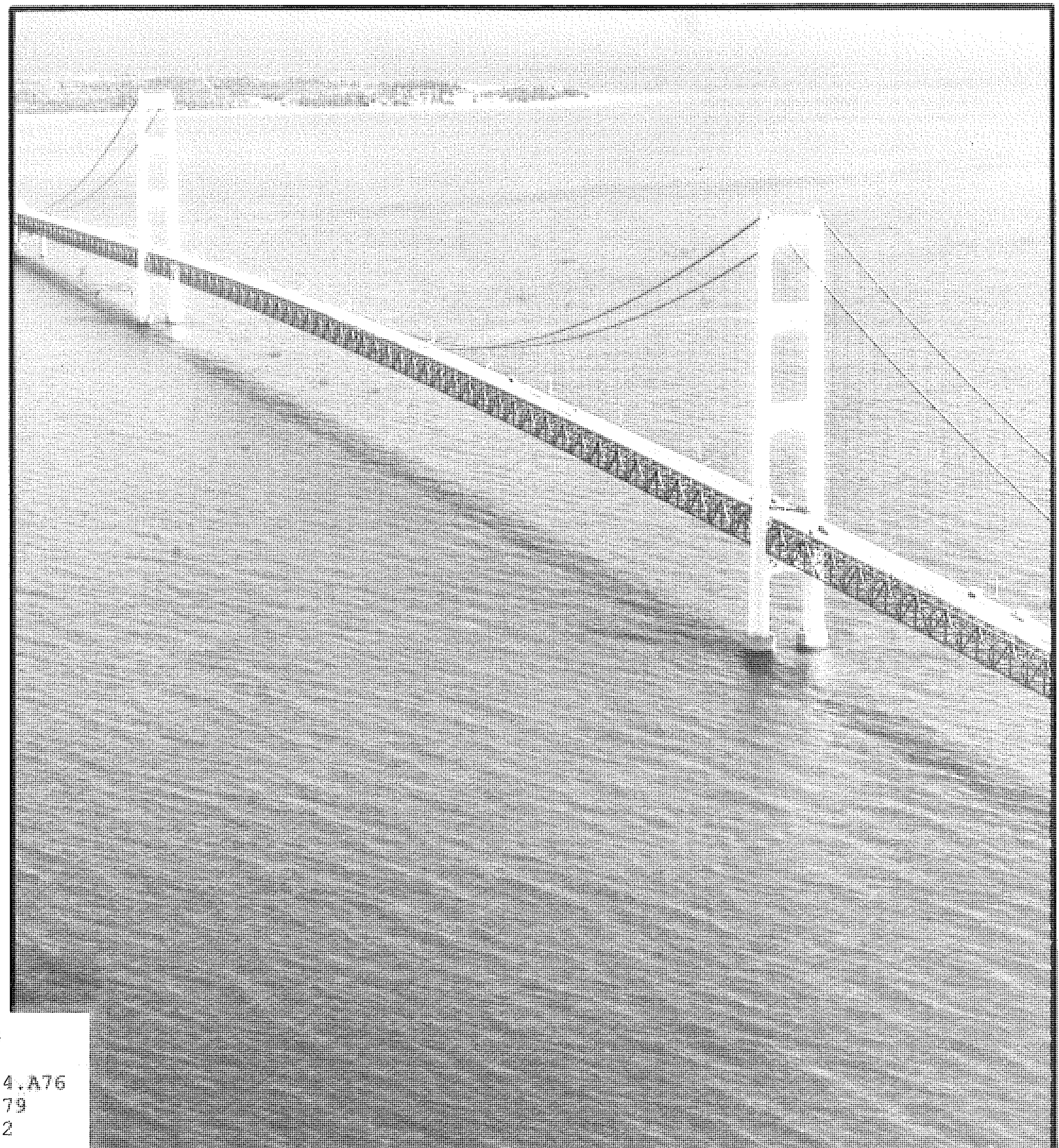
STRESS MEASUREMENTS AND LOAD DISTRIBUTION ON SELECTED PORTIONS OF THE MACKINAC BRIDGE



MICHIGAN DEPARTMENT OF TRANSPORTATION

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Stress measurements and
load distribution on
selected portions of the
Mackinac Bridge

STRESS MEASUREMENTS AND LOAD DISTRIBUTION ON SELECTED PORTIONS OF THE MACKINAC BRIDGE

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Research Laboratory Section
Testing and Research Division
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ACKNOWLEDGEMENTS

The experimental project was carried out by the Research Laboratory in close cooperation with the Design Division and the Mackinac Bridge Authority. Orlando Doyle, Chief Engineer for the bridge, was the project representative for the Bridge Authority, and the project was planned in cooperation with Lawrence Rubin, Executive Secretary. The Bridge Authority provided garage services, work space, traffic control, crane, radio communications, some supplies, additional personnel, and vehicles as needed.

W. D. Bullen's squad from the Department's Design Division provided all bridge design analyses, theoretical stress calculations, and determination of critical areas for strain gage application; M. E. Largo and E. T. Ho performed detailed calculations and checked design stresses. P. Whitlock was the representative at the site for Steinman, Boynton, Gronquist & Birdsall, designers of the bridge and consultants to the Bridge Authority. D. Woodend of the Safety Section of the Department, provided safety climbing harnesses and instructions in their use.

The Research Laboratory furnished materials, supplies, and all of the electronic equipment for the project. Research personnel did the steel sampling and strain gage work. The Instrumentation and Data Systems Group of the Laboratory, under the supervision of L. E. DeFrain, designed and built some of the complex electronic gear that was used, and also was responsible for the assembly and operation of the extensive instrumentation package. L. E. DeFrain and R. L. Downing did the actual gage application and connection of the leads to the gages as well as electronic equipment operation, alteration, and repair. J. R. Darlington worked in the assembly of the system and also in on-site operation. M. J. Fongers built the controller for the digital system, as well as on-site electronic alterations, repairs, and equipment operation. W. E. Casey did on-site wiring, equipment operation, and trouble-shooting on the assembly and system.

The Structural Mechanics Group of the Laboratory was responsible for beam preparation for gaging, steel sampling, and analysis. M. A. Chiunti and R. P. Dexter did the preparation and sampling work, and K. S. Bancroft worked in data gathering at the site and analysis of the traces.

Artwork and graphic presentation were done by J. Perrone and his group. Editing was done by J. B. Alfredson and typing by R. S. Flynn.

The work under the suspended span, high above the water, was an unusual and challenging assignment for the Research Laboratory team. Their performance on the high steel, under uncomfortable and unpredictable weather conditions, through extremely long workdays, was excellent and highly commendable.

The cooperation provided by all the groups involved in this project was exceptional, making it possible to complete the assignment on schedule, in spite of the many unexpected situations that arose.

SUMMARY

The primary purpose of the study was to confirm the feasibility of railcar transportation over the Mackinac Bridge, by making strain measurements on selected members of the bridge; also to gain additional insight concerning the overload capacity of the bridge, and to gather limited information on the effects of heavy commercial traffic that uses the bridge daily.

Calculations indicated that for the heavy vehicle types considered in this project, the critical or most highly stressed portion of the bridge would be the two-span unit at the very north end. Next in order came the adjacent four-span structure, followed by any of the 46 essentially identical two-span units between the main towers.

Strain gages were applied at 49 different points on the bridge; 20 on the above mentioned two-span structure on the north approach, 15 on the adjacent four-span, and 14 on one of the 46 two-span continuous portions of the main suspension span, near the north 1/4 point. Strains were measured and associated stresses calculated for the 49 gaged locations, as loads were applied by an experimental 11-axle, 74-tire, 80 ft by 12-ft, 249,000-lb vehicle. Limited data were also collected from commercial traffic on the bridge.

Measured stresses confirmed that the order determined by calculation mentioned in paragraph two above, was correct. The maximum live load stress determined by measurement and the calculated dead load stress was only 54 percent of the yield strength; well below the 65 percent limit recommended by the designers of the bridge.

Calculations indicated that the proposed railcar transporter would cause only about 3/4 as much stress as the experimental vehicle. Additional calculations indicate that the proposed transporter would not stress the floor system of the suspension bridge as highly as does the 77-ton legal commercial vehicle.

It was determined that there is interaction of the centerline stringer with the main supporting truss of the suspension bridge when very heavy vehicles or combinations of heavy vehicles use the bridge. Influence distances for such vehicles are 3,000 ft or more, due to changes in position of the main suspension cables. Within the limits of measurement, the suspension bridge returns to its original position once the heavy loads have passed.

Data from strains measured under commercial traffic show that stresses due to the 11-axle vehicles approach the levels caused by the experimental vehicle.

Evaluation of steel specimens removed from 13 locations on the instrumented spans determined that the strength of the steel exceeded specifications.

Conclusions

1) The bridge is capable of carrying 80-ton railcars on the proposed railcar transporter vehicle. This conclusion is related only to the physical strength of the bridge and stress distributions that occur. It is not a recommendation that such a program be undertaken, as there are many other factors to be considered.

2) The strength of the steel specimens removed from a total of 13 different locations on the northernmost two-span unit, the adjacent four-span unit, and one two-span unit of the main suspension span, all exceeded the specified minimum values.

3) In no location evaluated did the stresses (including calculated dead load stresses, and live load measured stresses) resulting from the 11-axle, 249,000-lb experimental vehicle, exceed 54 percent of yield stress. This is well below the recommended overload limit of 65 percent of yield stress, and is approximately equal to the 55 percent of yield strength used as an allowable design stress for ordinary use.

4) The greatest measured stress in the floor system of the suspension bridge was 34 percent of yield.

5) The maximum stress due to the proposed railcar transporter should be less than 3/4 of the stress that resulted from the experimental vehicle in the critical spans evaluated. Calculated stresses in the suspension bridge for the proposed transporter are lower than for the 11-axle, 77-ton legal commercial vehicle.

6) Heavy commercial traffic can cause maximum stresses nearly as high as those that resulted from the experimental vehicle.

I

INTRODUCTION

INTRODUCTION

Because of the diversity of backgrounds of those interested in this report, and the different purposes for which it is to be used by those readers, the report is divided into two major sections. Part I develops the background of the project and provides general information. It talks of the design considerations and outlines the experimental procedures. Part I closes with the final results of the entire study.

Part II of the report is devoted to detailed technical information and is included for the benefit of those interested in the area of structural mechanics from a more technically detailed point of view.

Background

The transportation of railroad cars across the Straits of Mackinac, is done by the State of Michigan using the ferry 'Chief Wawatam,' which has been in service for more than 60 years. During that time the railroad industry prospered and then declined. Records show approximately 35,000 cars transported in 1951, and a steady decline to about 400 in 1975. Cost to the State for the ferry operation has been in the vicinity of $700,000 per year during recent years; an increase to approximately $1.5 million in the past year, was due primarily to repairs required. At present, the ferry is in poor condition and the Department has been advised that the inefficient old coal-fired boilers must be replaced to reduce environmental pollution.

Reasons for or against maintaining rail service across the Straits are many and varied, and obviously are beyond the scope of this report. Similarly, reasons for the recent increase in railroad traffic at the Straits are related to Federally controlled freight rate structures, and also are beyond the scope of this report.

Railroad capabilities were omitted from the final design of the bridge because their inclusion would have approximately doubled the cost of the bridge, placing it far beyond the financing that could be obtained. However, since railroad volume has decreased so much in recent years, there have been numerous proposals and discussions concerning alternate methods of maintaining the rail link at the Straits, including the possibility of transporting railroad cars across the Mackinac Bridge on a rubber-tired transporter.

During 1977, a Departmental committee was formed and ordered to study the technical feasibility of hauling railroad cars over the bridge. The committee report, issued in March 1978, concluded that it would be techni-

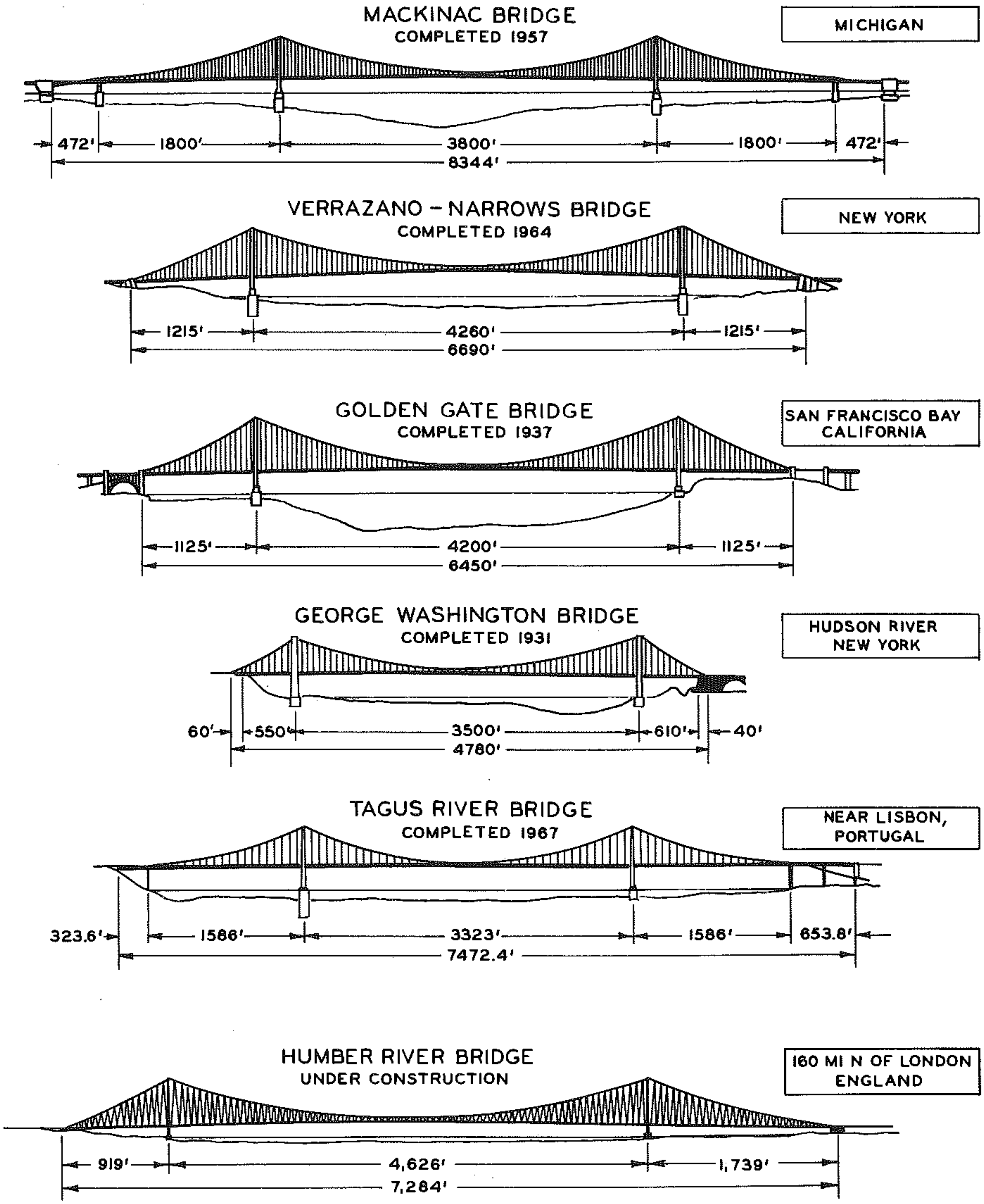


Figure 1. Comparative magnitudes of the world's greatest suspension bridge.

cally feasible to transport railroad cars, weighing up to approximately 80 tons, across the bridge, provided a special transporter vehicle was obtained and the bridge were closed to other commercial traffic as the transporter crossed. Cost estimates indicated that if the system was built and operated for two years, the decrease in cost as compared to the ferry operation, would pay for the new system. A practical maximum of about 20 cars per week could be transported each way. Note that although freight car weights of well over 80 tons are prevalent on many rail lines, the 80 ton figure was reasonable, based on the rail traffic at the Straits of Mackinac during the years immediately prior to the study.

The Committee's analyses were submitted to Steinman, Boynton, Gronquist & Birdsall, consulting engineers to the Mackinac Bridge Authority and designers of the bridge. The consultant suggested a more conservative approach than used for other Michigan bridges, but agreed that heavier loads were possible, provided that experimental loading and strain measuring were accomplished to document the actual stresses and load distributions that exist. The end result of the deliberations was an agreement within the Department that the subject study should be accomplished. Basic reasons for the study were to:

1) Confirm the feasibility of railcar transportation over the bridge,

2) Gain additional insight concerning the overload capacity of the bridge for unusually heavy vehicles with configurations different from the proposed railcar transporter,

3) Gather limited information on the effects of heavy commercial traffic that uses the bridge daily.

Field work was started on May 30, and completed on June 30, 1978, using a Research Laboratory crew of six to seven people, plus periodic help from the Bridge Authority.

General Information

The Mackinac Bridge stands as a landmark; a gateway to the north for transportation and recreation, an important link in Michigan's commerce, and one of the great engineering structures of the world. Figure 1, provided by the Bridge Authority, shows the dimensions of the bridge in comparison to other well-known structures. We have added at the bottom, a sketch of a new suspension bridge with a record 4,626-ft main span that is nearing completion in England, 160 miles north of London, over the Humber River, near Hull. However, the total length of that suspension bridge will

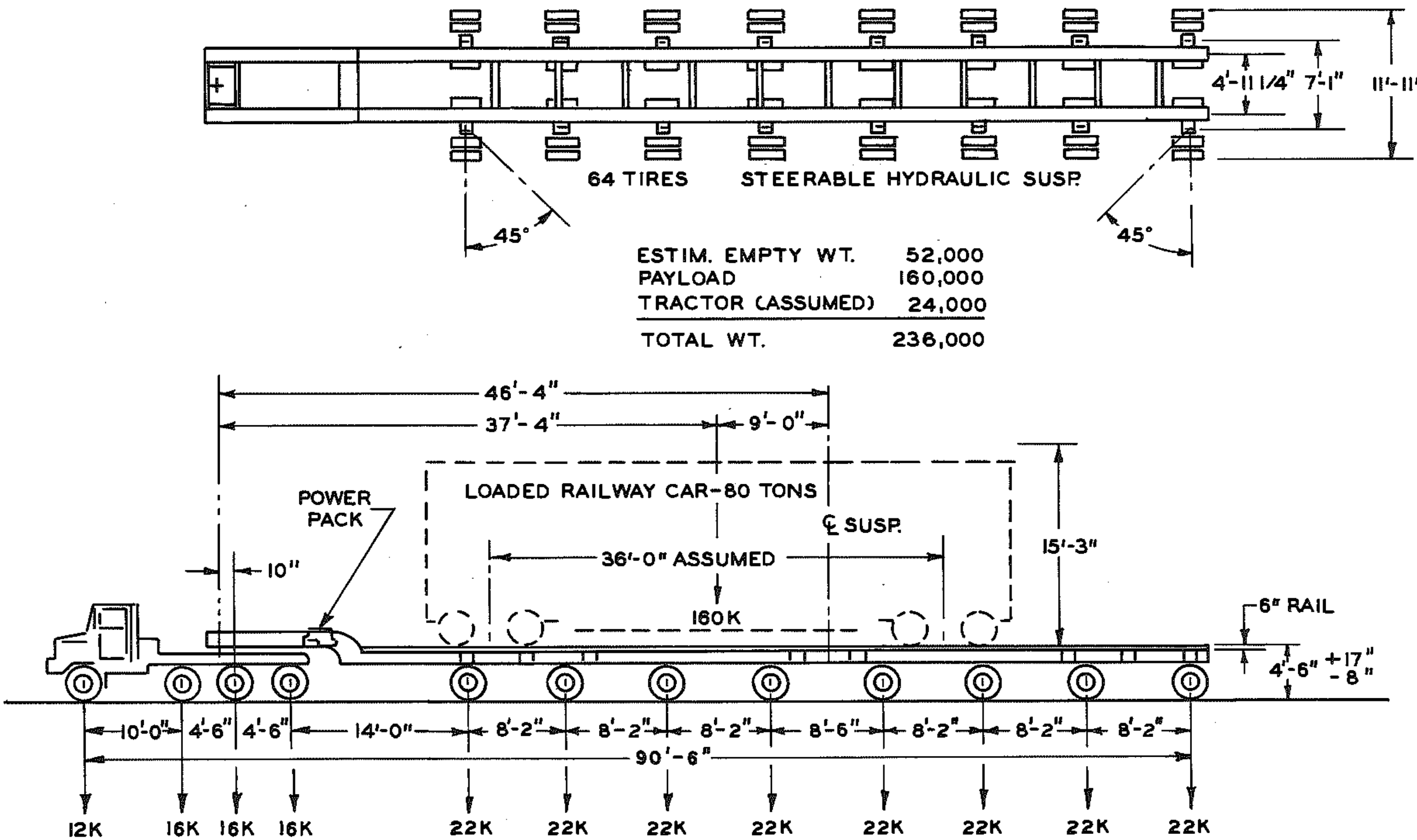


Figure 2. Proposed railcar transporter.

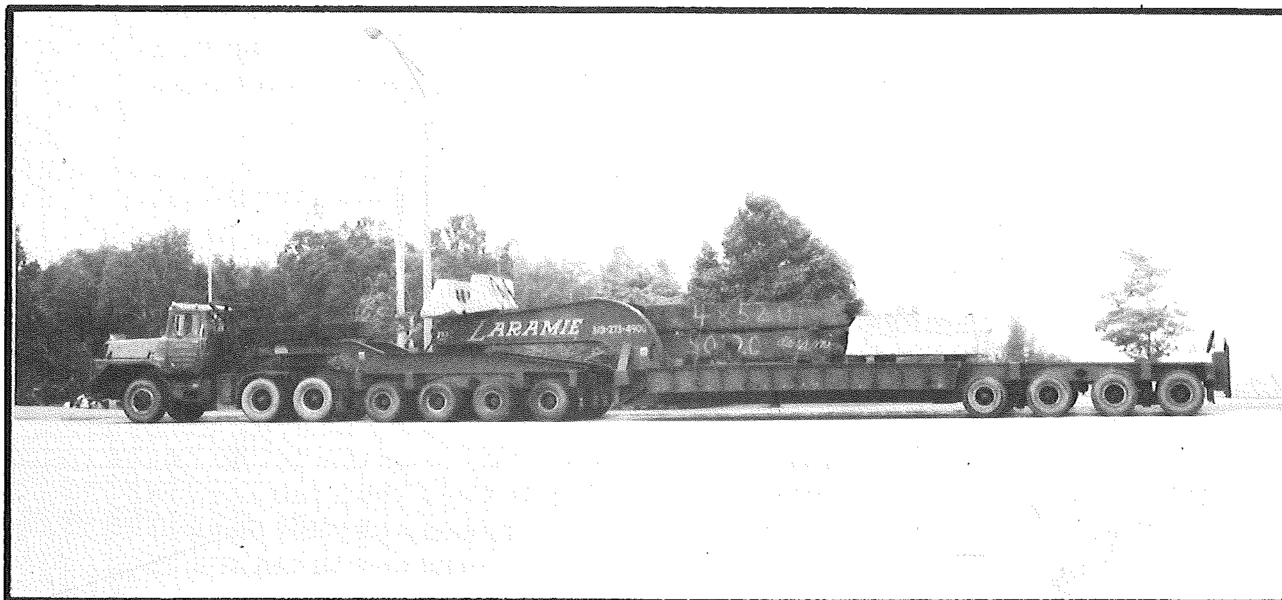
be 7,284 ft, still far short of Big Mac's total suspended span. In addition to the Mackinac suspension bridge shown in Figure 1, numerous approach spans on both ends of the structure bring the total length of the crossing to more than 19,200 ft.

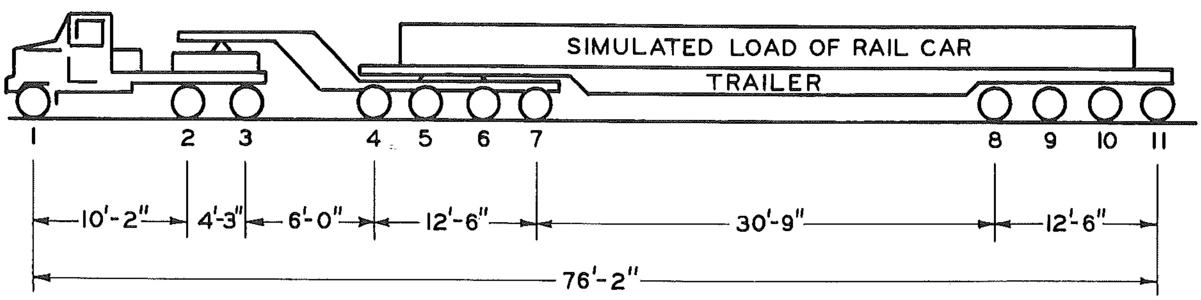
Floor systems for very large bridges are made as light as practical in order to decrease the total dead weight that must be supported by the structure. This factor resulted in a floor system somewhat lighter than that used for smaller Michigan freeway bridges. This does not affect the capability to handle cars and most normal commercial traffic, but it does require special consideration for very heavy vehicles that use the bridge.

Signs posted on the highways leading to the bridge specify Bridge Authority escort for vehicles weighing more than 40 tons. Such vehicles are required to travel at reduced speeds, and to maintain considerable distance from other heavy vehicles. Michigan law allows a maximum of 154,000 lb on an 11-axle rig, 8-ft wide, with 42 tires. Vehicles of this type cross the bridge daily. However, vehicles with extremely heavy wheel loads, such as earth movers or end loaders with 100,000 lb total weight on only four tires, are not permitted to drive across the bridge, even though they are lighter than the 11-axle highway truck. They put too much weight in one small place. The basic requirement is to distribute the loads as much as possible on the floor structure of the bridge. Figure 2 shows the vehicle that was proposed for use if railroad cars are to be transported across the bridge. The vehicle used for this study (Fig. 3) was shorter, with more closely spaced axles, and therefore, provided a more severe loading of the floor system than would the proposed vehicle; in other words, our measurements are on the 'safe' side. Calculations indicate that the maximum stress (live load plus dead load), caused by the proposed vehicle as shown in Figure 2, would be less than 3/4 of that which resulted from the experimental vehicle, even though the gross weight of the proposed vehicle is 95 percent of the weight of the experimental one. This is due to better distribution of load on the deck.

Strain gages were placed at 49 different locations on the bridge. This included 20 on a two-span continuous structure on the north approach, and 15 on a four-span continuous structure immediately south of the two-span (Fig. 4), plus 14 on one two-span continuous portion of the suspension bridge (Fig. 5).

Load was applied by an 11-axle, 74-tire rig, 80 ft long and 12 ft wide, loaded to 249,000 lb gross (Fig. 3), at speeds of creep, 10, and 20 mph. This is the heaviest vehicle ever known to have crossed the Mackinac Bridge.





	AXLE WEIGHT, THOUSANDS OF POUNDS											
	TRACTOR			TRAILER								TOTAL
	1	2	3	4	5	6	7	8	9	10	11	
SERIES 1	12	11	11	22	22	22	22	22.5	22.5	22.5	22.5	212
SERIES 2	12	18.5	18.5	25	25	25	25	25	25	25	25	249

NOTE: SERIES 2 WEIGHTS USED IN THIS REPORT, DESIGNATED AS "TV2" IN THE TABLES.

Figure 3. Load vehicle used for the study.

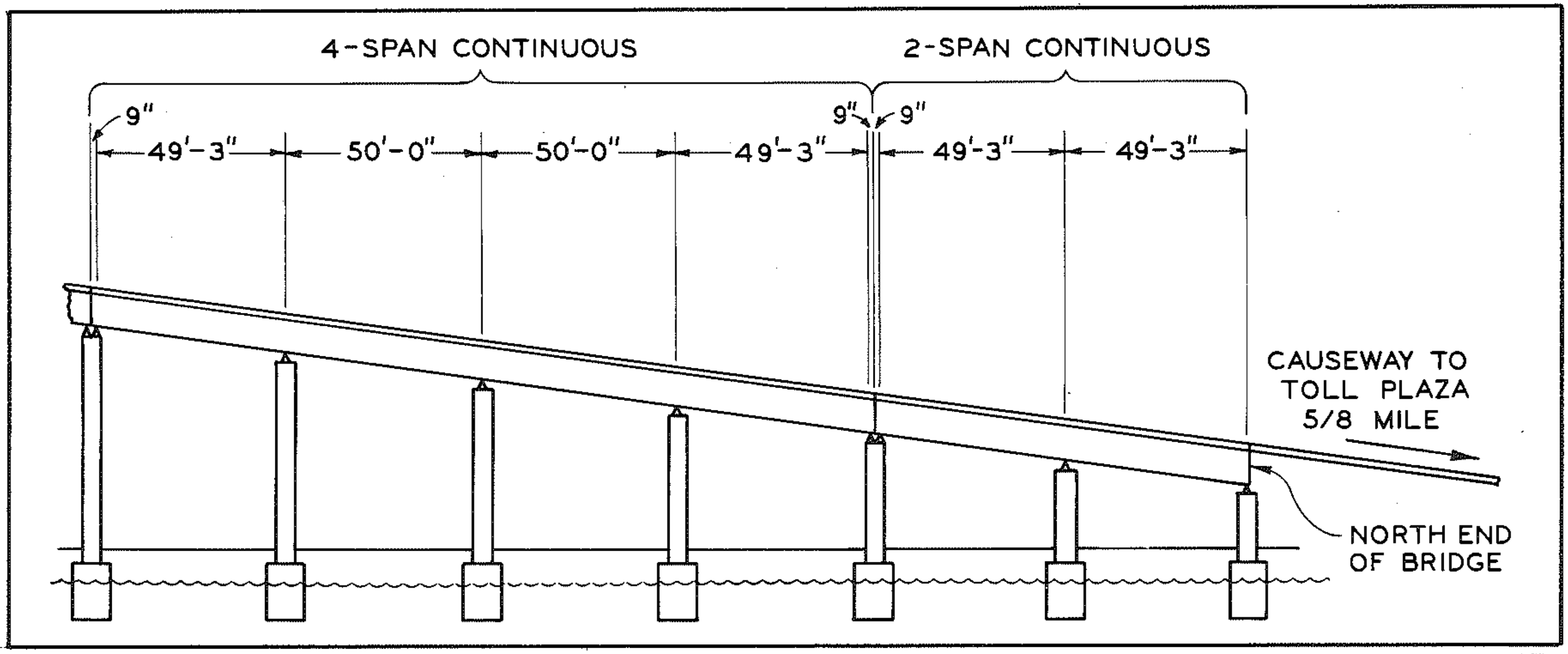
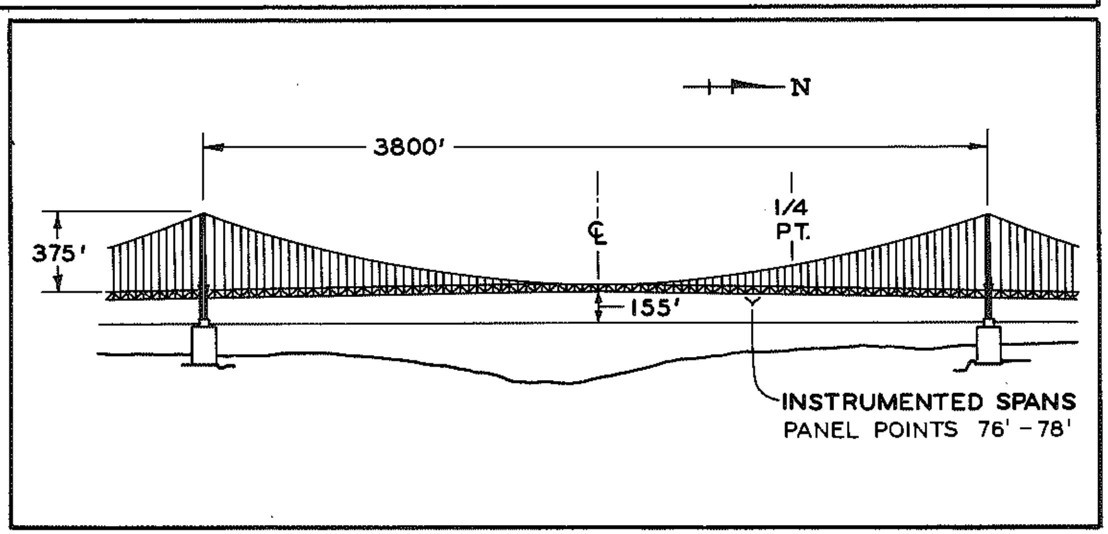


Figure 4. Two-span and four-span continuous structures that were instrumented on the north approach.

Figure 5. Location of the instrumented spans of the suspension bridge.



Design Considerations

The entire bridge system contains many spans of different types and lengths. When considering the capacity of the bridge to handle very heavy loads, the following must be carefully considered. Vehicles of equal weight but different configuration will cause different stresses in the various spans, depending to some extent on how the length of the vehicle and distribution of the axles compare to the lengths of the spans. Therefore, the 'controlling' span or spans (the one or more where the maximum allowable stresses occur) will be different for trucks of different configuration. For instance, in a two-span continuous portion (spans where the lengthwise beams are long enough to cover two spans in a row) a truck of a length such that the front group of axles is near the middle of one span when the rear group is near the middle of the other span will cause high bending stresses in the beams over the center support. The same truck weight on a longer or shorter configuration would cause a less severe loading of the beams at that point in that particular pair of spans, but might be worse somewhere else.

In general, the longer and wider the vehicle, the more axles and tires share the load, and the more the axles are separated, the greater the total load that the floor system of the bridge can support. As this concept is carried to an extreme, of course, the size of the truck becomes excessive for maneuvering, and the dead weight of the vehicle becomes so great that little payload can be hauled. Also, if the vehicle becomes large enough under this proposition, portions of the bridge other than the floor system may be overloaded. The vehicle proposed for railcar transport (Fig. 2) is a reasonable compromise of these various factors.

Calculations indicated that the two-span continuous structure at the very north end of the bridge (Fig. 4) would be the 'controlling' structure for the proposed transporter configuration, since it would cause stresses in the beams in that portion of the bridge that would be closest to the maximum allowable. Next in order came the adjacent four-span structure, and after that, any of the 46 essentially identical two-span continuous units between the main towers of the suspension bridge. (The two nearest the towers are of slightly different design.) Correspondence with the consultant, Steinman, Boynton, Gronquist & Birdsall, had revealed their concern over additional stresses in the floor system of the suspension bridge that result from shifting or changes in the 'sag' of the main suspension cables, as very heavy loads proceed across the bridge. While such effects are known to exist, they cannot be accurately calculated; therefore, they requested that instrumentation in the suspension bridge be done near the 1/4-point of the span where such effects appear to be the greatest. Access ladders are located every 14 spans on the suspension bridge, and the instrumented spans shown in Figure 5 were chosen adjacent to the access ladder closest to the 1/4 point. It was decided to place gages at that location to minimize interference with a painting contractor who was working in the same vicinity while strain gage application was in progress.

EXPERIMENTAL PROCEDURE

Loading

Experimental loading of the bridge for this study, required the use of a vehicle that was already in existence, within reasonable travel distance of the bridge, and of such configuration as to apply stresses similar to those that would result from the proposed transporter. The vehicle that was used was leased from a heavy hauling contractor in Detroit, and ballasted with steel and concrete to obtain the weights required. Initial measurements were made at a reduced loading (noted as 'Series One' in Fig. 3), to check the relationship between computed and measured loads. Then when the initial results had checked satisfactorily, loading was increased to the full 249,000-lb 'Series Two' magnitude. Only the results of the heavier Series Two loading ('TV 2' in the tables), will be discussed in this report.

Instrumentation

Stresses in the beams at various locations were determined by the application of electrical resistance strain gages to the beams, and wiring them into a complex system of electronic power supplies, signal conditioners, controllers, amplifiers, and recorders. The systems consisted of 10 channels of Astro-Data and four channels of Brush power supplies and amplifiers, and a Honeywell analog magnetic tape recorder; along with a Research Laboratory-designed and built, 16-channel digital interface (including power supplies, amplifiers, multiplexer, and analog-to-digital converter), coupled to a Digital PDP-8E mini-computer and a Wang Model 1045 tape deck. In the digital leg of the system, amplifications of 1,000 and 10,000 were provided, and the computer selected the proper output to obtain maximum sensitivity without distortion.

For those not familiar with strain gage applications, a strain gage is shown schematically in Figure 6. Gages used for this study were 1/4-in. long, but various special purpose gages are available with gage lengths down to less than 0.010-in. Gages are carefully formed from very thin foil of specially selected metals. The principal of operation is as follows: when

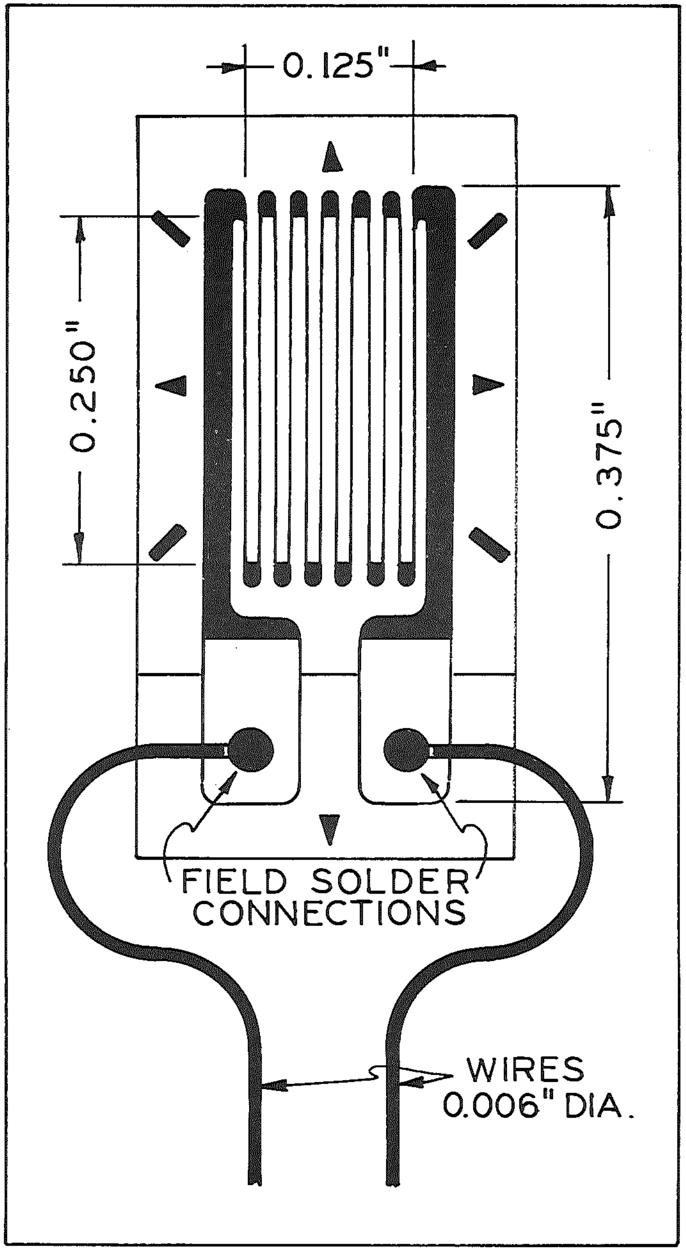
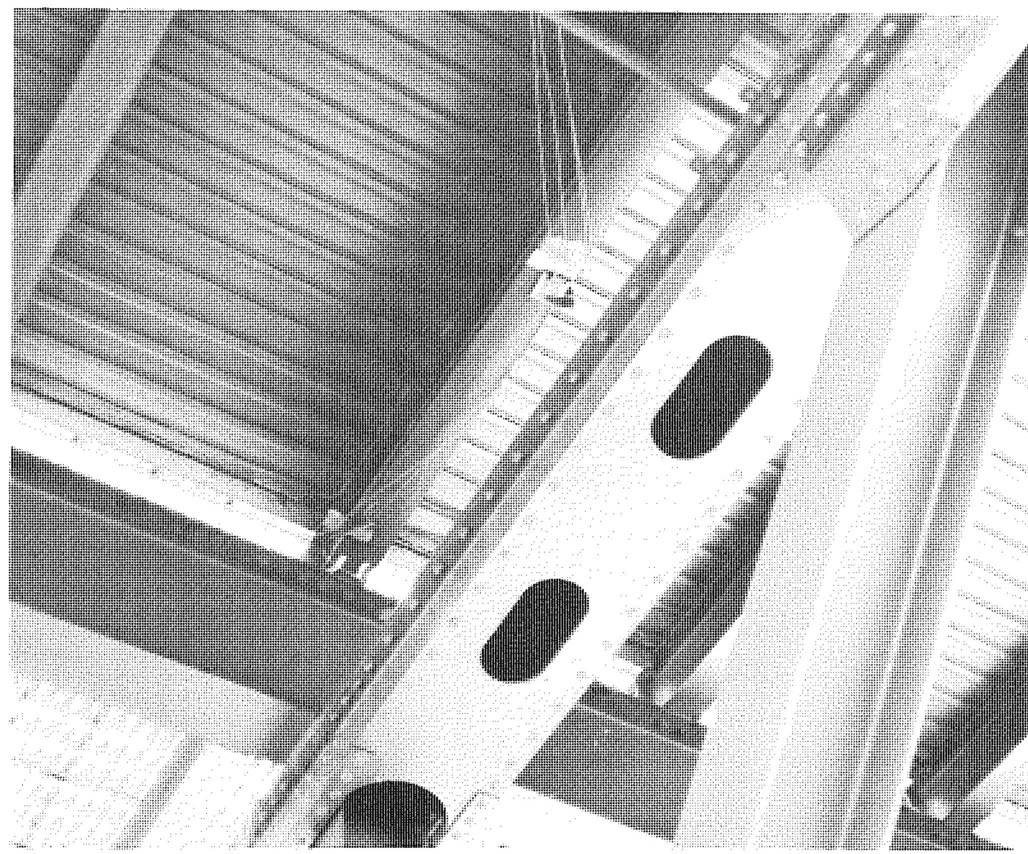


Figure 6. Drawing of strain gage as used on bridge. Micro-measurements Type EA-06-250BF-350 gages were applied with M-Bond 200 adhesive and M-Coat D, B, and G waterproofing.

Figure 7. Gages and wires attached to a floor beam of the suspension bridge.



the gage is cemented to a beam or other structural member, the thin foil will stretch or compress just as does the member to which it is attached. Strain (lengthening or shortening) in steel is usually indicated in micro-inches per inch, (millionths of an inch of stretch or compression for each inch of length). For each micro (millionth) of an inch of strain in the steel, there is an accompanying and accurately predictable number of micro-ohms of electrical resistance change in the strain gage. When the gage is connected to an accurate electrical power supply and a precise electronic amplifier, the strain in the bridge can be determined. Stress in the member can then be determined from the formula for steel: (stress in lb/sq in.) divided by (strain in in./in.) = 29,000,000.

Strain gages require a smooth surface for bonding, so all paint, scale, and pits must be removed from the beam, and a nearly polished flat steel surface established. Surface preparation was done with a disc sander or body grinder, followed by hand work with fine-grit paper. The surface was then cleaned carefully and neutralized before the adhesive was applied. Strain gage adhesive is a special cement, apparently a selected high grade of the Eastman 910 quick-setting type. During gage application, bridge traffic was stopped to allow at least one full minute of curing time for the cement before the beams were loaded.

Once the gages were cemented in place, tiny wires were soldered to the gages to connect them together in a two or four-arm 'bridge.' Four conductor shielded cables were then run from each gage location to a centrally located spot on the bridge railing for attachment to the instruments. Gages and wiring on a floor beam of the suspension bridge are shown in Figure 7.

Strain gages were placed on the selected spans at the locations indicated in Figures 8, 9, and 10 for the two-span continuous, north approach; four-span continuous, north approach; and two-span continuous suspended spans, respectively. The figures also show the locations where steel samples were removed. Two active and two temperature compensating gages formed a four-arm strain gage bridge at location Nos. 1 through 43, while one active and one compensating gage formed two-arm strain gage bridges at locations 44 and 45. Temperature compensating gages were placed on steel blocks of approximately the same thickness as the beam flange, and these blocks were clamped to the flange near the active gages. Active gages were placed one-fourth the flange width in from either flange tip. Active gages at the ends of cover plates (locations 28 and 31) were placed 5 in. away from the cover plate on the beam flange.

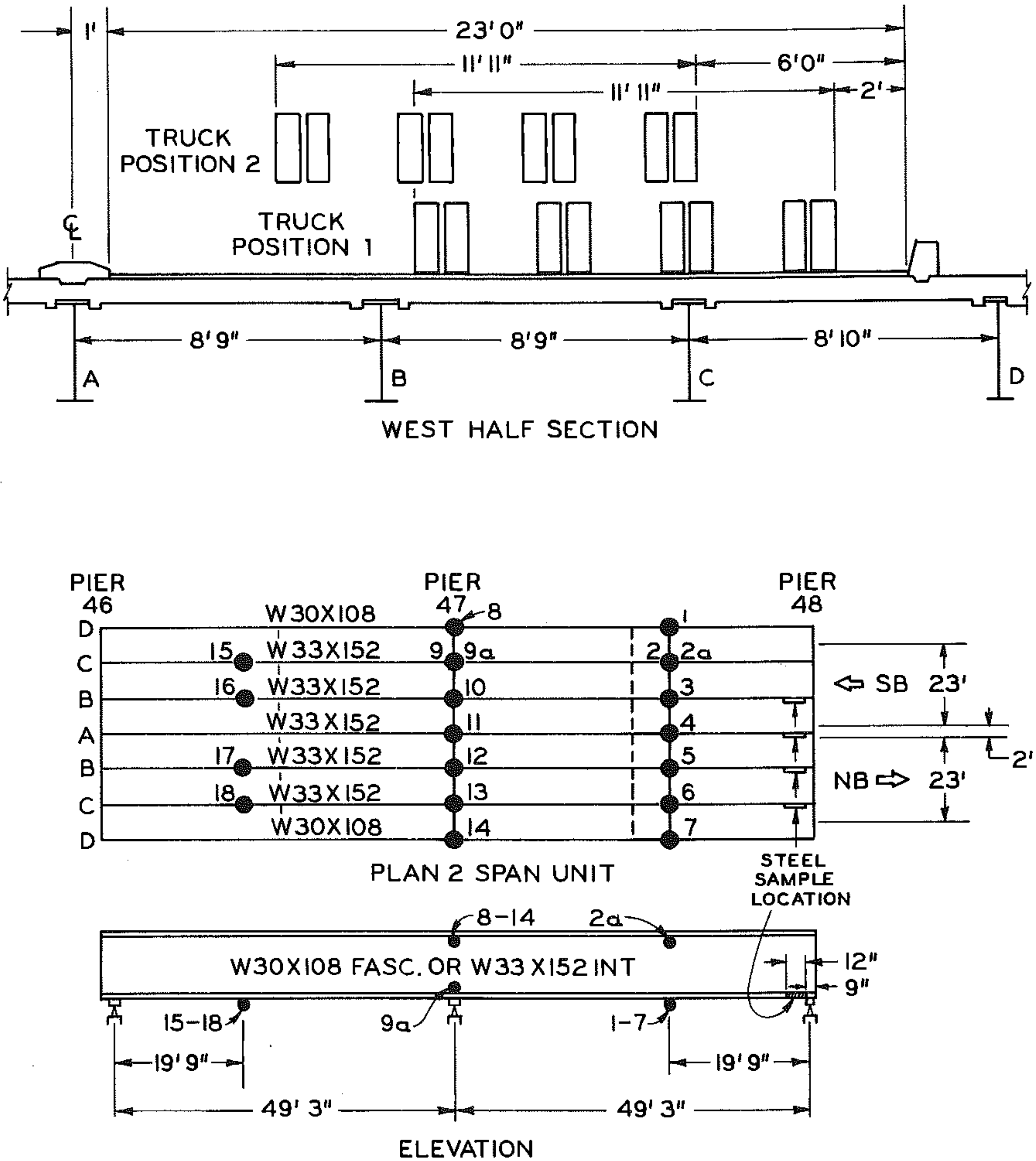


Figure 8. Strain gage and steel test sample locations for two-span continuous structure on north approach.

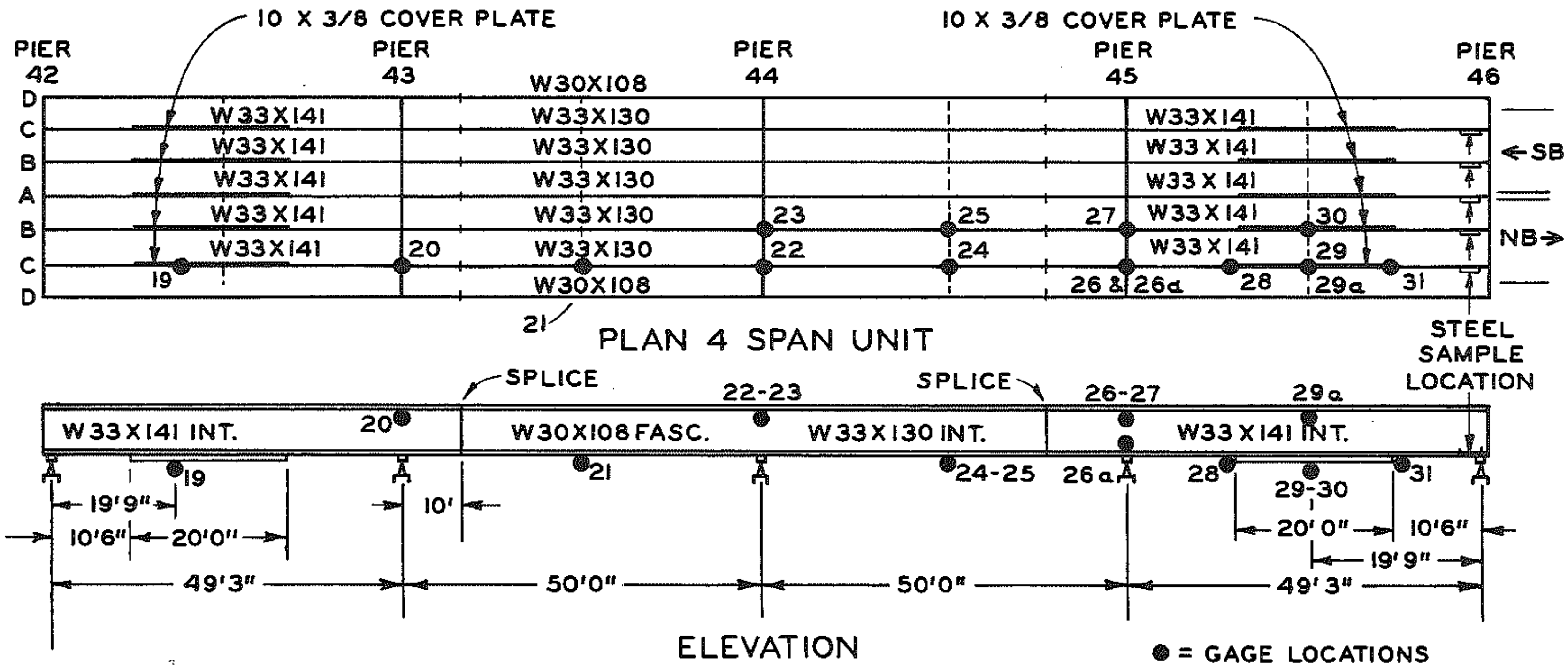


Figure 9. Strain gage and steel test sample locations for the four-span continuous structure on the north approach. Truck position, northbound, 2 ft from curb.

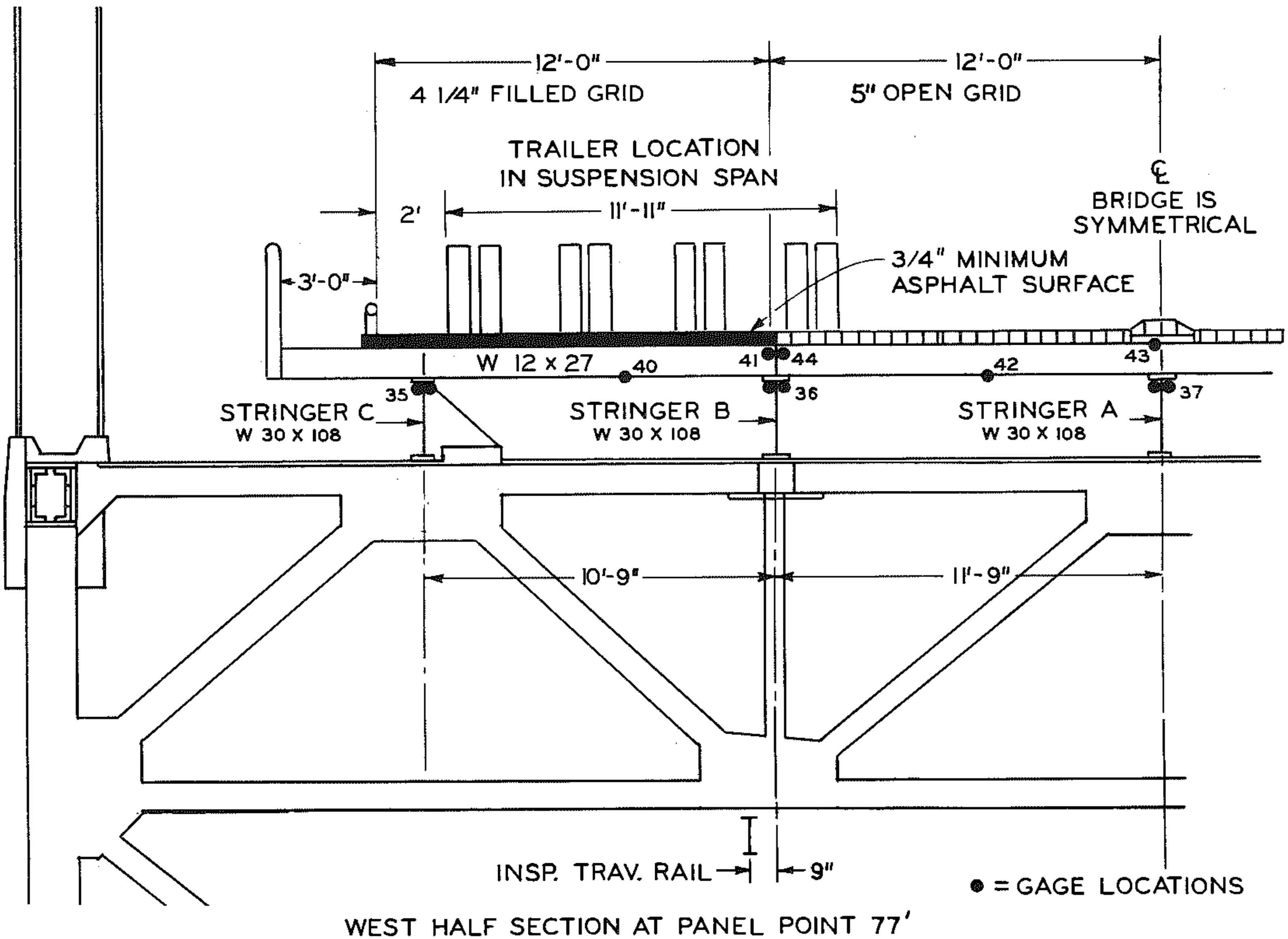


Figure 10(A). Stringers and cross-beams in suspension span.

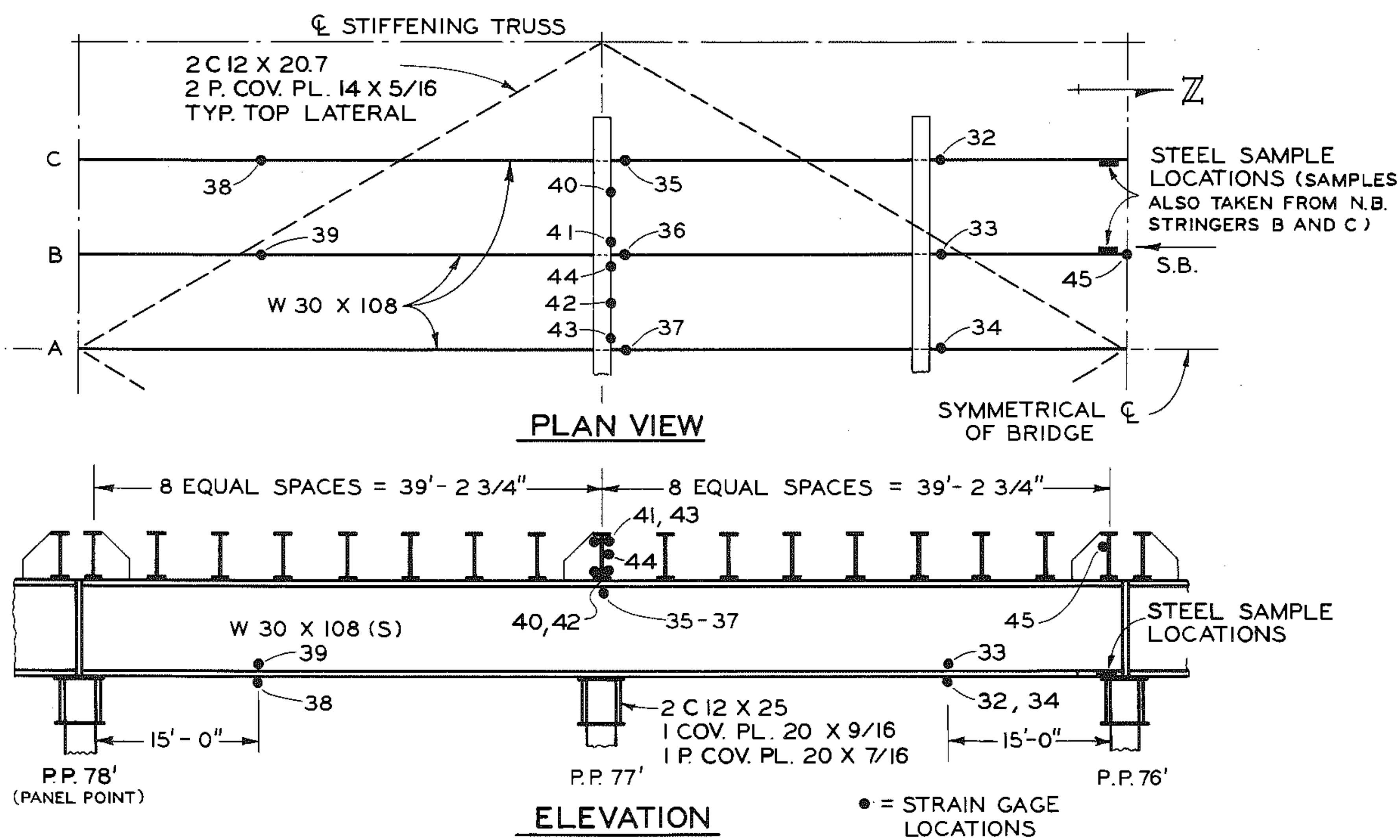


Figure 10(B). Stringers and cross-beams in suspension span.

Gages were Micro-Measurements Type EA-06-250BF-350, applied with M-Bond 200 adhesive, and weatherproofed with M-Coat Series D, B, and G materials, in order.

The electronic system was quite extensive; 14 data channels were available. Since the time frame for completion of the project was very limited and the rental cost of the loading rig was high, it was decided to use two separate, independent recording systems. A back-up system was then available in case of a malfunction of one of the components. Data were recorded on magnetic tape for both legs of the system: the more conventional leg of the system was analog, but the alternate leg was digital with on-board computer, control logic, analog-to-digital converter and multiplexer to sample, digitize, and store the data. Sampling was done at 50 and 100 Hz. Electric power was supplied by two portable motor generators. An oscilloscope, oscillograph, and x-y plotter were used to monitor the data at the site. The entire electronic system valued at approximately $40,000 was housed in a special 10-ft long box that was mounted to the guardrail at each of the three locations on the bridge (Fig. 11).

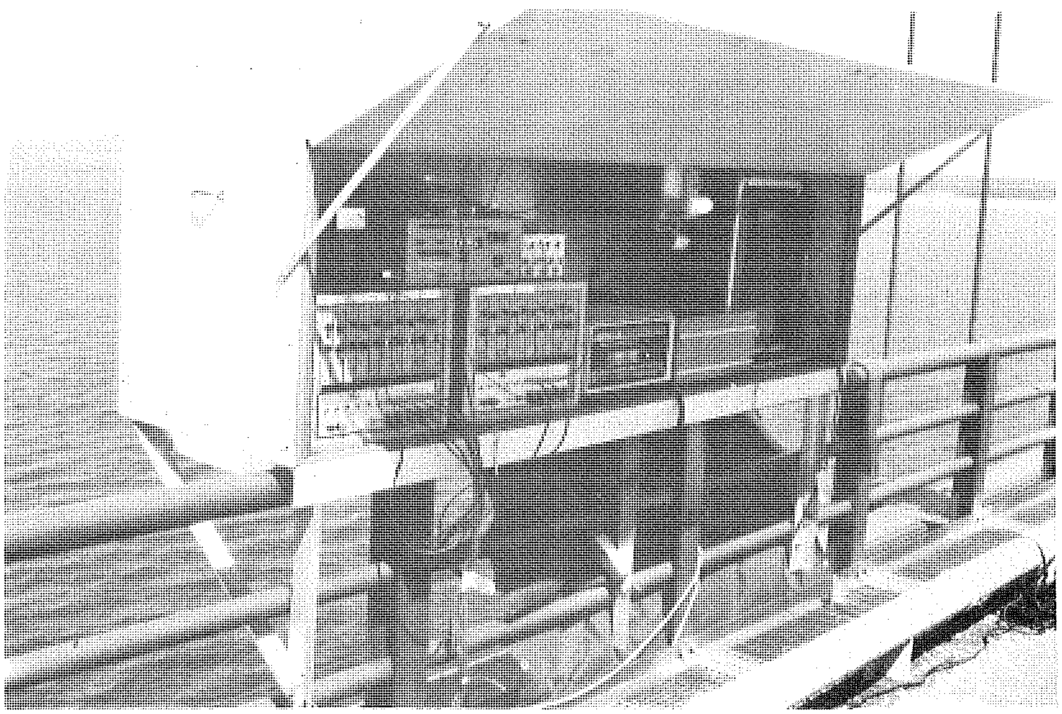


Figure 11. An instrument box, supported on the rail, contained the electronic gear used in the study. The picture shows the signal conditioning, amplifying, and control gear on the left; digital computer in the center; with the analog and digital magnetic tape recorders on the right. The two-channel direct writing oscillograph hangs from the top above the analog recorder.

Field Procedures

Work on the two-span and four-span structures at the north approach was quite readily accomplished, since these structures range from about 3 ft to less than 15 ft in height, and have firm, relatively level ground underneath. However, the suspended spans offer more of a challenge in this respect. Figure 12 shows a cross-section of the bridge, and the 'Maintenance Traveler' that is suspended within the truss for gaining access to various parts of the structure. The traveler is suspended on 32 wheels from a pair of I-beam tracks hung in the support trusses of the bridge; power is supplied by a motor generator on each traveler. (Several of these travelers exist within the suspended spans and heavy truss portions of the bridge.) Scaffolds were set up on a traveler in order to reach the floor members where gages were applied.

A special set of lightweight clip-together aluminum scaffolding aided greatly in accomplishing the many set-ups that were required high above the water in the suspended span. This equipment allowed all work below the deck on the suspended span to be completed in seven work days in spite of low temperatures and high winds.

The load vehicle travelled across the instrumented sections at creep and 10 mph speeds, with a few isolated runs at 20 mph. Bridge crossings by the rig during the experiment, were made at a nominal 10 mph. Commercial traffic was held at both ends of the bridge any time the load vehicle was making a crossing so that stresses due to another heavy vehicle were not added to the stresses caused by the experimental rig. Traffic control was provided by the Bridge Authority. All experimental strain measurements were made between midnight and 6:00 a.m. in order to coincide with the period of lowest traffic on the bridge. Work at night also aided in preventing 'zero drift,' variations in strain that result from rapid temperature changes, bright or intermittent sunshine. Three runs were made at each load position and speed whenever that was possible to do in the time available. Results included in the report are averages of the readings obtained.

RESULTS

Results of Stress Measurements and Calculations

In no location evaluated did measured stresses plus calculated dead load stresses exceed 54 percent of yield strength. Table 1 shows a summary of the data obtained from the experimental measurements as compared with calculated values. Details are given later in this report.

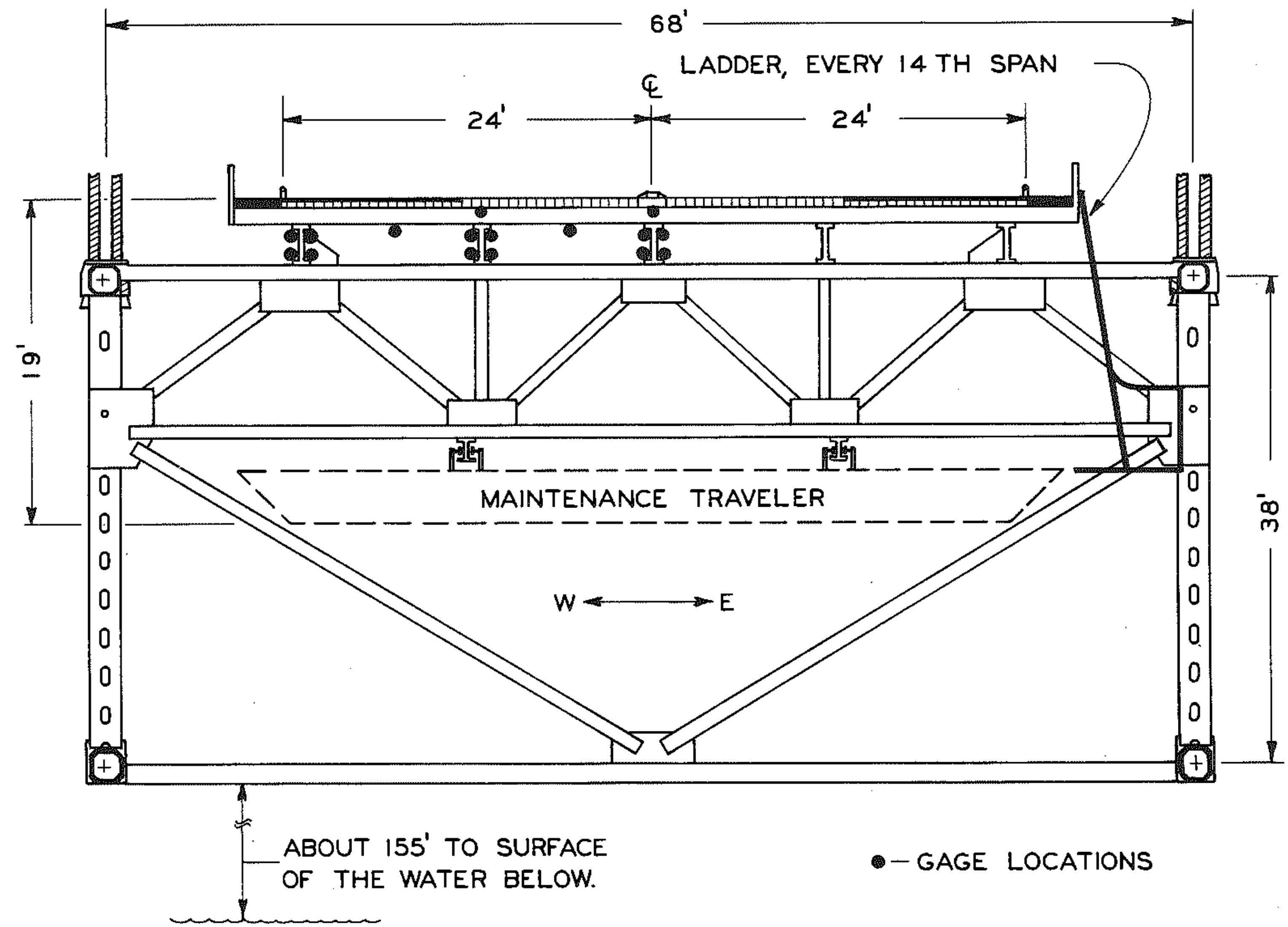


Figure 12. Cross-section of the suspension bridge showing the maintenance traveler.

TABLE 1
SUMMARY OF STRESS DATA
(Test Vehicle, Calculated vs. Experimentally Determined Stresses)

Location		Calculated	Experimentally Determined (includes dead load)	Measured Percent of Calculation	Measured Percent of Yield
Two-Span Non-Composite	Positive	22,000	12,300	56	37
	Negative	25,900	17,700	68	54
Four-Span Non-Composite	Positive	20,100	11,500	57	35
	Negative	25,800	17,500	68	53
Suspension Span Stringer B	Positive	25,900	10,300	40	23
	Negative	30,600	12,100	40	27
Suspension Span Stringer C	Positive	23,600	13,000	56	29
	Negative	25,700	15,100	59	34
Suspension Span Cross Beam	Positive	12,800	8,000	63	24
	Negative	10,100	1,900	19	6
Two-Span Partial Composite	Positive	18,100	12,300	68	--
	Negative	25,200	17,700	70	--
Four-Span Partial Composite	Positive	16,400	11,500	70	--
	Negative	24,900	17,600	70	--
End of Coverplate	Positive 1.6	15,600	11,600	75	35
End of Coverplate	Positive 1.2	14,000	10,300	74	31

The greatest calculated total stress for the 249,000-lb vehicle on the approach spans was on the two-span structure, amounting to 25,900 psi, or 79 percent of the yield stress if a non-composite deck is assumed. Measured stress plus calculated dead load stress at the same location was 17,700 psi which is only 54 percent of yield stress, and well below the recommended limit at 65 percent of yield stress. Additional calculations assuming partial composite action gave somewhat better correlation with the experimental results in the positive moment area (68 percent vs. 56 percent).

The measured distribution of loads is about 37 percent better than that used in the design calculations for positive moment, and 16 percent better for negative moment. Even assuming partial composite action, the measured values only amount to 79 to 84 percent of the total calculated positive moment in the two-span structure.

The greatest calculated total stress for the floor system of the suspended span was 30,600 psi or 68 percent of yield strength, while the measured stress at that location was only 27 percent of yield. Design calculations assume that the stringers are unsupported between transverse truss locations. However, the two stringers under each roadway rest upon heavy diagonal truss bracing members that aid in supporting the applied loads. The data also show evidence of interaction of the stringers with the stiffening truss.

The proposed 12-axle hydraulic suspension rail freight car transporter (PRT) would cause live load stresses only 73 percent of those caused by the experimental load vehicle (TV 2) and the total calculated stress for the PRT would be below 65 percent of yield in the worst case. As shown in the experiment for other types of vehicles, actual stresses under the proposed transporter would be lower than the calculated values.

Calculated total stresses in the suspension bridge for the proposed transporter are lower than for the legal 11-axle, 77-ton commercial vehicles now using the bridge. The greatest calculated stress for the proposed transporter is 52 percent of yield compared to 66 percent of yield for the 77-ton commercial vehicle.

Another factor that affects the comparisons is the magnitude of the conventional design impact factor at nearly 30 percent. Since the extremely heavy vehicle was required to cross the bridge at very low speeds, dynamic effects on the measured stresses are quite low, causing some additional difference between the measured and calculated values.

Strength of Steel

Figures 8, 9, and 10 show the location of steel specimens removed from the beams for determination of physical and chemical properties. Evaluation of the specimens gave the results shown in Table 2. Steel in the two-span and four-span structures was specified to be ASTM A 7, which at the time of bridge design was a fairly typical structural grade of steel with 33,000 psi minimum yield strength. However, the 30-in. beams in the suspended spans were specified to be ASTM A 94, a higher strength 'silicon' steel, to provide the needed strength with less weight. Comparison of the experimental results with the specified values listed in the table, shows that all specimens were well within the chemical and physical requirements of the specifications. In addition, all of the ASTM A 7 specimens evaluated also meet the requirement for the higher strength, more modern ASTM A 36 specification, with the single exception of specimen 4-5 having a carbon content 0.02 percent above the allowable maximum.

TABLE 2

RESULTS OF PHYSICAL AND CHEMICAL ANALYSIS OF STEEL SPECIMENS REMOVED FROM THE BRIDGE, AT LOCATIONS SHOWN IN FIGURES 8, 9, AND 10

	Sample No.	Location of Sample	Chemical Composition					Mechanical Properties				
			C	Mn	P	S	Si	Yield Strength, psi	Ultimate Strength, psi	Reduction of Area, percent	Elongation 2-in. Gage, percent	Average Charpy Impact Values, ft-lb at 40 F
Two-Span Continuous Pier 46 to Abutment 48	ASTM A7	Specification Requirement for the bridge	--	--	1	2	--	33,000 min	60,000 to 72,000	--	24 min	—
	ASTM A 36	Modern specification for comparison	0.26 max	--	0.04 max	0.05 max	--	36,000 min	58,000 to 80,000	--	21 min	--
	2 - 2	North Abutment, beam B west, east bottom flange	0.23	0.77	0.006	0.028	--	39,000	65,500	62	40	24
	2 - 3	North Abutment, beam A west bottom flange	0.20	0.77	0.006	0.025	--	36,500	66,000	64	44	30
	2 - 4	North Abutment, beam B east, east bottom flange	0.19	0.75	0.007	0.024	--	49,500	67,500	62	30	23
	2 - 5	North Abutment, beam C east, east bottom flange	0.21	0.81	0.007	0.023	--	39,000	66,000	64	40	19
Four-Span Continuous Piers 42 through 46	4 - 1	North End (Pier 46), beam C west, east bottom flange	0.18	0.77	0.010	0.029	--	42,000	65,500	64	38	20
	4 - 2	North End (Pier 46), beam B west, east bottom flange	0.17	0.79	0.010	0.025	--	41,000	63,500	64	39	29
	4 - 3	North End (Pier 46), beam A west bottom flange	0.19	0.79	0.010	0.002	--	42,500	64,500	64	39	35
	4 - 4	North End (Pier 46), beam B east, east bottom flange	0.24	0.75	0.010	0.033	--	40,500	63,500	64	39	17
	4 - 5	North End (Pier 46), beam C east, east bottom flange	0.28	0.79	0.010	0.031	--	42,000	63,500	64	36	17
Suspended Spans P. P. 76' through 78'	ASTM A95-54	Specification Requirement for the bridge	0.44 max	--	3	0.063 max	0.18 min	45,000 min	80,000 to 95,000	30 min	19 min	--
	S - 1	P. P. 76, beam C west, east bottom flange	0.36	1.05	0.018	0.029	0.29	58,500	92,500	57	28	18
	S - 2	P. P. 76, beam B west, west bottom flange	0.38	0.93	0.010	0.038	0.26	55,500	88,500	57	30	25
	S - 3	P. P. 76, beam B east, west bottom flange	0.38	0.95	0.010	0.034	0.26	56,000	88,500	56	31	13
	S - 4	P. P. 76, beam C east, west bottom flange	0.34	0.89	0.015	0.035	0.24	52,000	87,000	57	33	24

[1] Maximum percent open hearth or electric furnace: acid 0.075, basic 0.05; Bessemer acid 0.138.
[2] Maximum percent open hearth or electric furnace 0.063.
[3] Maximum acid 0.075, basic 0.05.

Tensile properties for new steel beams are determined from specimens taken out of the web of the section; whereas, the specimens evaluated in this project come from the toe of the flange. Limited experiments in the Research Laboratory have shown strength of the toe of the flange to be comparable to that in the web. However, the yield strength in the flange decreased from the toe toward the web, and was about 10 percent lower in the flange beneath the web than at the toe.

The initial location of yielding under load would be on the flange surface near the center. Therefore, it seems prudent to decrease the measured values by 10 percent when making calculations of bridge overload capacity based on samples from the tips of the flanges. Corrections of this magnitude applied to the controlling spans (the two-span continuous structure on the north approach) result in yield strengths approximately equal to the minimum required by specification, namely 33,000 psi. Similar correction would give about 36,000 psi for the four-span, and 47,000 psi for the suspended span beams. Note that the special silicon steel specified for the suspended spans was required to have a yield strength of 45,000 psi. Since there are so very many spans in the suspension bridge, and the sample so small, no calculated loads were based on a yield stress above the 45,000 psi minimum specified for that portion of the bridge. This table shows the relatively high values of ductility (elongation) obtained; all considerably above the specified minimum values.

Charpy impact values are included for information only, since they were not required by the bridge specifications. With the exception of one set of specimens from the suspended span, all Charpy results exceed the current AASHTO requirement of 15 ft-lb at 40 F, for redundant (multiple load-path) bridge members.

Comments by Gaston Arango of Steinman, Boynton, Gronquist, & Birdsall on the general conclusions of this report are presented in the Appendix.

II

TECHNICAL INFORMATION

TECHNICAL INFORMATION

Two-Span Continuous Structure, North Approach

Lateral distributions of the measured stresses for the two-span structure are shown in Figure 13. The figure indicates lower peak values for the northbound runs (downhill on the 3 percent grade) as compared to the southbound runs. This is due in large part to the fact that the maximum stress occurs when only the front seven axles of the rig are on the structure. When the vehicle reverses direction for the northbound runs, the gages in question are only loaded by the front of the vehicle while the back part is on the other span. This also may be affected by the rougher approach on the southbound runs, possible effects from tractive effort or braking and the associated load shift due to grade, and the possibility that the concrete deck section may vary from one roadway to another. While deck section was not checked as a part of this project, previous experience has shown that deck thickness and reinforcement position may vary considerably in typical bridge deck construction.

Calculated and measured values are shown in Table 3. Note that the Department's computerized bridge analysis and rating system (BARS) provides values that are within a few percent of those calculated by the Steinman organization when the bridge was designed, (designated as "Plan" in the tables). Previous calculations had indicated this two-span structure to be the controlling one for vehicles similar to the experimental vehicle, where stresses would be the closest to allowable maximum values. Limiting values have previously been proposed at 65 percent of the yield stress, which in this case is 65 percent of 33,000 psi, or 21,450 psi. Allowable live load stresses for the experimental vehicle on this structure were determined to be 16,600 psi in positive bending in the span, and 12,900 psi in negative bending over the center pier. If the spans are considered to be non-composite, as designed, calculation of live load plus impact, positive and negative moment stresses for the experimental vehicle gives 17,100 and 17,400 psi, respectively. Comparison of these critical values with the measured stresses shown in the figure and tables, reveals that the measured stresses are slightly less than one-half the calculated values in positive moment and approximately two-thirds in negative moment.

Additional calculations assuming some composite action (with an assigned value of n = 30) would reduce the computed live load plus impact stresses in the positive and negative moment areas to 13,100 and 16,600, respectively. This brings the measured stress plus calculated dead load stress to about 70 percent of the calculated values; with the positive moment stress at 37 percent of yield and negative moment stress at 54 percent of yield. This 54 percent figure is the maximum percentage of yield stress measured in the experiment.

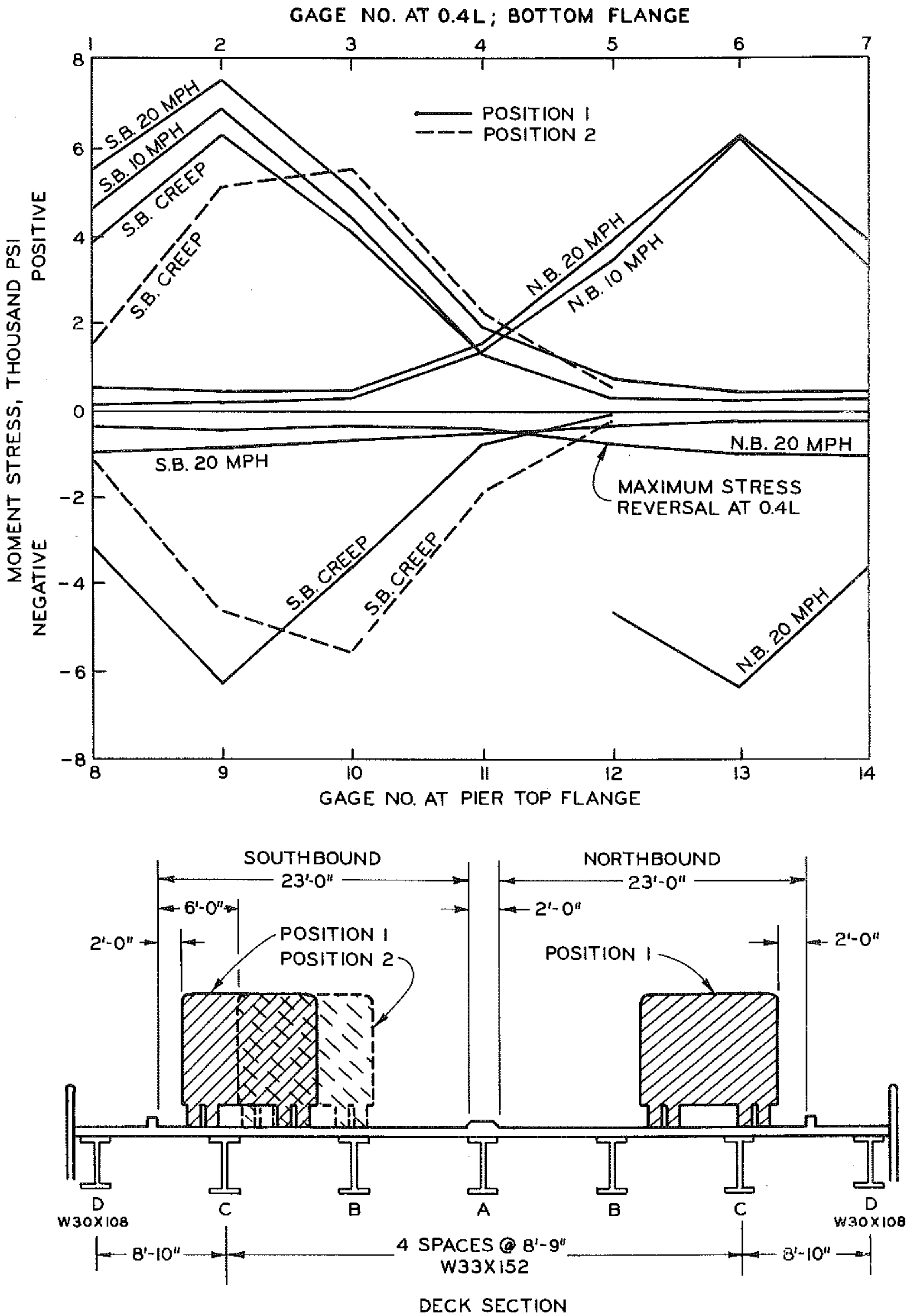


Figure 13. Lateral distribution of stresses in the two-span continuous structure on the north approach. Load vehicle is 11-axle, 249,000 lb.

TABLE 3
SUMMARY OF STRESS CALCULATIONS COMPARED TO YIELD AND EXPERIMENTAL DATA
(North Approach - Two-Span Continuous)

	Distribution Factor: Vehicle	Moment (Dead Load)/[1] Stress[2]	Moment (Live Load)/[1] Stress[2]	Moment (Impact)/[1] Stress[2] (Impact = 0.286)	Moment Total/[1] Stress Total[2]	Percent of Yield Stress (fy = 33,000)
	STRINGER C - NON-COMPOSITE (AS DESIGNED)					
Positive 1.4 W33 x 152, S = 486 gross, 478 net (cu in.)	S/10 (Plan moment): HS-20	198	431	123	752 18,900	57
	S/11 (MDOT BARS analysis): HS-20 (Inv. rating HS 20.7)	195 4,900	390	112	697 17,500	53
	S/14: 77-ton 11-axle commercial vehicle (Oper. rating 100.1T)	195 4,900	465	133	793 20,000	60
	S/17.4: TV 2	195 4,900	530 13,300	152 3,800	877 22,000	67
	S/17.4: PRT	195 4,900	320	91	606 15,200	46
	Test Results (TV 2)					
	Creep	4,900	6,300		11,200	34
	20 mph	4,900		7,400	12,300	37
	Percent of calculated		48	44	56	
Negative 2.0 W33 x 152, S = 486 gross, 478 net (cu in.)	S/10 (Plan moment): HS-20	-352	-303	-87	-742 18,600	56
	S/11 (MDOT BARS analysis): HS-20 (Inv. rating HS 20.2)	-344 8,600	-290	-83	-717 18,000	54
	S/14: 77-ton 11-axle commercial vehicle (Oper. rating 104.2T)	-344 8,600	-363	-104	-811 20,300	62
	S/17.4: TV 2	-344 8,600	-536 13,500	-153 3,900	-1,033 26,000	79
	S/17.4: PRT	-344 8,600	-391	-112	-847 21,200	64
	Test Results (TV 2)					
	Creep Top	8,600	7,000		15,600	47
	Creep Bottom		9,100		17,700	54
	Percent of calculated		67		68	

[1] Moment measured in ft-kips.
[2] Stress measured in psi.

TABLE 3 (Cont.)
SUMMARY OF STRESS CALCULATIONS COMPARED TO YIELD AND EXPERIMENTAL DATA
(North Approach - Two-Span Continuous)

	Distribution Factor: Vehicle	Moment (Dead Load)/[1] Stress[2]	Moment (Live Load)/[1] Stress[2]	Moment (Impact)/[1] Stress[2] (Impact = 0.286)	Moment Total/[1] Stress Total[2]	Percent of Yield Stress (fy = 33,000)
	STRINGER C - PARTIAL COMPOSITE - N = 30 (AS TESTED)					
Positive 1.4 W33 x 152, S = 628 gross, 478 net (cu in.)	S/11 (MDOT BARS analysis): HS-20	195 4,900	394 7,500	113 2,200	702 14,600	44
	S/14: 77-ton 11-axle commercial vehicle	195 4,900	472 9,000	135 2,600	802 16,500	50
	S/17.4: TV 2	195 4,900	536 10,200	153 2,900	884 18,000	55
	Test Results (TV 2)					
	Creep	4,900	6,300		11,200	34
	20 mph	4,900		7,400	12,300	37
	Percent of calculated		62	56	68	
Negative 2.0 W33 x 152, S = 628 gross, 478 net (cu in.)	S/11 (MDOT BARS analysis): HS-20	-344 8,600	-277 6,900	-79 2,000	-700 17,500	53
	S/14: 77-ton 11-axle commercial vehicle	-344 8,600	-347 8,700	-99 2,500	-790 19,800	60
	S/17.4: TV 2	-344 8,600	-512 12,900	-146 3,700	-1,002 25,200	76
	Test Results (TV 2)					
	Creep Top	8,600	7,000		15,600	47
	Creep Bottom	8,600	9,100		17,700	54
	Percent of calculated		71		70	

[1] Moment measured in ft-kips.
[2] Stress measured in psi.

Table 3 shows that the maximum calculated stress for the experimental vehicle is less than 30 percent higher than for 77-ton legal commercial vehicles, even though the experimental vehicle is more than 60 percent heavier. This is due to the greater distribution of the load on the deck. The table also shows that still greater distribution is obtained with the proposed railcar transporter (PRT, see Fig. 2), which is only 13,000 lb lighter than the experimental vehicle, but develops calculated stresses less than 5 percent above those for the 77-ton commercial vehicle.

Calculations were made to determine the distribution factor for the experimental vehicle and comparisons with the measured distribution were made. This Department normally uses a distribution factor of S/14 for calculations involving overloads on vehicles 8 ft wide, and makes a correction of [(width +8) divided by 16] in the denominator, for wider vehicles. Therefore, the 11-ft 11-in. width of the experimental vehicle gives a distribution factor of S/17.4. This value was used in the calculations of stresses for the experimental vehicle and the PRT shown in the tables.

Distribution factors (DF) from the experimental data were calculated from the sums of the maximum stresses in five beams multiplied by the beam spacing of 8-3/4 ft and divided by the maximum stress in the beam under consideration. Values of DF for positive moment stress varied from S/22.0 to S/25.7 and averaged S/23.9; which increases the distribution by 37 percent over the S/17.4 used in the calculations. Negative moment calculations for two different locations gave values of S/19.3 and S/21.2 for an average of S/20.2, which increases the distribution by 16 percent over the S/17.4 used in the calculations. Therefore, it is concluded that the Department's value of S/17.4 is a conservative value of distribution factor for this type of structure.

Figure 14 shows the values of stress that were measured at two locations to check on composite action in the positive and negative moment areas of this structure. The values of n, 32 for positive moment and more and 120 for negative moment (which means very little composite action in negative moment) are considerably different as is to be expected from the physical situation involved. Design considerations generally consider the deck to be cracked with no composite action present in negative moment areas.

Figure 15 shows the effects of increased speed, as well as the longitudinal distribution of stress in the beams, and the range of stress applied. In general, the figure shows the moderate increase in stress that occurs as speed is increased to 20 mph. This effect is demonstrated on the oscillograph traces by a higher frequency vibration superimposed on the main de-

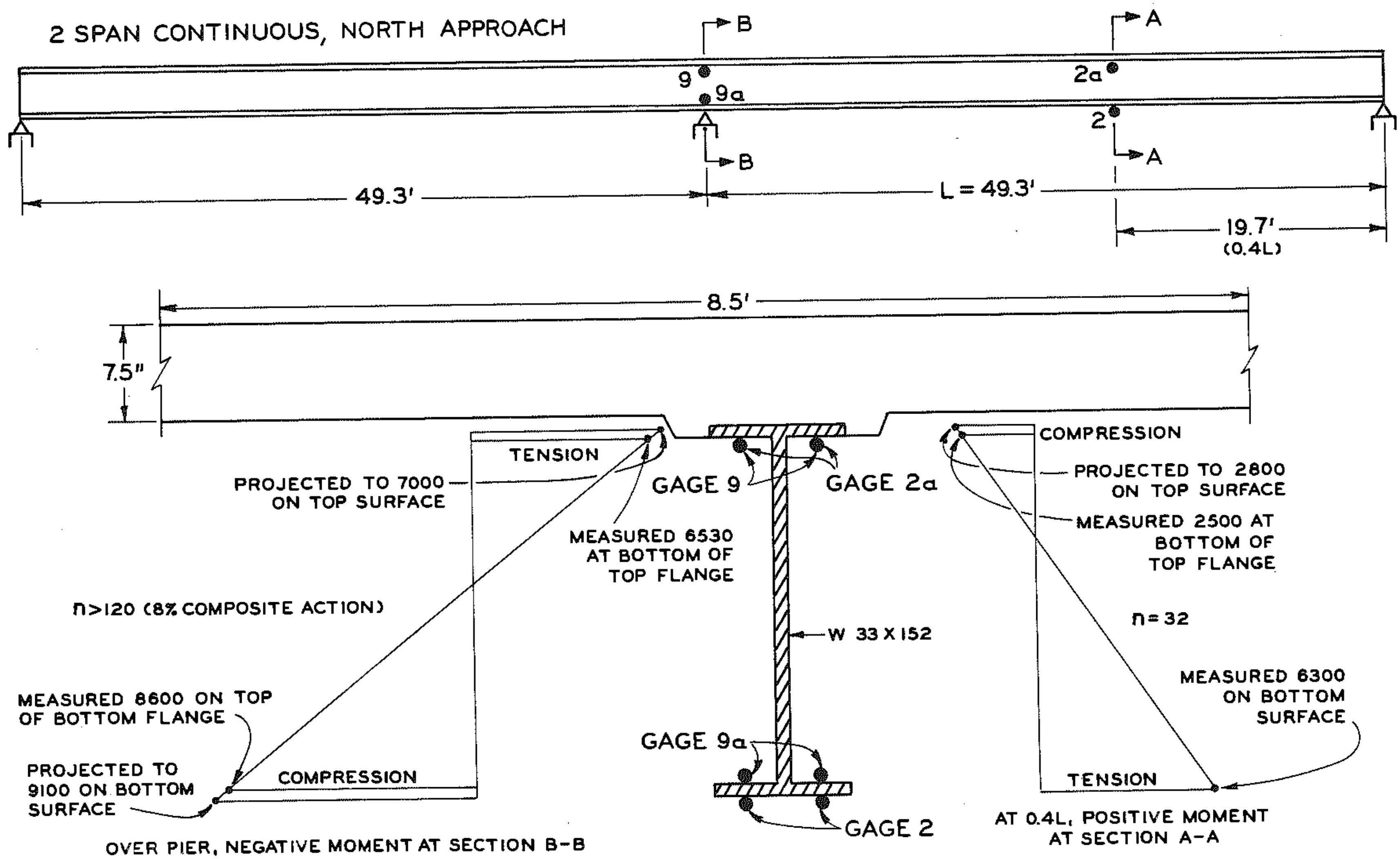
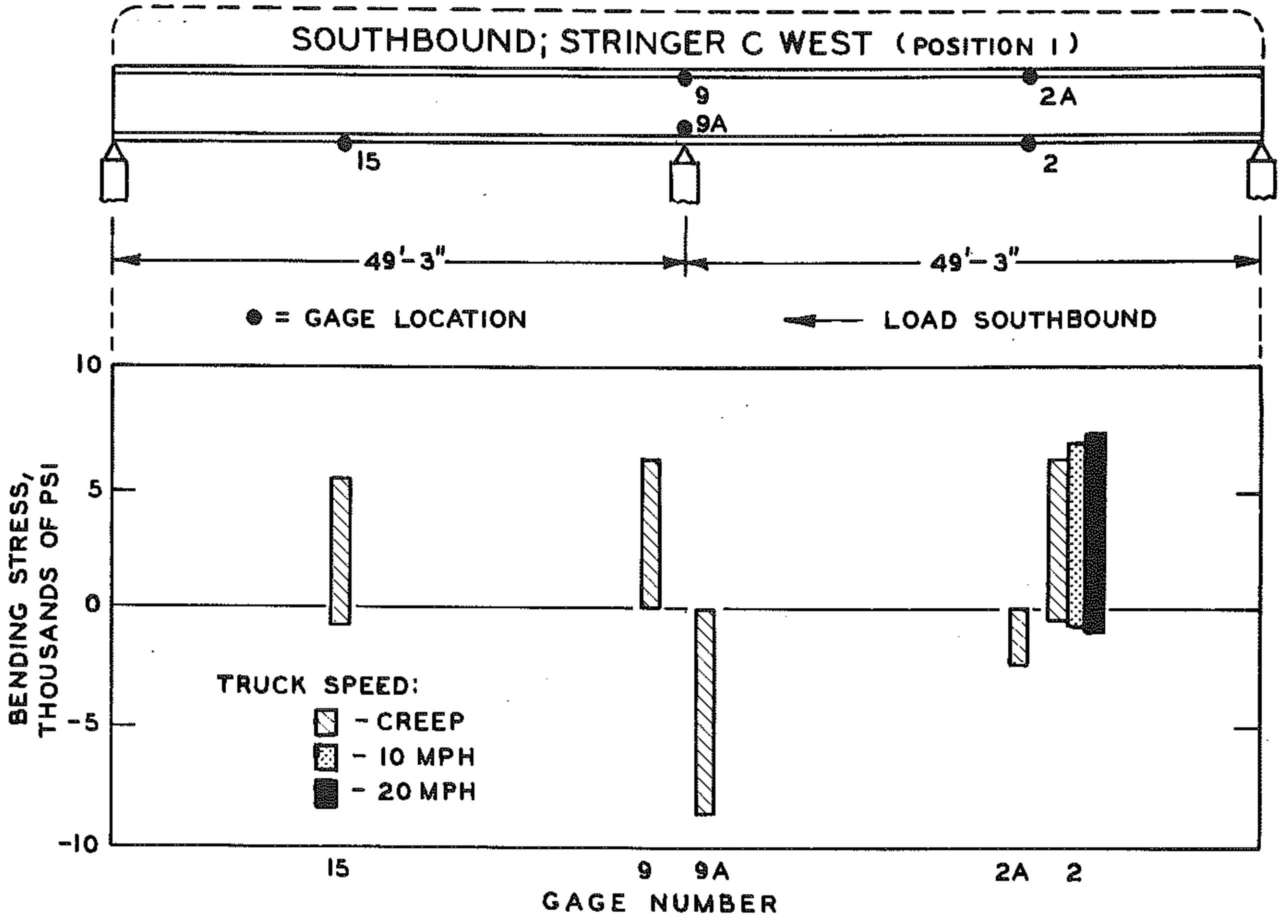


Figure 14. Stress measurements for composite action.



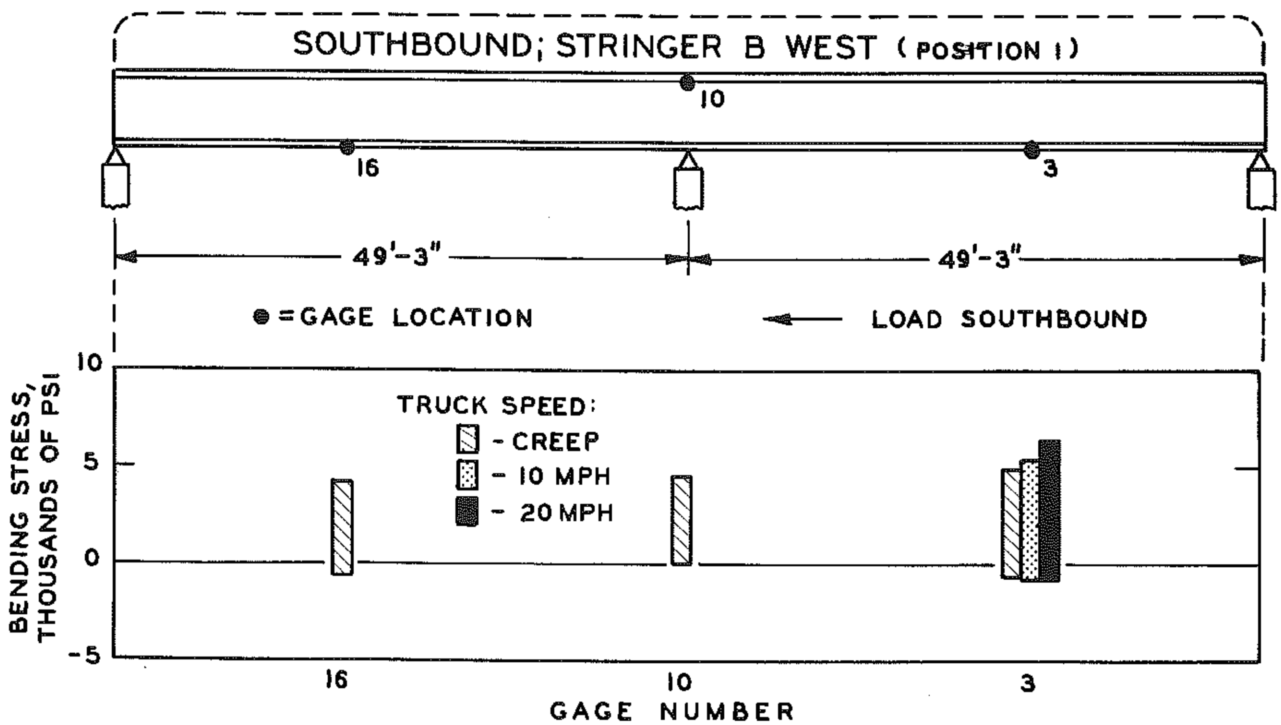


Figure 15. Longitudinal stress distribution and speed effects on two-span continuous structure on north approach.

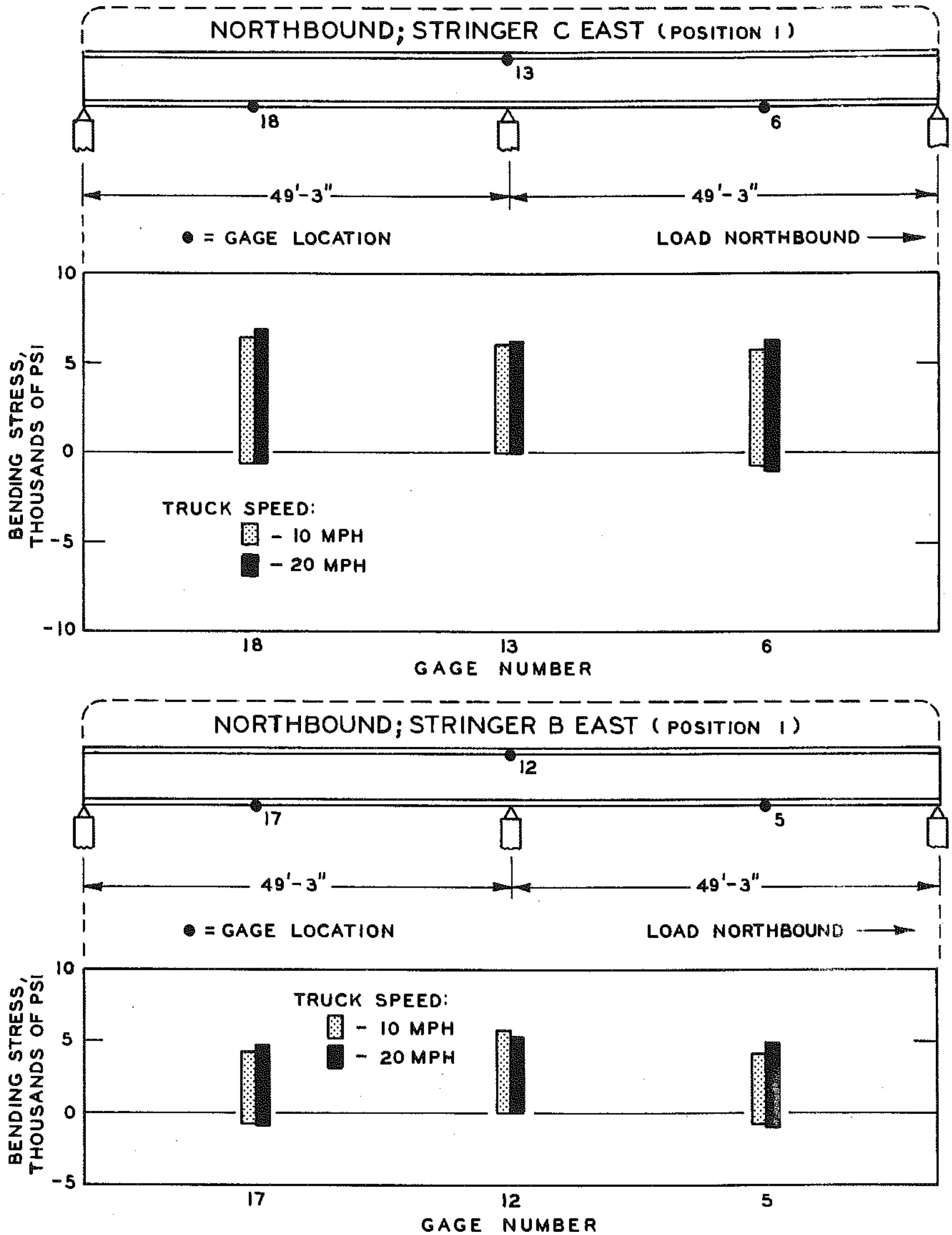


Figure 15 (Cont.) Longitudinal stress distribution and speed effects on two-span continuous structure on north approach.

flection curve. The single case of lower stress at higher speed (gage 12, stringer B east) is due to the higher frequency being directly out of phase with the deflection curve at the maximum excursion of the deflection curve. The design impact factor of 29 percent is quite conservative for the experimental vehicle at the relatively low speeds involved.

Since the tractor and four front trailer axles provide a heavier loading than the four rear trailer axles, and are on the lead span while the trailer is still on the approach, the lead span is stressed slightly more than the trailing span. Differences in stresses at symmetrical locations for northbound and southbound runs as mentioned above, are graphically illustrated in this figure.

Although initial calculations indicated that this two-span structure would be the controlling one for the experimental vehicle, the results detailed here have shown the actual stresses to be considerably lower than calculated. Based on previous experience, the values obtained are typical of results usually obtained for slab-on-beam multistringer bridges.

Four-Span Continuous Structure, North Approach

Table 4 shows stress calculations and experimental results for the four-span continuous structure. Here again, the BARS analysis is in excellent agreement with the original design. In general, the results are in a similar range, but numerically slightly lower than for the two-span structure discussed above. Similar comments would apply in this case, since the structures are very much alike except for the number of continuous spans.

Previous calculations had indicated that this structure would be the second most critical with respect to percentage of allowable maximum stress when loaded by the experimental vehicle. The more extensive calculations indicated here, and the measurements conducted, have indicated that preliminary evaluation was on target, and that the measured values were quite conservative when compared to the calculated stresses. As noted for the two-span structure above, such relatively low measured stresses are fairly typical for experiments with slab-on-beam multistringer bridges. Extrapolations of such conclusions to other types of bridges should not be made.

Figure 16 shows the range and distribution of stresses at several points spread longitudinally on the structure. Note that the effects of increased speed are not as precise as in the two-span structure. This is due to the interaction of the various spans, transmitting load and especially vibration,

TABLE 4

SUMMARY OF STRESS CALCULATIONS COMPARED TO YIELD AND EXPERIMENTAL DATA

(North Approach - Four-Span Continuous)

Section	Distribution Factor: Vehicle	Moment (Dead Load)/[1] Stress[2]	Moment (Live Load)/[1] Stress[2]	Moment (Impact)/[1] Stress[2] (Impact = 0.286)	Moment Total/[1] Stress Total[2]	Percent of Yield Stress (fy = 33,000)
	STRINGER C - NON-COMPOSITE (AS DESIGNED)					
Positive 1.4, W33 x 141 + two 3/8 x 10 PL, S = 562 gross, 543 net (cu in.)	S/10 (Plan moment): HS-20	216	426	122	764	
					16,900	51
	S/11 (MDOT BARS analysis):	220	395	113	728	
	HS-20 (Inv. rating HS 23.3)	4,900			16,100	49
	S/14: 77-ton 11-axle commercial	220	473	135	828	
	vehicle (Oper. rating 112.3T)	4,900			18,300	56
	S/17.4: TV 2	220	537	154	911	
		4,900	11,900	3,400	20,200	61
	Test Results (TV 2)					
	Creep	4,900	6,600		11,500	35
	20 mph	4,900		6,000	10,900	33
	Percent of calculated		55	40	54	
Negative 2.0, W33 x 141, S = 447 gross, 429 net (cu in.)	S/10 (Plan moment): HS-20	-305	-286	-82	-673	
					18,800	57
	S/11 (MDOT BARS analysis):	-286	-274	-78	-638	
	HS-20 (Inv. rating HS 20.2)	8,000			17,900	54
	S/14: 77-ton 11-axle commercial	-286	-347	-99	-732	
	vehicle (Oper. rating 101.9T)	8,000			20,500	62
	S/17.4: TV 2	-286	-495	-142	-923	
		8,000	13,800	4,000	25,800	78
	Test Results (TV 2)					
	10 mph Top	8,000	6,500		14,500	44
	10 mph Bottom	8,000	7,100		15,100	46
	20 mph Top	8,000		6,700	14,700	45
	20 mph Bottom	8,000		9,600	17,600	53
	Percent of Calculated Top		47	38		
	Percent of Calculated Bottom		52	54		
	At opposite end span, Top 10 mph	8,000	6,800		14,800	45
	At opposite end span, Top 20 mph	8,000		7,300	15,300	47

[1] Moment measured in ft-kips.
[2] Stress measured in psi.

TABLE 4 (Cont.)
SUMMARY OF STRESS CALCULATIONS COMPARED TO YIELD AND EXPERIMENTAL DATA
(North Approach - Four-Span Continuous)

Section	Distribution Factor: Vehicle	Moment (Dead Load)/[1] Stress[2]	Moment (Live Load)/[1] Stress[2]	Moment (Impact)/[1] Stress[2] (Impact = 0.286)	Moment Total/[1] Stress Total[2]	Percent of Yield Stress (fy = 33,000)
	STRINGER C - PARTIAL COMPOSITE - N = 14 (AS TESTED)					
Positive 1.4, W33 x 141 + two 3/8 x 10 PL, S = 723 gross, 543 net (cu in.)	S/11 (MDOT BARS analysis):	220	398	114	732	
	HS-20	4,900	6,600	1,900	13,400	41
	S/17.4: TV 2	220	542	155	917	
		4,900	9,000	2,600	16,500	50
	Test Results (TV 2)					
	Creep or 10 mph	4,900	5,700		10,600	32
	20 mph	4,900		6,600	11,500	35
	Percent of calculated		64	57	70	
Positive 1.6 (end of cover plate) W33 x 141 S = 633 gross, 447 net (cu in.)	S/11 (MDOT BARS analysis):	162	359	103	624	
	HS-20	4,300	6,800	2,000	13,100	40
	S/17.4: TV 2	162	461	132	755	
		4,300	8,700	2,500	15,500	47
	Test Results (TV 2)					
	Creep	4,300	6,600		10,900	33
	20 mph	4,300		7,300	11,600	35
	Percent of calculated		75	65	75	
Negative 2.0 W33 x 141, S = 447 gross, 429 net (cu in.)	S/11 (MDOT BARS analysis):	-286	-259	-74	-619	
	HS-20	8,000	7,300	2,100	17,300	53
	S/17.4: TV 2	-286	-471	-135	-892	
		8,000	13,200	3,800	24,900	76
	Test Results (TV 2)					
	Creep or 10 mph	8,000	7,100		15,100	46
	20 mph	8,000		9,600	17,600	53
	Percent of calculated		57	56	70	
Positive 2.5 W33 x 130, S = 542 gross, 388 net (cu in.)	S/11 (MDOT BARS analysis):	105	315	90	510	
	HS-20	3,300	7,000	2,000	12,200	37
	S/17.4: TV 2	105	359	103	567	
		3,300	7,900	2,300	13,500	41
	Test Results (TV 2)					
	Creep	3,300	6,500		9,800	30
	10 mph	3,300		6,900	10,200	31
	Percent of calculated		82	68	75	

[1] Moment measured in ft-kips.
[2] Stress measured in psi.

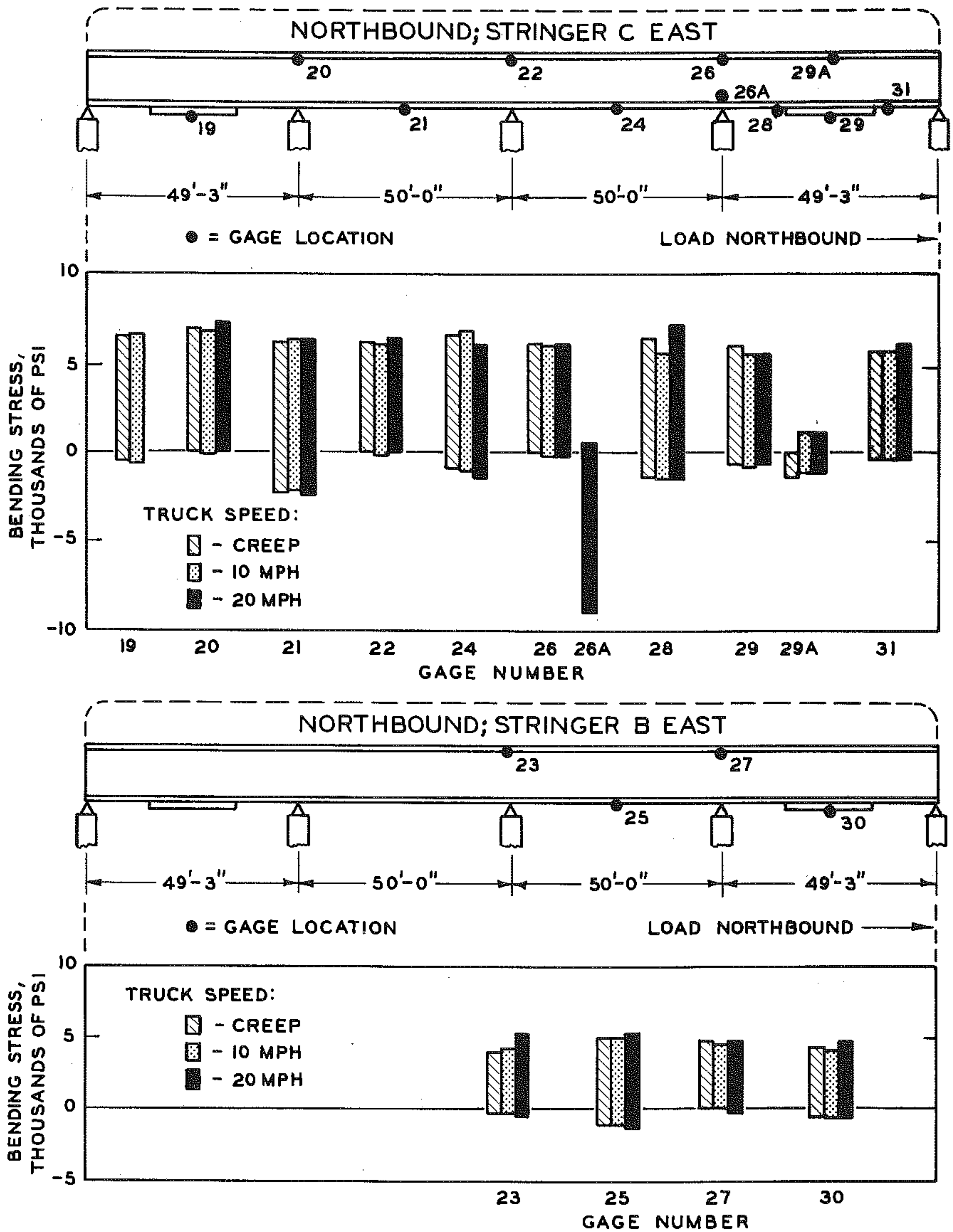


Figure 16. Longitudinal stress distribution and speed effects, four-span continuous structure on north approach.

from one span to the next. The different spans tend to vibrate at different phases and frequencies, and to affect one another through the continuous girders over the piers. Interaction of the vibration peaks can either tend to reinforce or cancel one another, and to beat in and out of phase. Since the spans are relatively light and short, and the load vehicle had many axles and traveled slowly, the vibrations died out quickly. Impact effects for the experimental vehicle were nominal, and considerably below the design values of 29 percent.

Main Suspended Span, Near the North One-Quarter Point, Two-Span Continuous Stringers

Table 5 summarizes the data collected on the suspension bridge, on the two-span continuous portion of the deck between panel points 76' and 78', near the north 1/4 point of the main span. The stringers at this location are typical of 46 of the 48 two-span units between towers (the two units adjacent to the towers are of slightly different design and would be stressed a little bit less than the others by vehicle loadings). Note that the stringers in the suspension bridge are made of a special silicon steel which has a yield strength of 45,000 psi, as opposed to the 33,000 psi yield strength of the cross-beams. The table indicates, again, the excellent agreement of the BARS analysis with the original bridge design.

The maximum calculated total stress under the experimental loading was the negative moment stress in stringer B which lies beneath the traffic lane, and amounted to 30,600 psi or 68 percent of the yield strength. The associated total positive moment stress was calculated to be 25,900 psi or 58 percent of the yield. Experimental results for these locations were considerably lower at 27 percent and 23 percent of yield, respectively, and only 40 percent of the calculated values. There appear to be three reasons for the relatively low measured stress values: 1) in most bridge structures there are many load paths that are not considered in the design, thereby causing greater distribution than assumed; 2) the four outside stringers bear upon the horizontal X-bracing in the top of the truss (Fig. 17 and refer back to Fig. 10). This support is not considered in the design of the stringers. Although the bracing members are not of large sections, the stress measurements show that there is a definite effect in reducing stringer stresses, especially at that location in stringer B since it is very close to the point of maximum moment; and, 3) there is interaction of the stringers with the truss, which tends also to reduce tensile stresses in the stringers. This factor will be discussed in greater detail later in the report.

Stringer C had calculated stresses slightly lower than B, but the gage placement for maximum positive moment was away from the support pro-

TABLE 5
SUMMARY OF STRESS CALCULATIONS COMPARED TO YIELD AND EXPERIMENTAL DATA
(Main Suspension Span at North 1/4 Point - Two-Span Continuous)

	Distribution Factor: Vehicle	Moment (Dead Load)/[1] Stress[2]	Moment (Live Load)/[1] Stress[2]	Moment (Impact)/[1] Stress[2] (Impact = 0.286)	Moment Total/[1] Stress Total[2]	Percent of Yield Stress (fy = 45,000)
	STRINGER B - NON-COMPOISTE (Silicon Steel)					
Positive 1.4 W30 x 108, S = 299 gross, 286 net (cu in.)	Plan moment: HS-20	70	384	115	569 23,800	53
	MDOT BARS analysis: HS-20 (Inv. rating HS 20.7)	70 2,900	386	116	572 24,000	53
	77-ton 11-axle commercial vehicle (Oper. rating 165.7T)	70 2,900	263	79	412 17,300	38
	TV 2	70 2,900	421 17,700	126 5,300	617 25,900	58
	PRT	70 2,900	245	73	388 16,300	36
	Test Results (TV 2)					
	Creep	2,900	7,400 (6,760 + 640)		10,300	23
	10 mph	2,900		7,400	10,300	23
	Percent of calculated		42	32	40	
Negative 2.0 W30 x 108, S = 299 gross, 256 net (cu in.)	Plan moment: HS-20	-124	-287	-86	-497 23,000	52
	MDOT BARS analysis: HS-20 (Inv. rating HS 21.3)	-123 5,800	-187	-86	-496 23,200	52
	77-ton 11-axle commercial vehicle (Oper. rating 148.4T)	-123 5,800	-239	-72	-434 20,300	45
	TV 2	-123 5,800	-407 19,100	-122 5,700	-652 30,600	68
	PRT	-123 5,800	-288	-86	-497 23,300	52
	Test Results (TV 2)					
	Creep	5,800	6,300 (5,700 + 600)		12,100	27
	10 mph	5,800		6,300	12,100	27
	Percent of calculated		33	26	40	

[1] Moment measured in ft-kips.
[2] Stress measured in psi.

TABLE 5 (Cont.)
SUMMARY OF STRESS CALCULATIONS COMPARED TO YIELD AND EXPERIMENTAL DATA
(Main Suspension Span at North 1/4 Point - Two-Span Continuous)

	Distribution Factor: Vehicle	Moment (Dead Load)/[1] Stress[2]	Moment (Live Load)/[1] Stress[2]	Moment (Impact)/[1] Stress[2] (Impact = 0.286)	Moment Total/[1] Stress Total[2]	Percent of Yield Stress (fy = 45,000)
	FASCIA STRINGER C - NON-COMPOSITE (Silicon Steel)					
Positive 1.4 W30 x 108, S = 299 gross, 244 net (cu in.)	Plan moment: HS-20	76 3,800	233	71	380 18,700	42
	MDOT BARS analysis: HS-20	76 3,800	235	70	381 18,800	42
	77-ton 11-axle commercial vehicle	76 3,800	395	118	589 29,000	65
	TV 2	76 3,800	309 15,200	92 4,600	477 23,600	52
	PRT	76 3,800	180	54	310 15,300	34
	Test Results (TV 2)					
	Creep	3,800	9,200		13,000	29
	10 mph	3,800		9,400	13,200	29
	Percent of calculated		61	48	56	
Negative 2.0 W30 x 108, S = 299 gross, 244 net (cu in.)	Plan moment HS-20	-135 6,700	-175	-52	-362 17,800	40
	MDOT BARS analysis· HS-20	-135 6,700	-175	-52	-362 17,800	40
	77-ton 11-axle commercial vehicle	-135 6,700	-358	-107	-600 29,500	66
	TV 2	-135 6,700	-299 14,700	-89 4,400	-523 25,800	57
	PRT	-135 6,700	-212	-64	-411 20,200	45
	Test Results (TV 2)					
	Creep	6,700	-8,300		15,000	32
	10 mph	6,700		-8,500	15,200	34
	Percent of calculated		57	45	59	

[1] Moment measured in ft-kips.
[2] Stress measured in psi.

TABLE 5 (Cont.)
SUMMARY OF STRESS CALCULATIONS COMPARED TO YIELD AND EXPERIMENTAL DATA
(Main Suspension Span at North 1/4 Point - Two-Span Continuous)

	Distribution Factor: Vehicle	Moment (Dead Load)/[1] Stress[2]	Moment (Live Load)/[1] Stress[2]	Moment (Impact)/[1] Stress[2] (Impact = 0.286)	Moment Total/[1] Stress Total[2]	Percent of Yield Stress (fy = 33,000)
	CROSS BEAM OVER TRANSVERSE TRUSS					
Positive 1.5 W12 x 27, S = 34.1 gross (cu in.)	Plan moment· HS-20	2 700	33	10	45 15,800	48
	MDOT analysis: HS-20 (Inv. rating HS 23.0)	2 700	32	10	44 15,500	47
	77-ton 11-axle commercial vehicle	2 700	24	7	33 11,500	35
	TV 2	2 700	27 9,300	8 2,800	37 12,800	39
	Test Results (TV 2)					
	Creep	700	7,300		8,000	24
	10 mph	700		7,300	8,000	24
	Percent of calculated		78	61	63	
Negative 2.0 W12 x 27, S = 34.1 gross (cu in.)	Plan moment: HS-20	-3 1,060	-37	-11	-51 18,000	54
	MDOT analysis: HS-20 (Inv. rating HS 20.0)	-3 1,060	-38	-11	-52 18,200	55
	77-ton 11-axle commercial vehicle	-3 1,060	-18	-6	-27 9,500	29
	TV 2	-3 1,060	-20 6,900	-6 2,100	-29 10,100	31
	Test Results (TV 2)					
	Creep	1,060	920		1,980	6
	10 mph	1,060		920	1,980	6
	Percent of calculated		13	10	20	

[1] Moment measured in ft-kips.
[2] Stress measured in psi.

vided by the X-bracing, so that measured stresses plus calculated dead load stresses were 56 to 59 percent of the calculated values, but still only 29 to 34 percent of the yield strength. The cross-beams had similar, relatively low stress values under the loading of the experimental vehicle.

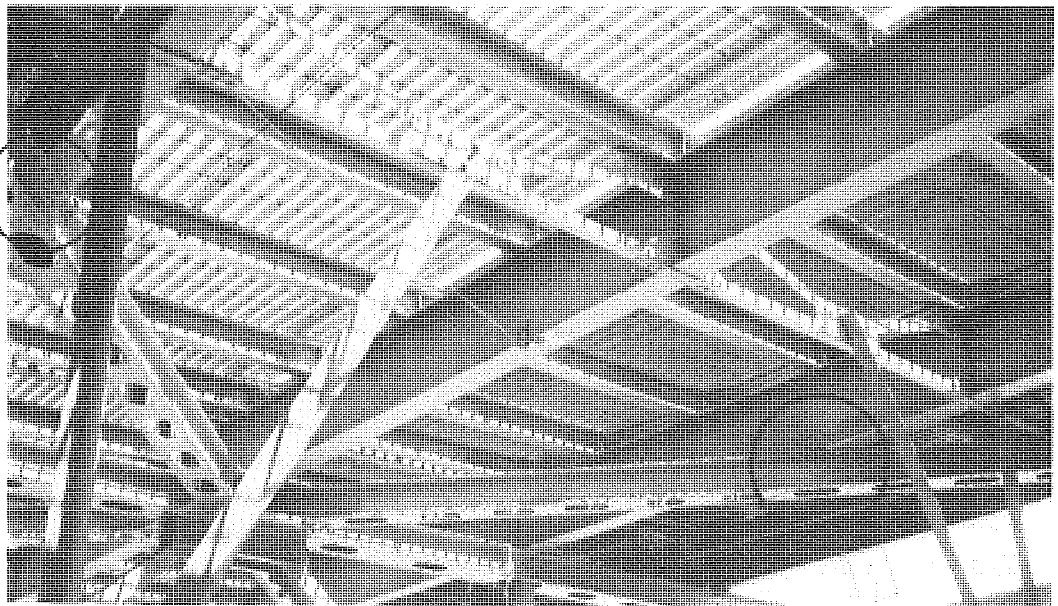


Figure 17. Horizontal X-bracing in the top of the truss supports stringers B near the point of maximum positive moment. The same bracing also supports the fascia stringers C at a symmetrical point near the center of the two-span unit.

The results shown for the legal 77-ton (Type 3S8), 11-axle commercial vehicle are of considerable interest, since for that vehicle the loading is heavier on the fascia stringer C, and is calculated to stress that stringer to approximately 65 percent of yield. This is higher than the calculated stress applied to that stringer by the experimental vehicle, and nearly as much as the calculated stress at maximum in stringer B due to the experimental vehicle. The 77-ton commercial vehicle is 4 ft narrower and considerably shorter than the experimental vehicle, and when run in the traffic lane, throws more load to the fascia stringer. Since these vehicles may use the bridge daily, it is of special interest to note that their effect on the stringer may be as great as the heavier experimental vehicle.

Also significant is the level of stress calculated for the proposed railcar transporter (PRT). The calculations indicate that the effect of the PRT at any location would be no more than 80 percent of that for the experimental vehicle at that same location. In addition, the maximum total stress calculated for the PRT is only 83 percent of the maximum for the 77-ton legal commercial vehicle. Although the experiment has shown that the actual stresses in the stringers would be somewhat lower than computed, the calculations give a good indication of the relative magnitudes of the applied bending moments which relate to the relative effects of the various vehicles.

Cable Shift and Deck-Truss Interaction on the Suspended Spans, and Effects of Commercial Vehicles

Only one of the two-span continuous units of the suspension bridge could be instrumented during this project. Many others, as previously mentioned, are practically identical in size and design. However, it is obvious that there could be differences in the tightness of fasteners that secure the stringers in place, and this could introduce some variation in the reaction of the units to indirect or secondary stresses discussed below. While these factors might cause the maximum stress to occur at a different location it does not appear that the maximum stress would be increased significantly.

Influence distances on the suspended spans are extremely long for heavy vehicles, as might be expected from the physical situation involved. Low amplitude vibrations can be felt at midspan due to the experimental vehicle or heavy trucks some 3,000 ft away on the tail span between the tower and the cable-bent support pier.

Although there was not much time in the schedule for monitoring the effects of commercial traffic on the bridge, it was possible to record data on the suspended span for a few hours. Also, a few additional data points were collected directly with the two-channel oscillograph, at times when the major instrument package was not available. Since these data are more closely related to the normal day-to-day operations of the bridge, they are presented here, even though they are not extensive or entirely definitive.

During nearly all of the time that commercial traffic data were recorded, the two outside lanes of the bridge were closed due to the operation of the painting contractor, and all traffic was on the open grating of the inner two roadways. Therefore, stringer A received a greater proportion of the direct or primary stress due to being subjected to nearby loadings and being unsupported by the X-bracing within the span. It also is subjected to the most indirect or secondary stress from cable shift and truss interactions, due to its framing into the bridge. Stringers B and C (west) showed a small amount of shift, but nowhere near as much as A. The heavy framing at the ends of stringer A, tying it in to the horizontal X-bracing, should account for much of the effect on that stringer.

In cases where several trucks traveled together, or when the experimental vehicle was in use, low amplitude strain gage output could be detected when the loads were beyond the towers. This occurred most noticeably on the centerline stringer (stringer A). As southbound loads passed the north tower, output from the gages on stringer A became increasingly negative (compressive) until the load actually moved onto the instrumented two-span unit. Then the tensile direct (primary) load stresses were superimposed over the compressive (secondary) values. When the load left the instrumented area and approached midspan the cable-effect stresses tended back to zero, and became tensile as the load passed the midspan of the suspension bridge. The tensile excursion then decreased as the loads reached the south tower. Slight effects from loads on the south sidespan of the suspension bridge could be determined sometimes, when other trucks were not near the instrumented site.

The experimental vehicle was run exclusively in the outer roadway, and therefore had much more direct effect on stringers C and B than A. However, stringer A still obtained nearly all of the indirect or secondary effects of load. In fact, under the experimental vehicle, the secondary compressive stress on gage 34, slightly exceeded the 'primary' tensile stress, so that the directly applied load did not quite bring the live load stress at that location back to zero. On the other hand, heavily loaded commercial traffic on the inside lane would give maximum compressive 'secondary' and maximum tensile 'primary' stresses of about equal magnitude. A few of the traces are included here to illustrate the situation. Figure 18 shows the strains at the positive and negative moment areas of stringer A for a heavily loaded commercial vehicle. Note that the tensile stress excursion would have been about twice as large if the secondary effects had not shifted so far into compression. Similarly, the compressive stress in the opposite flange of the beam would be increased by more than 1/3. Also note the 'blips' from smaller vehicles following the heavy one. Figure 19 shows the slightly different pattern from a 3S3-5 type vehicle where the tensile and compressive excursions are nearly identical. The effect of two heavy vehicles traveling together is shown in Figure 20. Comparison of these traces for gage 34 with those in Figure 19 for a similar vehicle alone, shows the additional downward (compressive) shift caused by the combined effects of the two loads on the secondary strain. The upper trace in Figure 20, gage 42 on a cross-beam, shows no zero shift from the secondary effects. Please note that the calibrations or 'scale' for the two traces in this figure are different from each other, and that both are different from the calibrations used in the other figures.

Figure 18. Trace from a truck type 3S8 loaded with steel. This was one of the heaviest trucks recorded and because it is somewhat shorter than the 3S3-5 type, it causes relatively high stress. Vehicle enters instrumented two-span unit at (a) and leaves at (b).

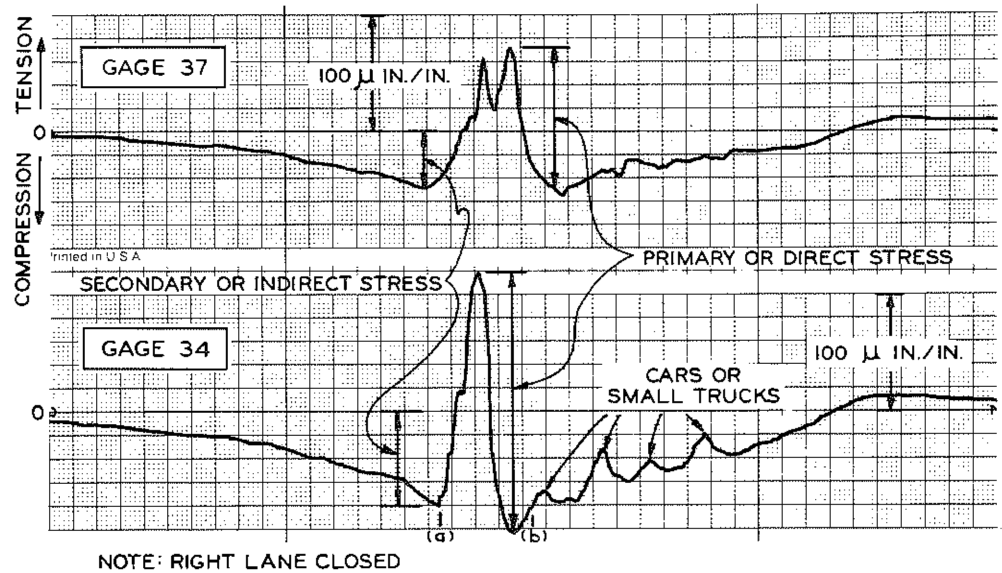
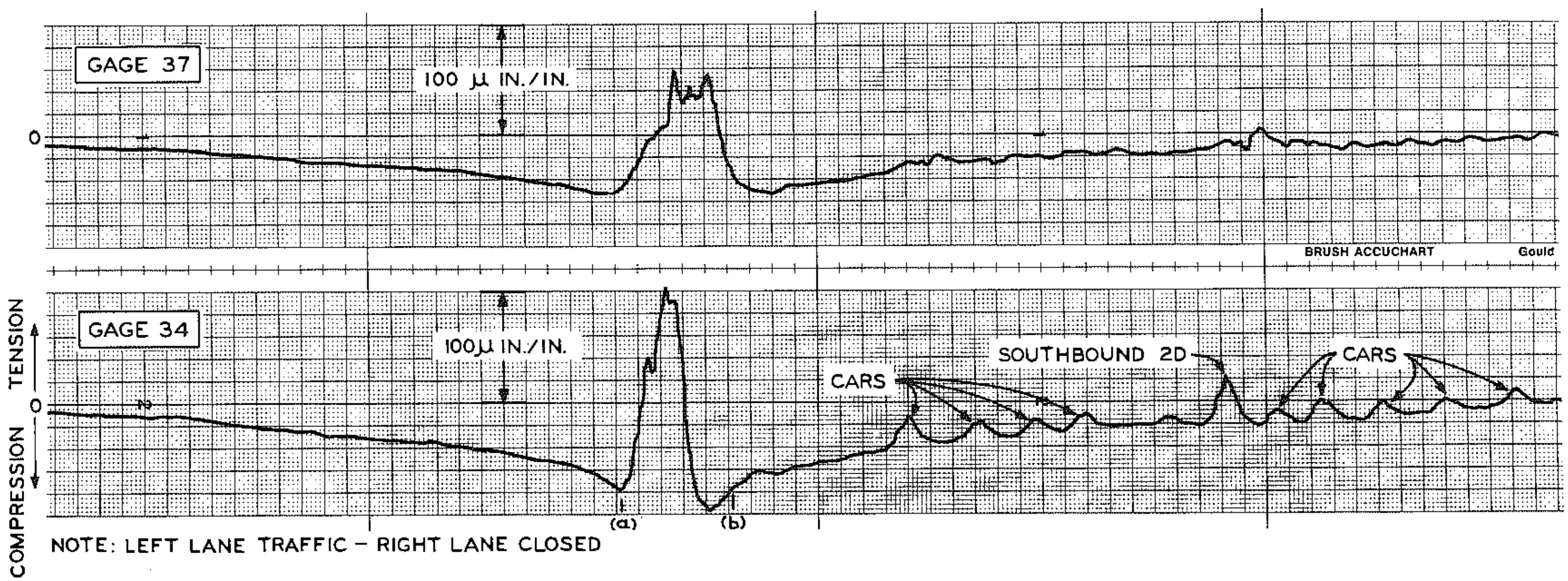


Figure 19. Trace from truck type 3S3-5 and several smaller vehicles, showing the positive and negative strain excursions of practically identical amounts.



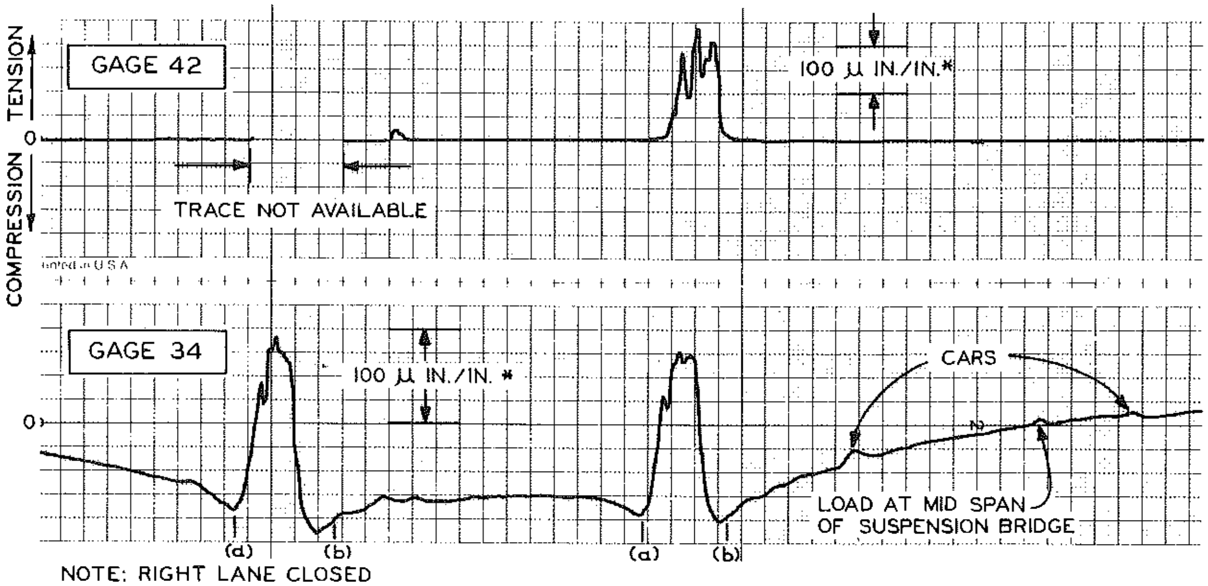


Figure 20. Trace from two gravel trains, type 3S3-5 traveling together. The superposition of the two vehicles shifts the maximum compressive secondary strain on gage 34 so that the live load compressive exceeds the live load tensile. The trace above, from gage 42 on the cross-beam shows no effect from the cable shift or truss interaction phenomena. (*) Note that the calibration for gage 34 on this trace is different from previous and following figures, and that calibration for gage 42 is still another value.

Figure 21 shows a comparison of strains from a 10-axle type 3S7 vehicle, with those from two 6-axle type 3S3 'permit loads.' Note the double and triple peaks for the 3S3 vehicles, due to the more concentrated application of the loads, and the considerable magnitudes of the loadings applied by the 6-axle permit vehicles in comparison with the 10 and 11-axle vehicles shown here and in the other figures. The more typical 3S2 (5-axle semi) traces are shown in Figure 22 for comparison, and Figure 23 shows results for 9 and 10-axle logging rigs that use the bridge quite regularly. Figure 24 indicates the net effect of many light vehicles moving across the bridge together. Their total weight on the suspension bridge is sufficient to cause the secondary strains to swing far enough below zero that the primary strains from a relatively small commercial vehicle do not cross the zero line. This figure shows also that there is a net secondary compressive live load strain in both gages 34 and 37 on stringer A. Gage 34 is a positive moment gage on the bottom flange and No. 37 is a negative moment gage on the top flange (refer back to Fig. 10 for location). Therefore, this strain pattern indicates a net thrust or compression in stringer A. Although this effect tends to decrease the magnitude of the tensile strains in gages

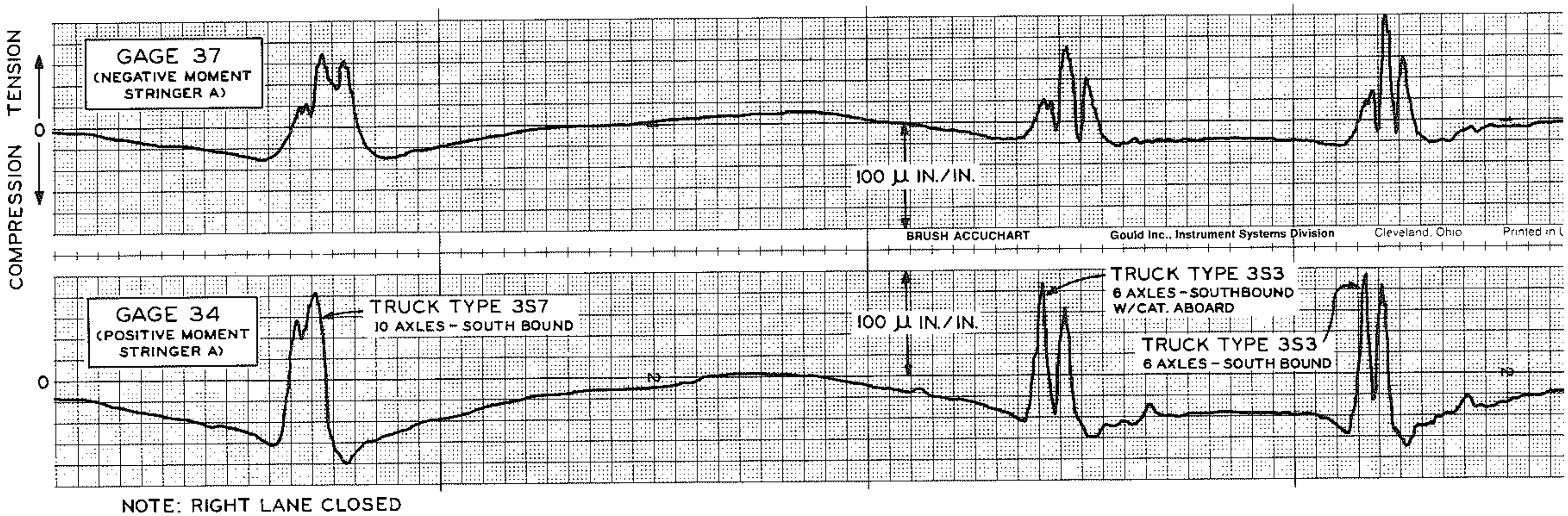


Figure 21. Comparison of traces from different truck types. Note that the tensile strains due to the six-axle trucks are greater than for the ten-axle truck, and that the shape of the trace is considerably different. Gross vehicle weights are unknown.

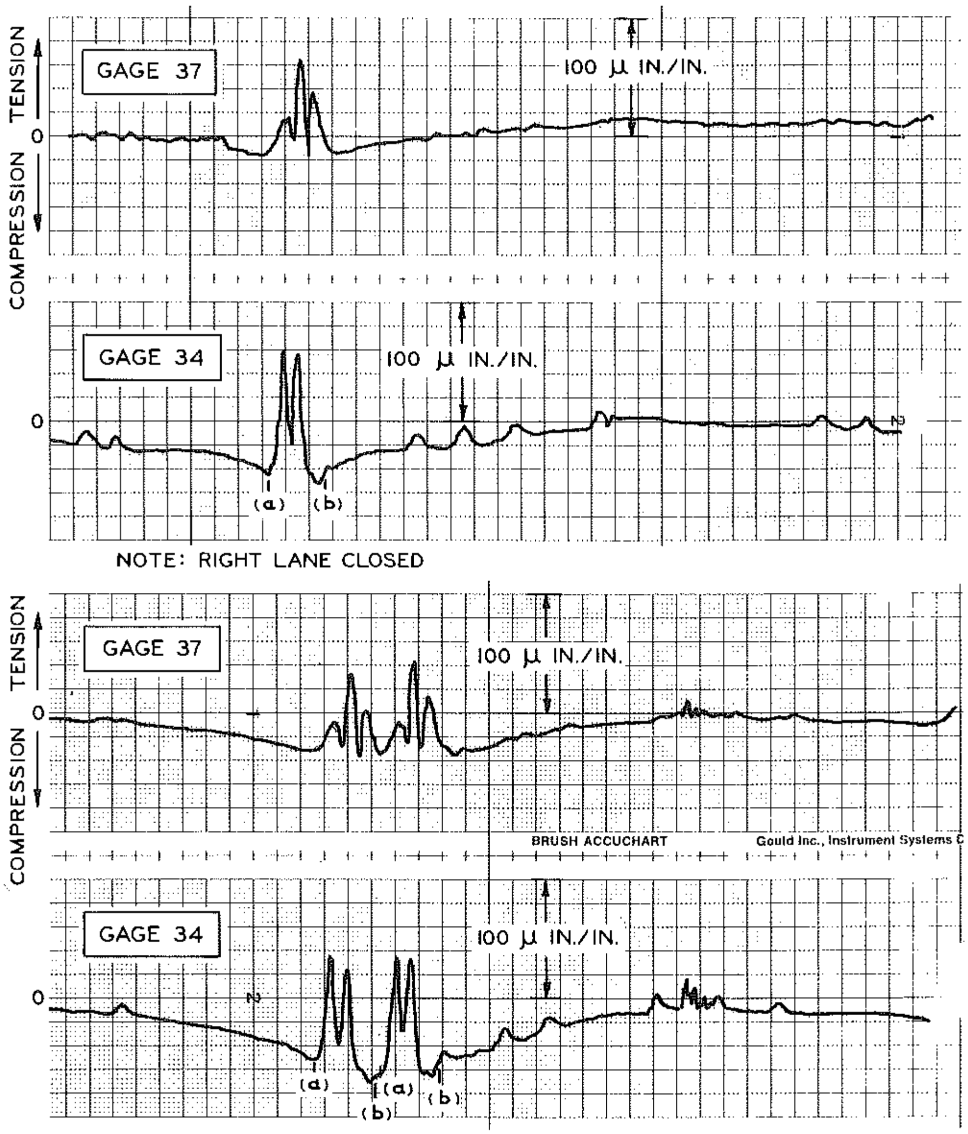


Figure 22. Typical traces for 3S2 type (five-axle semi) vehicles, loaded; singly and two together.

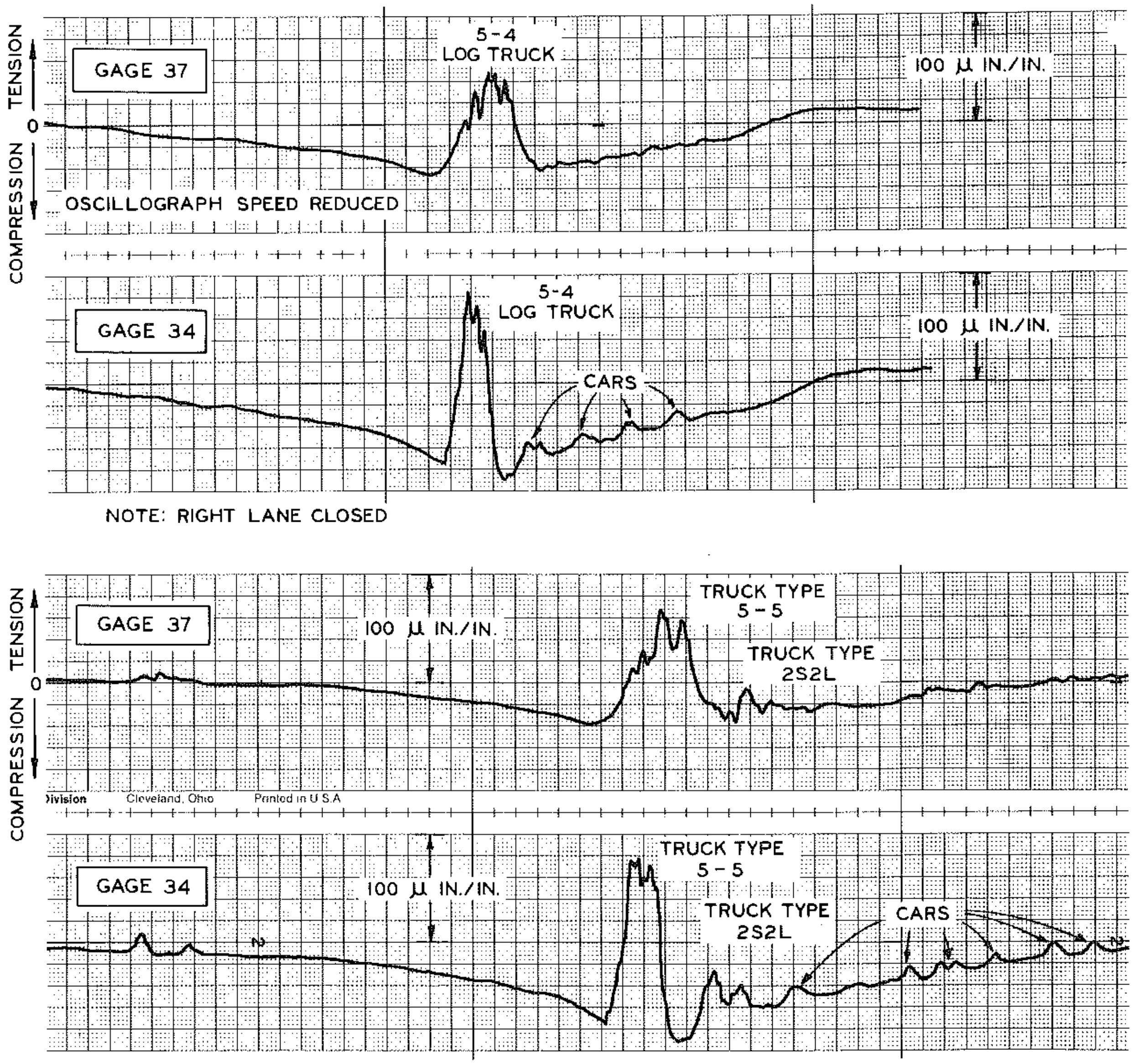


Figure 23. Typical traces for nine and ten-axle log and lumber trucks that use the bridge regularly.

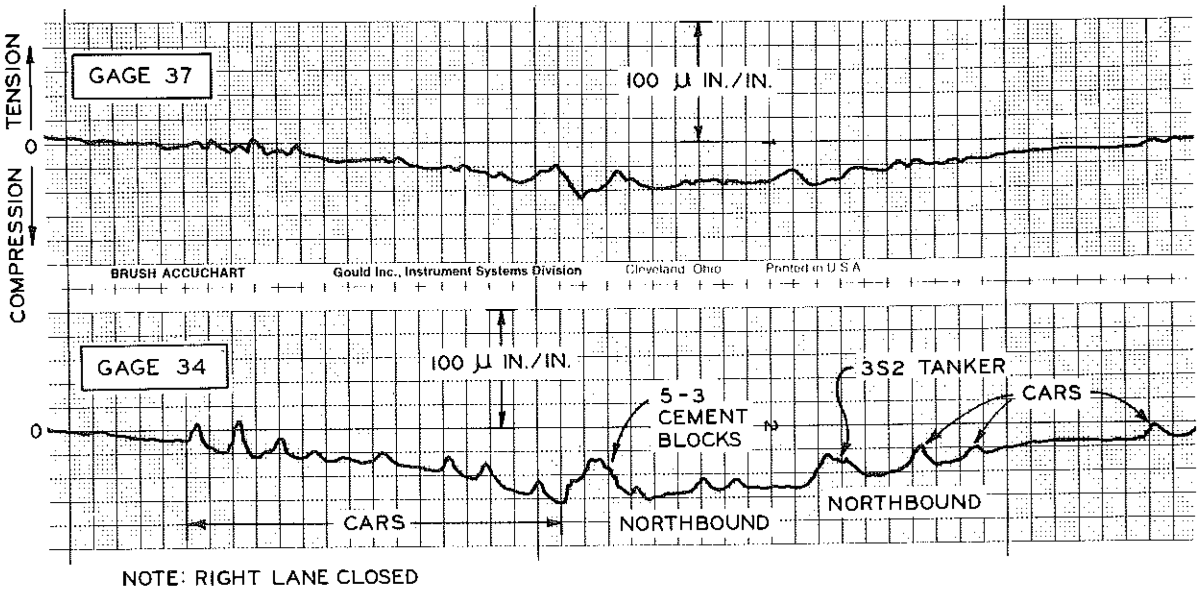


Figure 24. Traces showing the effect of the combination of numerous, relatively light vehicles. The shift is so far negative that the primary live load strains do not bring the resultant back up to zero.

34 and 37, the effect would be additive in the compression flanges immediately above or below those same locations on the stringer. This factor has been considered in the computations of stresses and included in the values presented in this report. Although there is some effect of the deck grating acting compositely with the stringers, an assumption of non-composite action was made so that the results are conservative.

Table 6 shows a tabulation of the strains in the various gages for selected strain events due to heavy commercial vehicles. The experimental vehicle is included for comparison. The four events listed in the first four pairs of columns in the left side of the table were caused by trucks operating in the passing lane, while on the right hand side of the table one group of trucks and the experimental vehicle were in the traffic lane. The 3S3-5 and 3S2 vehicles, listed in the first two pairs of columns, were quite close together, one following the other, so that each caused some additional secondary stress for the other. The event recorded in the fifth pair of columns included the total effects of three vehicles in a row, quite close together.

Comparison of the values for gages 32 to 34, all the way across the table, shows the shift in strain from stringer A (gage 34), to stringer C (gage 32) when the vehicles operate in the traffic lane (right hand two pairs of columns) instead of the passing lane. The data for the rows marked 34'

TABLE 6

MEASURED LIVE LOAD STRAIN (in μ in./in.) CAUSED BY SELECTED HEAVY COMMERCIAL VEHICLES ON THE SUSPENSION BRIDGE*

Gage No.	Truck Type 3S3-5 SB Passing Lane		Truck Type 3S2 SB Passing Lane		Truck Type 3S3-5 SB Passing Lane		Truck Type 3S8 + 2S2L SB Passing Lane		Truck Type 3S5L + 3S3LL +3S3LL SB Traffic Lane		Experimental Vehicle TV2, 10 mph SB Traffic Lane	
	Maximum	Range	Maximum	Range	Maximum	Range	Maximum	Range	Maximum	Range	Maximum	Range
32	30	45	15	30	55	70	45	65	120	145	310	330
33	75	135	50	110	140	175	65	125	0	80	230	270
34	75	185	50	115	95	185	90	215	0	115	30	145
(34')	(-290)	(290)	(-175)	(175)	(-260)	(260)	(-290)	(290)	(-175)	(175)	(-210)	(210)
35	15	30	15	20	25	35	30	45	140	145	290	310
36	130	150	80	95	135	150	140	160	110	130	200	225
37	130	175	85	120	90	140	170	225	100	155	20	125
(37')	(-220)	(220)	(-150)	(150)	(-185)	(185)	(-200)	(200)	(-200)	(200)	(-110)	(110)
38	45	70	40	45	35	55	85	90	135	170	245	300
39	95	145	65	100	110	155	130	185	85	120	185	255
40	0	55	5	50	0	55	0	60	75	75	255	255
41	95	110	90	105	110	120	105	125	15	25	30	55
42	205	205	165	165	215	215	215	215	15	30	25	45
43	80	115	55	75	90	120	50	85	15	20	20	45

* With the experimental vehicle included for comparison. Values for locations (34') and (37') have been added, based on projections of what the strain would be in the opposite beam flange at locations 34 and 37.

and 37' are values determined from the traces from gages 34 and 37 for locations on the opposite (compression) flanges of the beams at those locations. Since the opposite flange in each case is in compression, the primary stresses would be compressive and would add to the compressive secondary stresses. (Reversing the primary stress peak so that it acts downward from point (a) in Figure 18 gives an idea of how this projection is made.) An assumption of non-composite action was made in these projections. Any composite action existing due to the deck grating would reduce these stresses slightly. Since stringer A is the only one with significant secondary stresses, no projections have been added for the other gages.

Comparison of the maximum values in the table shows that the maximum strain caused by a heavy commercial vehicle in the passing lane is little different from the maximum caused by the experimental vehicle in the traffic lane. The main difference is that the maximum strain occurs at a different location. The reader is reminded here that the wider stringer spacing in the left hand lane and the lack of intermediate supports under stringer A affect this situation to some extent. However, a repeated reference to Table 5 will also show that the calculated stresses for the 77-ton commercial vehicle (3S8) in the traffic lane gives a projection of maximum stress in stringer C only slightly lower than that for the experimental vehicle, TV 2.

Figure 25 shows a summary of the types of trucks that passed by while the limited sampling of strains from commercial vehicles was in progress. Note that 14 percent of the vehicles are in the 8 to 11-axle class where legal loads can run from the vicinity of 100,000 lb up to a maximum of 154,000 lb.

The limited data from commercial traffic that have been evaluated here seem to indicate the following: Depending upon their lateral position on the bridge and their proximity to other heavy vehicles, commercial vehicles in the heaviest classes can cause stresses in the floor system of the suspension bridge that are quite high. The location of the maximum stress may vary considerably.

Superposition of the stresses from another similar vehicle going in the opposite direction would amplify the effects in the spans where the two vehicles meet. This is especially true during times that the 'traffic' or outside lanes of the structure are blocked for some reason. The results of this study have shown cable-shift or truss-interaction stresses in stringer A that would seem to warrant additional caution in the transit of heavy vehicles over the suspension bridge from both directions simultaneously when the traffic is confined to the inner lanes.

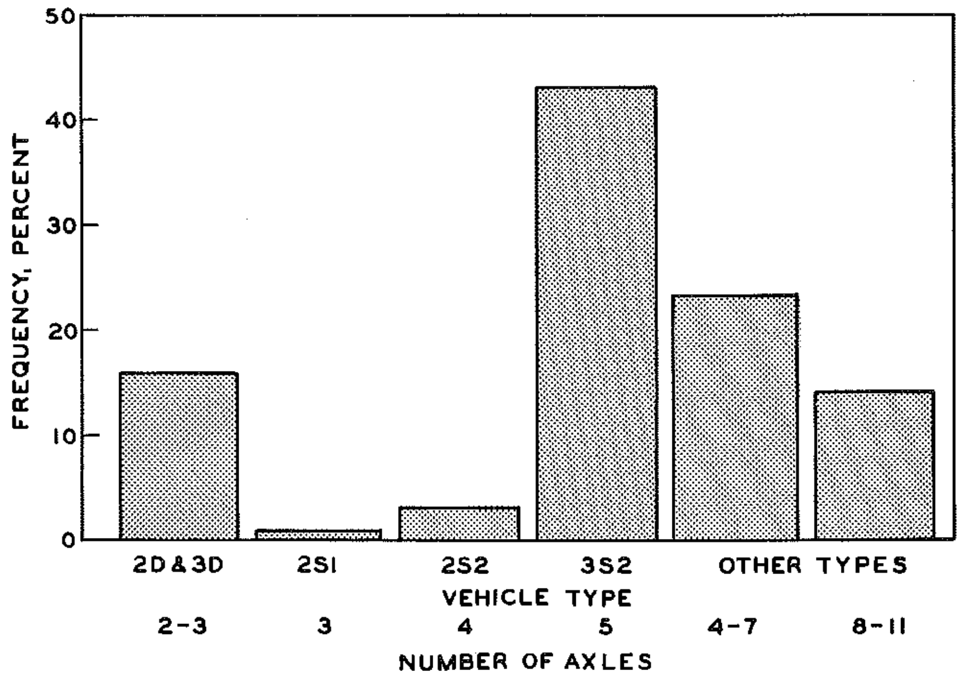


Figure 25. Truck type frequency distribution composite for June 22, 25, and 26, 1978 (209 trucks total).

APPENDIX

The quotations that follow are the pertinent portion of a letter of October 15, 1979, from Gaston Arango of Steinman, Boynton, Gronquist & Birdsall to the Executive Secretary of the Mackinac Bridge Authority, Lawrence A. Rubin.

"We are generally in agreement with the conclusions of the report. It has been demonstrated to our satisfaction that the measured stresses caused by the transporter vehicle carrying the 80-ton railroad cars are below the recommended overload stress of 65% yield stress.

"It is furthermore reassuring that test specimens have confirmed the good quality of the steel, all specified minimum values having been exceeded.

"The fact that stresses due to commercial vehicles approach the stresses caused by the experimental vehicle show that the larger total weight of the transporter has been successfully redistributed transversely as well as longitudinally.

"With all this, we also agree that stress levels are by no means the only factor to be considered when making recommendations to allow railroad cars to be transported across the bridge.

"The tests confirmed again the fact that the specified design parameters are, in the majority of cases, conservative. Specifically, as regards the design of stringers and floorbeams the basic assumptions concern the lateral distribution of wheel loads, the amount of impact, and the degree of composite interaction with the deck.

a. The lateral distribution of the wheel loads to each stringer, representing the reactions of a floor-grid. In turn, this distribution is based on the assumption of simultaneous loading of more than one lane by axles with a 6' c.t.c. wheel-base, or a single lane with the same wheel-base. AASHTO does not specify a reduction formula to take into account wider wheel-bases. The Michigan DOT does use such a formula, which applied to the twelve foot o.t.o. wheel-base of the test vehicle yields a distribution factor of $\frac{S}{7} \times \frac{14}{18} \times \frac{S}{9}$; the actual measured distribution was S/9.6 for the negative moment and S/12 for the positive moment for the stringers of the two-span continuous spans. This gives a reduction of at least 6-1/4% and as much as 25% with reference to the MDOT formula, and between 27% and 42% with respect to the AASHTO Load Table. It would have been of interest to test the actual distribution for a vehicle with a 6' c.c. wheel-base, but even in the absence of such a test it may be assumed to be as conservative and to have as wide a margin of safety as the MDOT formula indicated for the 12' o.t.o. wheel-base.

b. As for the percentage of impact, it was shown that at least for the slow speeds of the test vehicles it was 7% lower than given by the AASHTO formula.

c. The tests revealed a pronounced reduction of stresses due to composite action between the concrete deck and the stringers at the approach span. Similar interactions were shown at the suspended spans, between the I-Beam Lok deck and the crossbeams on the one hand and the I-Beam-Lok - Crossbeam system and the stringers on the other hand.

This composite behavior kept the primary stresses in the floor system below 34% of yield stress.

The report also postulates additional elastic support of the stringers by the laterals at the suspended spans. Even though the readings cannot be disputed, it is nevertheless difficult to conceive that the 30" deep stringers can derive significant support from the 12" deep laterals. In addition the effective span of the stringers is only 70% of the lateral's span. The combined effect of relative stiffness and span lengths as computed seems to account for no more than 6% of support.

It is stated in the report that the stresses caused by the proposed rail transporter vehicle are comparable to the 11 axle, 77 ton legal commercial vehicle. There remains however the question of the relative frequency of crossing of the two types. It is possible that a 77 ton truck crosses only infrequently, whereas the transporter is proposed on a regular schedule. This assumption may be refuted through a traffic count of heavy vehicles.

On the other hand, the experimental finding that stresses caused by regular commercial multi vehicle, multi lane traffic approach the stress levels of the trial vehicle, does provide a reasonable argument for permitting the passage of railroad transporters.

But, inasmuch as the reserve strength of the bridge as revealed by the tests is shared by most structures and is, indeed, very often called upon to make up for unplanned and unforseen loadings, as well as for the ravages of time, it must be invoked with caution."