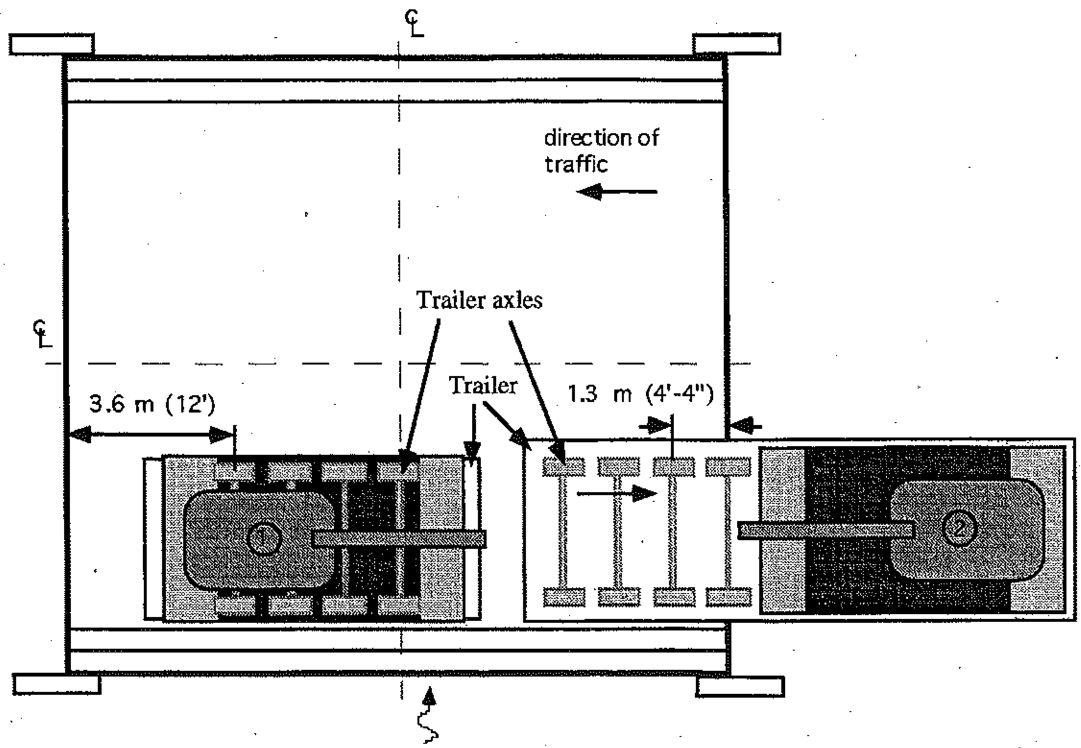


(a) Load Case 1 - Upstream Case.

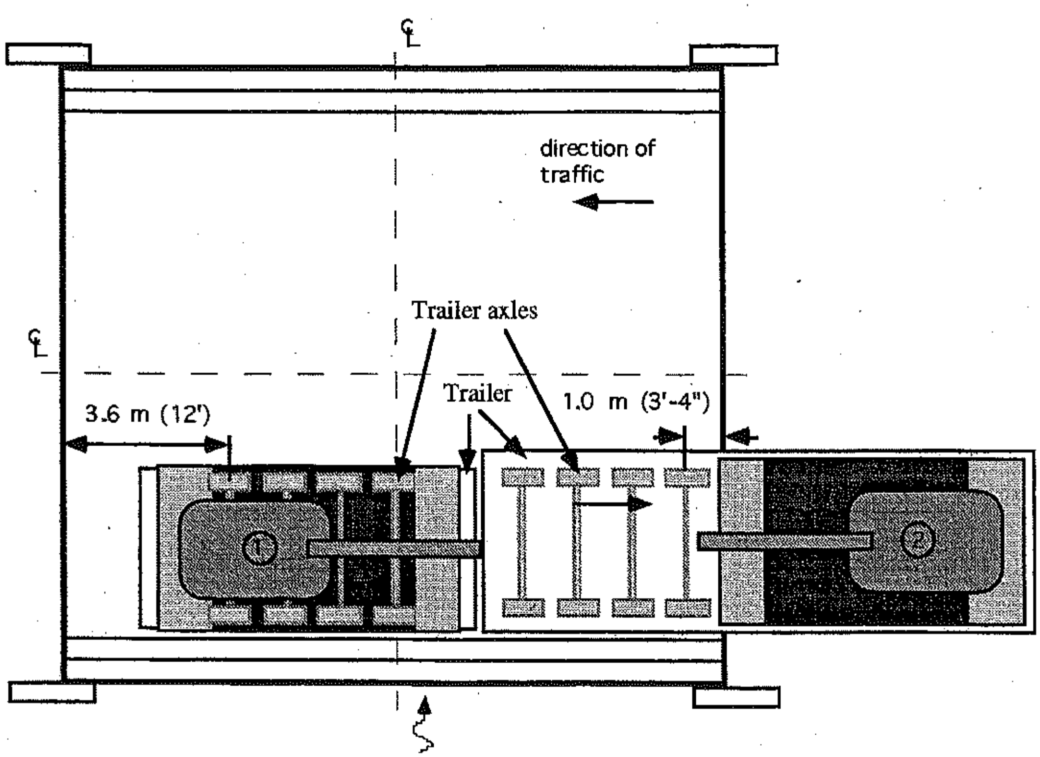


(b) Load Position 2 - Upstream Case.

Figure 7-6 : Longitudinal Load Positions for Upstream Case.



(c) Load Position 3 - Upstream Case.



(d) Load Position 4 - Upstream Case.

Figure 7-6 : Longitudinal Load Positions for Upstream Case. (cont'd)

Legends for Figure Nos. 7-4 to 7-6.

① denotes M-60 Tank on Military Trailer (see Figure 3-5).

② denotes M-60 Tank on Commercial Trailer (see Figure 3-6).

7.5 Proof Load Test Results

Deflections at the mid-point of all interiors girders and at the quarter points of selected girders were measured for each load case. The maximum deflection was only 4.7 mm (0.19 in) for girder no. 4, which corresponds to the maximum applied lane moment of 2,120 kN-m (1560 kip-ft), for upstream loading. The applied proof load moment was over 2.6 times that corresponding to HS20 loads and 1.7 times the two-unit 11-axle moment. Figure 7-7 shows the deflection at the mid-span of girder no. 3.

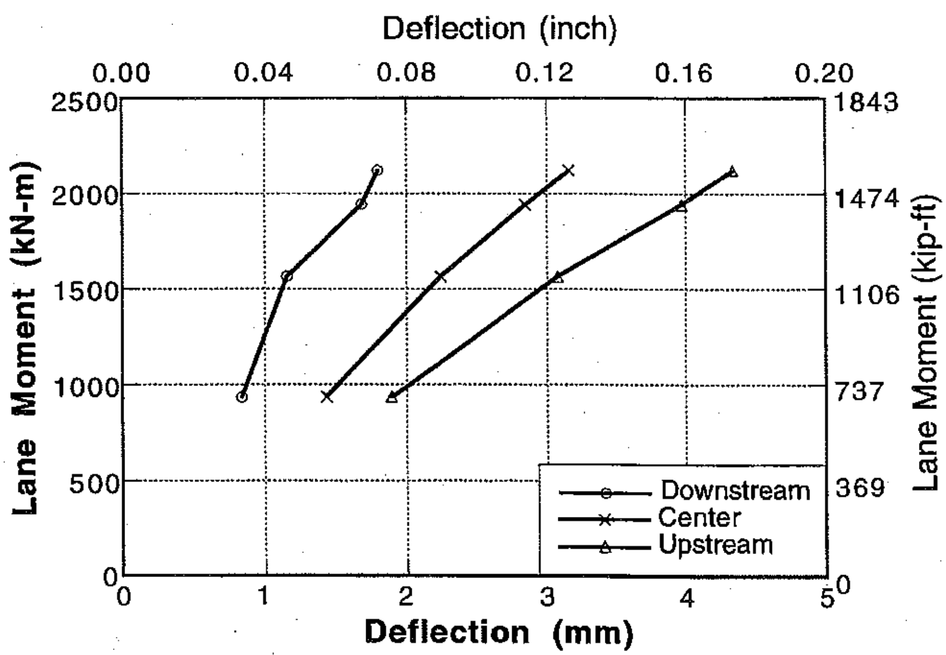


Figure 7-7 : Deflections at Mid-Span of Girder 3 of Bridge No. 2.

The relation between the applied load and the deflection is close to linear. Observed experimental deflections are considerably smaller than those predicted using the analytical model as well as the AASHTO deflection limits. The maximum analytical deflection for the non-composite model was 16 mm (0.63 in). As shown in Figure 7-8, the actual deflections in Girder no. 6 are even smaller than the composite model, which also includes the effects of non-structural members.

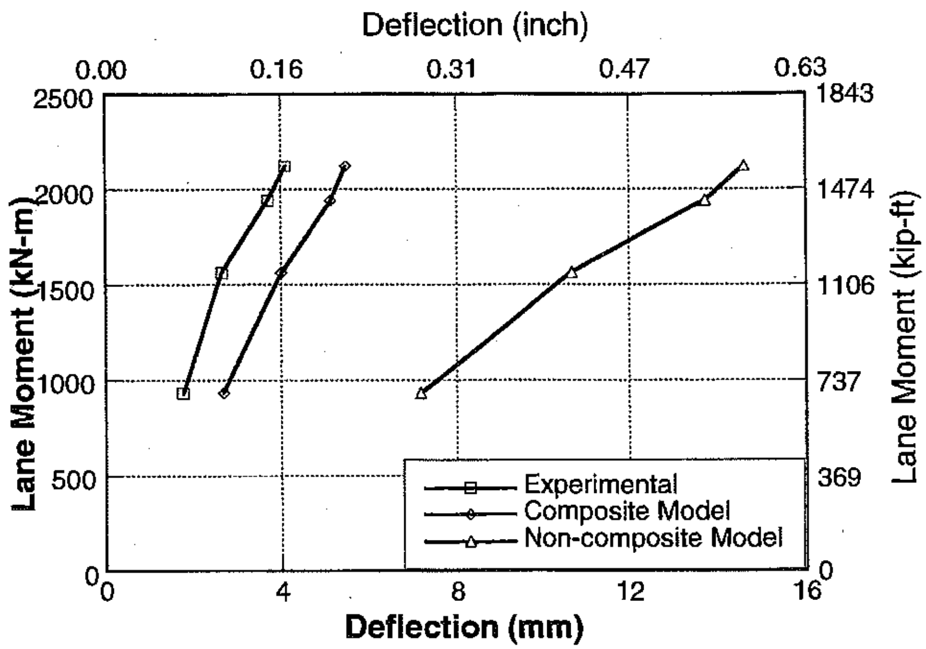


Figure 7-8 : Deflections at Mid-Span of Girder 6 for Downstream Loading on Bridge No. 2.

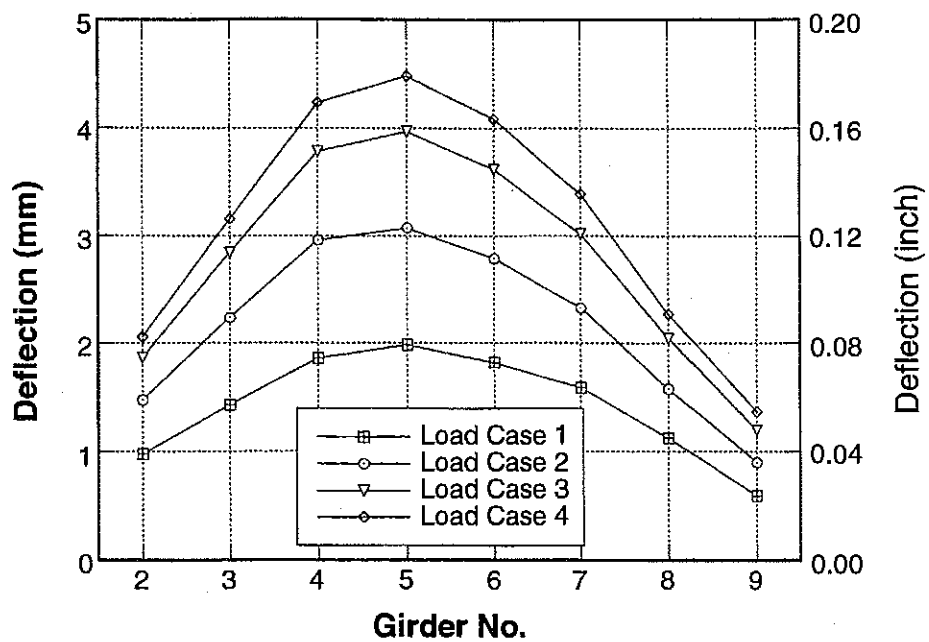


Figure 7-9 : Girder Distribution of Deflections due to Center Loading on Bridge No. 2.

This behavior indicates the presence of an unintended composite action. The difference between experimental deflections and those from the composite model, is attributed to the restraint provided by end supports. Similar behavior was observed for other girders. Lateral distribution of mid-span deflections for center loading is shown in Figure 7-9.

Stresses in the lower flanges of the steel girders were also very small, although the strain transducers were placed over areas with potential for stress concentration. Figure 7-10 shows the stresses observed in the lower flange of girder no. 3. The maximum observed stress was only 19.4 MPa (2.8 ksi), which is less than 0.1 of the yield strength of steel. The maximum stress of 48 MPa (6.9 ksi) was expected based on the non-composite model. The experimental stresses are proportional to the applied moment. A nearly linear stress-moment behavior and small values are indications of an adequate safety reserve.

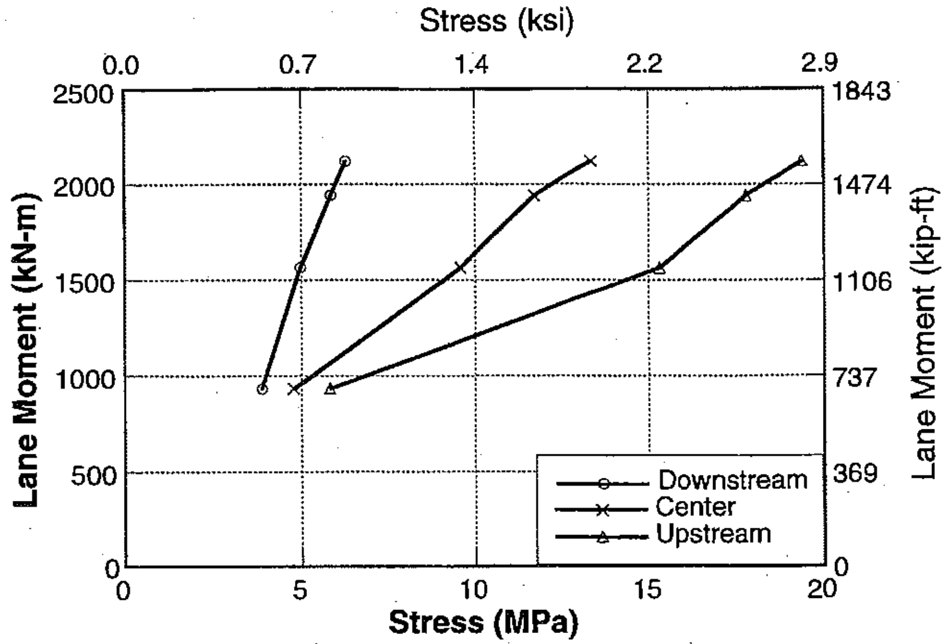


Figure 7-10 : Stresses at Mid-Span of Girder 3 of Bridge No. 2.

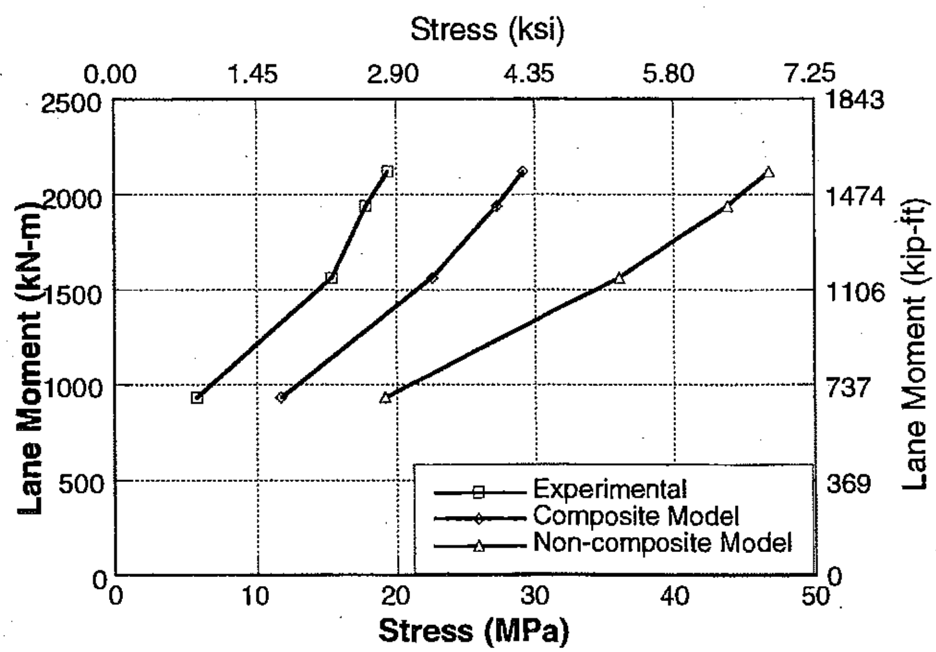


Figure 7-11 : Stresses at Mid-Span of Girder 3 for Upstream Loading on Bridge No. 2.

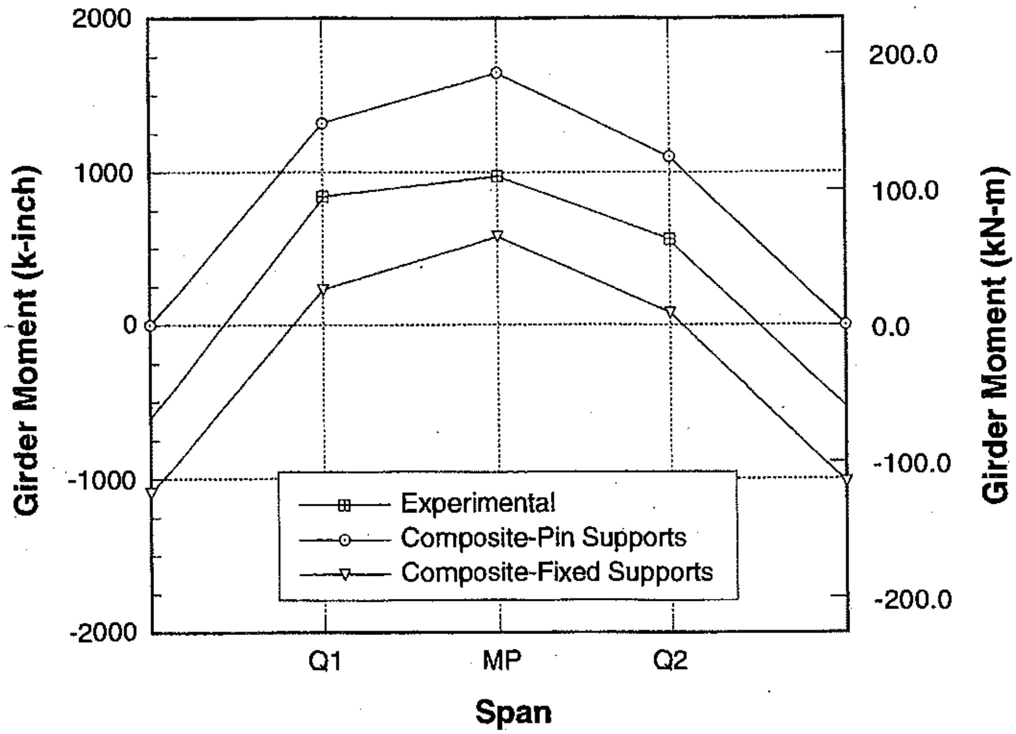


Figure 7-12 : Longitudinal Distribution of Stresses in Girder No. 9 due to the Downstream Loading on Bridge No. 2.

Stresses in other girders also confirm this observation. Experimental stresses were also compared with those from non-composite and composite models. Comparison of stresses in girder no. 3 under upstream loading is shown in Figure 7-11. The composite model gives results closer to the actual stresses as compared to the non-composite model. It confirms that the unintended composite action is present even at the high proof load level. However, the actual stresses are still smaller than those from the composite model. This is possible because the supports provide some restraint to rotation, which in turn reduces the mid-span moment. The longitudinal profile of measured strains was compared with the results from the composite model with pin supports and the composite model with fixed supports. As shown in Figure 7-12, the actual response of girder no. 9, lies between the results form the composite model with pin supports and the composite model with fixed supports. Stresses in other girders also show the same pattern. Lateral distribution of load between different girders is also shown for downstream loading in Figure 7-13. The actual distribution is

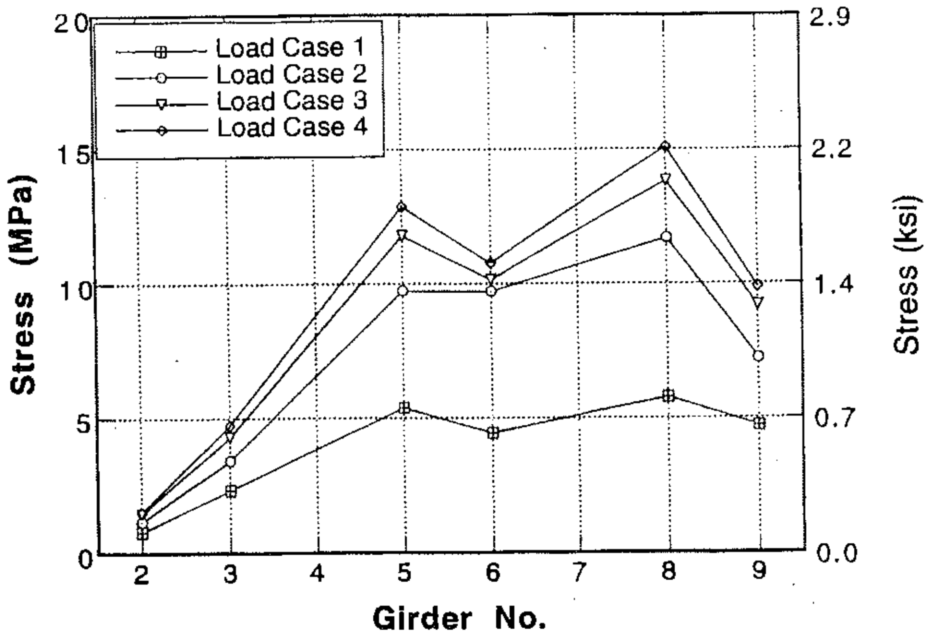


Figure 7-13 : Girder Distribution of Stresses due to Downstream Loading on Bridge No. 2.

nearly uniform, which indicates better load sharing between different members of the structure and ensures the safety of bridge under normal traffic. No sign of distress was observed during the test and the pre-determined proof load level was successfully reached. The operating rating factor for the two-unit 11-axle truck is 1.01 after the test. Therefore, the bridge was considered safe for legal truck traffic.

Note:

Intentionally left blank

8. BRIDGE NO. 3 (B01 of 78011, M-103 over White Pigeon River)

8.1 Description

This bridge has two identical simply supported spans. It is located in St. Joseph County, Michigan, near Mottville. The side elevation of this bridge is shown in Figure 8-1. It was built in 1931 and carries state highway route M-103 over the White Pigeon River with ADT of 5,300. There are ten steel girders with a 165 mm (6.5 in) thick concrete slab on top. The thickness of the concrete wearing surface is 150 mm (6 in) at the center and 75 mm (3 in) on the sides. Each span has a span length of 14.8 m (48.5 ft) and about 35° skew. The total width of the structure is 14.1 m (46.3 ft) and the total length is about 36.6 m (120 ft). Figure 8-2 shows the cross-section and dimensions for the exterior and interior girders for both spans.

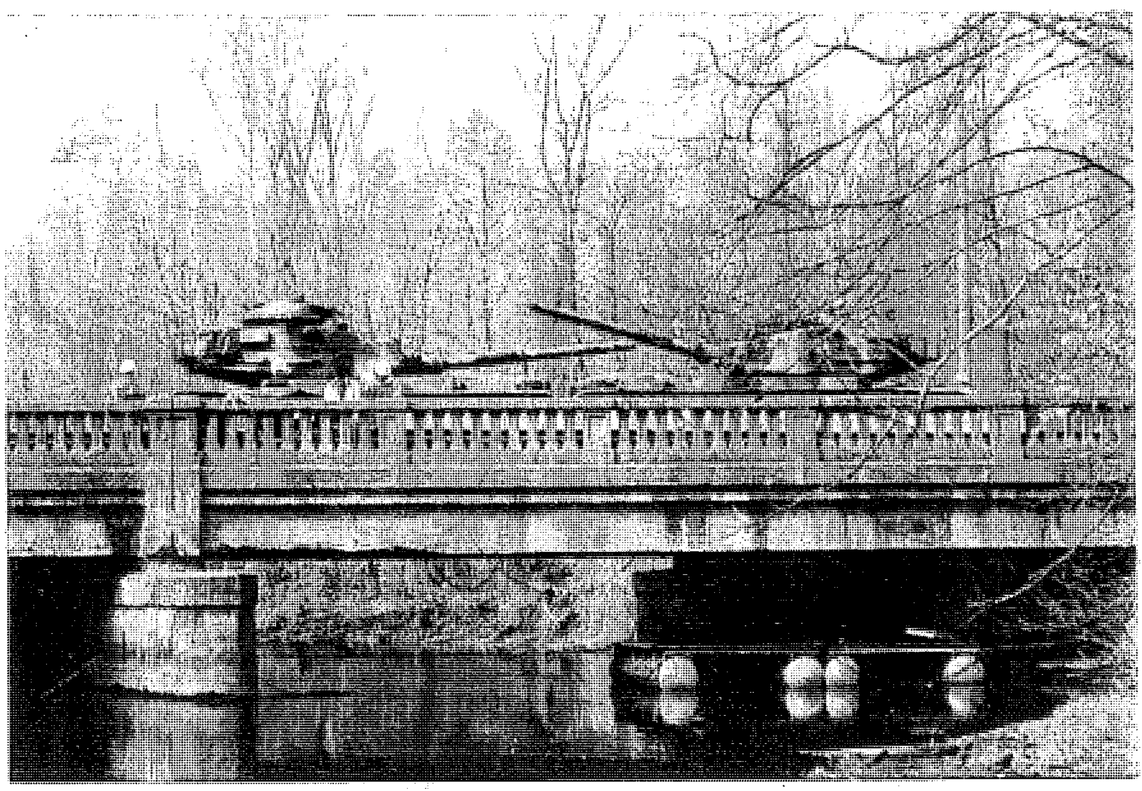


Figure 8-1 : Side Elevation of Bridge No. 3.

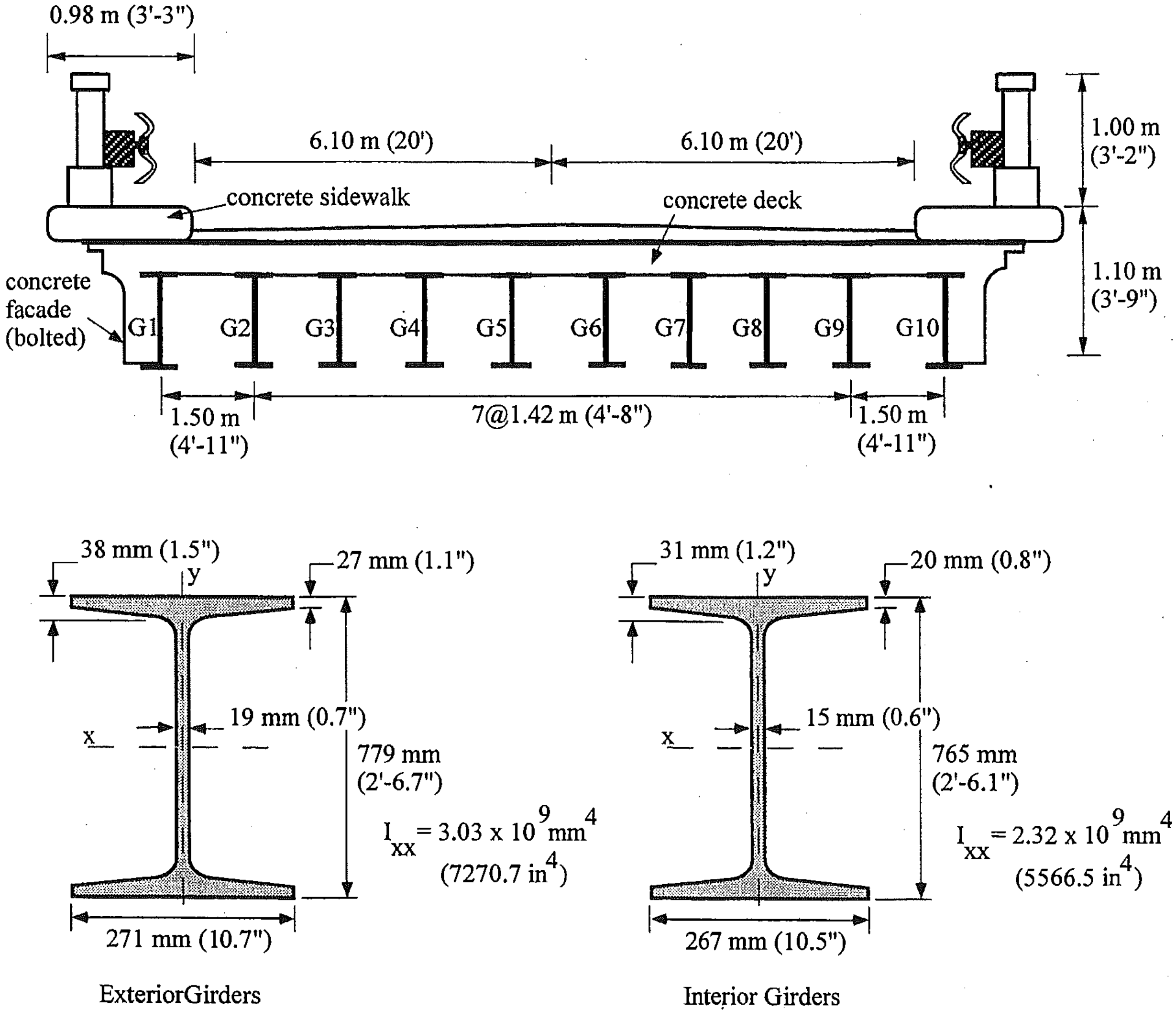


Figure 8-2 : Cross-Section of Bridge No. 3.

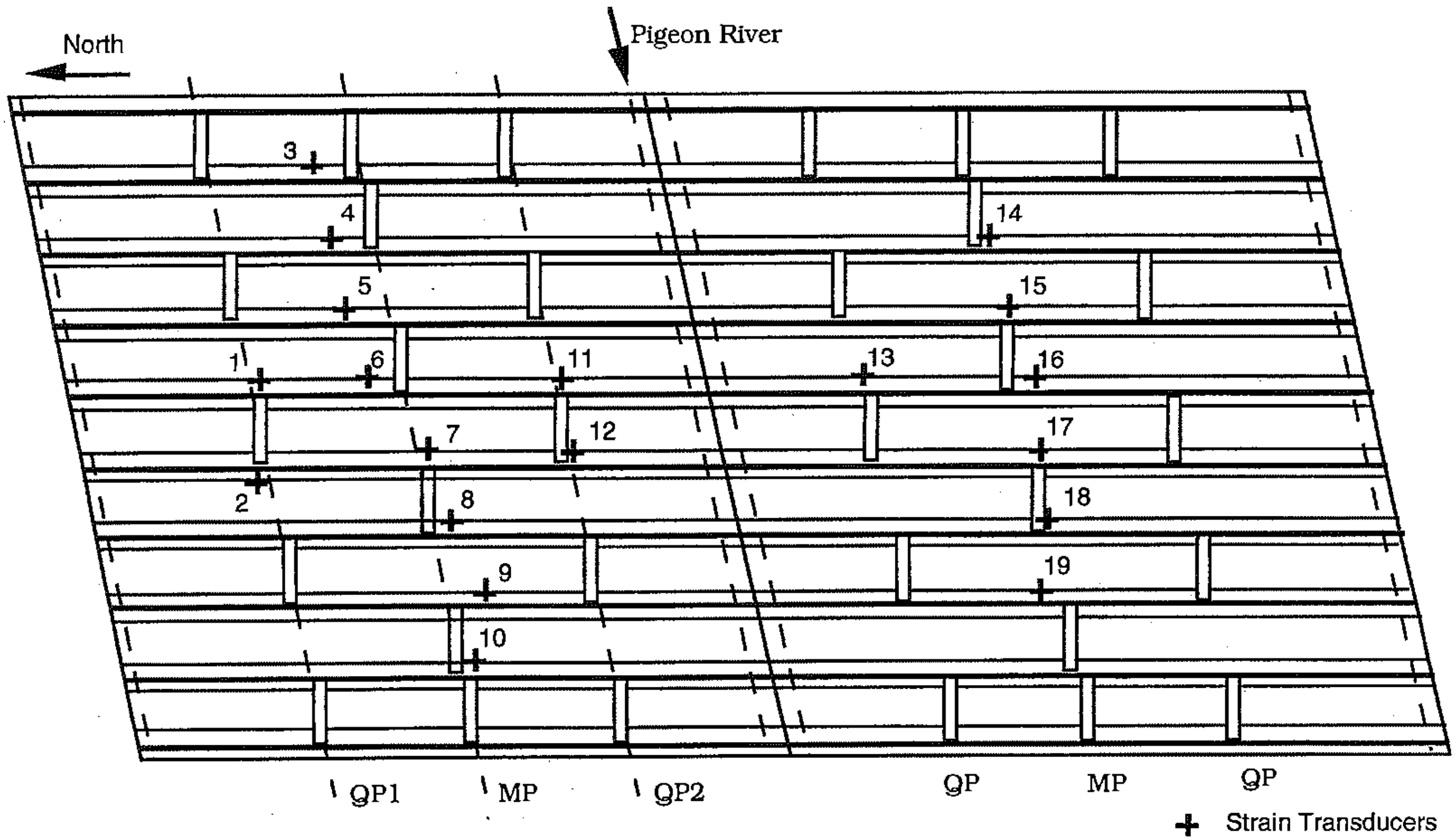
8.2 Pre-Test Inspection

In both spans, the steel girders and diaphragms were severely corroded before the repair. At the time of field investigation, the bridge was being repaired. At some locations, the reduction in flange thickness was up to 70 percent. The deterioration in the south span was more severe than that in the north span. As part of the repair, the deteriorated portions of the bridge were cleaned, painted, and angles (see attached details in Appendix F with hand calculations) were bolted to the lower flanges of the steel girders. Several diaphragms were also replaced.

The proof load test was carried out soon after the repair work was completed. The objective of the proof load test was to confirm the adequacy of the repaired structure and to verify the capacity to carry legal load. Based on the Michigan Bridge Analysis Guide (1983), the design compressive strength of concrete was 17.3 MPa (2.5 ksi), and the yield strength of steel was 207 MPa (30 ksi). The rating calculations by MDOT using BARS were available for the unrepaired section prior to the test. The inventory and operating rating factors for HS20 truck loading were 0.28 and 0.60, respectively. After repair, the remaining operating level capacity was calculated to be 1,261 kN-m (929 k-ft). The pre-test analytical results from non-composite and composite models are shown in the form of graphs and compared to the experimental results (Appendix C).

8.3 Instrumentation

Both spans of the bridge are simply supported and are not designed to transfer any moment over the pier. Therefore each span was tested separately. The north span was tested before the south span. However, during the test of each span, the unloaded span was also instrumented to check if any load was being transmitted from one span to the other. Nineteen strain transducers were used to measure strains in the lower flanges of the steel girders at the mid-span and quarter points. The locations with the smallest flange thickness were selected for instrumentation. The water level in White Pigeon River was quite high, therefore, floating platforms were required to install instrumentation. Deflections of steel girders could not be measured due to high water level, which exceeded the height of the tripod that would hold the LVDT's. Instrumentation layout for testing of the north and south spans are shown in Figures 8-3 and 8-4, respectively.

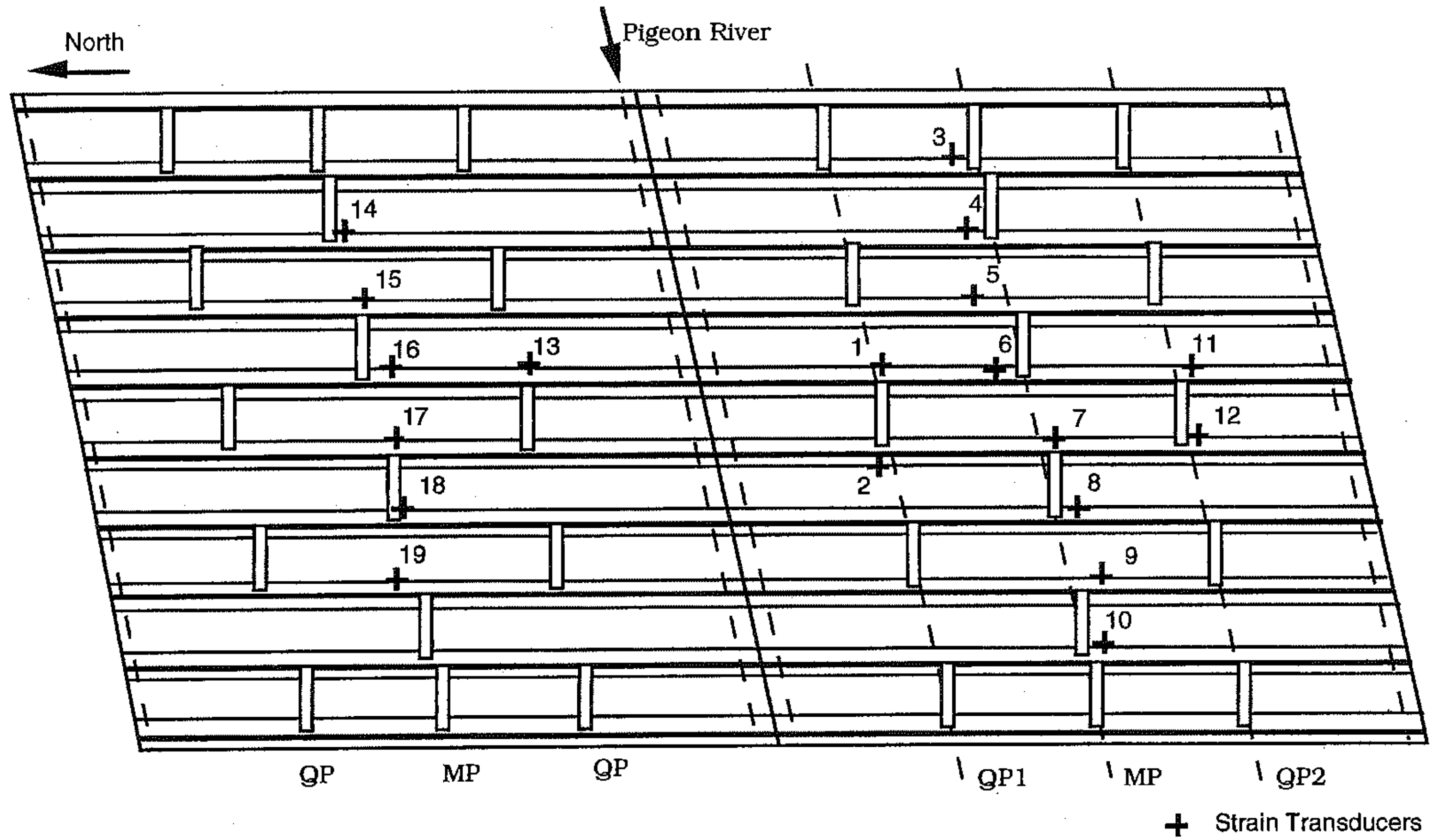


MP : Mid Point.

QP1 : Quarter Point 1.

QP2 : Quarter Point 2.

Figure 8-3 : Instrumentation Layout for North Span of Bridge No. 3.



MP : Mid Point.

QP1 : Quarter Point 1.

QP2 : Quarter Point 2.

Figure 8-4 : Instrumentation Layout for South Span of Bridge No. 3.

8.4 Proof Load Positions

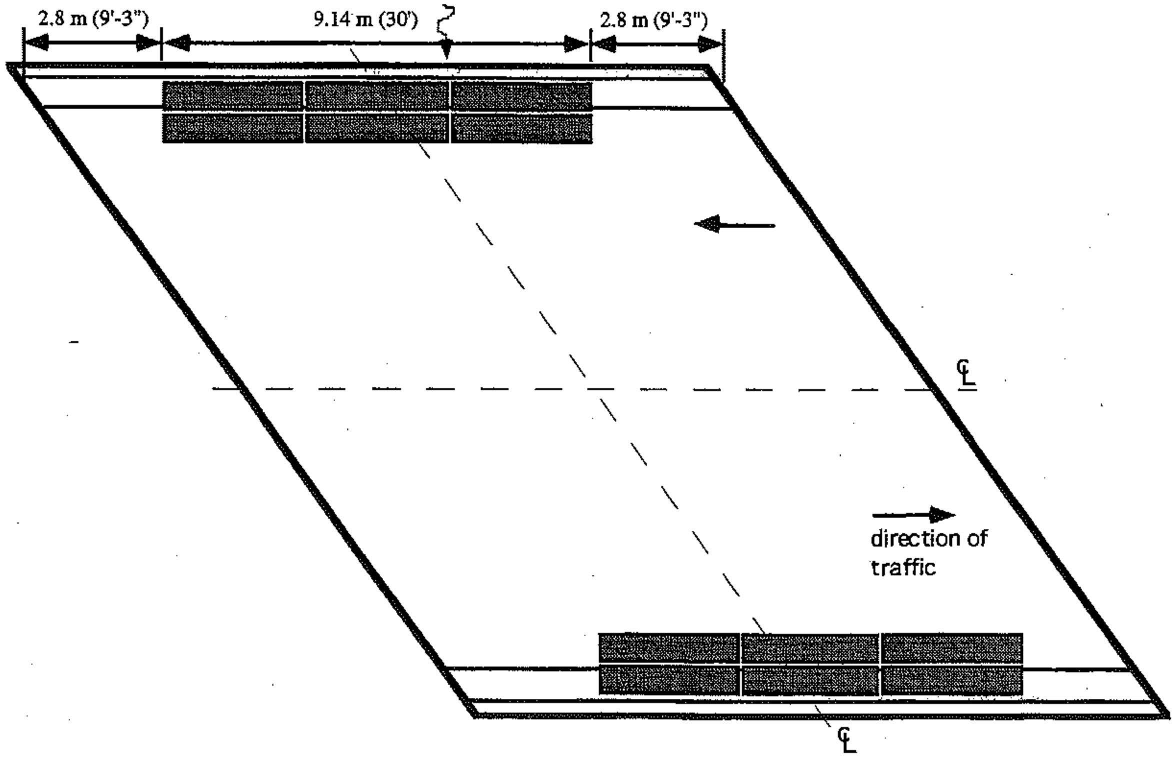
Two M-60 tanks, placed directly on the pavement, were used to load the bridge. In addition to these tanks, several concrete barrier blocks, weighing 22 kN (5 kip) each, were also used. For the test on the north span, six blocks were placed on each curb. Then, it turned out that there was a possibility to place more blocks. Therefore, eight blocks were placed on each curb for the south span test. The lane moment caused by these blocks was calculated. The concrete blocks were considered to be the first load step. Then, the tanks were placed on the bridge. Three load cases were considered, by gradually moving the tanks towards the mid span until the target proof load lane moment was attained. The actual load positions during proof load testing of each span are shown in Figures 8-5 to 8-10. The applied lane moments from all load positions are listed in Tables 8-1 and 8-2. The target proof load moment was 2,145.8 kN-m (1,582 k-ft). The traffic was allowed over partial width during the test, and it was fully stopped only at critical times during maximum load placement, i.e. Load Position 4.

Table 8-1 : Applied Proof Load Lane Moments for North Span.

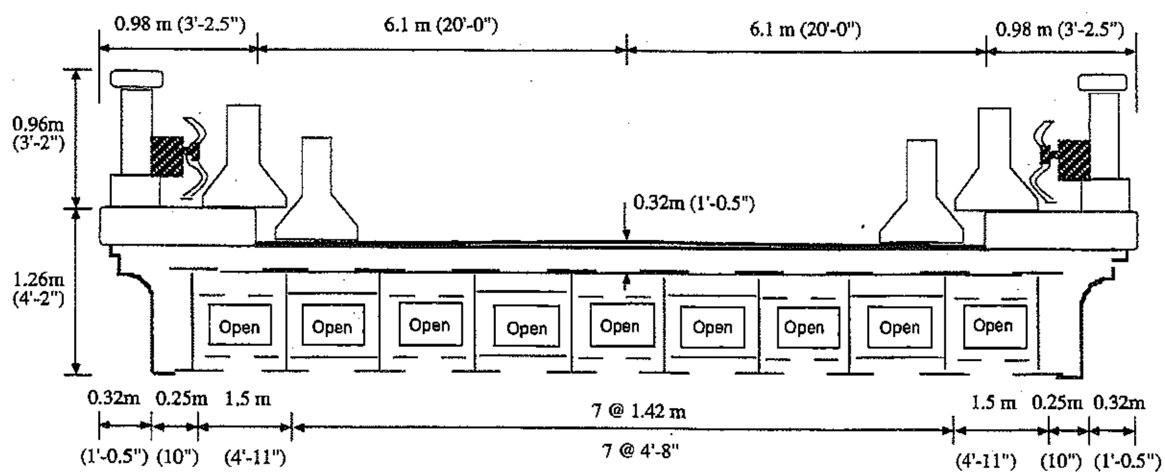
Load Position. No	Applied Lane Moment in kN-m (k-ft)		
	Quarter Point 1	Mid Point	Quarter Point 2
1	500 (368)	710 (524)	500 (368)
2	1670 (1231)	1820 (1342)	1055 (778)
3	1660 (1224)	2015 (1486)	1150 (848)
4	2040 (1504)	2765 (2039)	2150 (1584)

Table 8-2 : Applied Proof Load Lane Moments for South Span.

Load Position. No	Applied Lane Moment kN-m (k-ft)		
	Quarter Point 1	Mid Point	Quarter Point 2
1	670 (494)	950 (700)	670 (494)
2	1835 (1353)	2055 (1515)	1220 (900)
3	1830 (1349)	2250 (1659)	1315 (970)
4	2205 (1626)	3000 (2212)	2320 (1710)



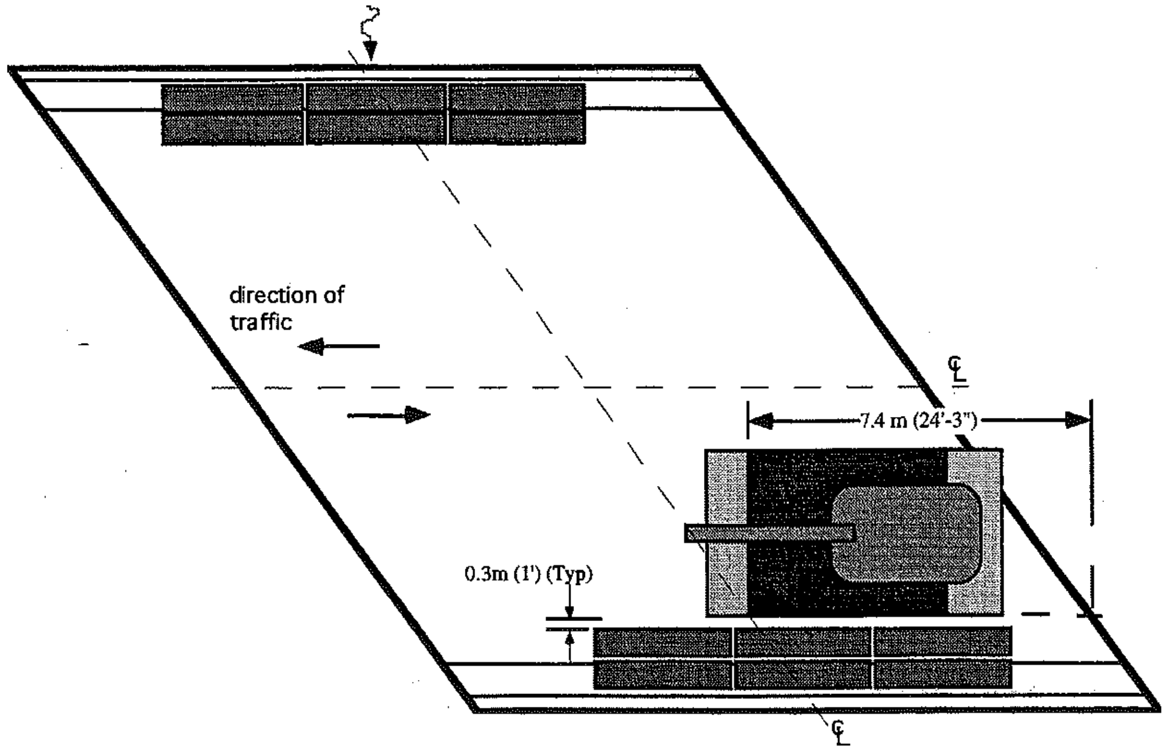
(i) Plan View.



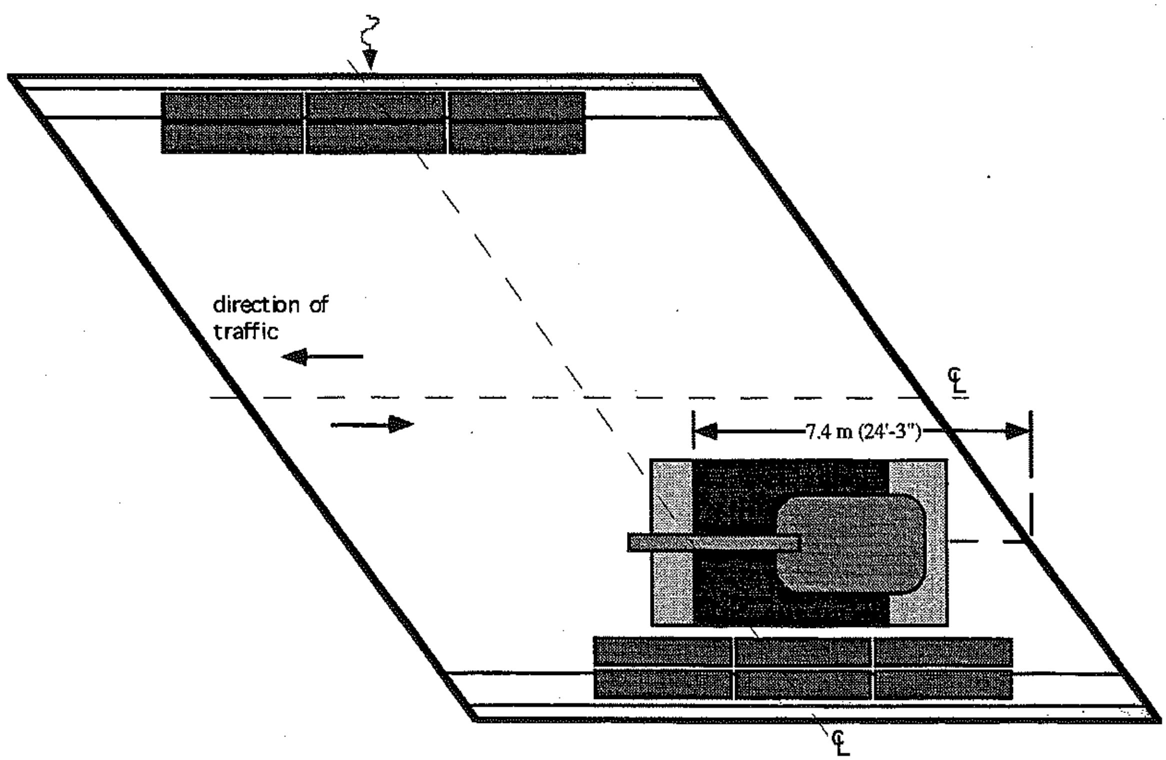
(ii) Cross-Section.

(a) Load Position 1 - All Cases.

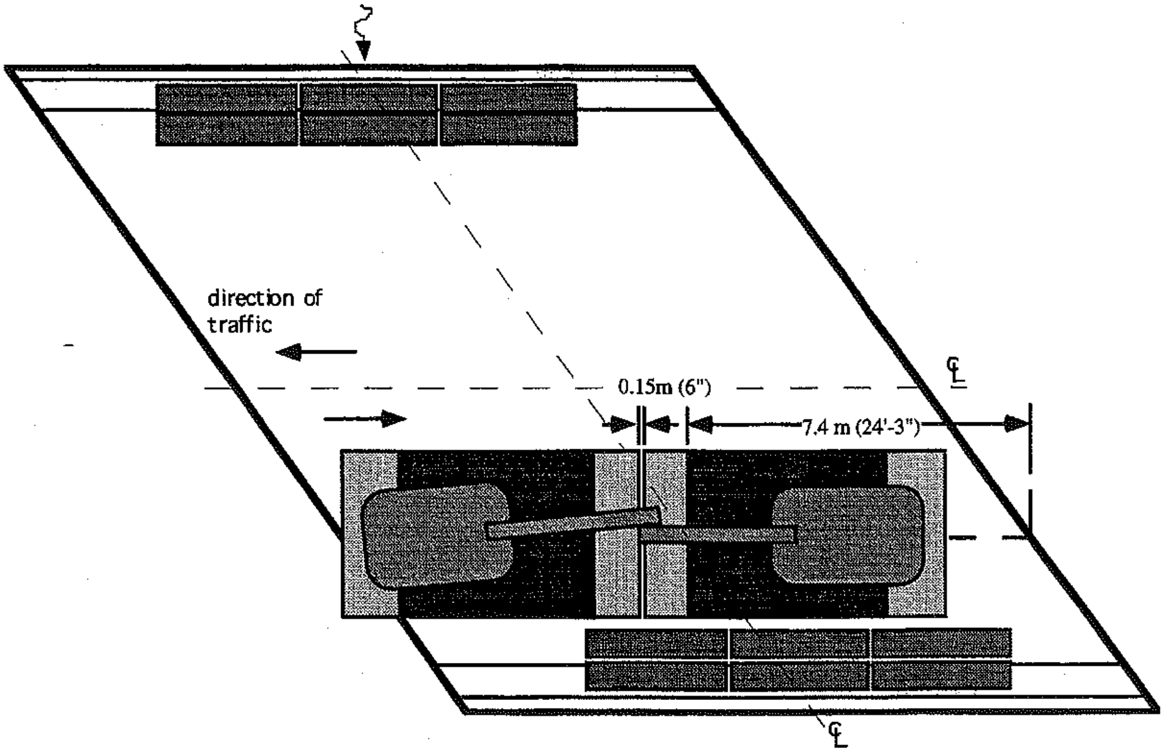
Figure 8-5 : Longitudinal Load Positions for Downstream Case for North Span (N-S).



(b) Load Position 2 - Downstream Case.

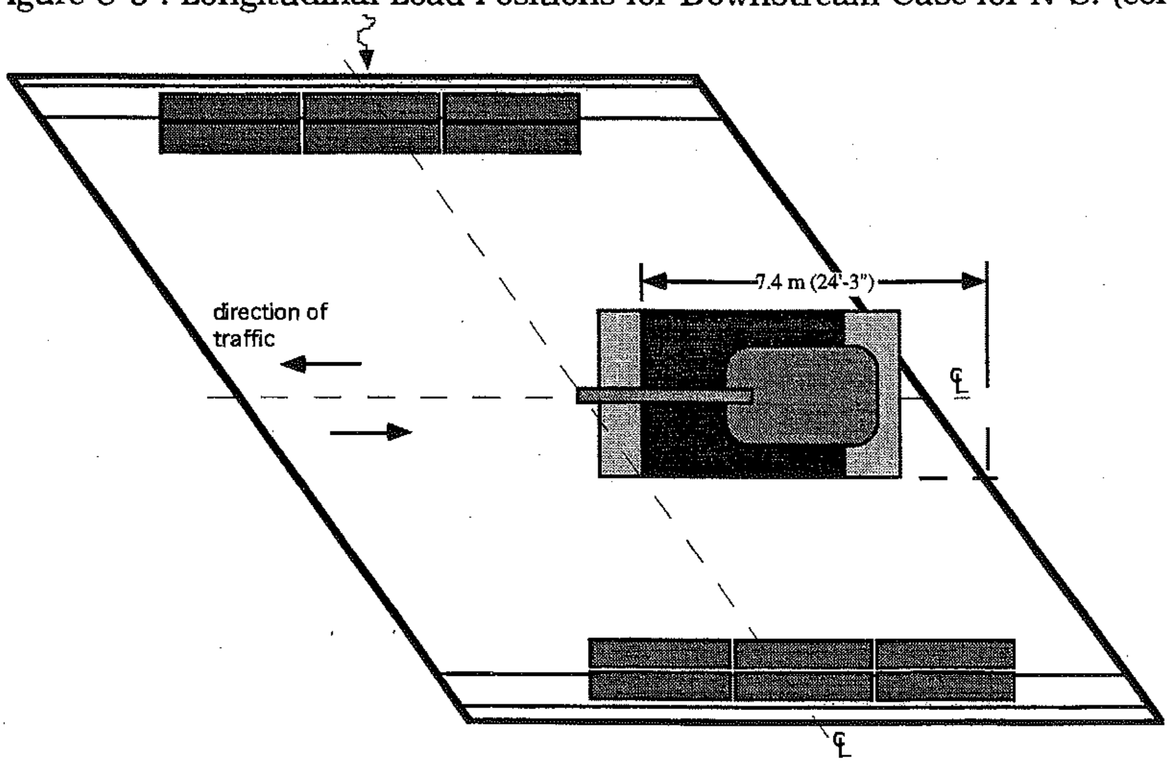


(c) Load Position 3 - Downstream Case.
Figure 8-5 : Longitudinal Load Positions for Downstream Case for N-S. (cont'd)



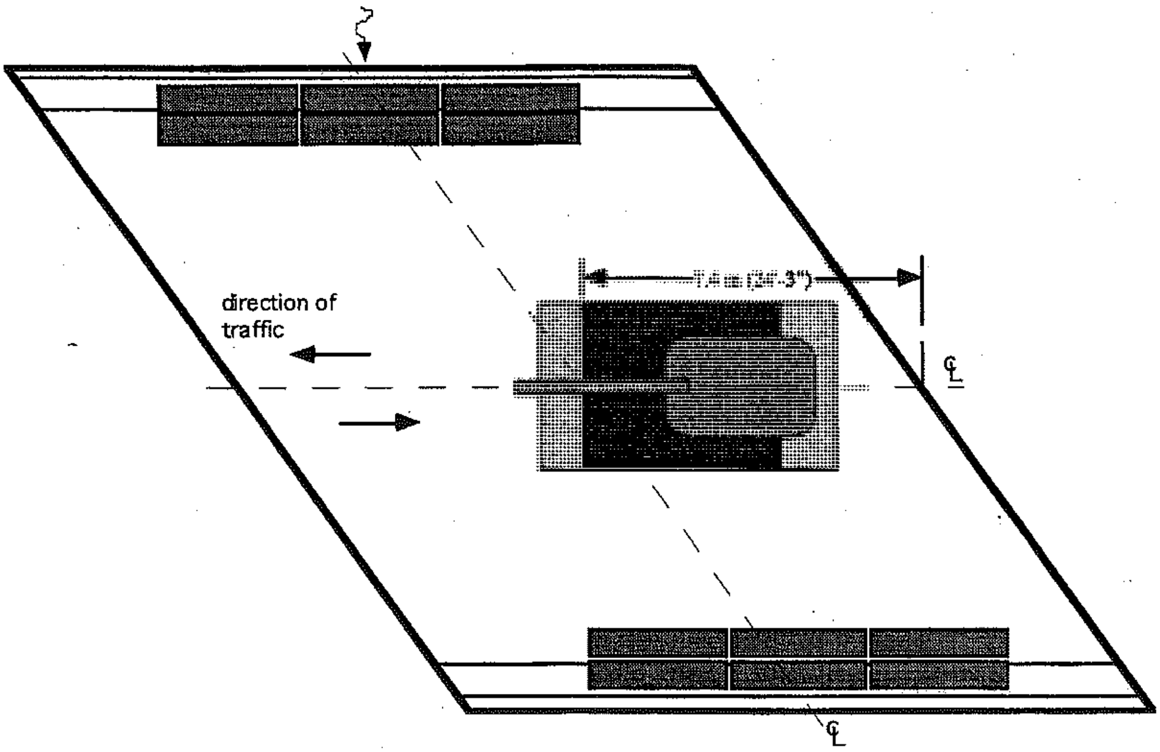
(d) Load Position 4 - Downstream Case.

Figure 8-5 : Longitudinal Load Positions for Downstream Case for N-S. (cont'd)

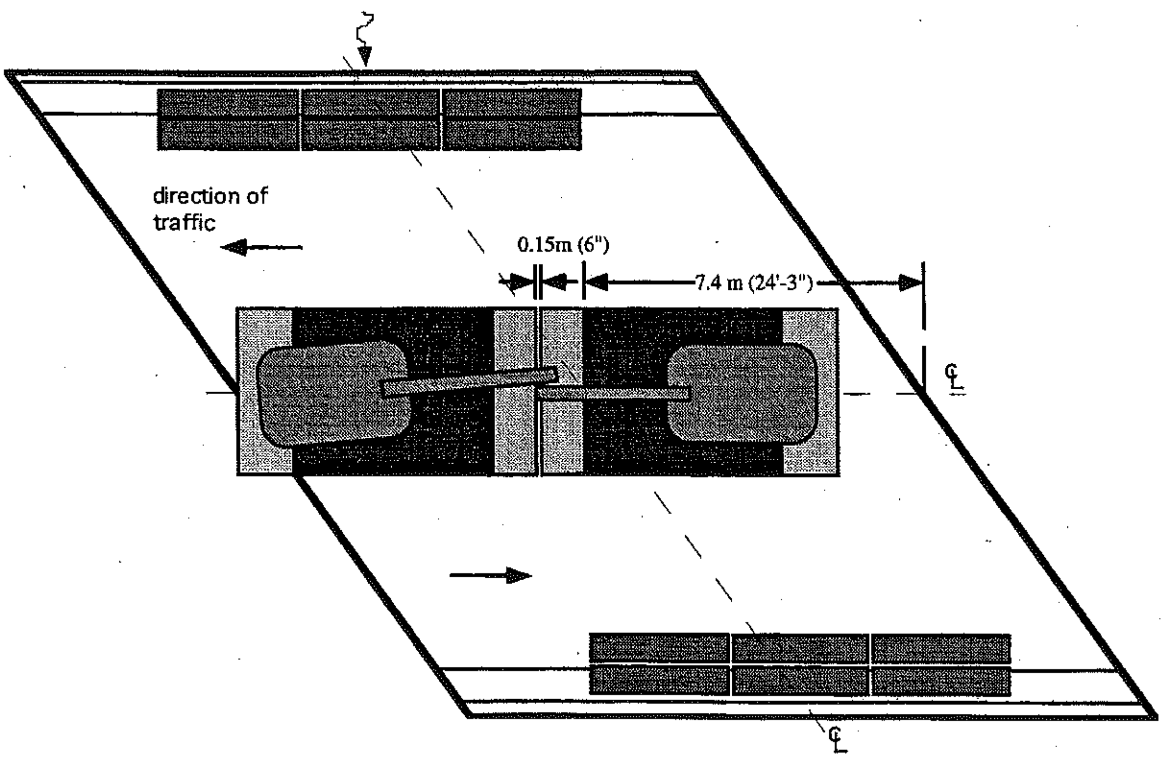


(a) Load Position 2 - Center Case.

Figure 8-6 : Longitudinal Load Positions for Center Case for N-S.

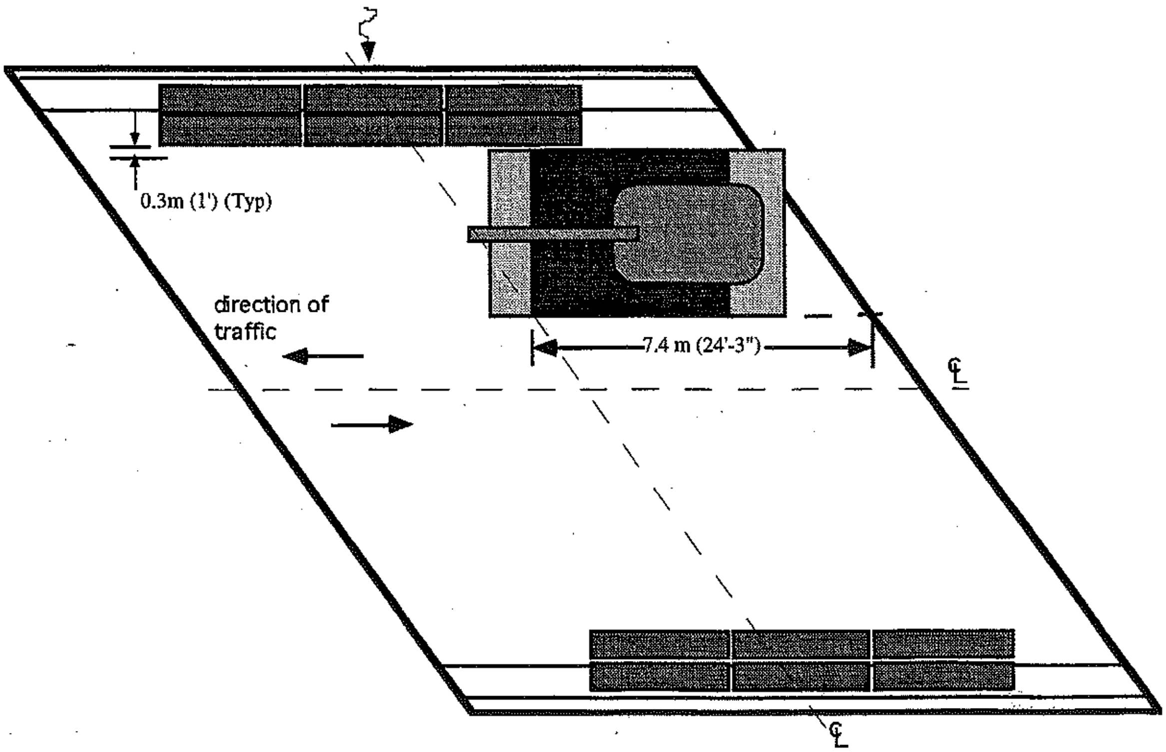


(b) Load Position 3 - Center Case.

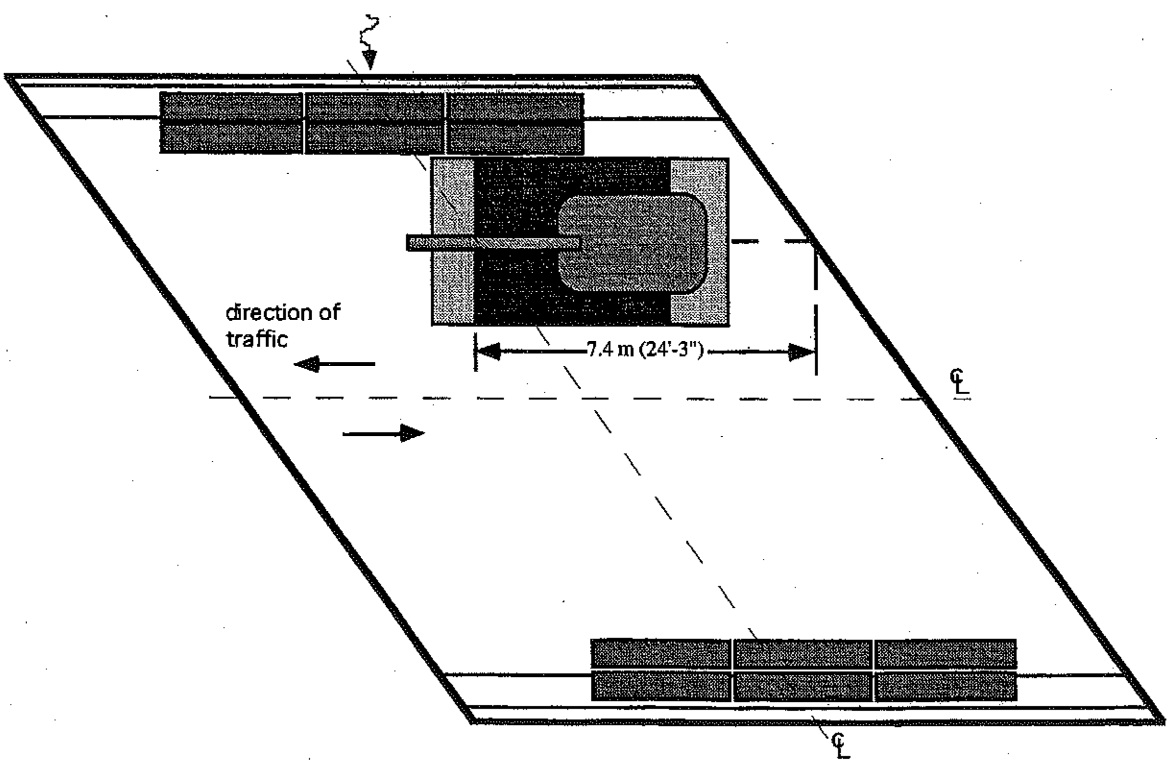


(c) Load Position 4 - Center Case.

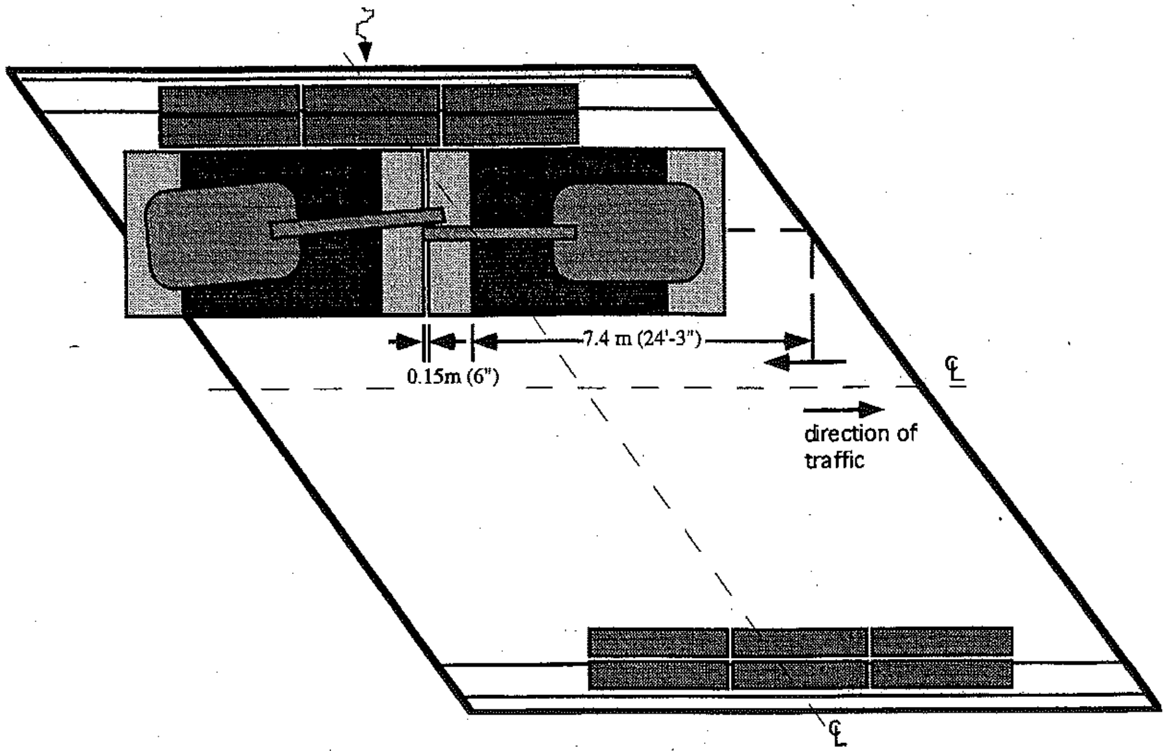
Figure 8-6 : Longitudinal Load Positions for Center Case for N-S. (cont'd)



(a) Load Position 2 - Upstream Case.

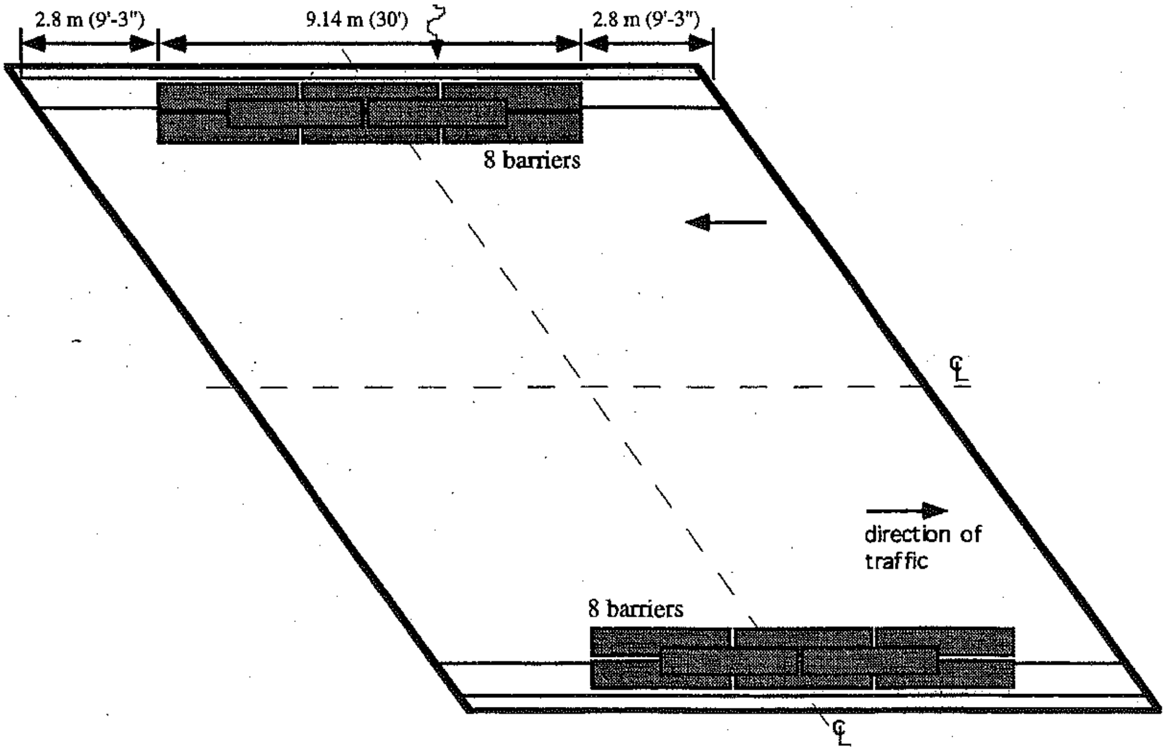


(b) Load Position 3 - Upstream Case.
Figure 8-7 : Longitudinal Load Positions for Upstream Case for N-S.

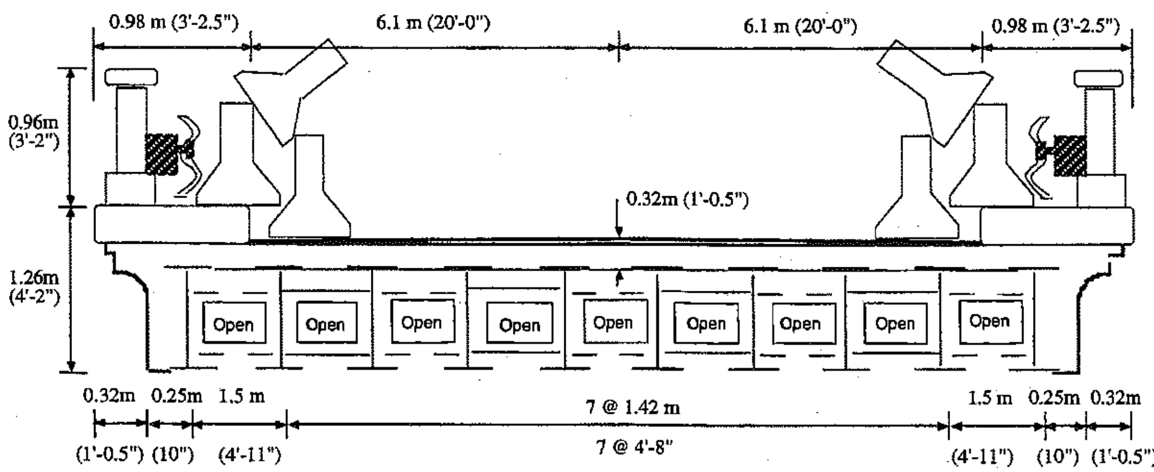


(c) Load Position 4 - Upstream Case.

Figure 8-7 : Longitudinal Load Positions for Upstream Case for N-S. (cont'd)



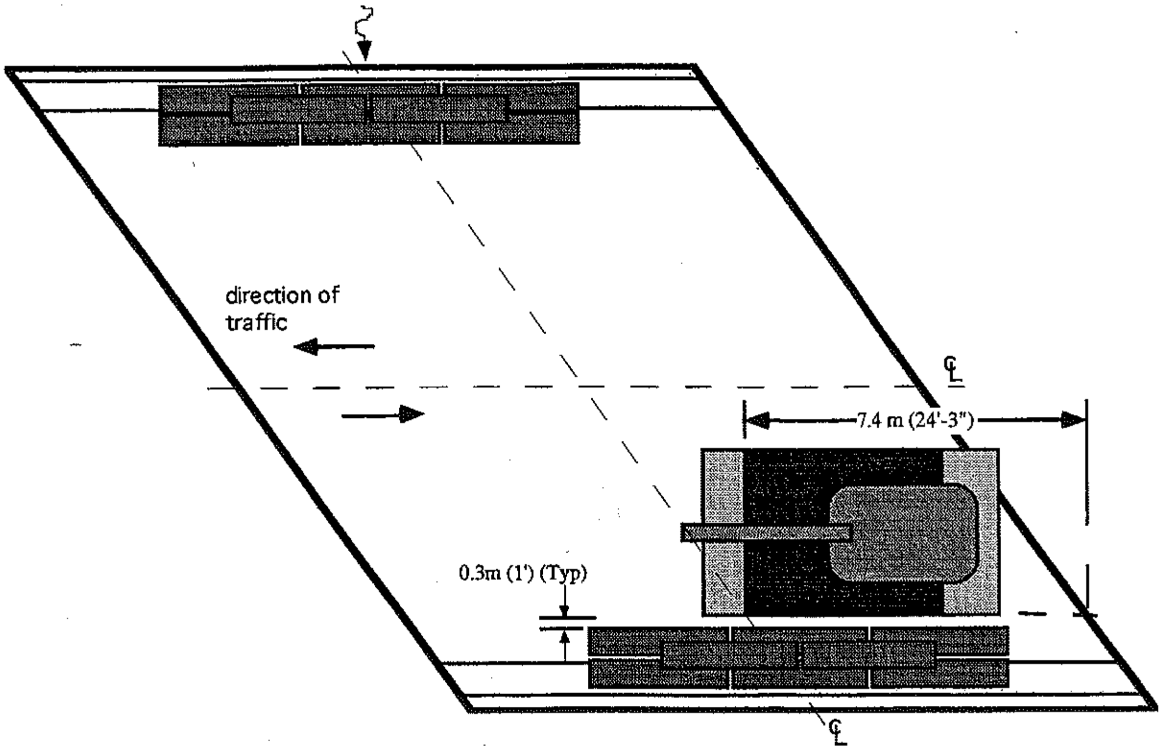
(i) Plan View.



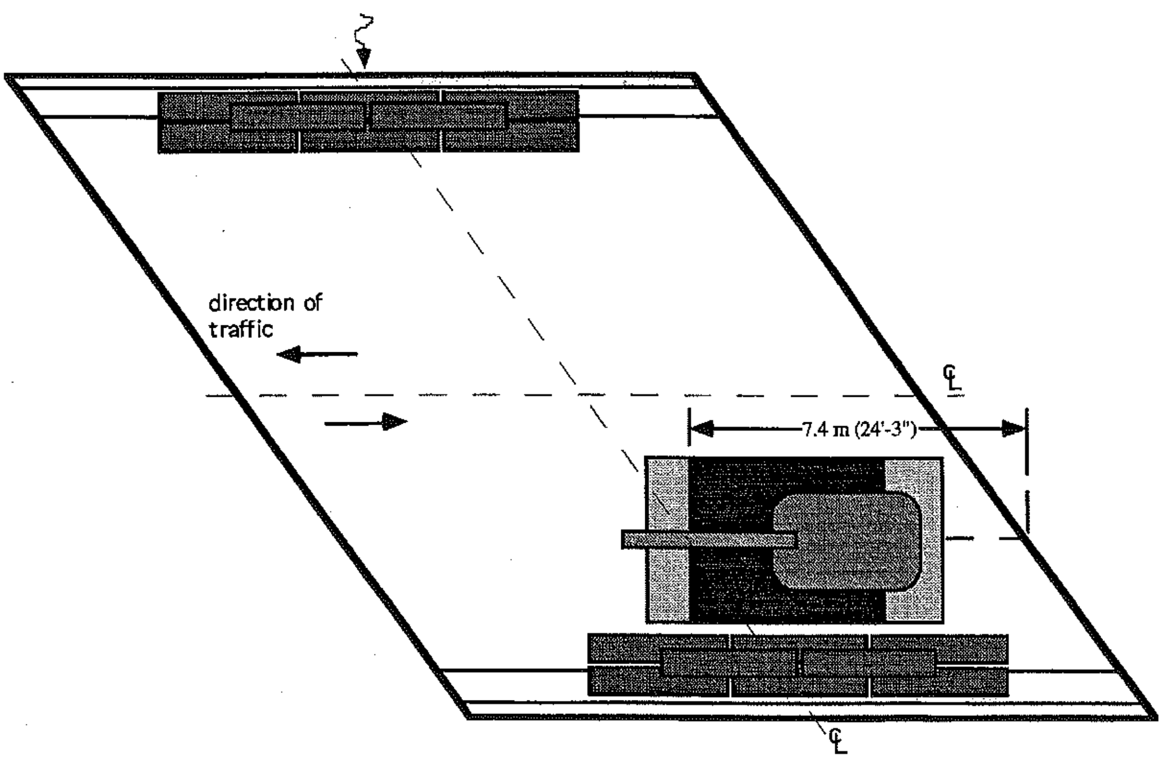
(ii) Cross-Section.

(a) Load Position 1 - All Cases.

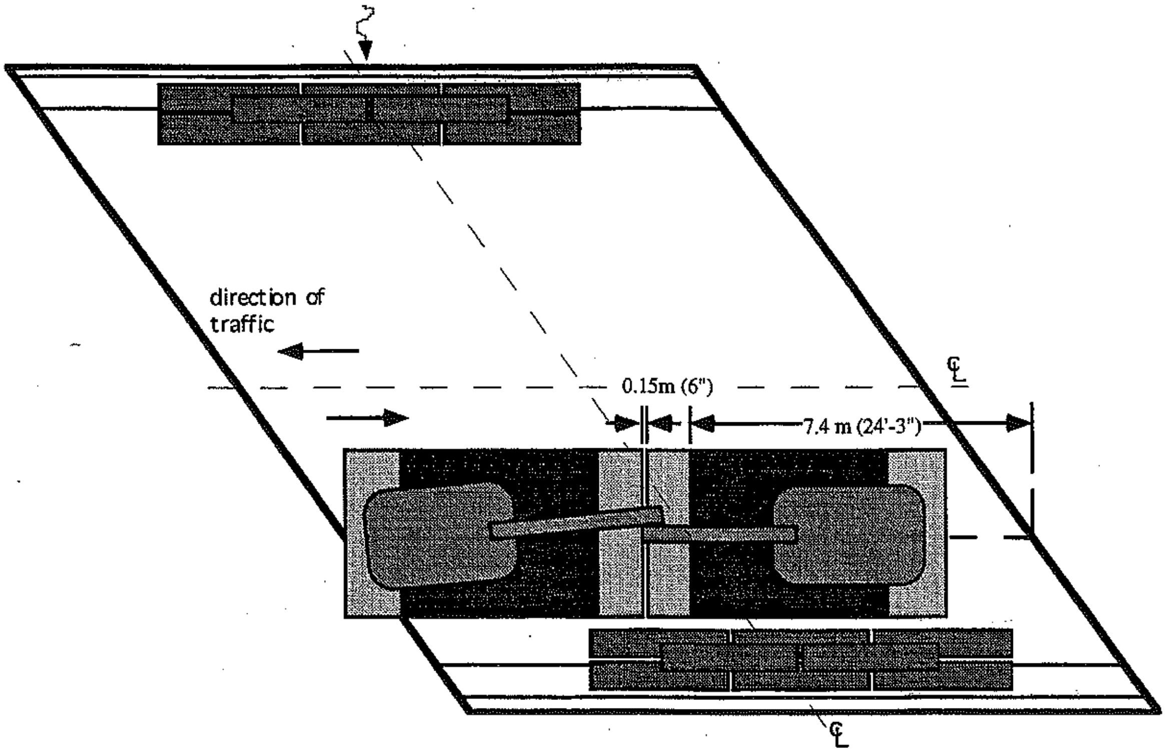
Figure 8-8 : Longitudinal Load Positions for Downstream Case for South Span (S-S).



(b) Load Position 2 - Downstream Case.

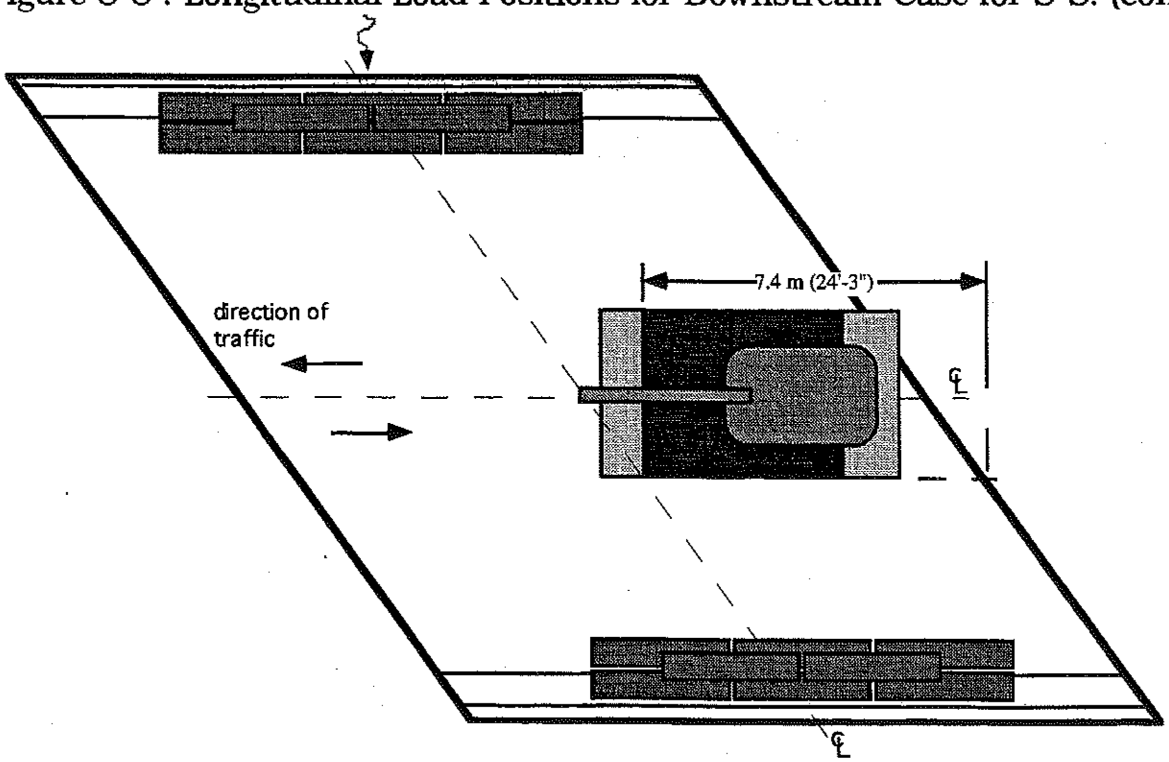


(c) Load Position 3 - Downstream Case.
Figure 8-8 : Longitudinal Load Positions for Downstream Case for S-S. (cont'd)



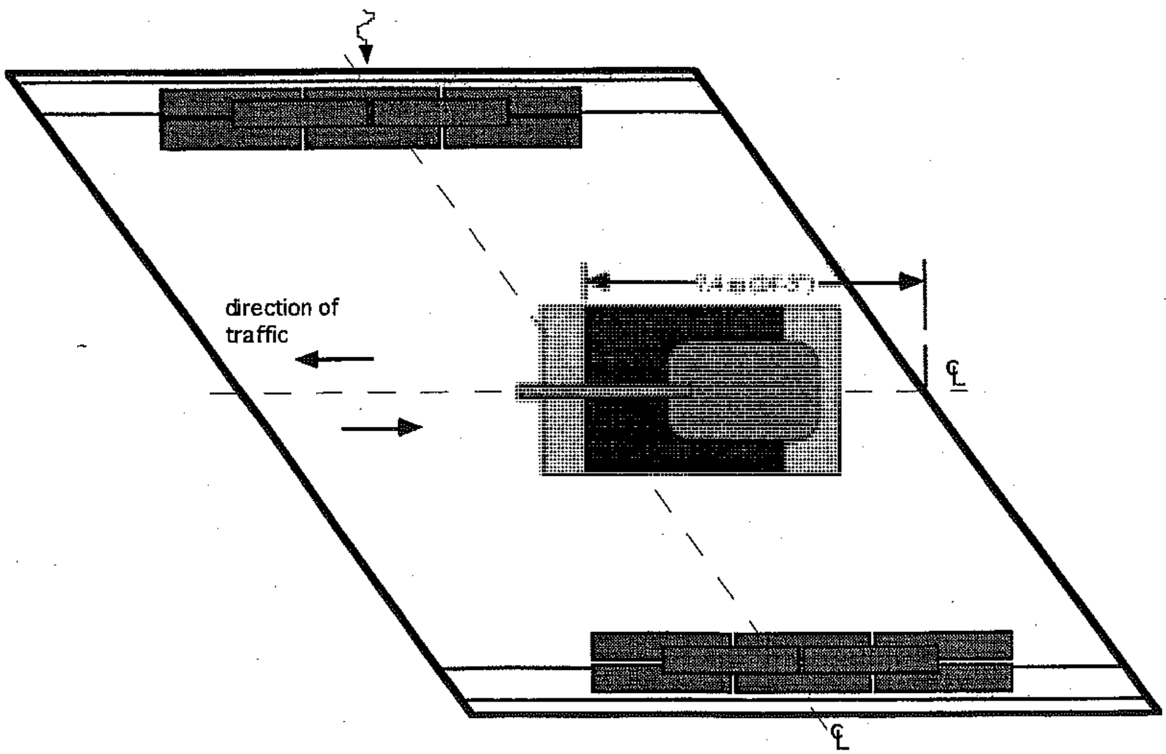
(d) Load Position 4 - Downstream Case.

Figure 8-8 : Longitudinal Load Positions for Downstream Case for S-S. (cont'd)

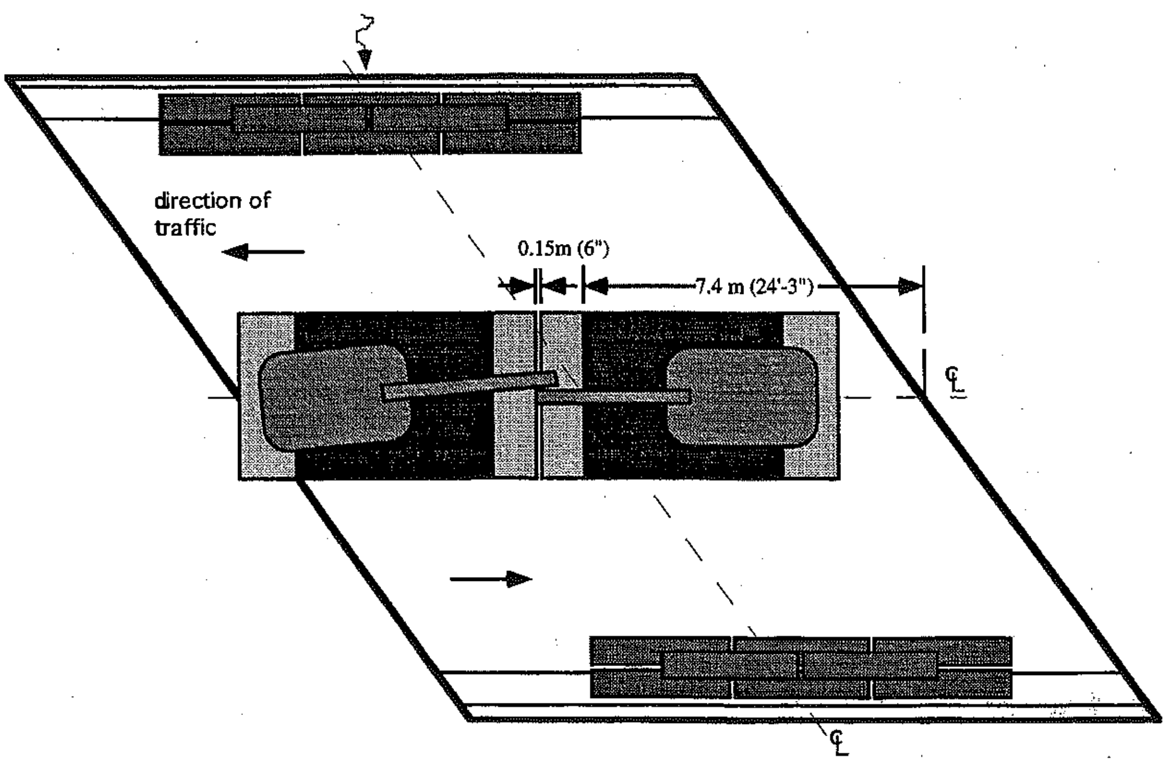


(a) Load Position 2 - Center Case.

Figure 8-9 : Longitudinal Load Positions for Center Case for S-S.

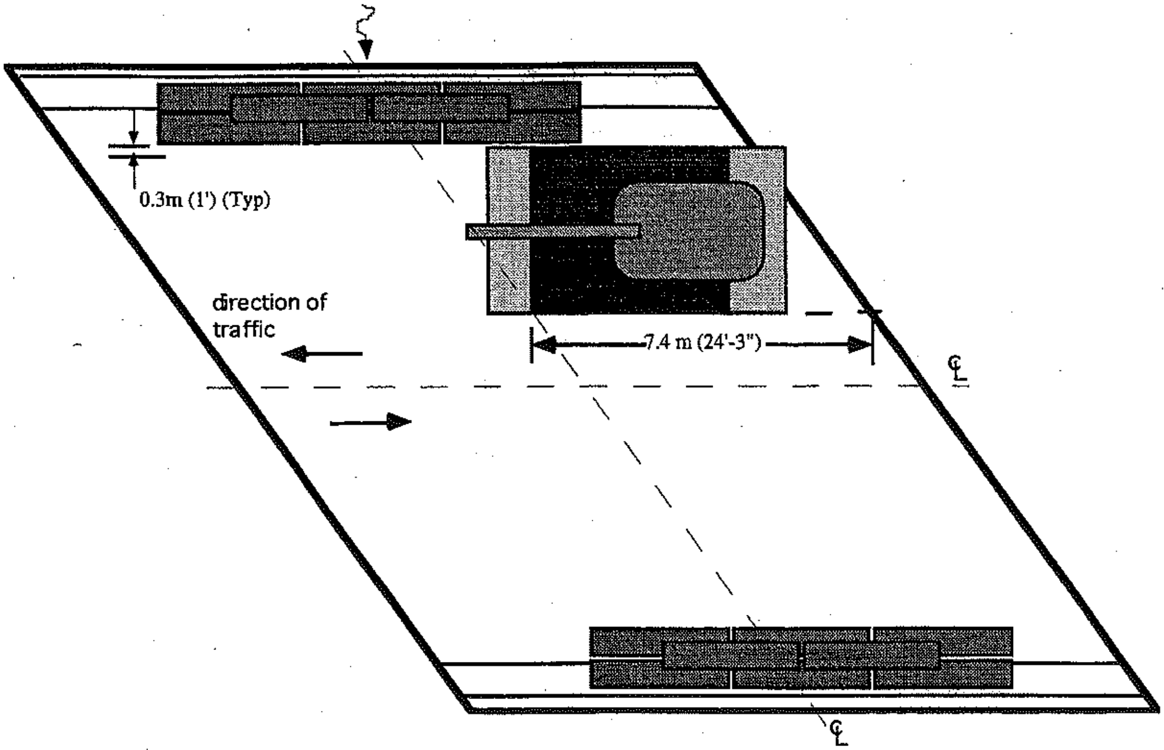


(b) Load Position 3 - Center Case.

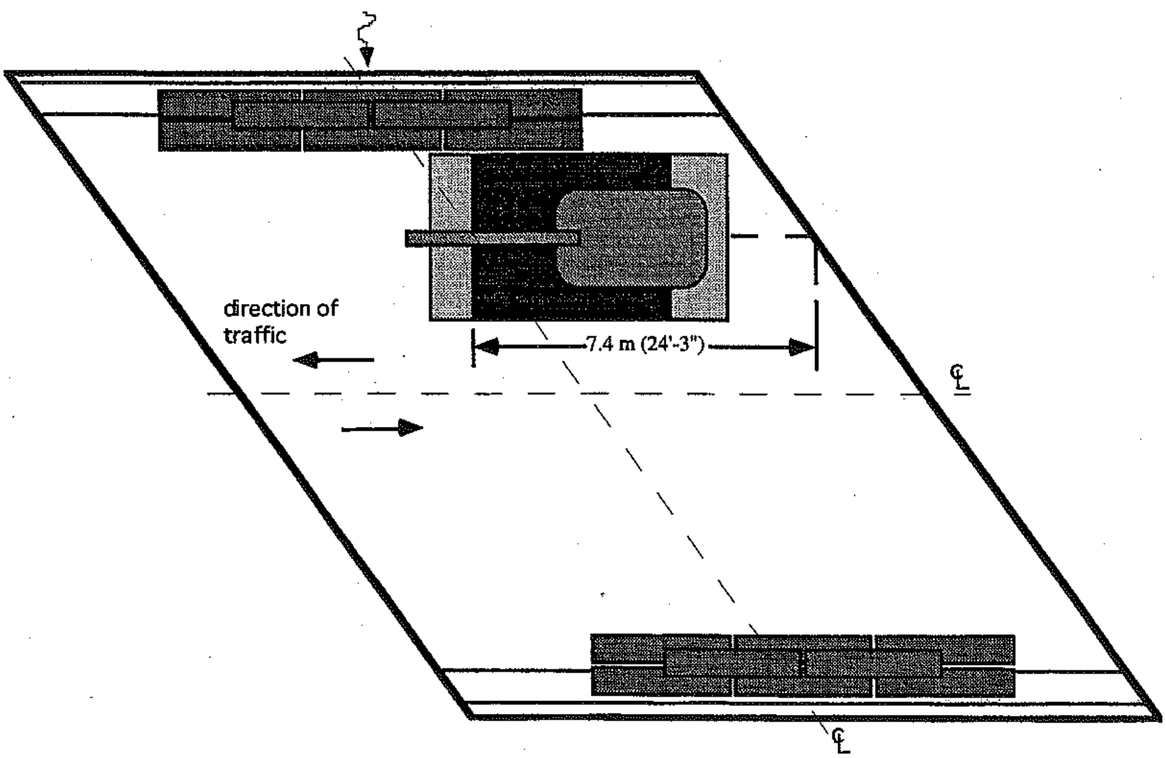


(c) Load Position 4 - Center Case.

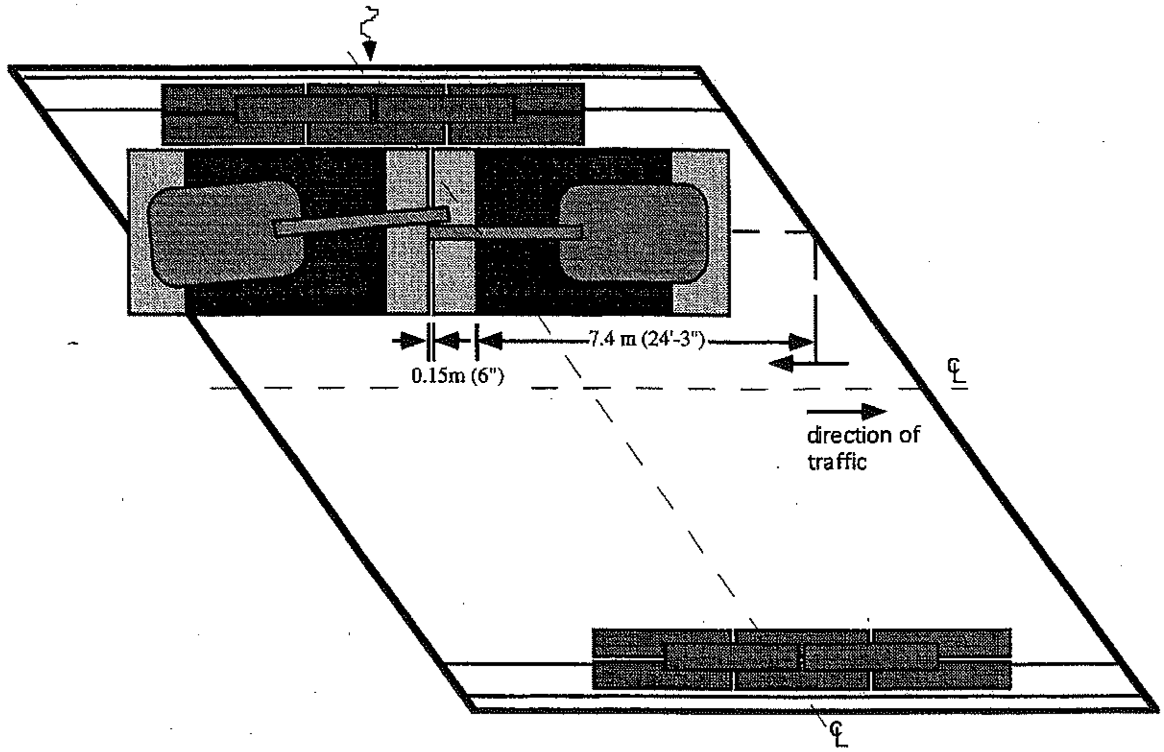
Figure 8-9 : Longitudinal Load Positions for Center Case for S-S. (cont'd)



(a) Load Position 2 - Upstream Case.



(b) Load Position 3 - Upstream Case.
Figure 8-10 : Longitudinal Load Positions for Upstream Case for S-S.



(c) Load Position 4 - Upstream Case.

Figure 8-10 : Longitudinal Load Positions for Upstream Case for S-S. (cont'd)

8.5 Proof Load Test Results for the North Span

Only the strains in the lower flanges of the steel girders were measured at locations close to the mid-span and quarter points (Figure 8-3). The maximum experimental stress was 22.8 MPa (3.3 ksi) in girder no. 3 corresponding to the maximum proof load level of 2,765 kN-m (2,039 k-ft) in the upstream position. The actual stresses in girder no. 3 are shown with the applied lane moment in Figure 8-11. The relationship between measured stress and applied lane moment is linear. The magnitude of stress is about 0.1 of the yield strength, which reveals the extra capacity of this bridge. Small stresses also show that the repair of the previously deteriorated flanges is effective. During the testing of this span, the response of the south span was also monitored to check possible continuity over intermediate support. However, no evidence of continuity was observed.

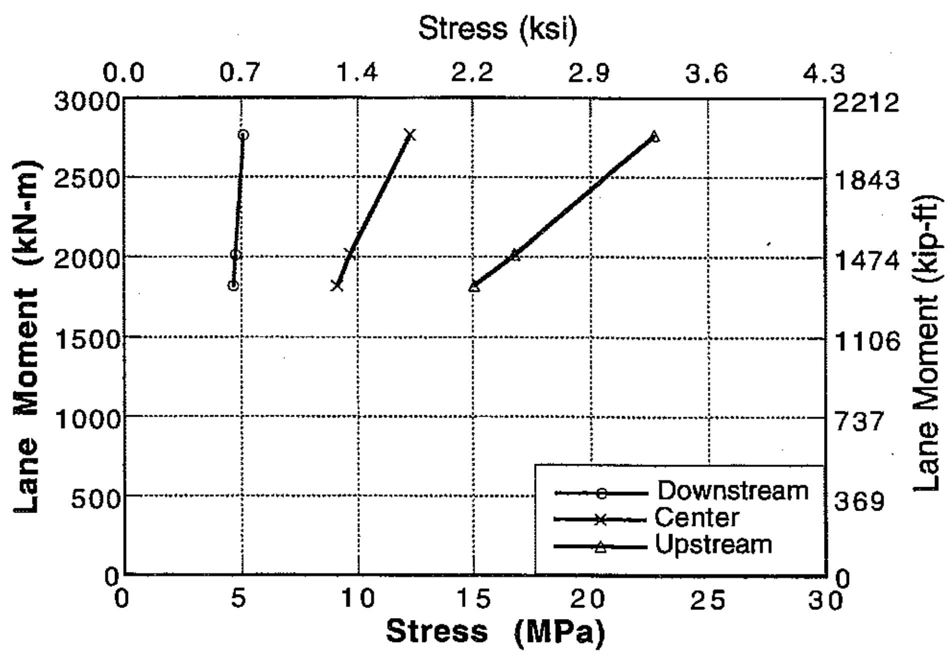


Figure 8-11 : Stress at Mid-Span of Girder 3 of Bridge No. 3 - North Span.

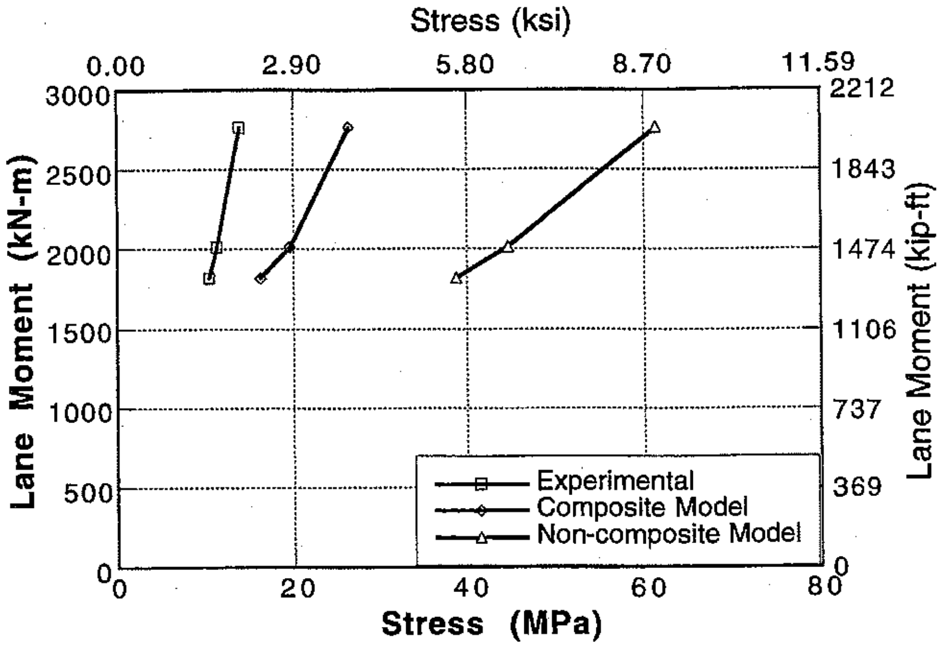


Figure 8-12 : Stress in Girder 3 for Upstream Loading on Bridge No. 3 - North Span.

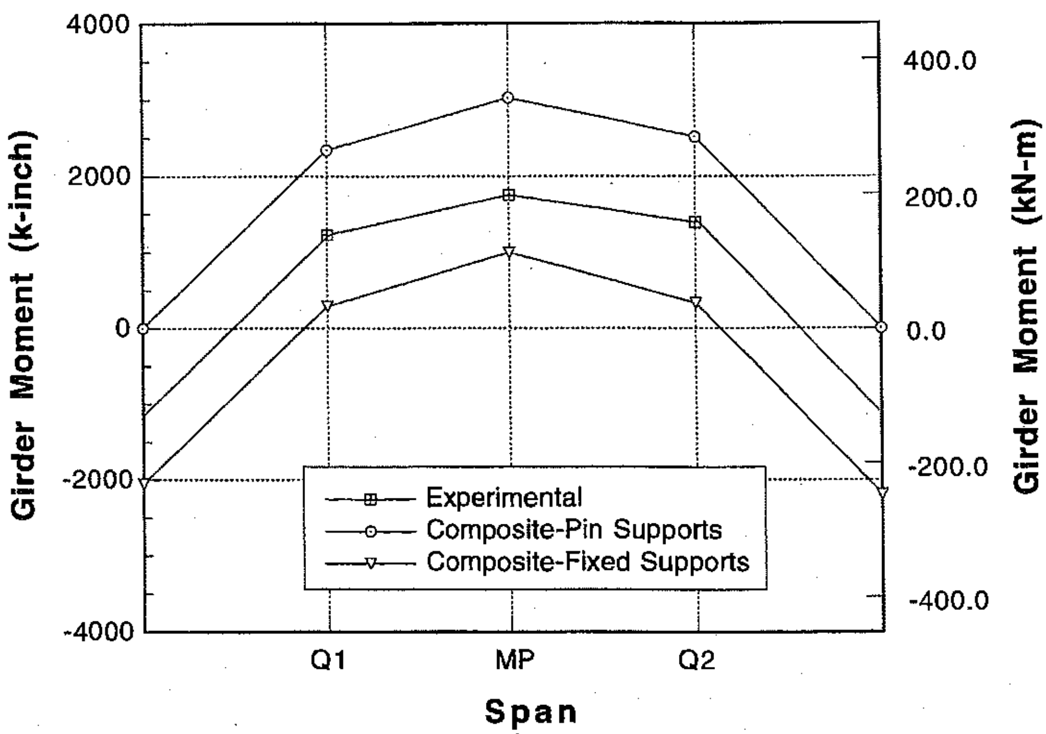


Figure 8-13 : Longitudinal Distribution of Stresses in Girder No. 5 due to the Center Loading on Bridge No. 3 - North Span.

Comparison of the measured and calculated stresses shows that the trends are similar to the previous bridge (Figure 8-12). Although this bridge was designed to behave as a non-composite structure, the actual results are closer to the values from the composite model than those from the non-composite model. Therefore, unintended composite action and the effect of non-structural members contribute to the flexural stiffness. It was observed that the moments at mid-span are reduced by some restraint at the bearings. Figure 8-13 shows the longitudinal moment distribution for girder no. 5 under center loading. It includes the experimental results and analytical results from the composite model with pin supports and the composite model with fixed supports. The experimental response lies between the pin and the fixed analytical models, confirming the presence of partial fixity at supports. The actual location of strain transducers was a few inches away from the midspan.

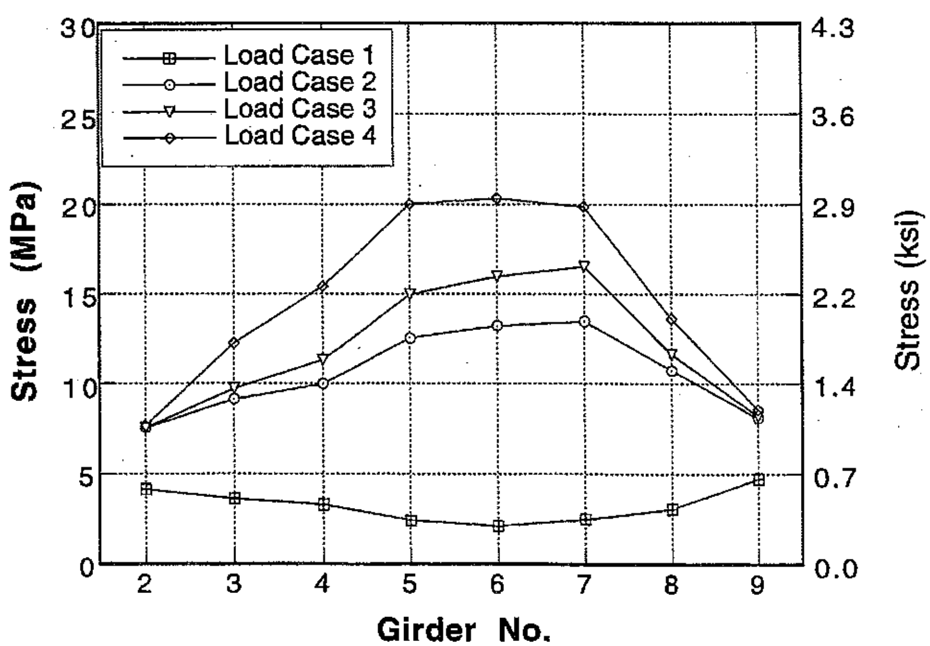


Figure 8-14 : Girder Distribution in terms of Stress due to Center Loading on Bridge No. 3 - North Span.

Lateral load distribution is shown in Figure 8-14 for center loading. The load distribution is much more uniform than expected. This again proves the effectiveness of new diaphragms. Due to the excellent load sharing, as well as low and linear stress response, this bridge was also considered to be safe. After the test, the operating rating factor for a two-unit 11-axle truck is 1.29. Additional graphs are shown in Appendix C.

8.6 Proof Load Test Results of South Span

Analytical results for this span were identical to the north span of the same bridge, since the original designs and the repair process for both spans were similar. However, during testing a higher proof load level of 3,000 kN-m (2,212 k-ft) was reached for this bridge. The maximum measured stress of 29.98 MPa (4.34 ksi) was recorded in girder no. 5 under the center load position. The maximum stress is about 0.15 of the yield strength of steel girders. Figure 8-15 shows the response of girder no. 3. The stress-moment response is linear, which indicates that the bridge is strong enough to support more load than the applied proof load. Stresses at the mid-span and quarter points of other girders also point to the same conclusion. Once again, the repair work in the lower flanges of this bridge appears to be effective.

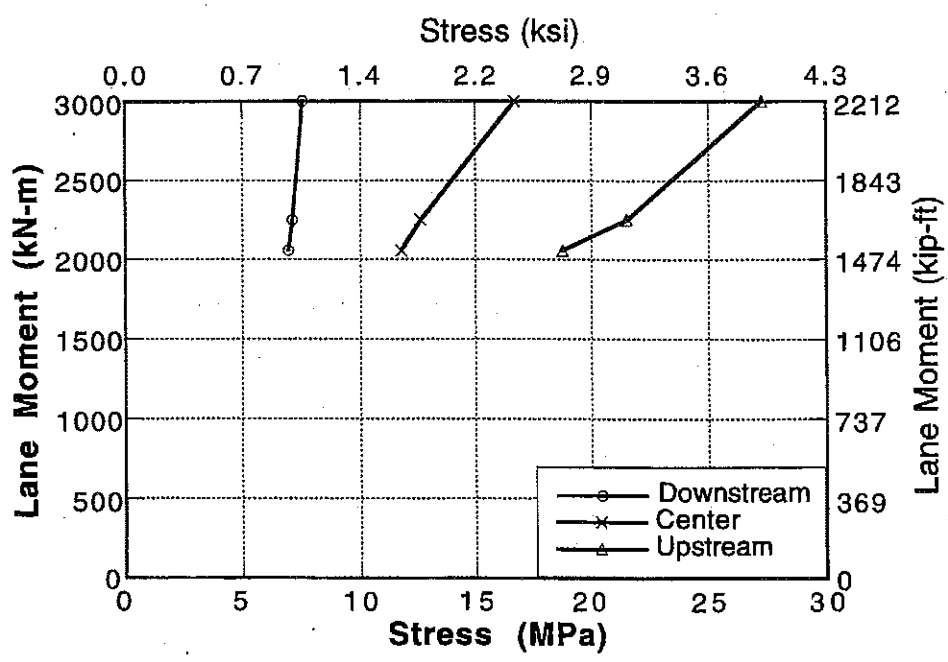


Figure 8-15 : Stress in Girder 3 on Bridge No. 3 - South Span.

Maximum expected stress from the non-composite model was about 65 MPa (9.42 ksi), which is almost twice the actual maximum

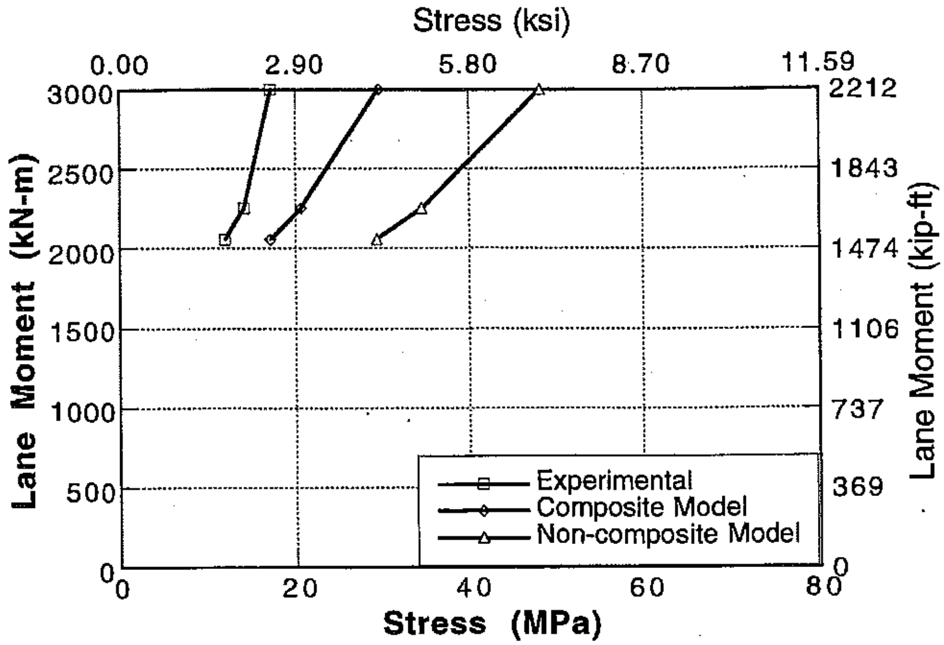


Figure 8-16 : Stress in Girder 6 for Downstream Loading on

Bridge No. 3 - South Span.

stress. This considerable difference is again due to the unintended composite action and the contribution of non-structural members such as parapets, railings and concrete facade, which are bolted to the exterior girder. As shown in Figure 8-16, for girder no. 6, the measured response is closer to the results of the composite model. However, the magnitude of the measured stresses is smaller than those from the composite analytical model. All girders at the mid-span and quarter points show the same behavior. As in the case of the other bridges, the partial restraint at the supports reduces the moment at the mid-span. Figure 8-17 shows the longitudinal moment distribution for girder no. 5 under downstream loading. It includes the experimental results and analytical results from the composite model with pin supports and the composite model with fixed supports. The experimental response lies between the pin and the fixed analytical models, confirming the presence of partial fixity at supports. The actual location of strain transducers was a few inches away from the midspan.

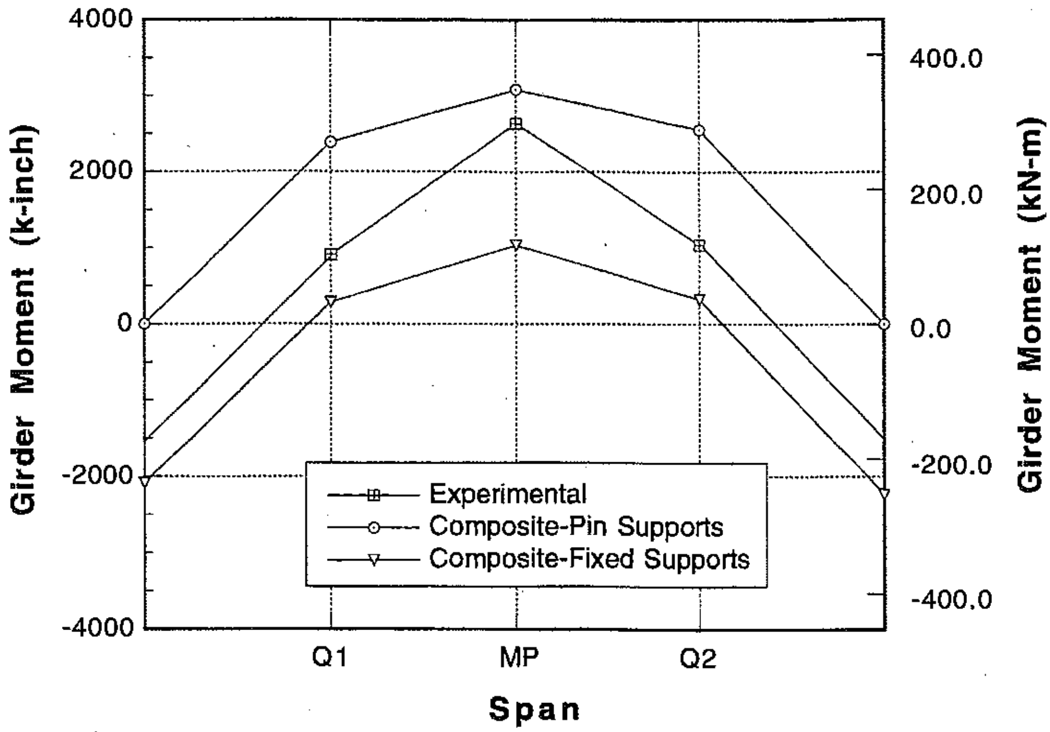


Figure 8-17 : Longitudinal Distribution of Stresses in Girder No. 5 due to the Center Loading on Bridge No. 3 - South Span.

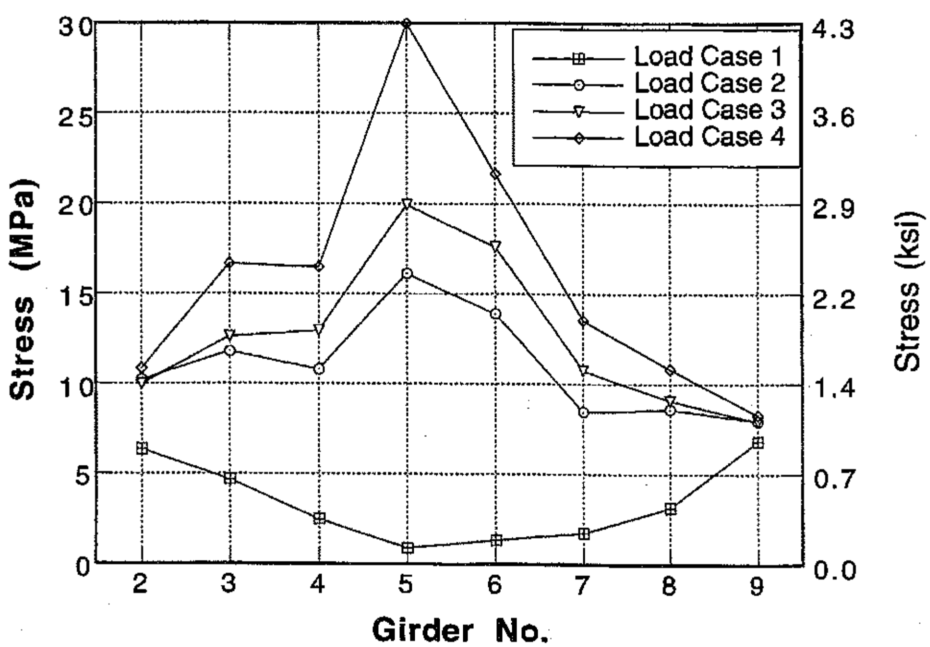


Figure 8-18 : Girder Distribution of Stress for Center Loading on Bridge No. 3 - South Span.

The lateral load distribution for the center load position is shown in Figure 8-18. In contrast to the previous bridge, the load distribution in this bridge is not uniform. The load on girder no. 5 is not transferred adequately to the adjacent girders. Later it was found that the diaphragms between girder 4 and 5 and between 5 and 6 were left unrepaired by mistake. These diaphragms were replaced after the tests and the bridge was then opened for normal traffic. The operating rating factor was found to be 1.40 using Equation 3-6. Additional graphs and processed data are listed in Appendix C.

Note:

Intentionally left blank

9. BRIDGE NO. 4 (B01 of 19062, M-21 over Little Maple River)

9.1 Description

This bridge is located in Clinton County, Michigan, close to Ovid. It carries state highway M-21 over Little Maple River. The side elevation of the bridge is shown in Figure 9-1. It was built in 1929 and has ten steel girders with a 165 mm (6.5 in) thick concrete slab. The thicknesses of the concrete wearing surface and the asphalt are 152 mm (6 in) and 76 mm (3 in), respectively. The width of the structure is about 22.2 m (74 ft) and the span length is 11.7 m (38.5 ft). The ADT on this bridge is about 5,100. The cross-section and the girder details are shown in Figure 9-2.

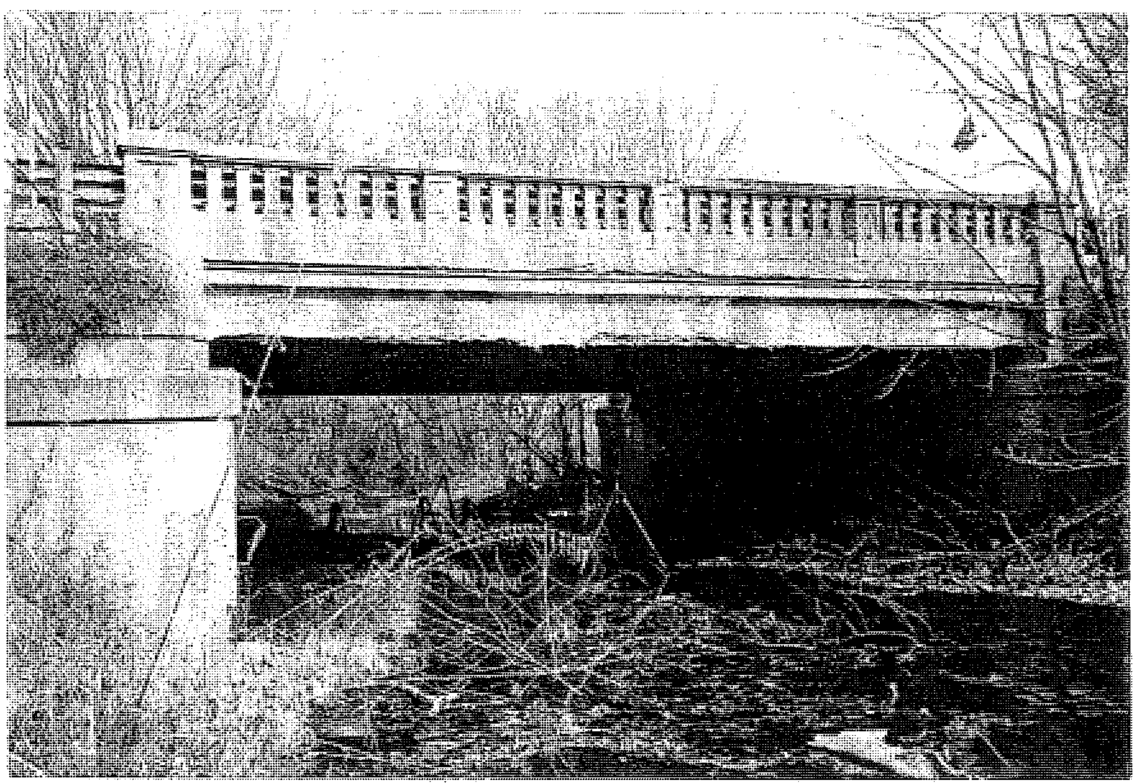
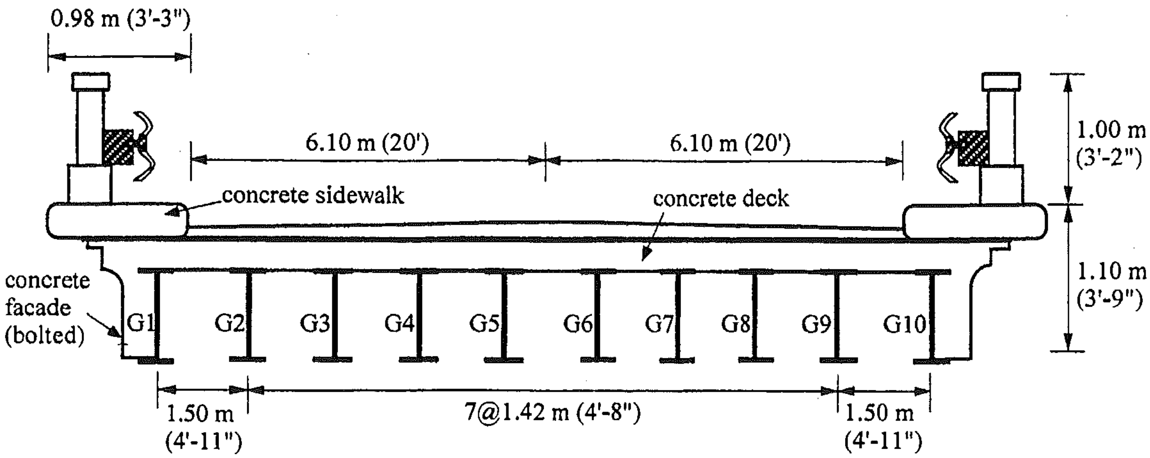
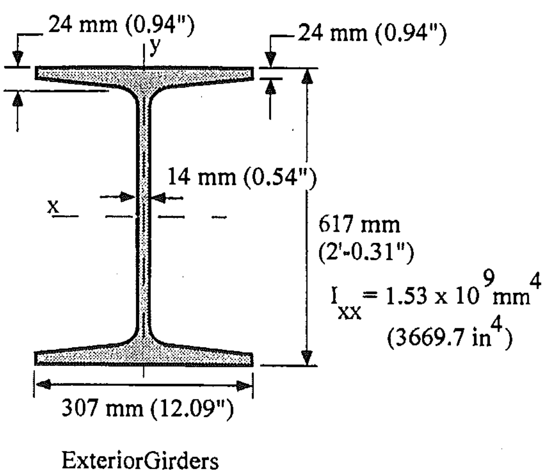


Figure 9-1 : Side Elevation of Bridge No. 4.





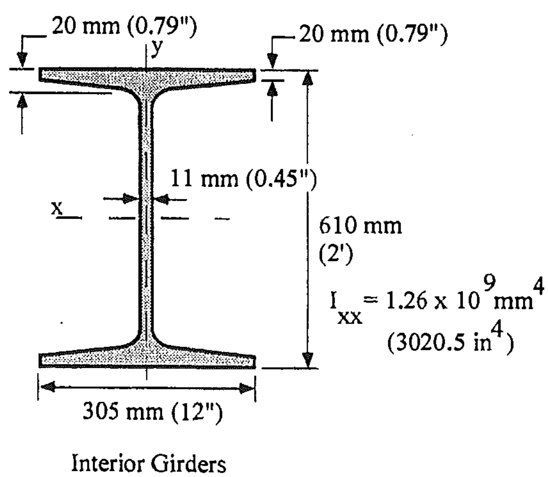


Figure 9-2 : Cross-Section of Bridge No. 4.

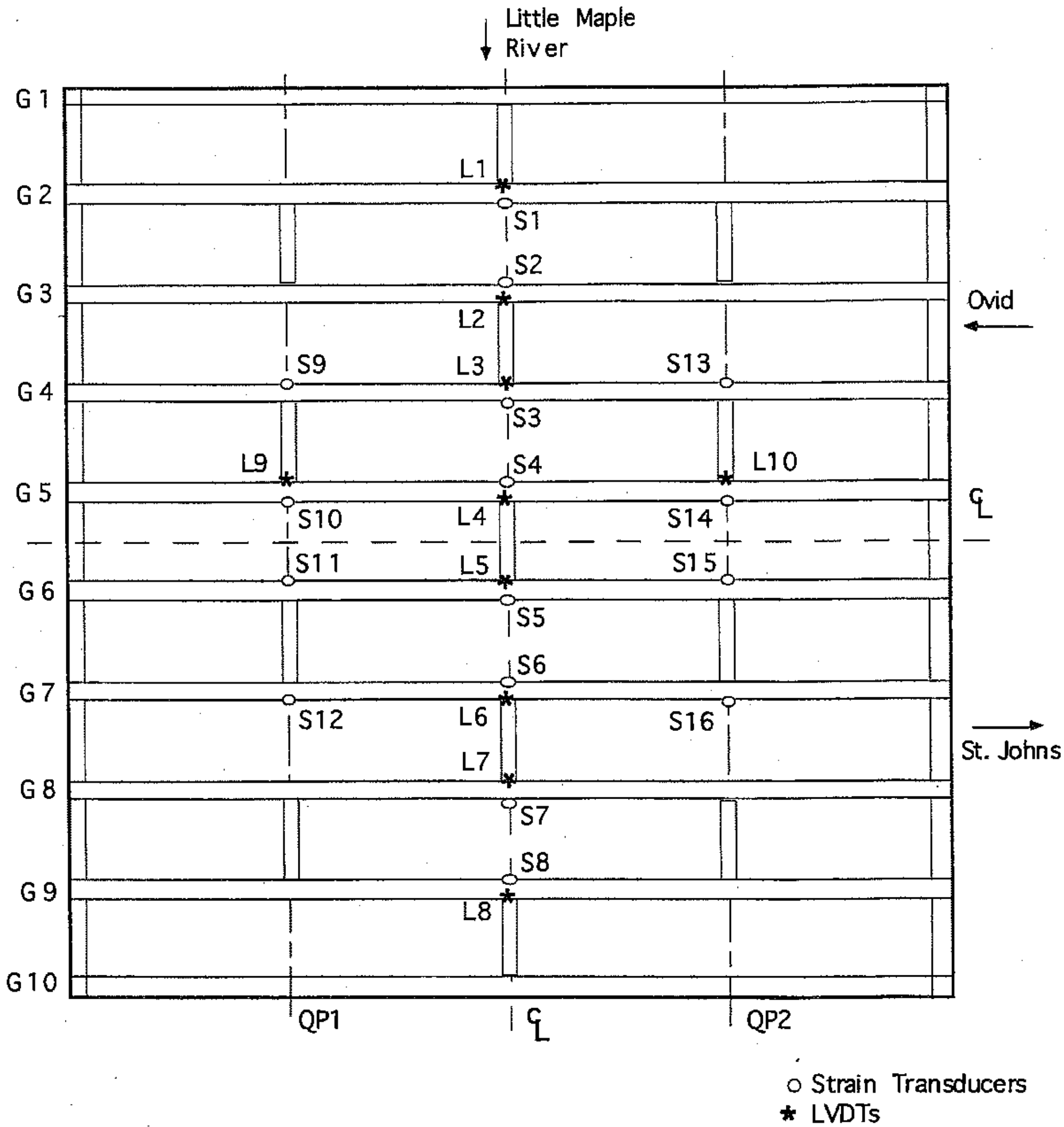
9.2 Pre-Test Inspection

In comparison with the first three bridges, this bridge had little deterioration. Only two interior girders were heavily corroded due to the seepage of water. The slab and the diaphragms close to these girders were also deteriorated. Other girders and the deck were in relatively good condition. Both abutments, parapets and top of the deck were also in fair condition. The critical limit state for this bridge was the mid-span moment.

Based on the Michigan Bridge analysis Guide (1983), the design compressive strength of concrete was taken to be 17.3 MPa (2.5 ksi), and 207 MPa (30 ksi) was used as the yield stress of steel. The remaining capacity to carry live load and impact corresponding to the operating rating was determined to be 1,103 kN-m (813 k-ft) per lane by MDOT. The pre-test rating calculations performed by MDOT indicated an inventory rating factor of 1.06 for H15 load and a operating rating factor of 0.98 for two-unit 11-axle truck. The pre-test analytical results from non-composite and composite models are shown in the form of graphs and compared to the experimental results (Appendix D).

9.3 Instrumentation

All interior girders of the bridge were instrumented at the mid-span using LVDT's and strain transducers. The exterior girders were not instrumented, since the response was expected to be very small due to the presence of the bolted concrete facade. In addition, for selected girders the instruments were mounted at the quarter points. In total, sixteen strain transducers and ten LVDT's were used. The instrument layout for this bridge is shown in Figure 8-3.



MP : Mid Point.

QP1 : Quarter Point 1.

QP2 : Quarter Point 2.

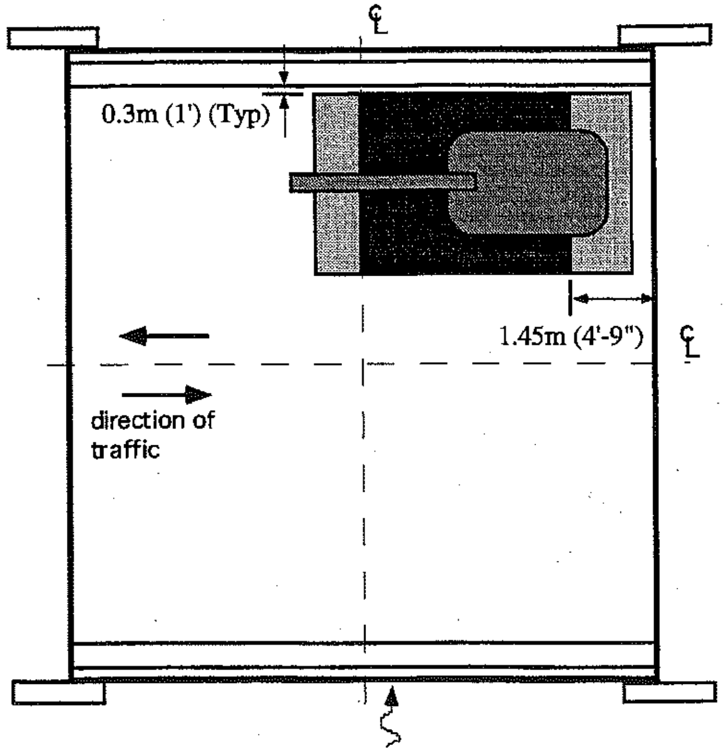
Figure 9-3 : Instrumentation Layout for Bridge No. 4.

9.4 Proof Load Positions

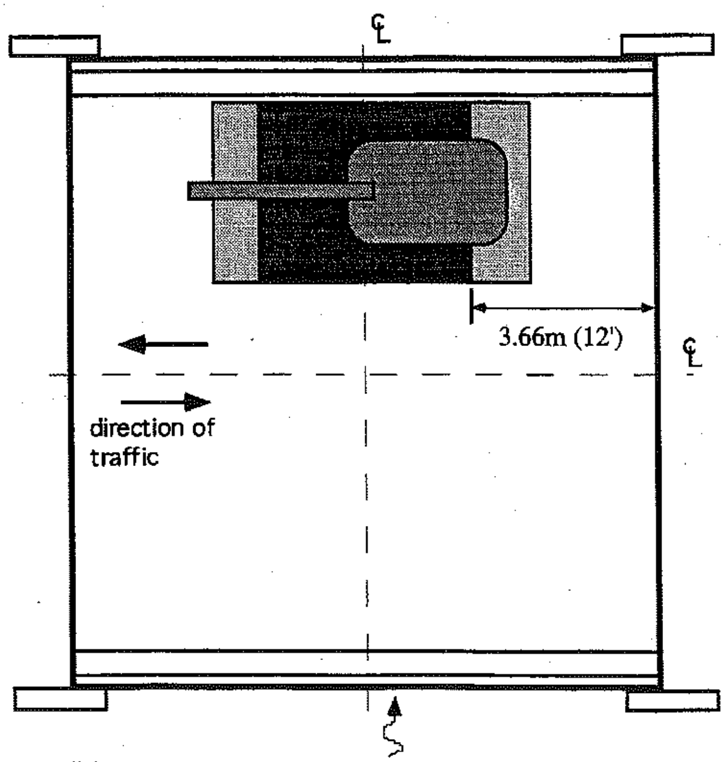
The target proof load level was achieved in three steps by using two M-60 tanks, and placing them directly on the pavement. Three different load positions were used in the transverse direction as well. The load positions are shown in Figures 9-4 to 9-6. The lane moments resulting from these load positions are shown in Table 9-1. The target proof load moment was 1,464.9 kN-m (1,080 k-ft). The traffic was allowed over partial width during the test, and it was fully stopped only at critical times during maximum load placement, i.e. Load Position 3. The maximum applied proof load moment at the mid-span was 1,330 kN-m (980 k-ft). However, a proof load moment of 1,397 kN-m (1,030 k-ft) was achieved at 0.76 m (2.5 ft) away from mid-span during the load position no. 3.

Table 9-1 : Applied Proof Load Lane Moments.

Load Position. No	Applied Lane Moment in kN-m (kip-ft)		
	Quarter Point 1	Mid Point	Quarter Point 2
1	895 (660)	935 (690)	465 (345)
2	730 (540)	1,200 (885)	735 (540)
3	1,095 (805)	1,330 (980)	1,030 (760)

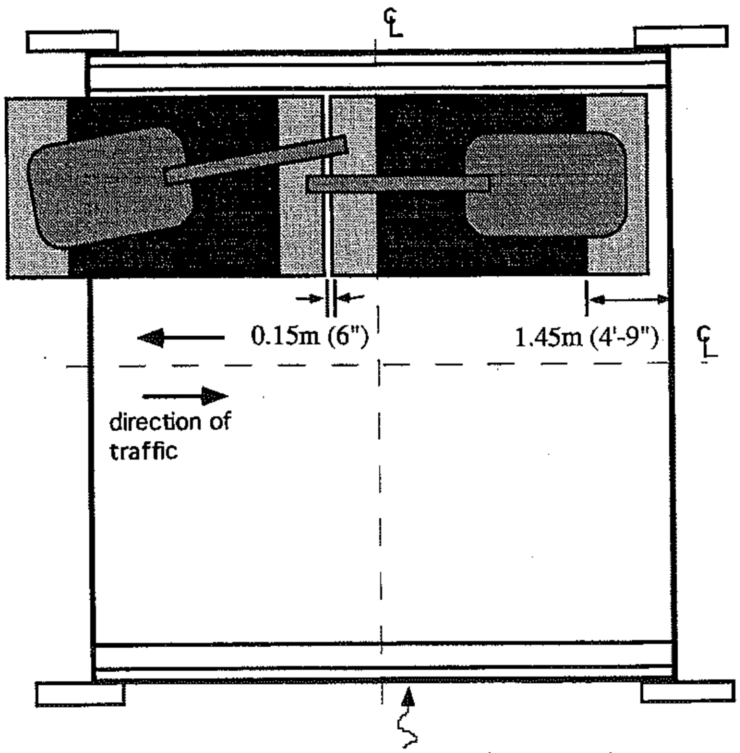


(a) Load Position 1 - Downstream Case.



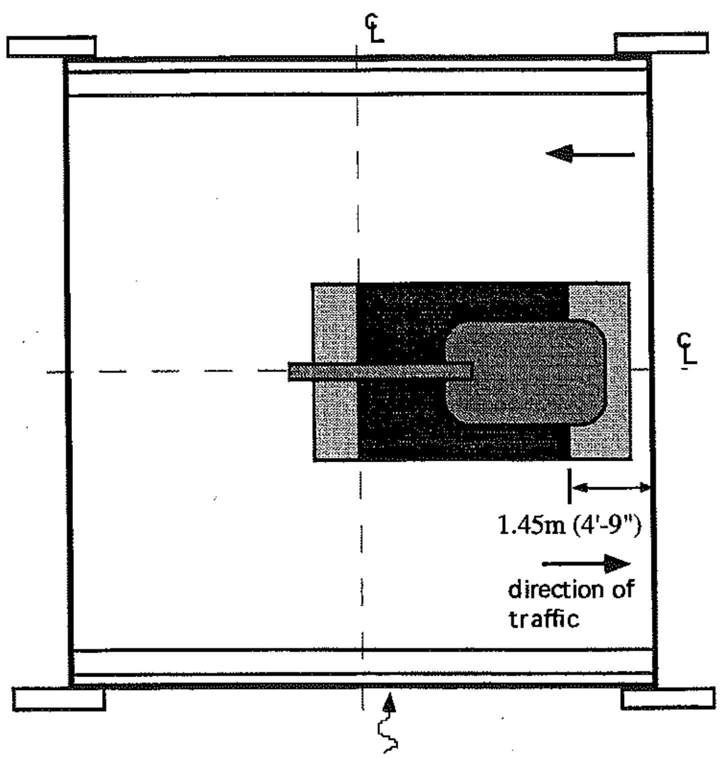
(b) Load Position 2 - Downstream Case.

Figure 9-4 : Longitudinal Load Positions for Downstream Case.



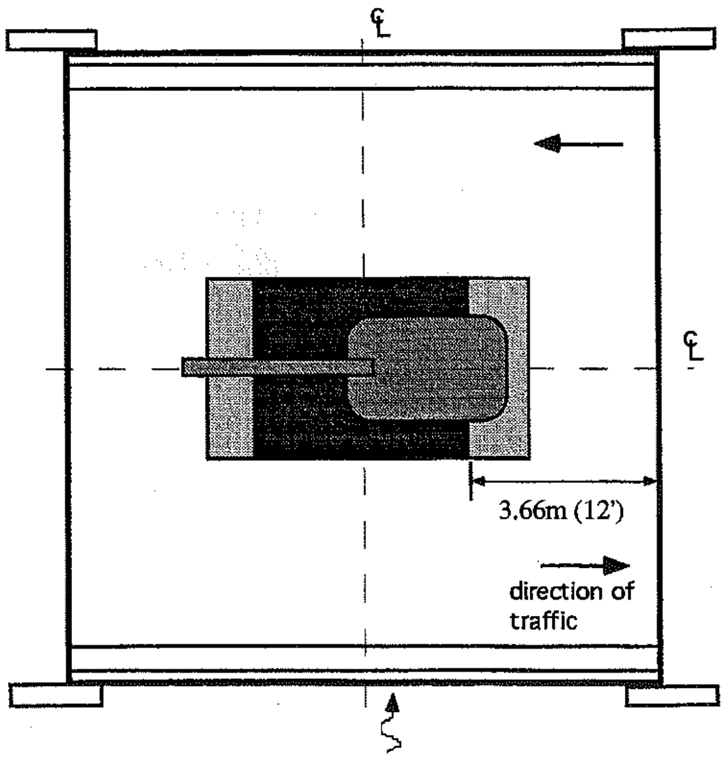
(c) Load Position 3 - Downstream Case.

Figure 9-4 : Longitudinal Load Positions for Downstream Case.(cont'd)

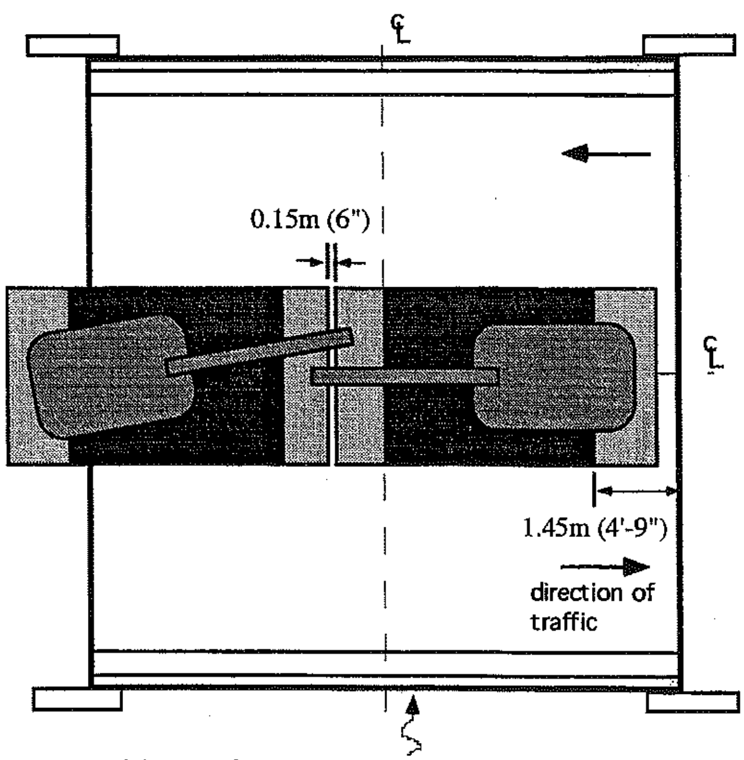


(a) Load Position 1 - Center Case.

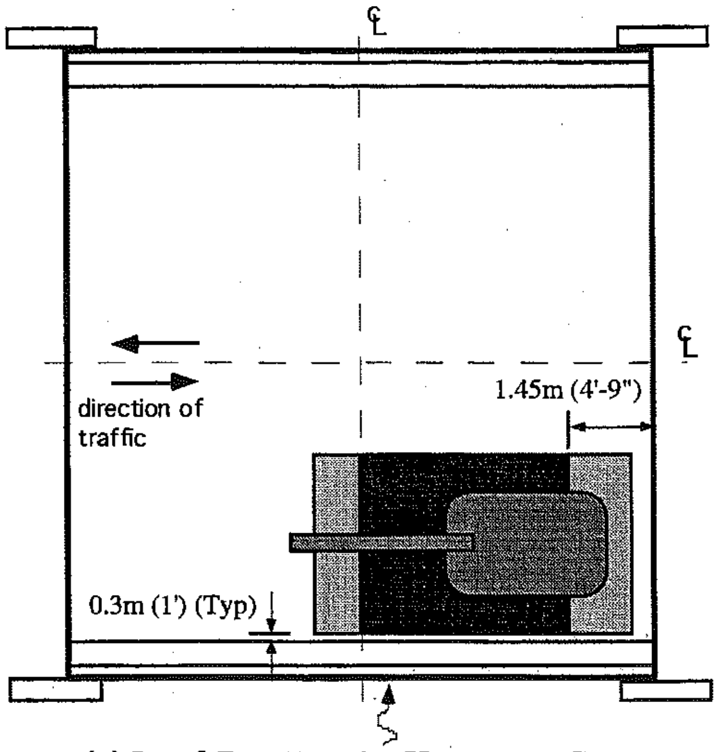
Figure 9-5 : Longitudinal Load Positions for Center Case.



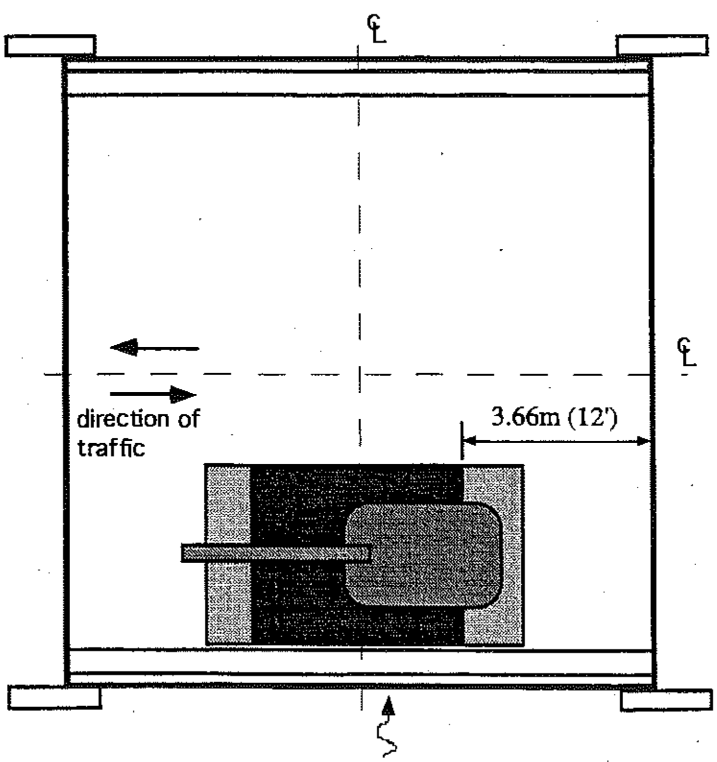
(b) Load Position 2 - Center Case.



(c) Load Position 3 - Center Case.
Figure 9-5 : Longitudinal Load Positions for Center Case. (cont'd)

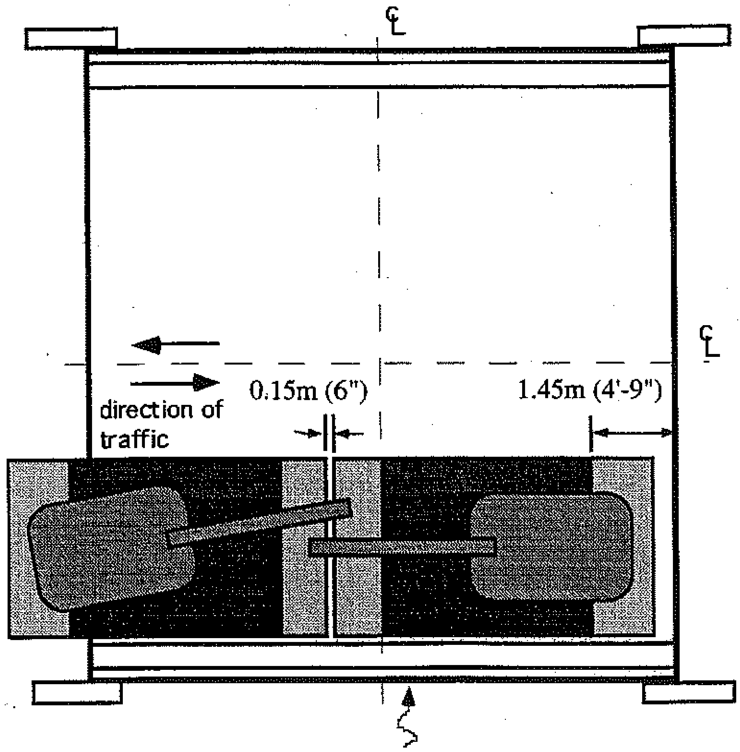


(a) Load Position 1 - Upstream Case.



(b) Load Position 2 - Upstream Case.

Figure 9-6 : Longitudinal Load Positions for Upstream Case.



(c) Load Position 3 - Upstream Case.

Figure 9-6 : Longitudinal Load Positions for Upstream Case. (cont'd)

9.5 Proof Load Test Results

A proof load moment of 1,330 kN-m (980 k-ft) was reached in three steps without any inelastic (nonlinear) behavior, which is over 2.43 times the HS20 moment and 1.53 times the two-unit 11-axle moment. The maximum measured deflection of 2.36 mm (0.094 in) was measured in girder no. 6 for center loading. As shown in Figure 9-7, the deflection increased nearly linearly with the applied lane moment.

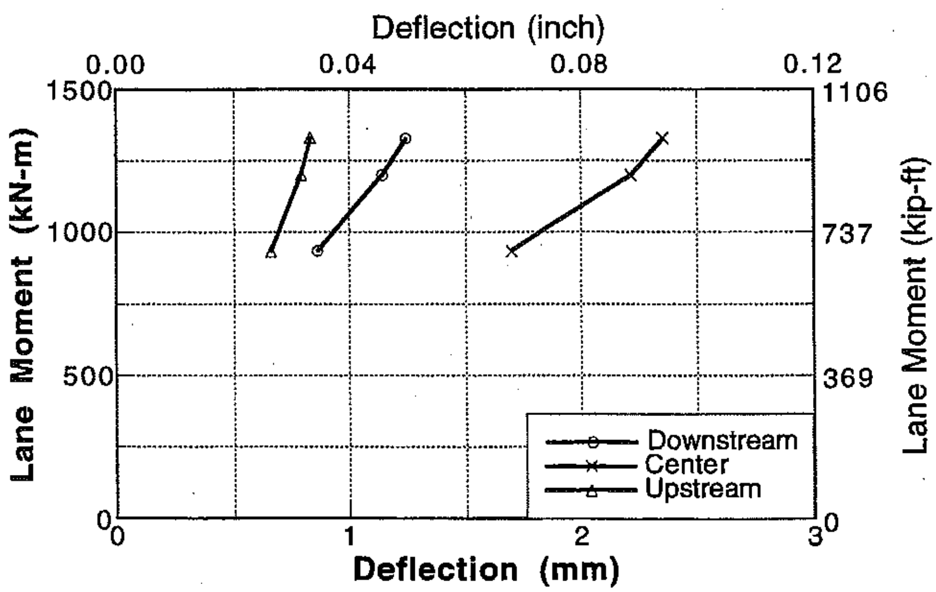


Figure 9-7 : Deflections at Mid-Span of Girder 6 of Bridge No. 4.

The maximum analytical deflection was about 13.5 mm (0.53 in) for the non-composite model and 3.5 mm (0.14 in) for the composite model. Figure 9-8 shows the comparison of analytical and measured deflections at the mid-span of girder 5 for center loading. The measured deflections are significantly smaller than the analytical deflections using the non-composite model. Even the composite model deflections were larger than those from proof load testing. This clearly indicates that the structure behaves as a composite section with flexural stiffness increased by the effect of non-structural members. The additional decrease in deflection is due to the rotational restraint provided by supports. The actual flexural stiffness of the structure is much higher than the analytically predicted values. The magnitudes of deflections at quarter points were even smaller. Deflections at other locations are shown in Appendix D. They also show the similar behavior.

The lateral load distribution among girders is shown in Figure 9-9, in terms of deflections at the mid-span for downstream loading.

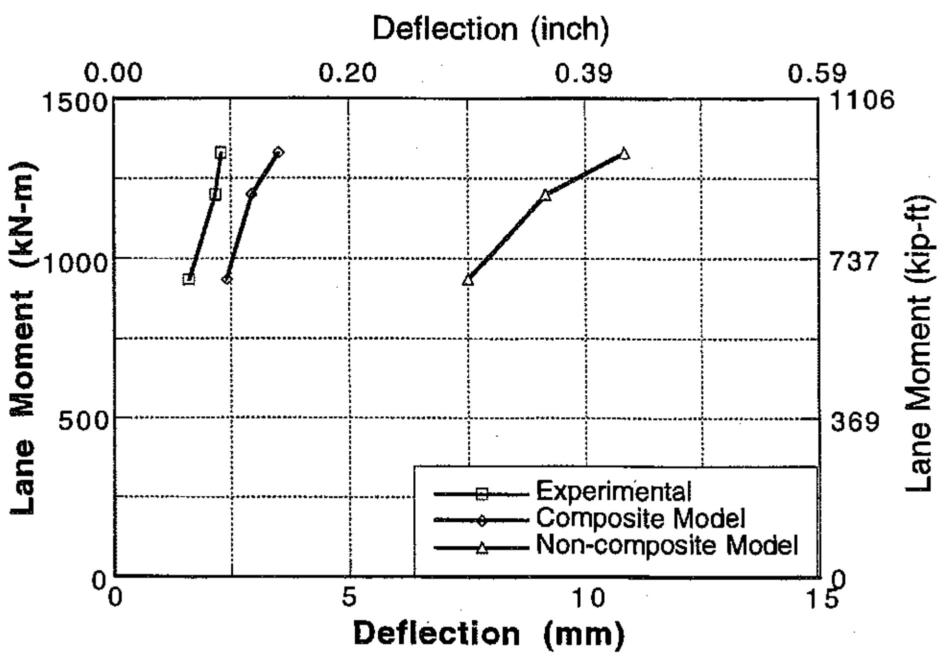


Figure 9-8 : Deflections at Mid-Span of Girder 5 for Center Loading on Bridge No. 4.

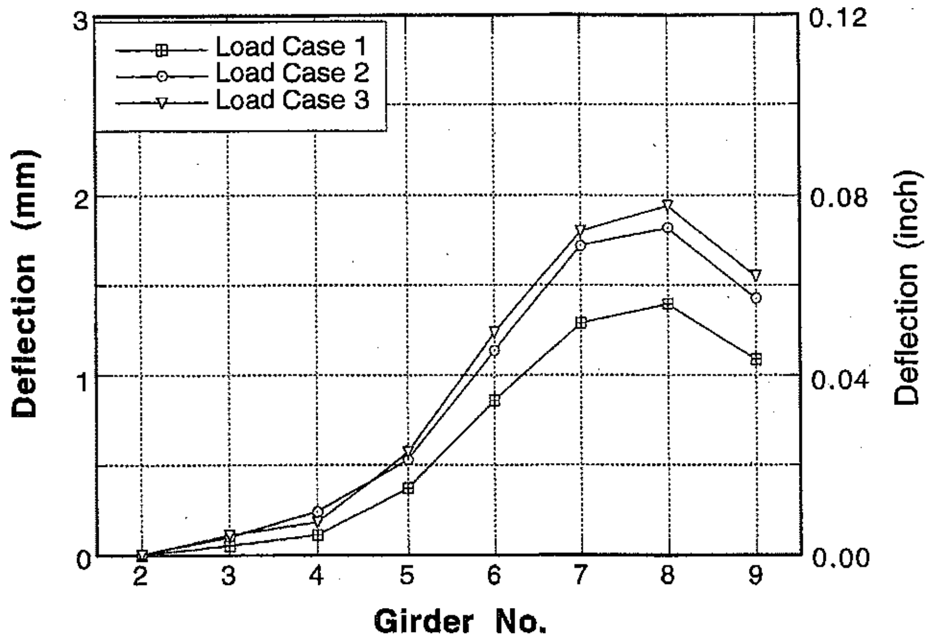


Figure 9-9 : Girder Distribution of Deflections due to Downstream Loading on Bridge No. 4.

Figure 9-10 shows the measured stress and the applied lane moment at the mid-span of girder 6. The maximum measured stress was only 15.9 MPa (2.3 ksi), which is much smaller than the yield strength of 207 MPa (30 ksi). The stress for load case 3 in center loading decreased while the applied lane moment increased. One tank was enough to produce moments for load case 1 and 2. For load case 3, the first tank was kept at the same position as in load case 2, while a second tank was placed close to the east support to increase the lane moment. Observed stresses should have increased if the supports were behaving as pin connections. A decrease in stress due to the placement of additional load close to the support was observed because the supports provide some moment resistance. This decrease in stress due to bearing restraint disappears for the girders which are further away from the applied load. Therefore, the measured stress for downstream and upstream loading did not show a decrease for load case 3.

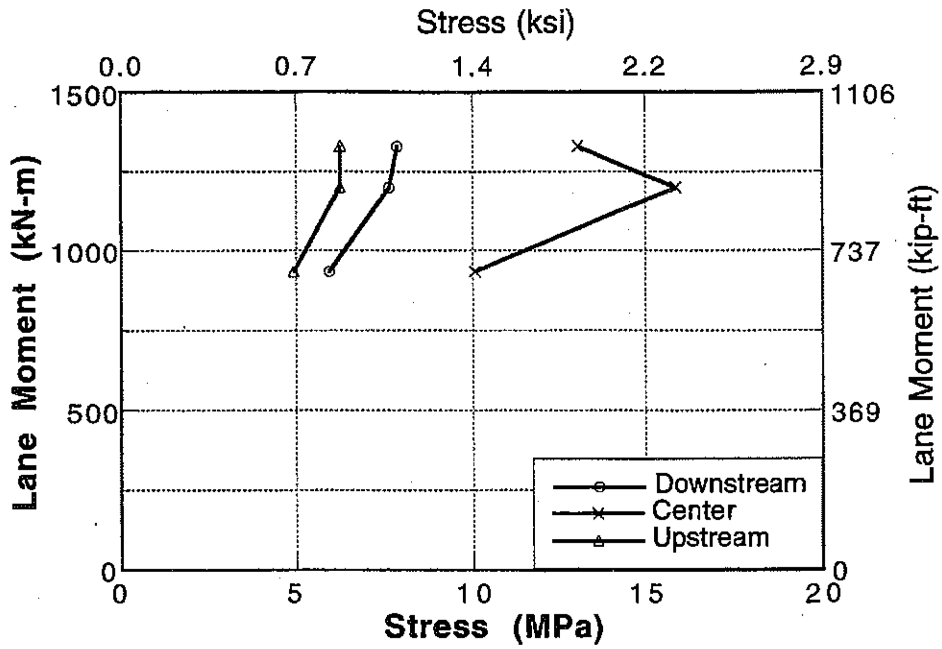


Figure 9-10 : Stresses at Mid-Span of Girder 6 of Bridge No. 4

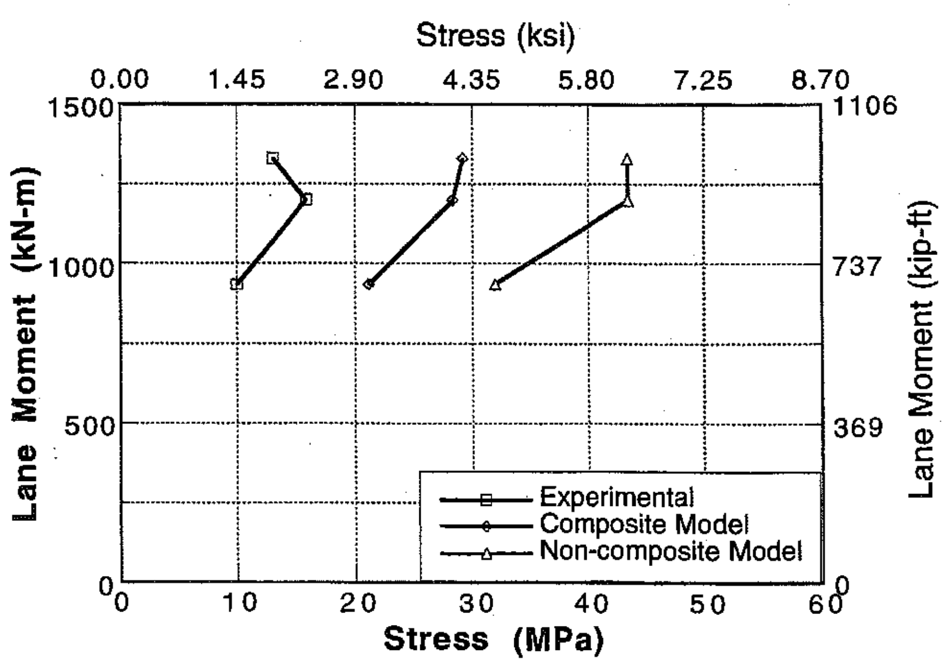


Figure 9-11 : Stresses at Mid-span of Girder 6 for Center Loading on Bridge No. 4.

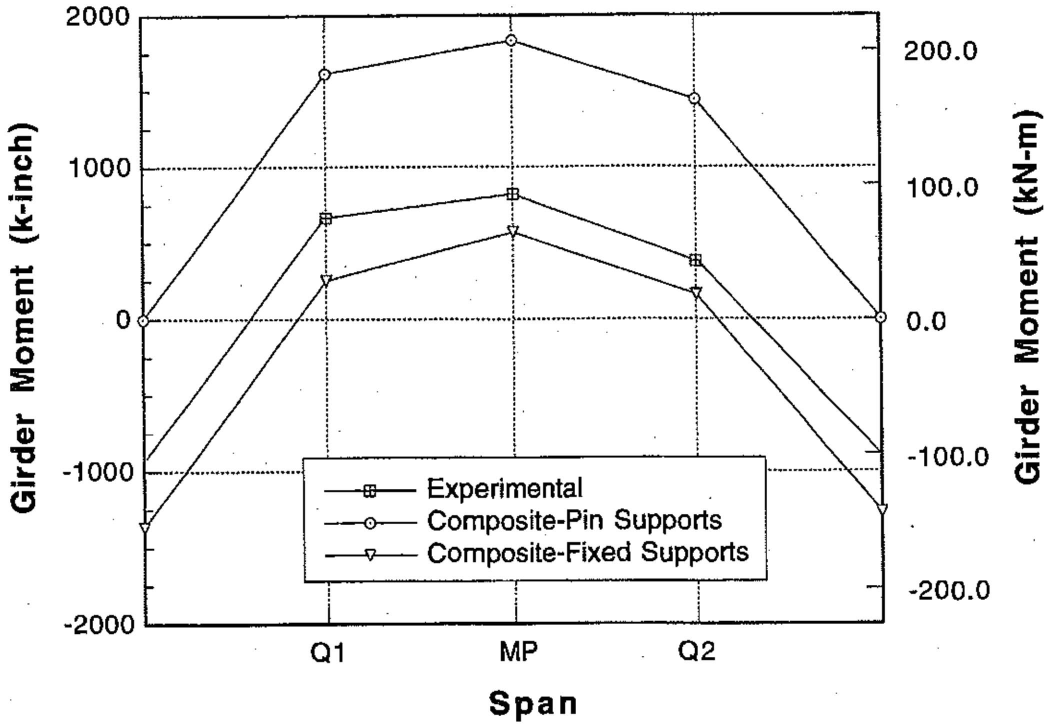


Figure 9-12 : Longitudinal Distribution of Stresses in Girder No. 6 due to the Center Loading on Bridge No. 4.

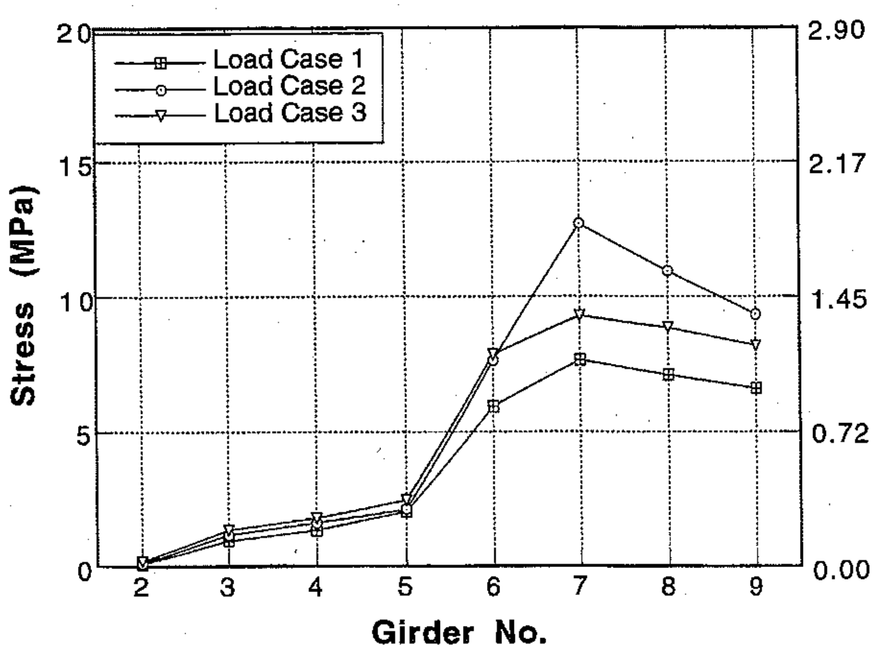


Figure 9-13 : Girder Distribution of Stresses due to Downstream Loading on Bridge No. 4.

Comparison of measured and analytical stresses at the mid-span of girder no. 6 for center loading is shown in Figure 9-11. The measured stresses are smaller than both the non-composite and composite model stresses. The analytical model shows a small increase in stress due to the increased lane moment for load case 3, while the measured stress shows a significant reduction. This reduction of stress for increased load is expected for partially fixed supports. Figure 9-12 shows the longitudinal moment distribution for girder no. 5 under center loading. It includes the experimental results and analytical results from the composite model with pin supports and the composite model with fixed supports. The experimental response lies between the pin and the fixed analytical models, confirming the presence of partial fixity at supports. The girder distribution of stress for downstream loading is shown in Figure 9-13. The measured stress in other girders also shows a similar behavior. The results are listed in Appendix D. Although the structure did not show any sign of distress, the actual maximum moment applied during the test was slightly smaller than the target moment, due to the unavailability of the extra load. Hence, the factor K_0 in Equation (3-5) was considered to be 1.0. Based on Equation 3-6, the operating rating factor for a two-unit 11-axle truck was only 0.95. However, if the composite action observed during the proof load test is incorporated in the original rating calculations, then the operating rating factor would be 2.31, according to the Michigan Bridge Analysis Guide (1983). Therefore, the bridge should be considered safe for the legal truck traffic.

10. BRIDGE NO. 5 (B01 of 82081, M-153 over Fellows Creek)

10.1 Description

This is a single span structure over Fellows Creek, located in Canton Township, Michigan. It carries five lanes of state highway M-153 (Ford Road), with a total ADT of 32,400. The original structure was built in 1920, which consisted of nine reinforced concrete T-beams, with a 150 mm (6 in) thick slab and a 150 mm (6 in) thick concrete wearing surface. It was widened in 1979, and 300 mm (12 in) precast concrete slabs were added on each side. The span length of the current structure is 7.8 m (25.5 ft) and the width is 24.4 m (80.2 ft). The side elevation of the bridge is shown in Figure 10-1. The cross-section of the interior structure with RC beams is shown in Figure 10-2.

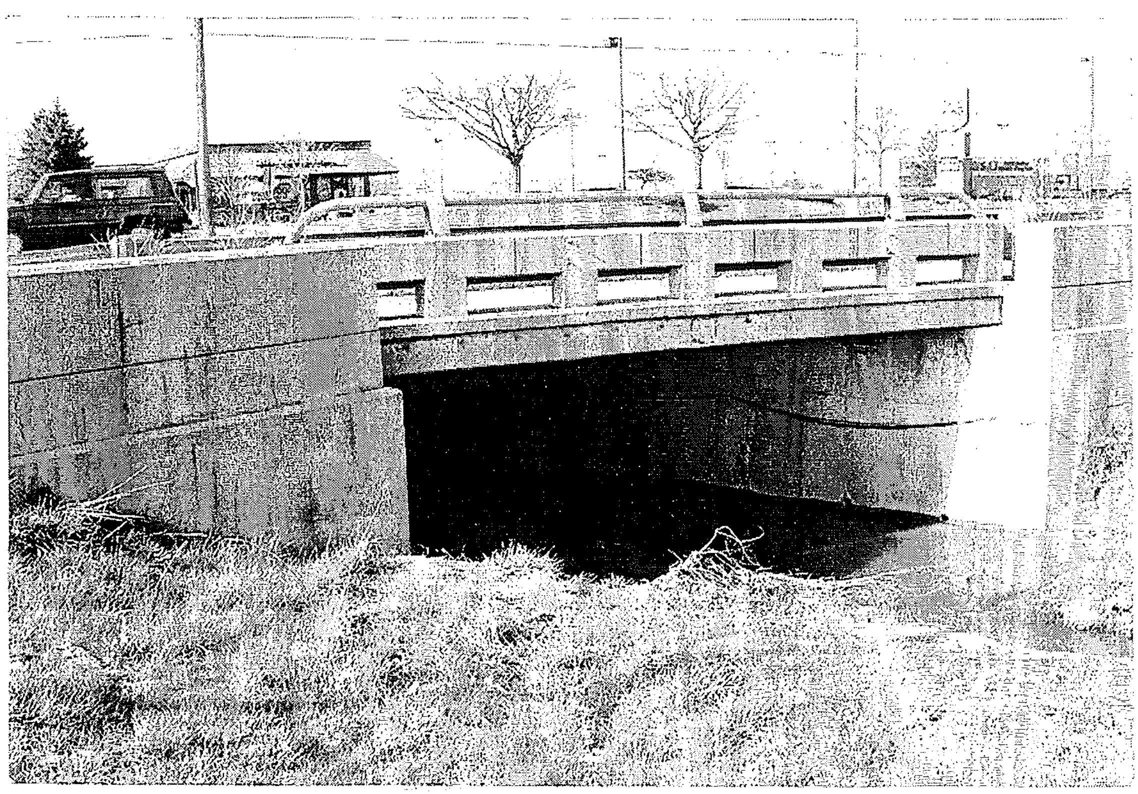


Figure 10-1 : Side Elevation of Bridge No. 5.

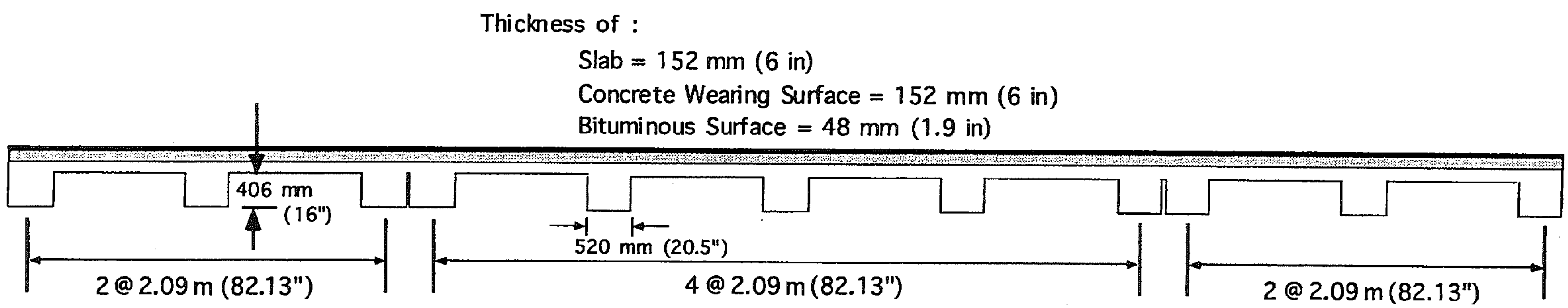


Figure 10-2 : Cross-Section of Bridge No. 5 - Older Part.

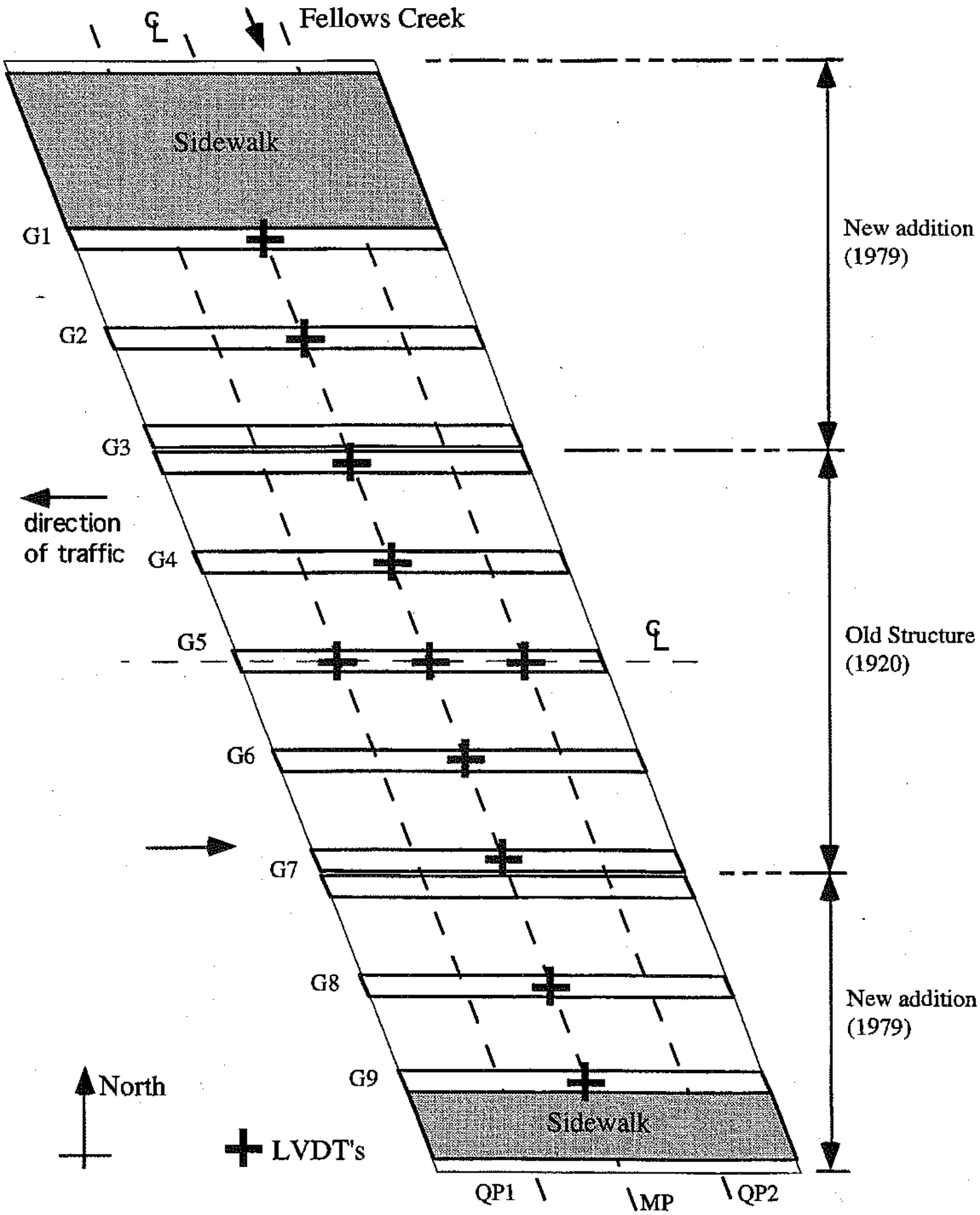
10.2 Pre-Test Inspection

During the site inspection prior to the test, several longitudinal and transverse cracks were observed in the older reinforced concrete beams, that were built in 1920. Spalling of the concrete was also noticed at several locations. The newly added precast members were in good condition. Several leach stains were also observed on the old structure. The objective of the proof load test was to check the adequacy of the older part of the structure to carry legal truck traffic.

According to the Michigan Bridge Analysis Guide (1983), the compressive strength of concrete was taken to be 14.0 MPa (2.0 ksi). The pre-test rating calculations by MDOT show an inventory rating factor of 1.24 for H15 truck and an operating rating factor of 1.06 for two-unit 11-axle truck. The operating level remaining capacity to carry live load and impact was 575.1 kN-m (424 k-ft). The pre-test analytical results using SECAN (see Section 5) are shown in the form of graphs and compared to the experimental results (Section 10.5 and Appendix E).

10.3 Instrumentation

Based on the experience during the test of Bridge No. 1, it was decided not to measure the strains in concrete girders. Therefore, the interior structure with reinforced concrete beams was instrumented using LVDT's only. The deflection was measured at the mid-span of all nine girders. In addition, for girder no. 5, the deflections at the quarter points were also measured. The instrumentation layout is shown in Figure 10-3.



MP : Mid Point.

QP1 : Quarter Point 1.

QP2 : Quarter Point 2.

Figure 10-3 : Instrumentation Layout for Bridge No. 5.

10.4 Proof Load Positions

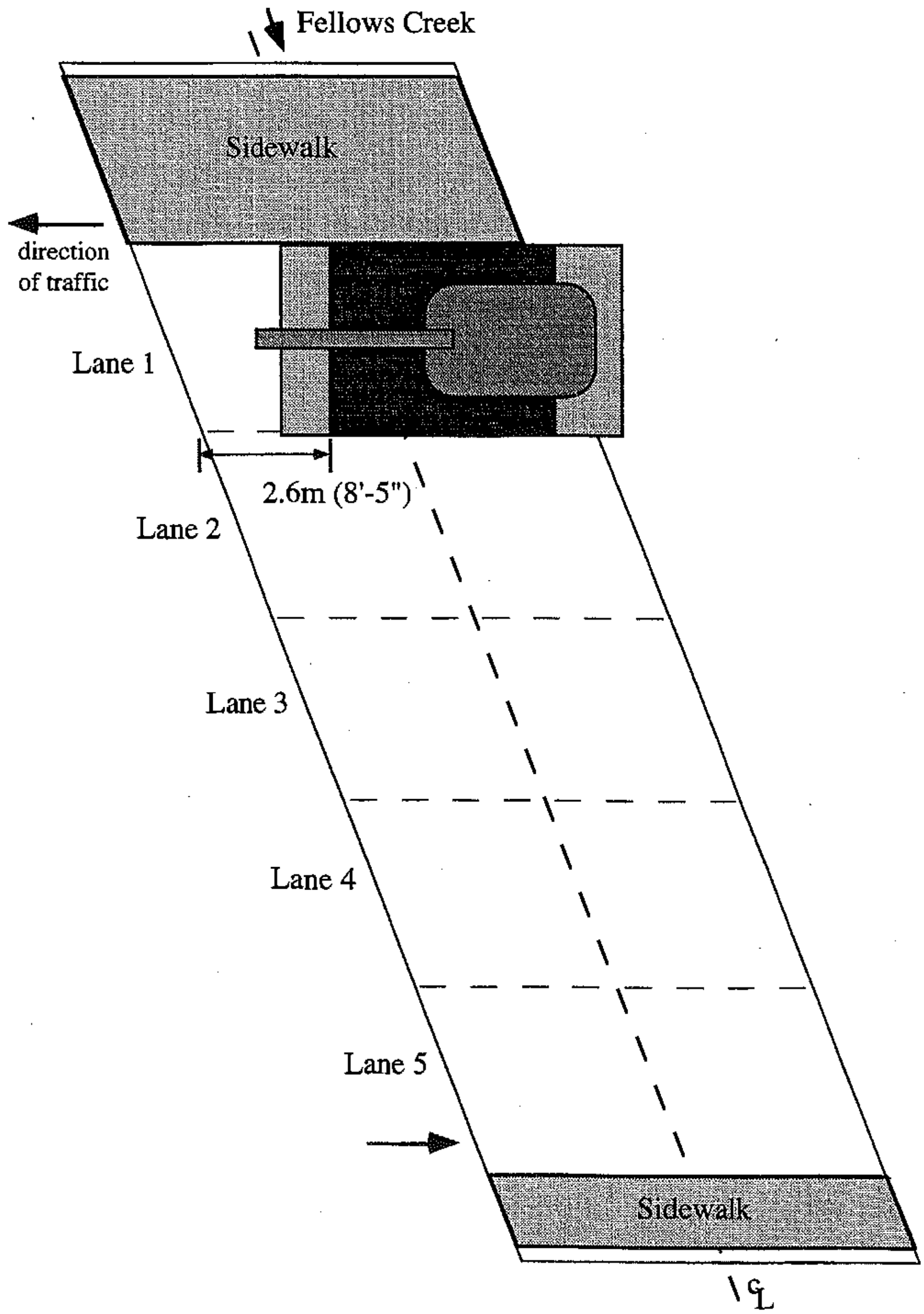
Two M-60 tanks were used as proof load. The span length of the bridge was not enough to fit two tanks in one lane. Therefore, for higher load levels the tanks were placed adjacent to each other. Three different longitudinal and four different transverse positions were used. For this bridge, the tanks were placed directly on the pavement. Figures 10-4 to 10-7 show the longitudinal load position for each transverse load position. The resulting mid-span moment at the mid-span and quarter points are listed in Table 10-1. The target proof load moment was 610.4 kN-m (450 k-ft). The Load Position No. 3 corresponds to two tanks placed in adjacent lanes, however, the applied moments shown in Table 10-1 are on a per lane basis. Therefore, values corresponding to Load Position No. 2 and 3 are the same. The traffic was allowed over partial width during the test, and it was fully stopped only at critical times during maximum load placement, i.e. Load Position 3.

Table 10-1 : Applied Proof Load Lane Moments.

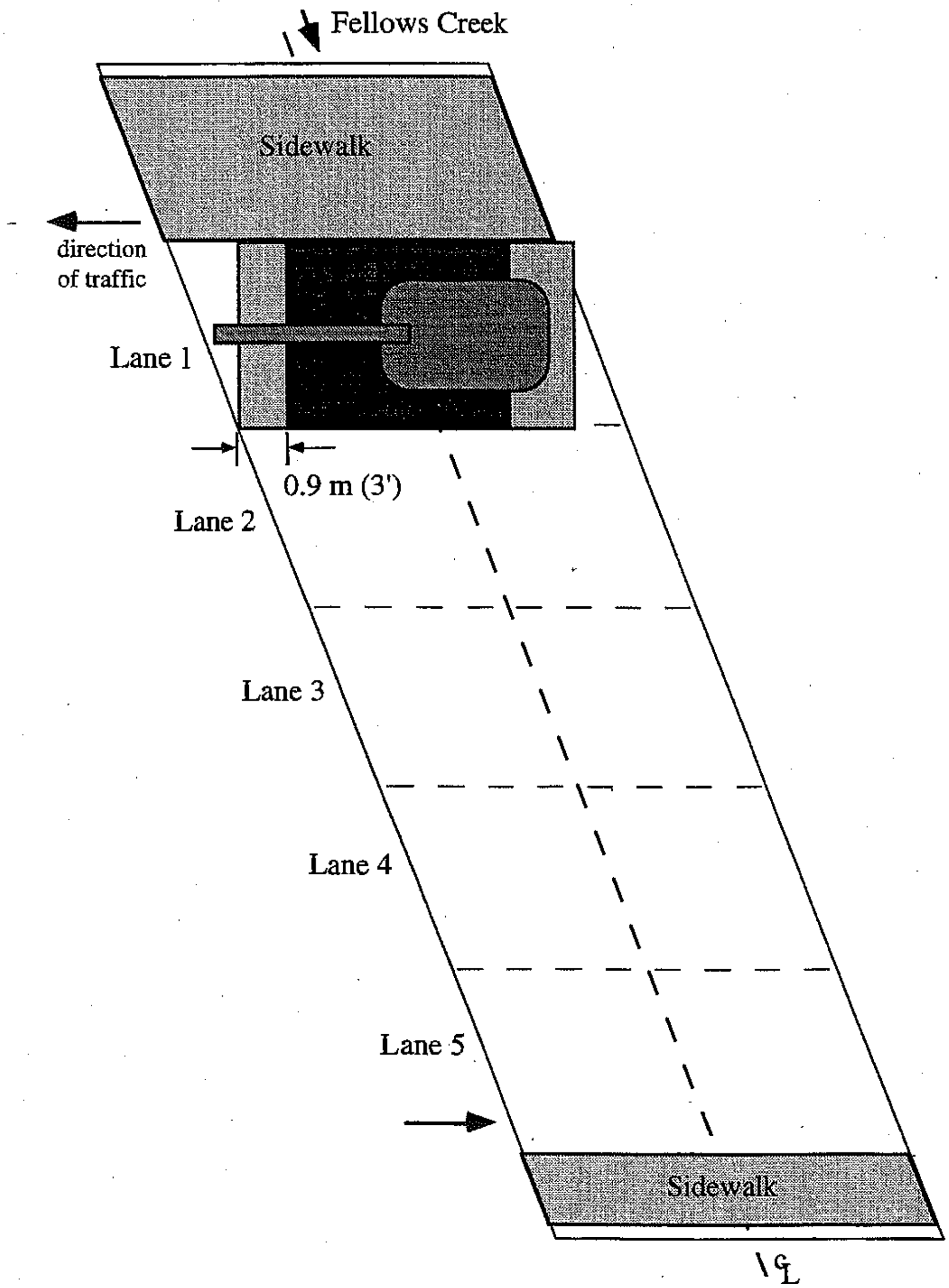
Load Position No.	Applied Lane Moment in kN-m (kip-ft)		
	Quarter Point 1	Mid Point	Quarter Point 2
1	270 (199)	525 (385)	470 (350)
2*	470 (350)	685 (500)	470 (350)
3**	470 (350)	685 (500)	470 (350)

* one lane loading
** two lane loading

For Bridge No. 5, the maximum load was obtained with two tanks placed side-by-side. For other tested bridges, the tanks were placed in one lane at a time. Then, the relationship between the applied load moment and deflection was plotted. For Bridge No. 5, the deflection is also plotted as a function of applied load moment, but, in order to be consistent with the terminology used in previous sections, an equivalent 'moment per lane' was calculated for the second tank. It was assumed that this additional moment (due to the second tank) is proportional to the increase in deflection (as compared to the case with one tank).

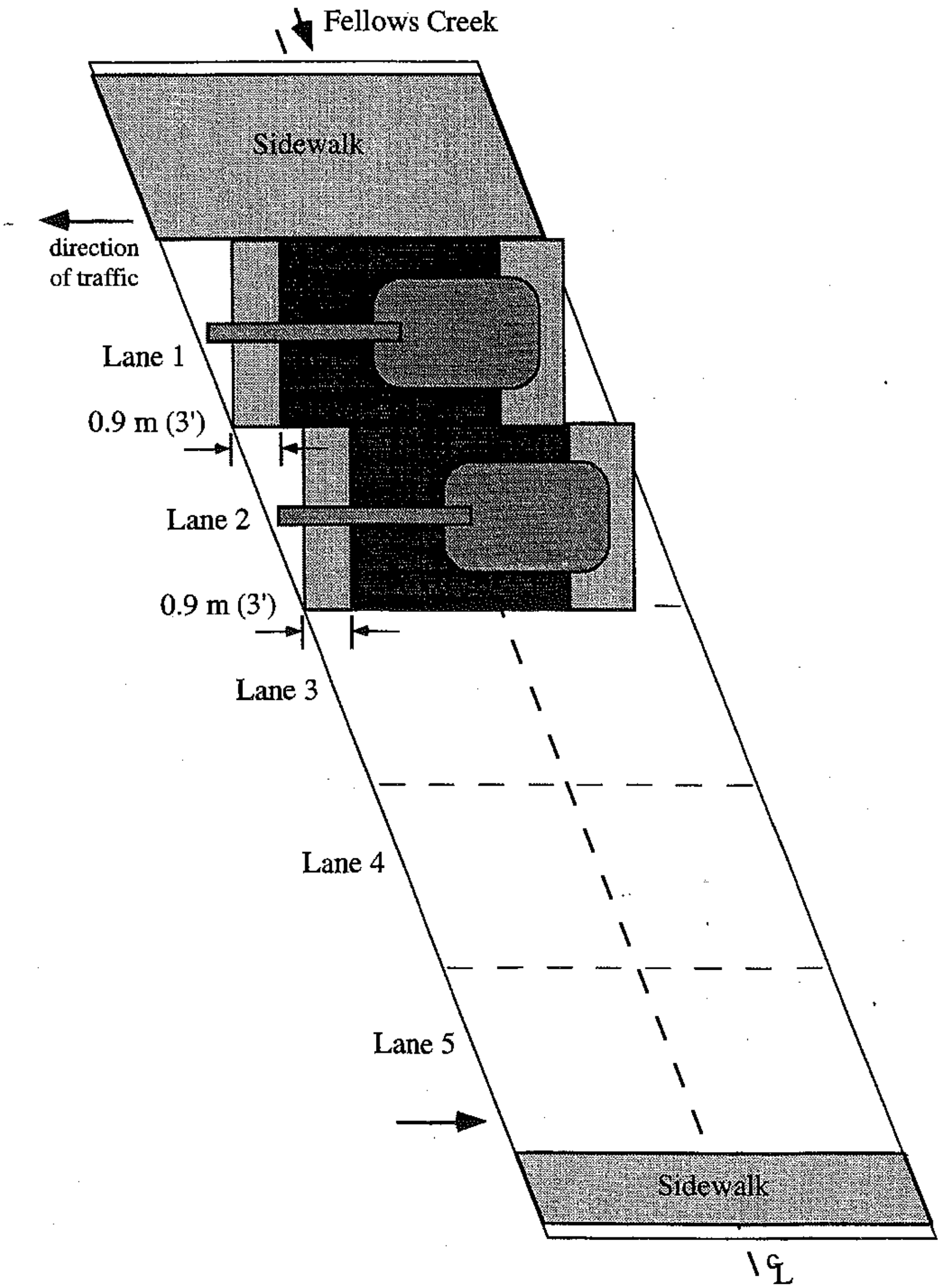


(a) Load Position 1 - Lane 1 and 2 Loading.
Figure 10-4 : Longitudinal Load Positions for Lane 1 and 2 Test.



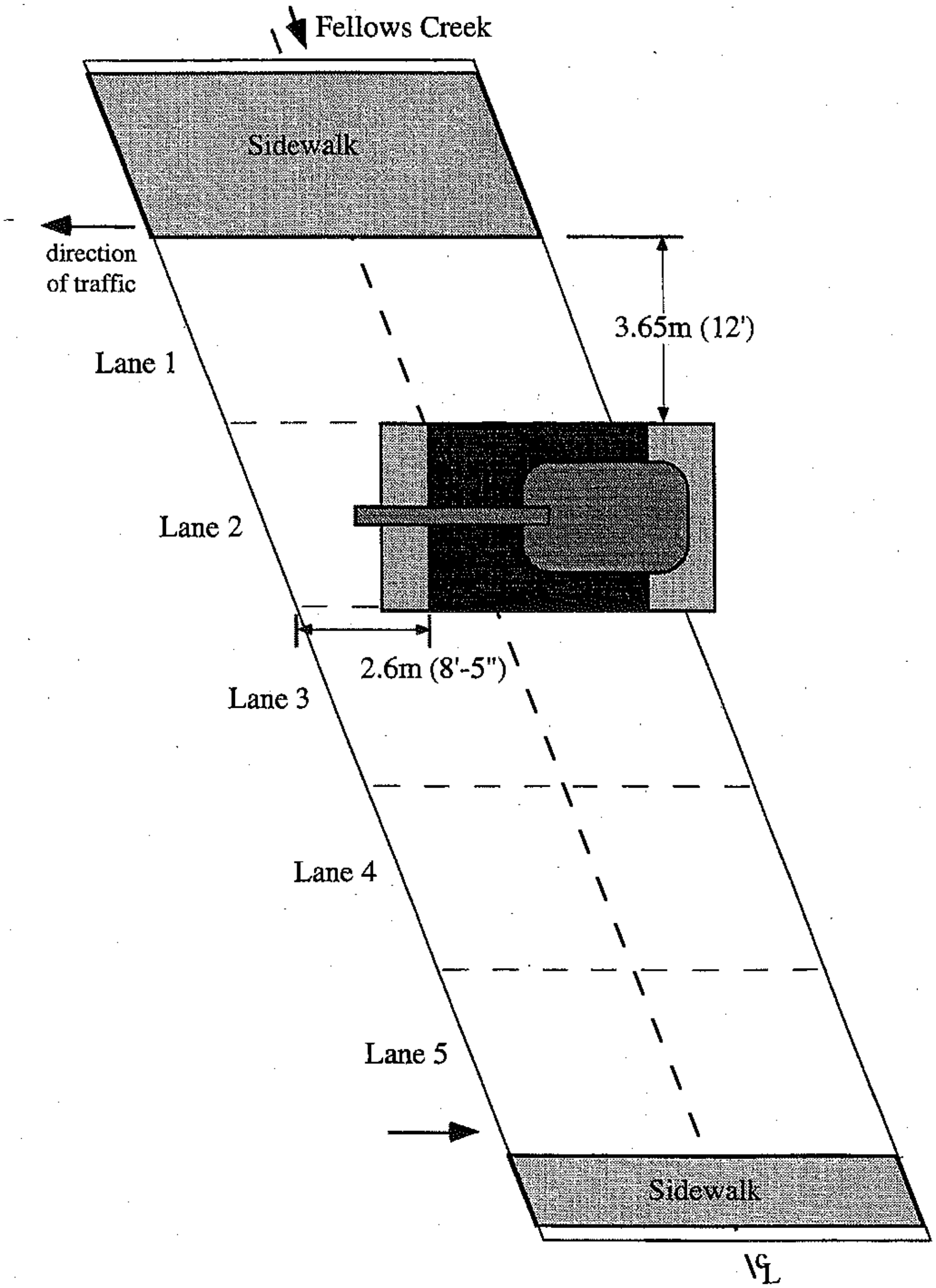
(b) Load Position 2 - Lane 1 and 2 Loading.

Figure 10-4 : Longitudinal Load Positions for Lane 1 and 2 Test. (cont'd)



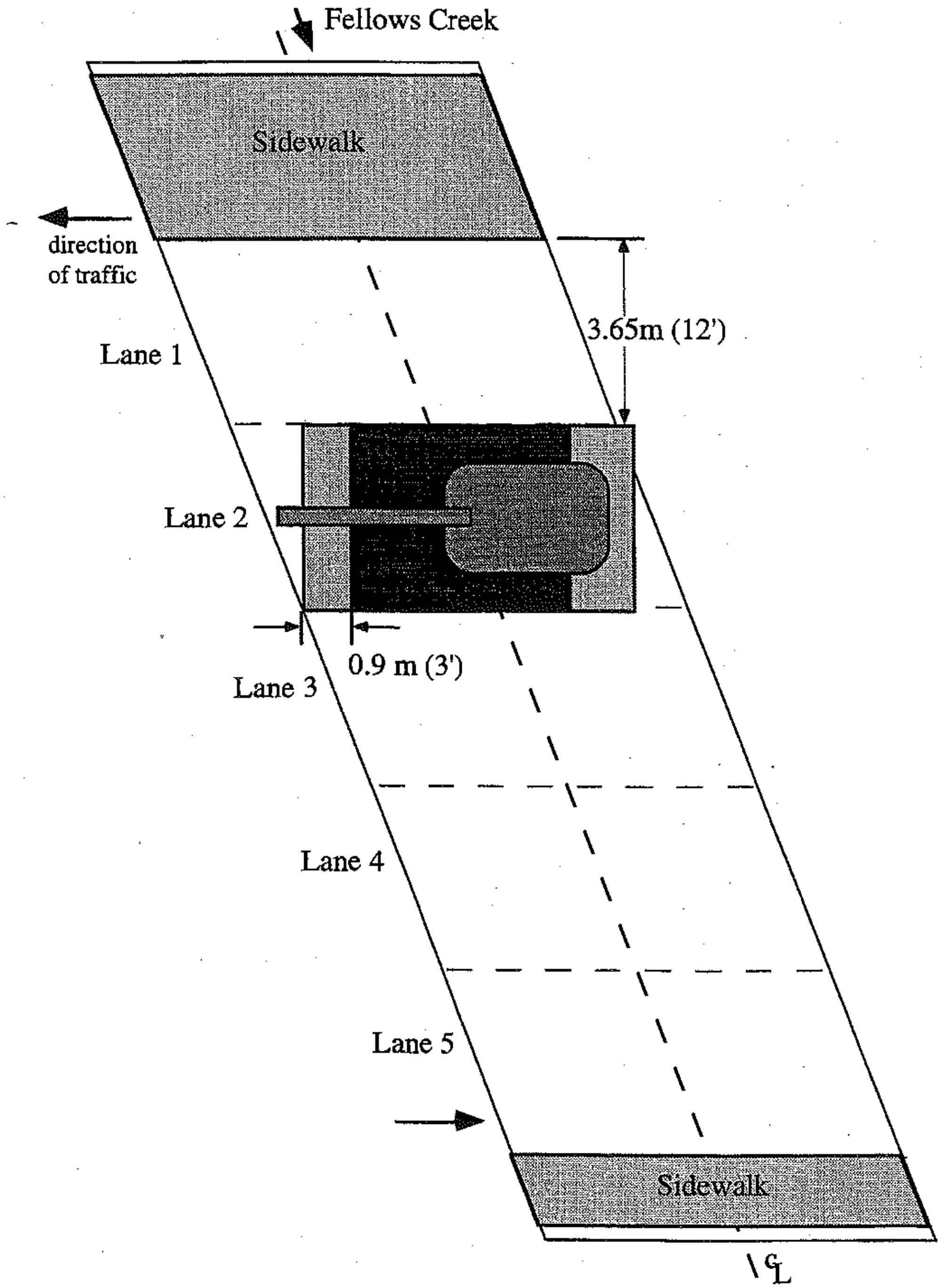
(c) Load Position 3 - Lane 1 and 2 Loading.

Figure 10-4 : Longitudinal Load Positions for Lane 1 and 2 Test. (cont'd)



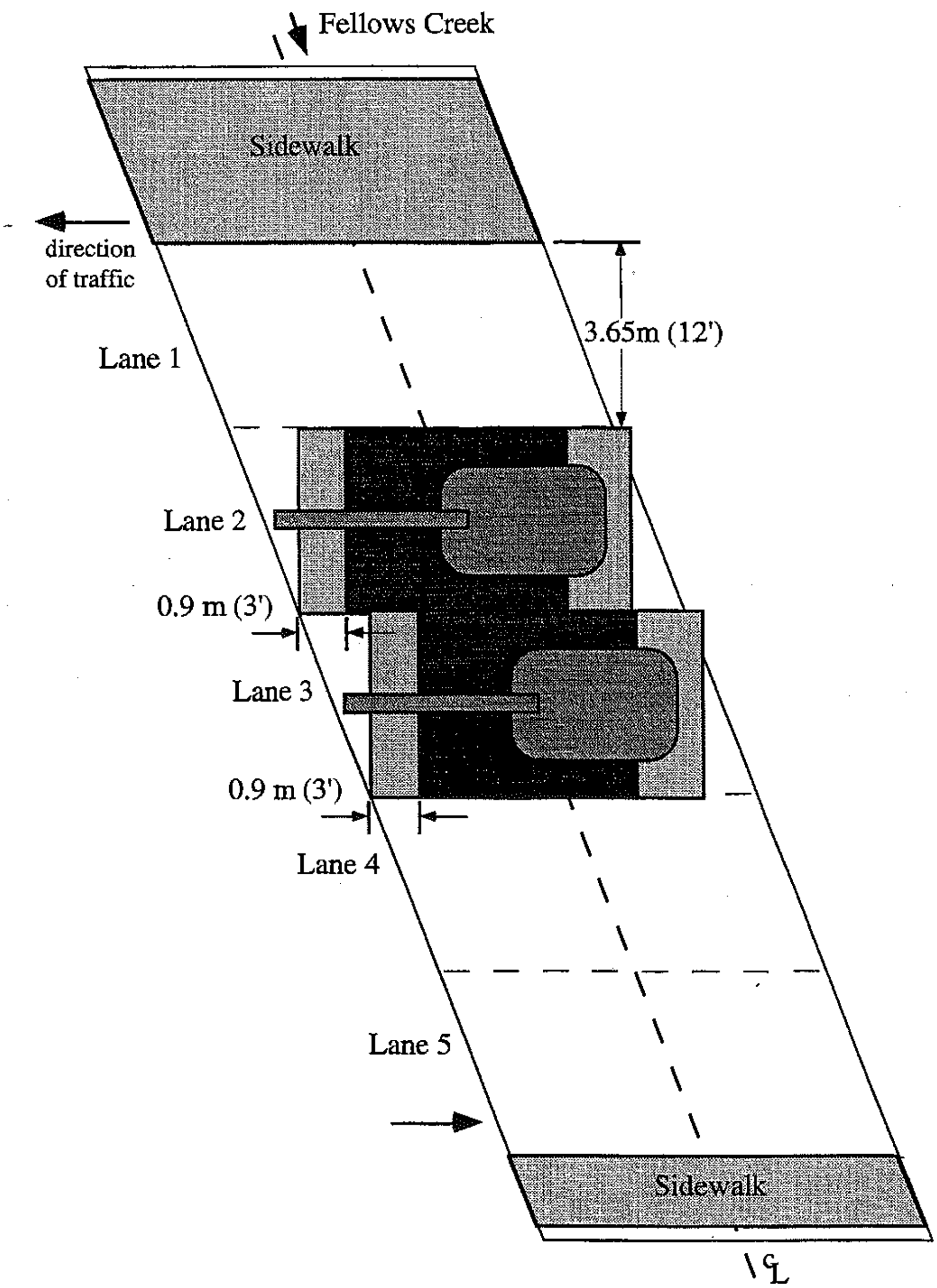
(a) Load Position 1 - Lane 2 and 3 Loading.

Figure 10-5 : Longitudinal Load Positions for Lane 2 and 3 Test.



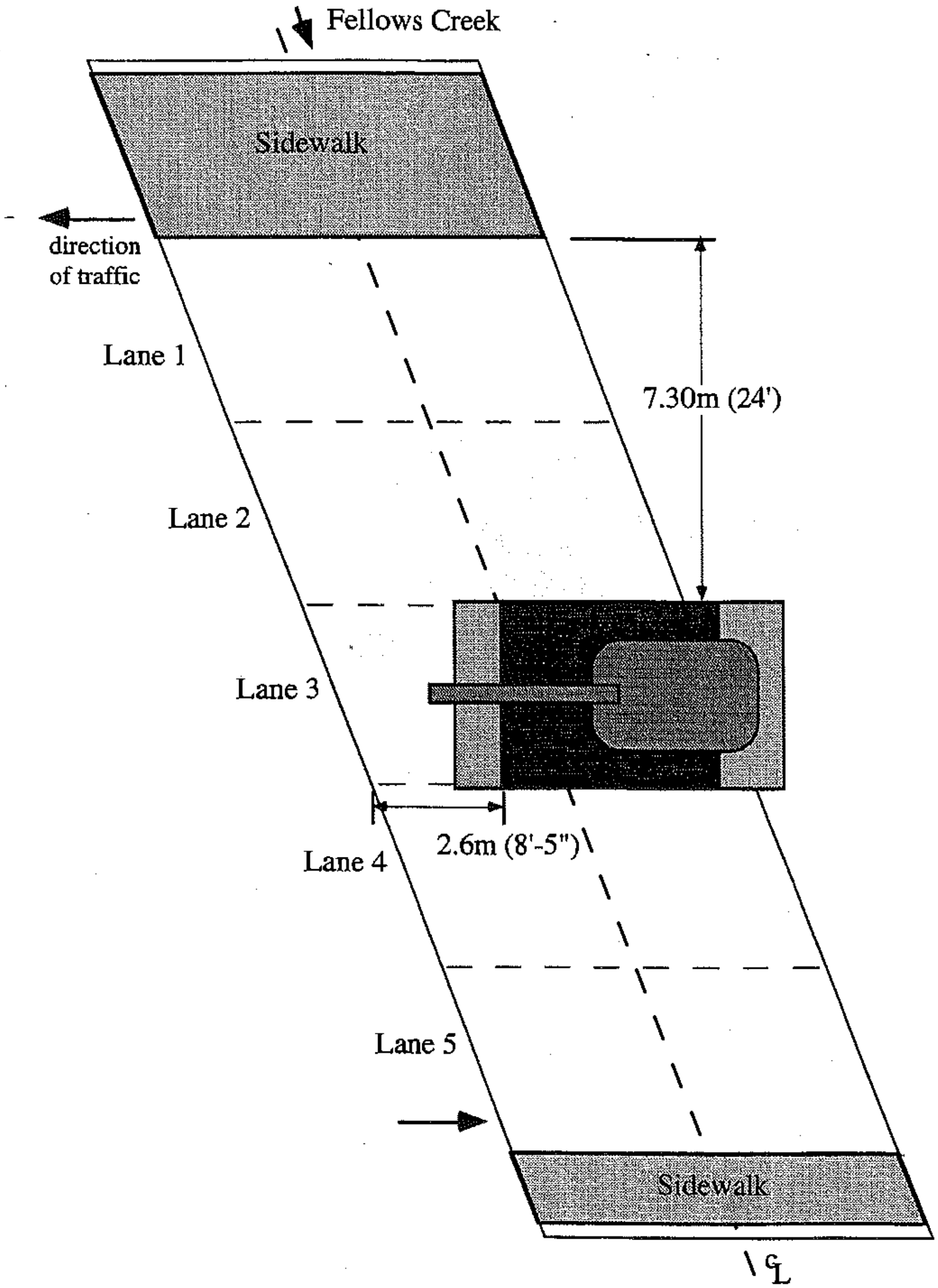
(b) Load Position 2 - Lane 2 and 3 Loading.

Figure 10-5 : Longitudinal Load Positions for Lane 2 and 3 Test. (cont'd)



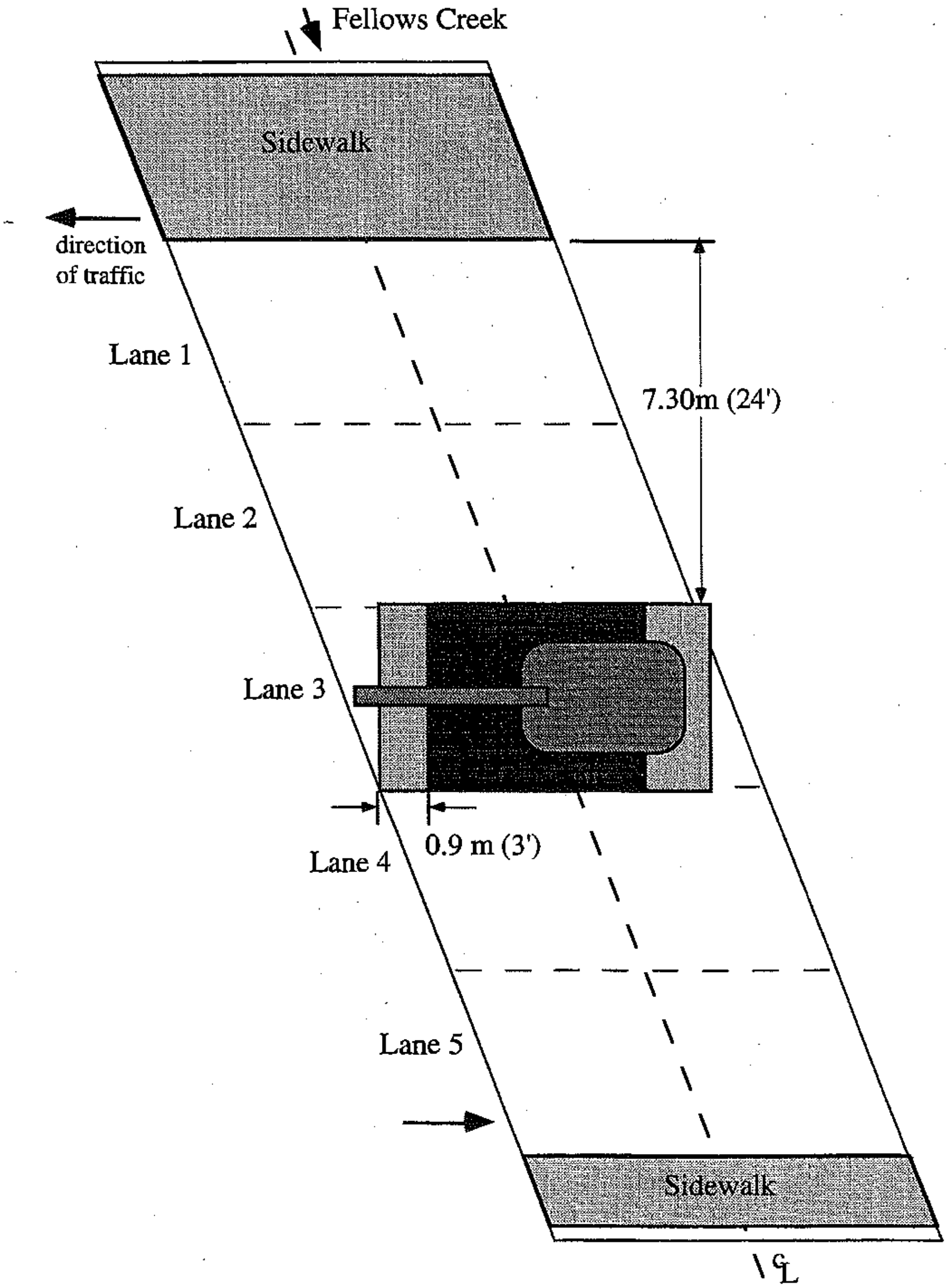
(c) Load Position 3 - Lane 2 and 3 Loading.

Figure 10-5 : Longitudinal Load Positions for Lane 2 and 3 Test. (cont'd)



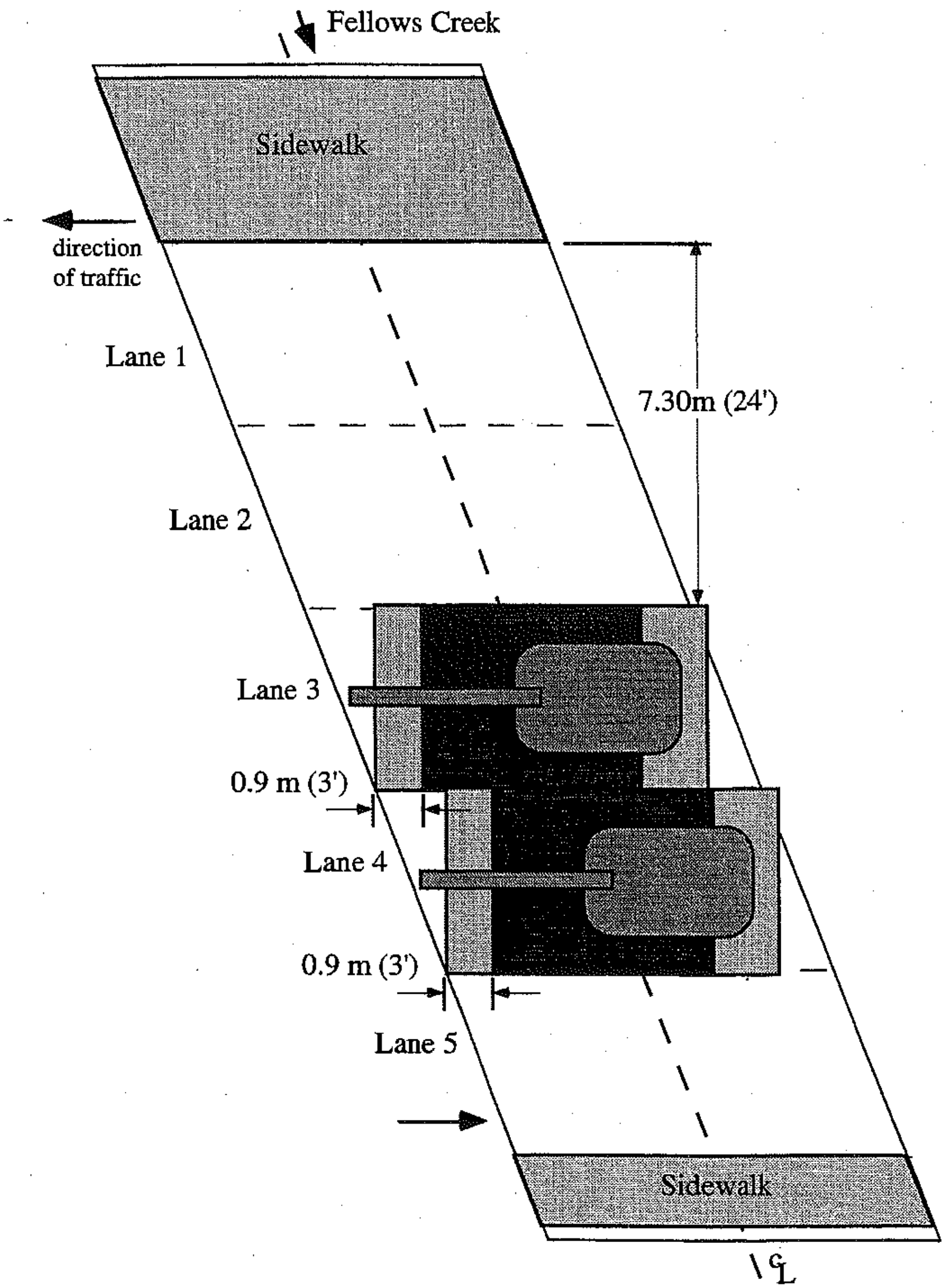
(a) Load Position 1 - Lane 3 and 4 Loading.

Figure 10-6 : Longitudinal Load Positions for Lane 3 and 4 Test.



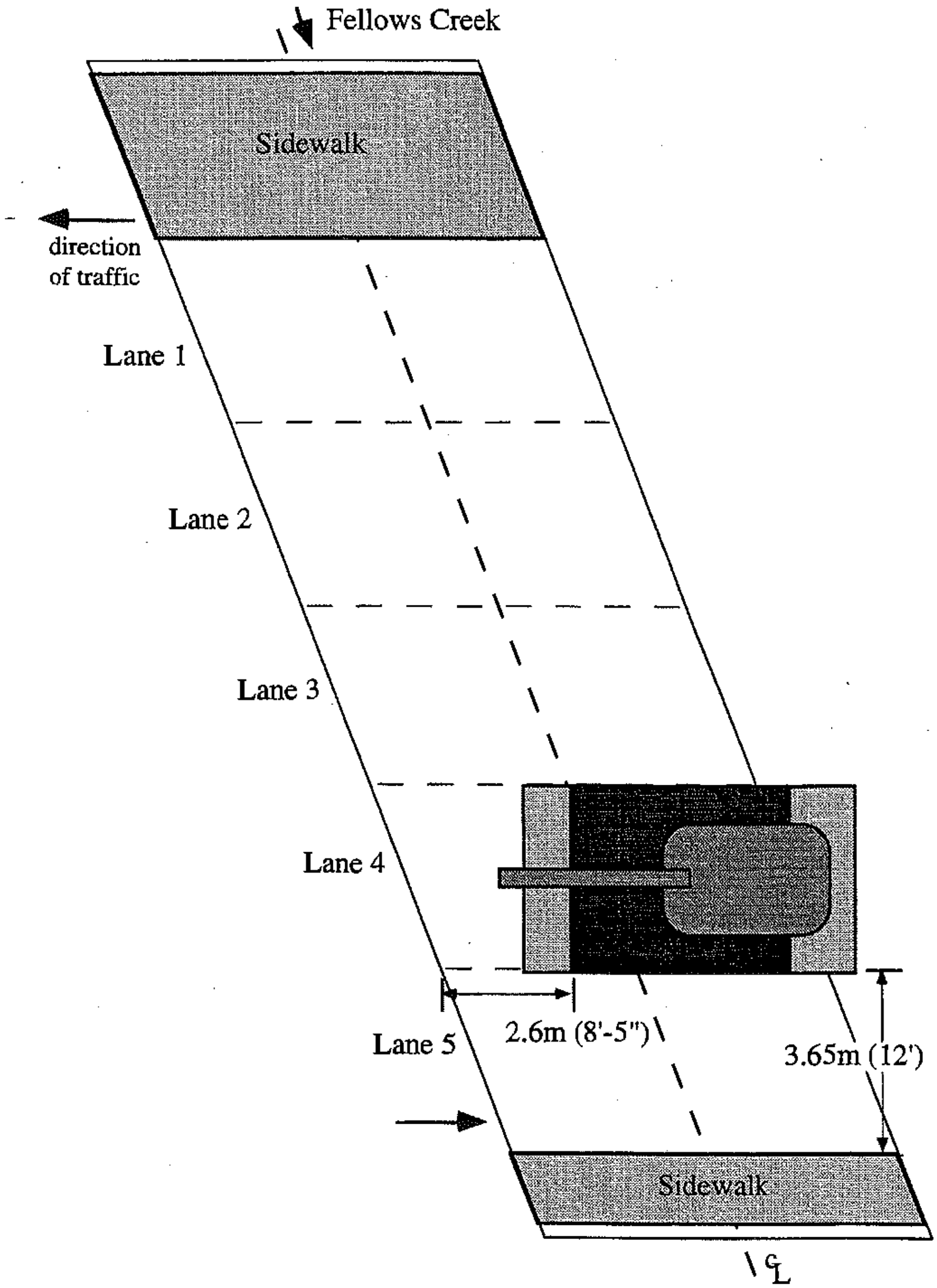
(b) Load Position 2 - Lane 3 and 4 Loading.

Figure 10-6 : Longitudinal Load Positions for Lane 3 and 4 Test. (cont'd)



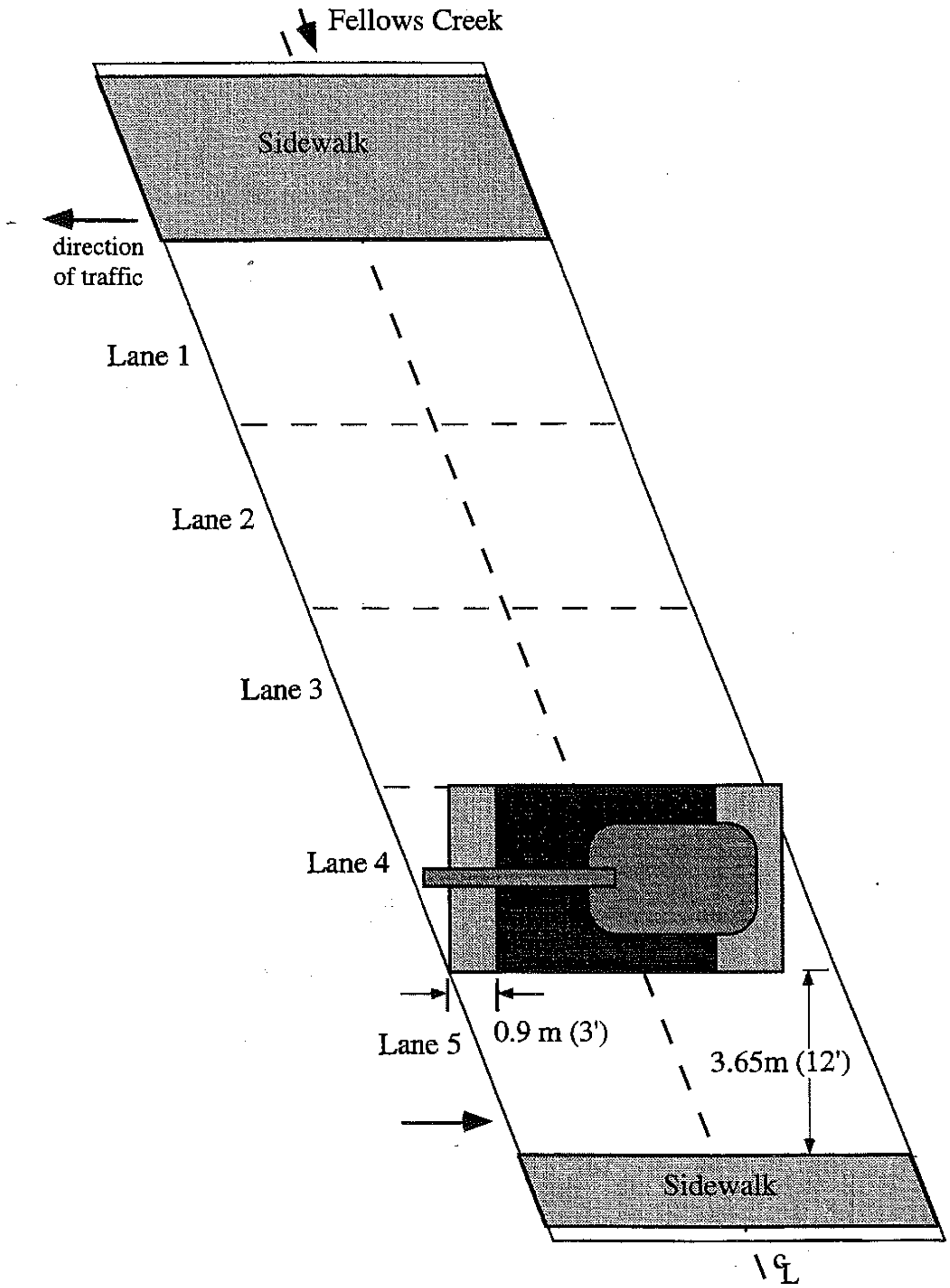
(c) Load Position 3 - Lane 3 and 4 Loading.

Figure 10-6 : Longitudinal Load Positions for Lane 3 and 4 Test. (cont'd)



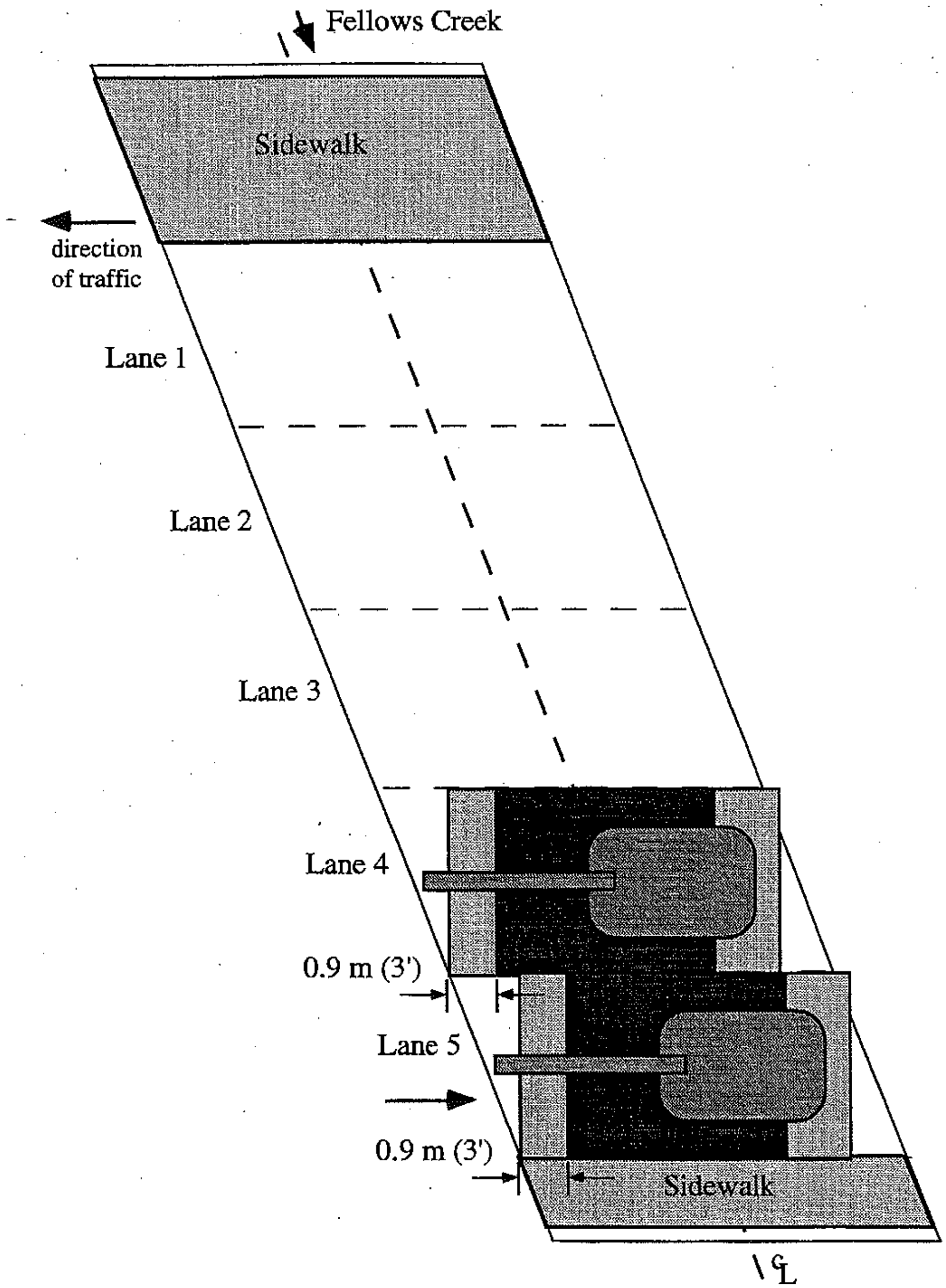
(a) Load Position 1 - Lane 4 and 5 Loading.

Figure 10-7 : Longitudinal Load Positions for Lane 4 and 5 Test.



(b) Load Position 2 - Lane 4 and 5 Loading.

Figure 10-7 : Longitudinal Load Positions for Lane 4 and 5 Test. (cont'd)



(c) Load Position 3 - Lane 3 and 4 Loading.

Figure 10-7 : Longitudinal Load Positions for Lane 4 and 5 Test. (cont'd)

10.5 Proof Load Test Results

The proof load test was successfully carried out without any sign of distress. Although the structure was heavily cracked before the test, no additional spalling was observed during the test. The maximum measured deflection due to the applied load was only 1 mm (0.04 in).

For Bridges No. 1-4, deflections were plotted versus applied lane moments. The maximum load case for Bridge No. 5 corresponds to two tanks side-by-side. Therefore, for consistency of presentation and easier comparison with other bridges, an equivalent lane moment is calculated for two tanks side-by-side. The equivalent lane moment is taken equal to the lane moment due to one tank multipled by a factor, δ, defined as follows,

$$\delta = \Delta_{max2} / \Delta_{max1} \tag{10-1}$$

where Δ_{max1} = maximum measured deflection corresponding to one lane loaded (one tank); Δ_{max2} = maximum measured deflection corresponding to two lanes loaded (two tanks). Deflections Δ_{max1} and Δ_{max2} are determined for the same beam (at the same location).

An example of the response (deflection) of girder no. 2 versus the calculated lane moment is shown in Figure 10-8. The response of the structure is practically linear.

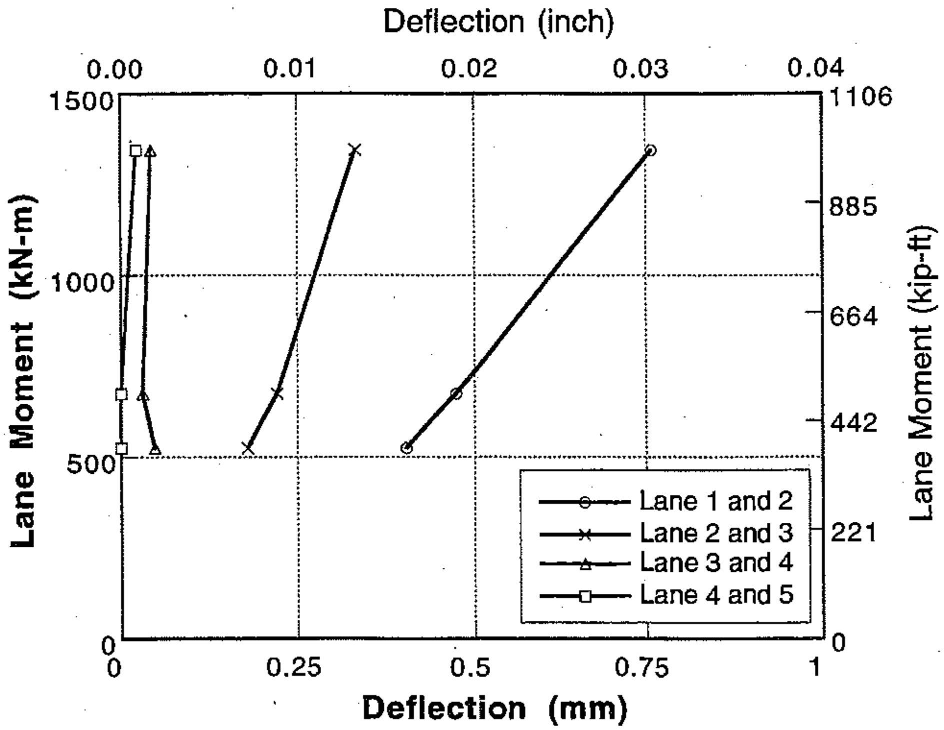


Figure 10-8 : Measured Deflections at Mid-span of Girder 2.

The measured deflection was also compared with the analytical results. For almost all cases the measured values are smaller than those from analysis. This indicates the extra stiffness available to the structure, possibly due to several reasons, such as the influence of the newer part of the structure and some rotational restraint at supports. Figures showing the response of the other parts of the bridge are in Appendix E.

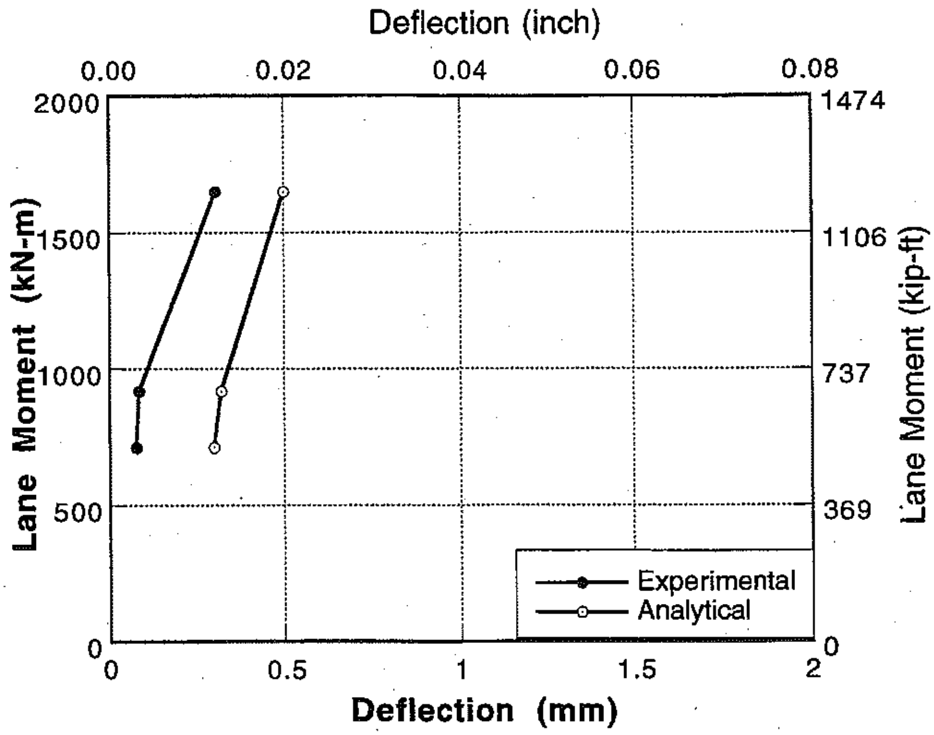


Figure 10-9 : Comparison of Measured and Analytical Deflections at Mid-span of Girder 5 due to Loading in Lane 1 and Lane 2.

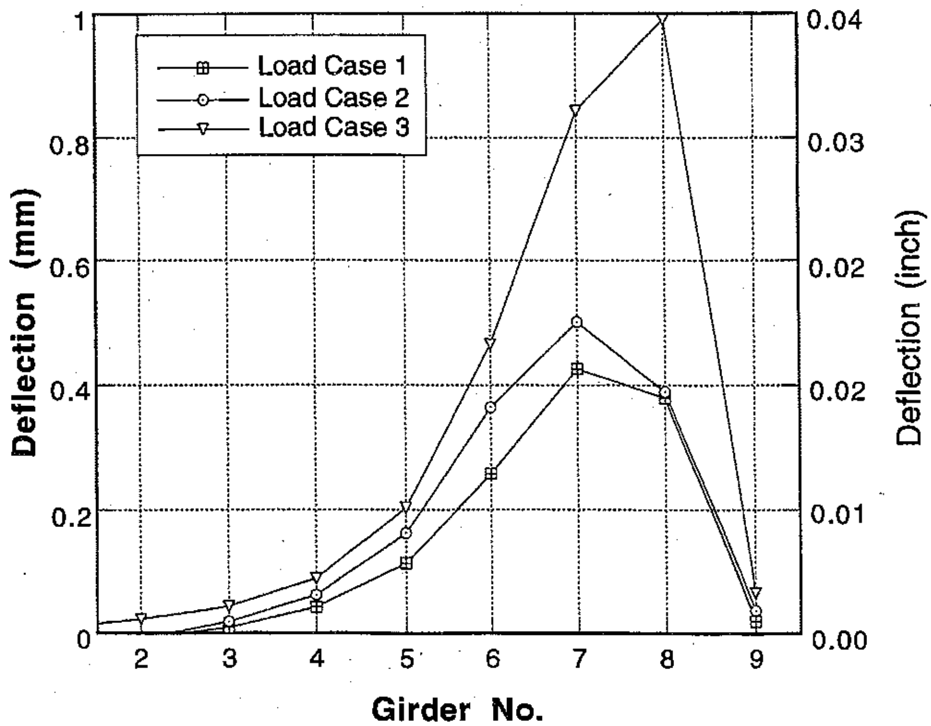


Figure 10-10 : Girder Distribution of Deflections for Loading in Lane 4 and 5.

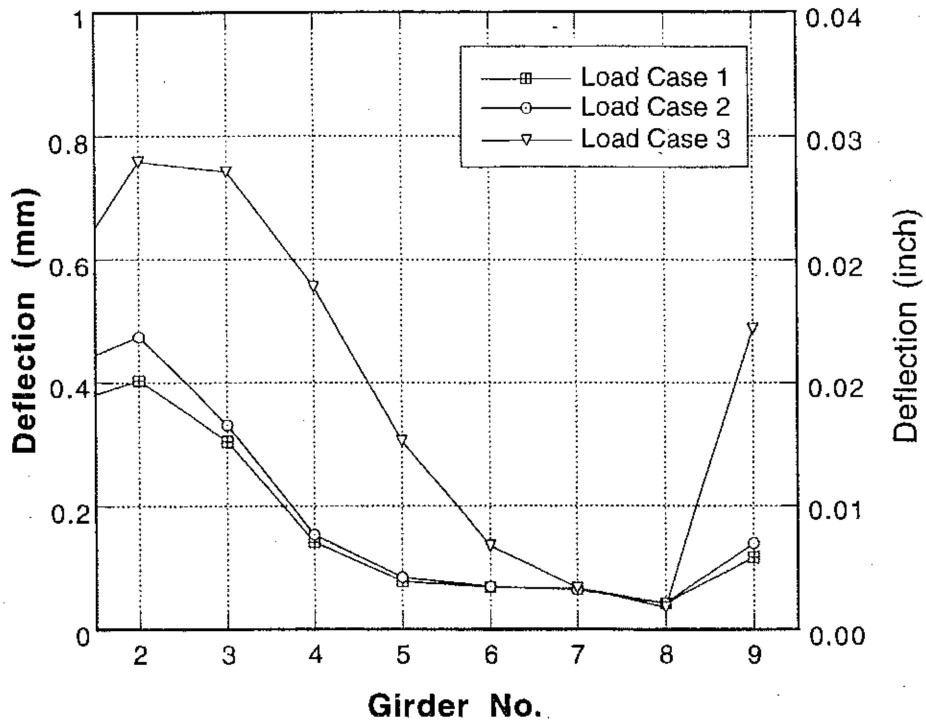


Figure 10-11 : Girder Distribution of Deflections for Loading in Lane 1 and 2.

The girder distribution plots of the deflection are also shown in Figures 10-10 and 10-11 under downstream (Lane 4 and 5) and upstream (Lane 1 and 2) loading, respectively. These figures indicate that the north end of the structure is much more stiffer than the south end. It was also noticed in both figures that the transfer of load between girder 8, 9 and the rest of the structure is very poor. Strengthening of girder nos. 8 and 9 may be considered. The applied proof load moment was 2.33 times the HS20 moment and 1.62 times the two-unit 11-axle moment. The operating rating factor of 1.11 was obtained for a two-unit 11-axle truck. Therefore, the bridge is considered safe to carry the legal truck traffic.

Note:

Intentionally left blank

11. SUMMARY AND CONCLUSIONS

A procedure has been developed for proof load testing of bridges. The test can be used to verify the minimum load carrying capacity of the structure. Load testing can be very efficient in the case of bridges difficult to analyze due to deterioration, repairs, and/or lack of documentation. The developed procedure is based on the use of military tanks as proof load. The tanks are heavy and the load is concentrated over a short length of the tracks.

The procedure is demonstrated on five selected bridges. Structural response is measured in terms of strains/stresses and deflections. Tested structures are instrumented using strain transducers and LVDT's. The load is applied gradually until the predetermined proof load level is reached.

- For all bridges, the target proof load lane moments were reached with minimal signs of non-linearity or distress. Therefore, the operating rating factors for a two-unit 11-axle two unit truck is 1.0 after the test, for all selected bridges, except of Bridge No. 4 for which it is 0.95. However, even for Bridge No. 4, the measured stresses and deflections are very small clearly pointing to a considerable safety reserve in the load carrying capacity. Therefore, all tested bridges were found adequate to carry legal highway traffic.

- As listed in Table 11-1, the applied lane moments were more than 3.5 times larger than HS20 design moments and 2.3 times larger than the two-unit 11-axle moment, still the maximum measured stress in lower flanges of steel girders was only 30 MPa (4.3 ksi) for all bridges. The deflections were also very small and much lower than the AASHTO limits.

- The actual stiffness of the structure turned out to be higher than the values predicted analytically. The primary reasons for this were the unintended composite action and partial fixity of supports for steel bridges; and partial fixity of supports for concrete bridges.

Table 11-1 : Maximum Measured Stresses and Deflections.

Bridge No.	Applied / 11-axle Moment	Applied / HS20 Moment	Maximum Stress in Steel Flange, MPa (ksi)	Maximum Girder Deflection, in mm (inch)	New Operating Rating Factor
1	2.16	2.71	-	3.3 (0.132) (L/2000)	1.21
2	1.70	2.63	19.40 (2.81)	4.70 (0.188) (L/3000)	1.01
3-North	2.18	3.39	22.80 (3.30)	-	1.29
3-South	2.36	3.68	29.98 (4.34)	-	1.40
4	1.53	2.43	15.90 (2.30)	2.36 (0.094) (L/5000)	0.95/2.31
5	1.62	2.33	-	1.00 (0.040) (L/7500)	1.11

- Non-structural members such as a concrete facade, parapets, and railing also contribute to the flexural stiffness of the structure.

- The use of tanks as proof load was found to be safe, economical, and a fast way to verify the minimum load carrying capacity of bridges.

12. REFERENCES

1. *The State of the Nation's Highway and Bridges: Condition and Performance and Highway Bridge Replacement and Rehabilitation Program,* Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1989.

2. Laman, J.A., (1995), *Load Spectra for Girder Bridges*, Proceedings of Fourth International Bridge Engineering Conference, Transportation Research Board, Vol. 1, pp. 314-323, San Francisco, CA, August, 1995.

3. Bakht, B., and Jaeger, L.G., *Ultimate Load Test of Slab-on-Girder Bridge,* Journal of Structural Engineering, ASCE, Vol. 118, No. 6, pp. 1608-1624, June, 1992.

4. MDOT - *Michigan Bridge Analysis Guide*, Michigan Department of Transportation, Lansing, MI, December 1983.

5. Lichtenstein, A. G. *Bridge Rating Through Nondestructive Load Testing,* NCHRP report no. 12-28(13) A, June 1993.

6. AASHTO - *AASHTO Standard Specifications for Bridge Design,* American Association of State and Transportation Officials, Washington, DC, 1992.

7. Nowak, A.S., *Live Load Model for Highway Bridges*, Structural Safety, Vol. 13, pp. 53-66, 1993.

8. Chan, T.H.T. and O'Connor, C.O., *Wheel Loads from Highway Bridge Strains: Field Studies*, Journal of Structural Engineering, ASCE, 116 - 7, 1751-1771 (1990).

9. Chan, T.H.T. and O'Connor, C.O. - *Vehicle Model for Highway Bridge Impact*, Journal of Structural Engineering, ASCE, Vol. 116 - 7, 1772-1793, 1990.

10. Nowak, A.S., Laman, J.A., and Nassif, H., *Effect of Truck Loading on Bridges*, Research Report UMCE 94-22, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI, 1994.

11. Nassif, H.H., and A.S. Nowak, *Dynamic Load Spectra for Girder Bridges*, Transportation Research Record 1476, TRB, National Research Council, Washington, D.C., 1995, pp. 69-83.

12. Juntunen, D.A., and M.C. Isola, *Proof Load Test of R01 of 61131 M-37 over CSX Railroad, South of Bailey. Michigan*, Michigan Department of Transportation. Lansing, MI, April 1995.

13. Mufti, A.A., et al., (1992), *User Manual for Computer Program SECAN*, Ministry of Transportation of Ontario, Ontario, 1992.

14. Jaeger, L.G., and B. Bakht, *Bridge Analysis by Microcomputer*, McGraw-Hill Book Company, 1989.