

EXTENSION OF LIFE FOR PERLEY BRIDGE

H. Vaidyanathan, M.M. Dillon Limited, Consulting
Engineers & Planners, Ottawa
M.S. Cheung, and J.C. Beauchamp, Public Works
Canada

This paper describes a comprehensive evaluation procedure undertaken to decide the future of the 45-year old Perley Bridge, a multi-span, 696.16 m (2284 foot) steel structure, traversing the Ottawa River between Hawkesbury, Ontario and Grenville, Quebec. The bridge was originally designed for approximately H-15 live load and this was to be increased substantially. The bridge consists of an assortment of trestle, deck-truss, through-truss and bowstring-truss spans. The evaluation procedure included a number of phases, such as a review of past performance records and field inspections, theoretical analyses, field and laboratory tests etc. which resulted in recommendations for repairs and/or replacements of deficient members of the bridge. Each phase of the study is discussed in detail, in the hope that it might prove useful as a guide in the evaluation and upgrading of other existing bridge structures. Special attention is drawn to the need for field and laboratory tests to complement the theoretical analyses in assessing the validity of the original design assumptions and procedures in order to develop more realistic analytical models. The load factor design method was employed to assess the theoretical capacity of the bridge. Strengthening measures proposed would extend the life of the bridge at the current traffic level (up to 722.80 kN (162,500 pounds)), while allowing for increases in the future.

The Perley Interprovincial Bridge is a two lane high-level structure spanning the Ottawa River between Hawkesbury, Ontario and Grenville, Quebec. The only crossing from Ottawa to Montreal Island, it links Highway 148 in the Province of Quebec to Highways 17 and 34 in the Province of Ontario (Fig. 1a). Considerable industrial traffic uses the bridge, usually to and from the paper mill in Hawkesbury. A petition initiated by Sir George Perley, M.P., in 1909, first stressed the need for a road link across the Ottawa River at Hawkesbury. Though the design of Perley Bridge was done prior to 1919, the actual construction

was completed in 1931.

History

Structural

From the scant records [1] it appears that the original structure remained untouched, except for some routine painting and minor alignment improvements to the approach roadway, until 1961, when the Carillon Dam development necessitated raising the structure to provide a navigational clearance of 12.802 m (42 feet). A section of about 548.64 m (1800 feet) was raised, to heights varying from 2.44 m (8 feet) to 3.05 m (10 feet), and a new through-truss replaced one of the original deck trusses for navigational purposes.

The steel superstructure, as it stands today, is 696.16 m (2,284 feet) long between abutments, and is made up of an assortment of 27 trestle spans, 5 deck-truss spans, 1 through-truss span and 1 bowstring-truss span (Fig. 1b). The deck is an exposed concrete slab, carrying a clear roadway with a width of 7.315 m (24 feet) between curbs, and a 1.524 m (5 feet) sidewalk on the west side only, resting on supporting steelwork with no shear connectors. The deck over the relatively new through-truss span (1961) is of corrugated metal planks tack welded to the supporting steel work and filled with asphalt to form the riding surface.

Most of the superstructure is of open-hearth steel and assembled together by power driven shop and field rivets. The substructure piers and pedestals are 20.648 MPa (3,000 psi) concrete founded on solid rock.

More than 300 contract, shop and erection drawings prepared during the period 1914-1961 were obtained from various sources, and from them it was determined that the structure was built according to C.E.S.A. standard A6-1929 [2,3,4,5]. The design live-load was specified as U-100 (4.79 kN/m² (100 psf) for spans up to 30.48 m (100 feet), dropping linearly to 3.822 kN/m² (80 psf) for spans 60.96 m (200 feet) and greater), or one 177.9 kN (20-ton) road roller or two H-15 trucks side-by-side. The through-truss span was

designed for two lanes of HS-20 loading in 1961.

Maintenance

Improvements to the south approaches were made in 1965-66 and the superstructure was painted in 1963 and 1972-73.

Performance

During the winter of 1972-73, a header angle (Fig. 2) on one of the south trestle spans failed, causing one corner of the span to fall several inches. Repairs were rapidly carried out and the structure was put back in service. This failure prompted serious considerations about the ability of the structure to carry modern traffic, and a program to assess the safety of the structure was begun in May 1973.

Investigation Program

A flow chart indicates the sequence of operations conducted under this investigation (Fig. 3).

The three main groups (apart from various individual specialists) involved in the safety assessment program were Public Works Canada, M.M. Dillon Ltd., Consulting Engineers, Ottawa and the Ministry of Transportation & Communications, Ontario. Each phase of activity undertaken by the various groups would form a chapter by itself, and hence it is suggested that reference be made to the individual reports of each of these groups [6, 7, 8, 9, 10 and 11].

The investigation program was conducted in several stages; field inspection (underwater, visual and non-destructive), collection of traffic data, theoretical estimates of the carrying capacity of the existing structure based on the 1970 and 1974 AASHTO manual for maintenance inspection of bridges [12, 13], field tests to supplement the theoretical work, laboratory tests to determine the various properties of materials in the structure and determination of the various strengthening measures needed to accommodate the more realistically observed Ontario loads [14, 15]. This paper briefly reviews the salient features of this program.

Program Description

Field Inspection

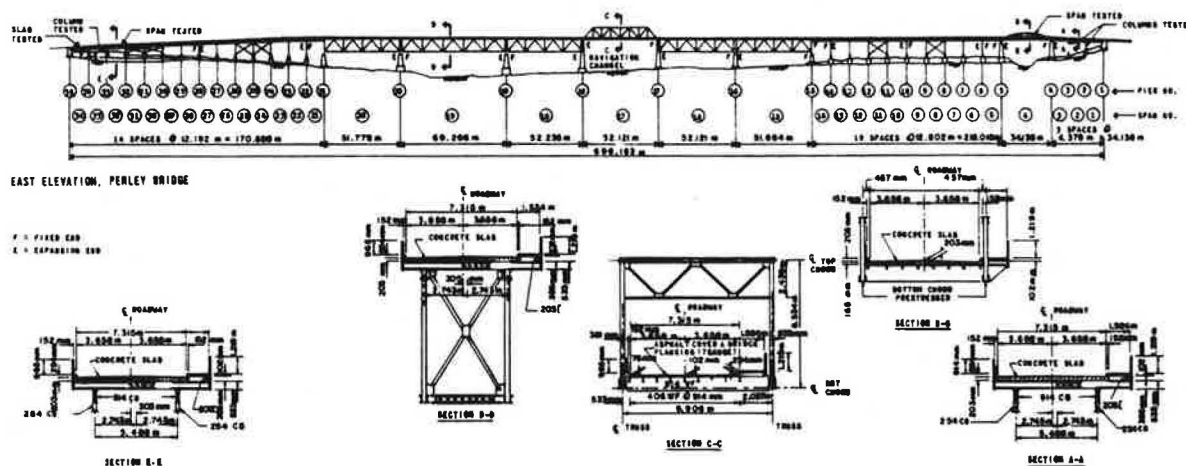
Underwater Inspection. The piers and foundations under water were inspected visually for major structural damage such as ice damage, cracking, scouring and shifting of foundations. Noted were small cracks or concrete honeycombing that might indicate causes for possible failure in the future. Depth soundings were taken at the four faces of each pier and the existing river bottom around the footings was inspected for erosion or undermining. Underwater photography was not required.

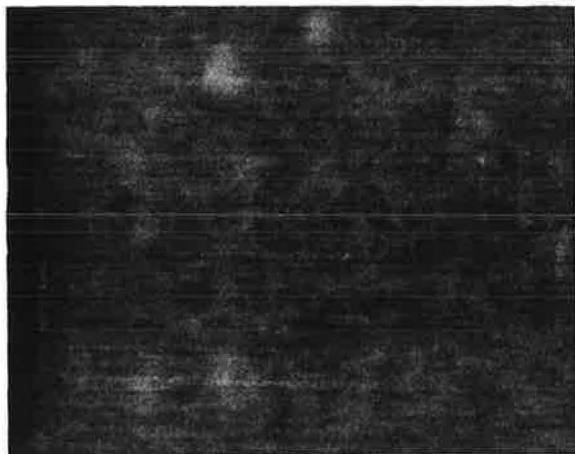
Generally, the substructure below the waterline appeared to be in good condition. The approximate velocity of the current varied from 0.5 m/sec to 3 m/sec (1 to 6 knots) and no cause for concern of erosion was noted. Minor damage in the form of cracks or honeycombed areas were observed.

Figure 1a. Perley Bridge.



Figure 1b. General arrangement (existing).





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graph TD
    A[INVESTIGATION PROGRAMME] --> B[APPRAISAL OF EXISTING STRUCTURE]
    B --> C[FIELD INSPECTION]
    B --> D[STUDY OF EXISTING TRAFFIC ON STRUCTURE]
    C --> E[VISUAL UNDERWATER MECHANICAL NON-DESTRUCTIVE]
    D --> F[TYPE MAGNITUDE INTENSITY]
    E --> G[ANALYTICAL STUDY]
    F --> G
    G --> H[METHOD 1  
RATE EXISTING STRUCTURE  
BY WORKING STRESS  
DESIGN ACCORDING TO AASHTO 1970  
MAINTENANCE MANUAL]
    G --> I[METHOD 2  
LOAD FACTOR DESIGN FOR  
ACTUALLY OBSERVED  
LOADS USING AASHTO 1974 MANUAL & MTC  
BRIDGE]
    H --> J[RESULTS OF METHOD 1 - UNSATISFACTORY  
AND HENCE  
ADOPT ANALYTICAL WORK BY METHOD 2]
    I --> J
    J --> K[PREFABE IMMEDIATE STRENGTHENING  
MEASURES FOR SHORT TERM  
CAPABILITY OF 356 kN ≈ 80 kips  
LOADING ON STRUCTURE]
    J --> L[POST REPAIR  
FOR  
LOAD LIMITS]
    J --> M[EXAMINE STRUCTURE ADEQUACY ON A LONG  
TERM BASIS FOR PROPOSED ONTARIO BRIDGE  
DESIGN LOADING OF 723.0 kN]
    M --> N[DOUBTS ABOUT  
VALIDITY OF  
ANALYTICAL WORK]
    N -- NO --> L
    N -- YES --> O[CONDUCT PROTOTYPE FIELD  
TESTS IN PROBLEM AREAS]
    O --> P[MODIFY ANALYTICAL  
WORK]
    P --> M
    L --> Q[EXECUTION OF  
REHABILITATION  
WORK]
    L --> R[STEEL SAMPLES  
1. MECHANICAL  
2. CHEMICAL  
3. BRITTLENESS  
4. WELDABILITY]
    L --> S[CONCRETE SAMPLES  
1. COMPRESSIVE  
STRENGTH  
2. CHLORIDE  
CONTENT]
    R --> Q
    S --> T[PREFABE DETAILS OF REPAIR WORKS  
TO UPGRADE STRUCTURE FOR LOAD  
CAPABILITY OF 723.0 kN]
    T --> M
  
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1. A visual inspection was carried out at all piers and abutments above water or ground level. Apart from some minor cracks, spalls and honeycombed areas, the substructure appeared generally sound (Fig. 4a).
2. Bearings varied from good to severely corroded or completely frozen (Fig. 4b).
3. Some header angles connecting girders to columns appeared to be undergoing possible fatigue problems. While no apparent evidence of surface cracking was observed, the heavy magnetic particle equipment detected possible fatigue problems at the top of the heel of some header angles.

5. The steel decking of the relatively new through-truss was flexible, and as a result the asphalt surfacing had been deteriorating badly. Neither surfacing nor decking were watertight, and this caused corrosion of the decking and floor system.

6. Expansion joints in the various spans had not been functioning properly and they appeared to suffer from poor detailing, poor installation and bad corrosion (Fig. 4d).

7. Concrete deck and sidewalk, from a visual inspection and the chain-drag method, showed localized areas of spalling and delamination over 5 per cent of the deck. The spalls were mainly confined to the top of deck slab, but also occurred on the underside of the slab (Fig. 4e).

8. The railing generally appeared to be in poor to fair condition, and did not meet current traffic rail standards (Fig. 4f).

Traffic Study

It was considered prudent to obtain and collect information on the traffic using the bridge before settling down to any theorizing. Based on the traffic data gathered in 1972 and a series of one-hour and half-hour counts in October 1973, the SAWDT (Summer Average Weekday Traffic) was estimated to be about 11,000 to 12,000 vehicles per day, 20 to 30 per cent being trucks. Apart from logging trucks bound for the Canadian International Paper mill (C.I.P.), other heavy vehicles using the bridge included liquid and solid bulk carriers, floats with heavy earth-moving equipment, and flat-beds with loads of concrete blocks or concrete pipe (Figs. 5a, 5b). The following information was extracted from C.I.P. weight records in 1973 (Fig. 6).

Clearly, the bridge structure had been subjected to loads much heavier than those for which it was designed.

Theoretical Estimate of Structural Capacity - Analysis and Design

An elastic analysis with assumptions consistent with common practices according to existing standards [12, 13, 16] had been made as a first step in the evaluation procedure. These assumptions have been summarized below.

1. Deck slab continuous over transverse floor beams, considered non-deflecting.
2. Floor beams simply supported on longitudinal girders and act without any composite action from the deck slab. In estimating loads on floor beams, no wheel load distribution has been allowed for in the lateral direction, while in the longitudinal direction, distribution has been determined based on the deck slab acting as continuous span, all wheel loads being considered as point loads.
3. Longitudinal girders span between the trestle columns as simple spans. Note the absence of any connection between the flanges of girders and the columns.
4. Columns fixed in position and direction

Figure 4a. Local spall areas in pier.

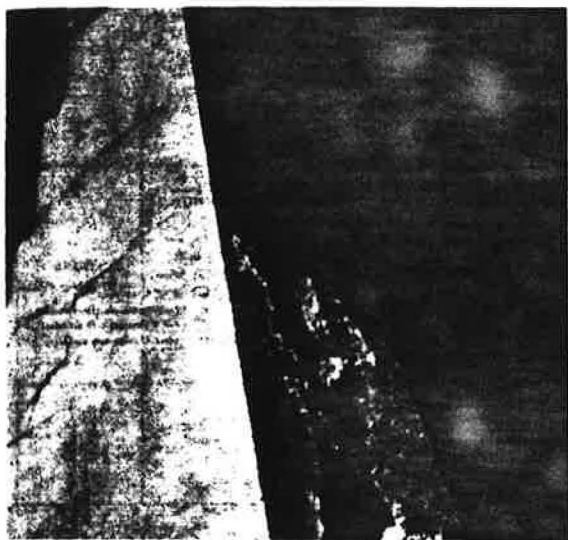


Figure 4b. Bearing assembly.

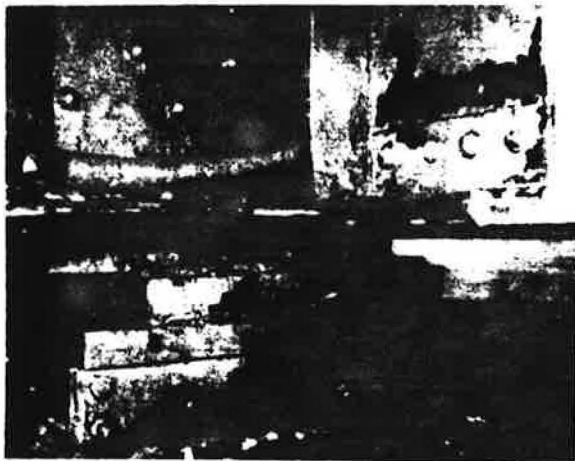


Figure 4d. A typical expansion joint.

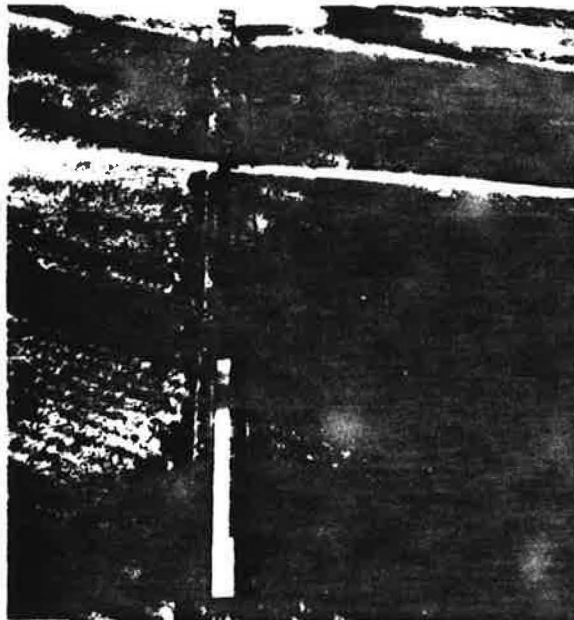


Figure 4e. Spall areas (underside of the deck)



Figure 4c. Severe corrosion of floor beam.



Figure 4f. Inadequate railing.



Figure 5a. Floats with construction equipment.

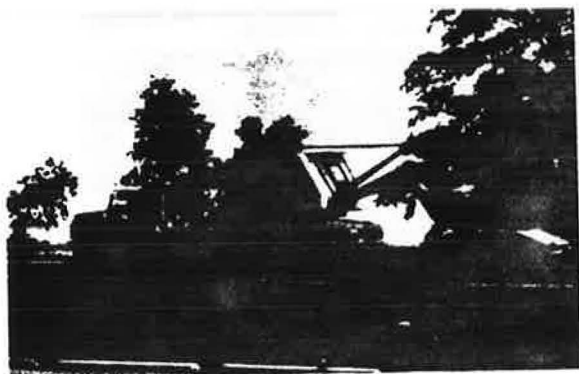
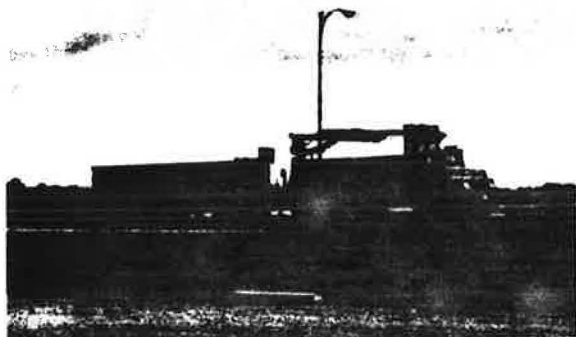


Figure 5b. Flat-beds with concrete blocks.



at the base, and held in position at the top by restraints provided by bracings in the longitudinal direction.

5. Trusses analysed as both pinned and rigid jointed plane frames.

Design Methods. The theoretical capacity for design employed two methods; Method I - Working Stress Design (WSD) and Method II - Load Factor Design (LFD).

Working Stress Design. This is based on "AASHTO Manual for Maintenance Inspection of Bridges - 1970" [12].

1. Allowable stresses specified in [12] were used for both operating and inventory load rating. These are summarized in Table 1 for the inventory rating.

2. The Secant formula, as in [12], was used to deal with cases of axial compression and bending.

3. Load capacities of the various elements of the structure were computed as fractions of AASHTO loadings. For example, a member capable of carrying 65 per cent of H-20 loading was rated at H-13, and so on. Table 2 provides a summary of ratings of some of the items in the structure. It is clear from the table that some structural elements rated in the order of AASHTO H-12 (60 per cent of AASHTO loading H-20). According to the 1931 specifications the structure was designed for a loading similar to AASHTO H-15 (75 per cent

Figure 6. Status of traffic on bridge at time of inspection (May, 1973) - abstract from C.I.P. weight records.

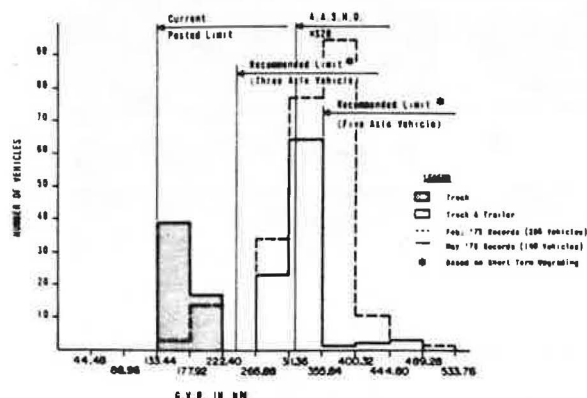


Table 1. Allowable stresses used for inventory rating.

Type of Stress	Allowable stresses
Tension in Structural steel (0.55 times yield point)	113.76 MPa (16.6 ksi) - 1931 steel 125.49 MPa (18.2 ksi) - 1961 steel
Tension in reinforcing steel	125.49 MPa (18.2 ksi)
Shear in rivets	91.01 MPa (13.2 ksi)
Compression in Concrete	12.41 MPa (1.8 ksi)
Safety factor in Compression member	1.48

Table 2. Summary of load rating.

Members	Load rating using AASHTO (Manual 1970)	(Inventory Level)
	HS - Trucks	H - Trucks
Columns	7-9 ^a (55%) ^b	11-13 (45%)
	9-11 (45%)	13-16 (55%)
Deck Trusses	8-16 (52%)	12-16 (40%)
	16-20 (24%)	16-19 (12%)
	20-25 (24%)	>19 (48%)
Bowstring Truss	13-15 (48%)	17 (34%)
	15-20 (52%)	19 (14%)
		>19 (52%)
Girders	10	13
Floorbeams	13	
Decks/ab	20	

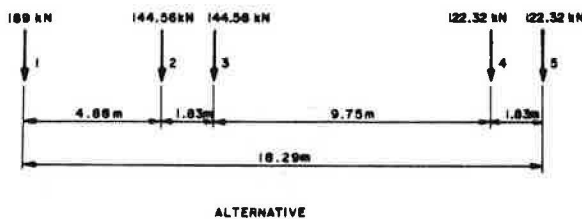
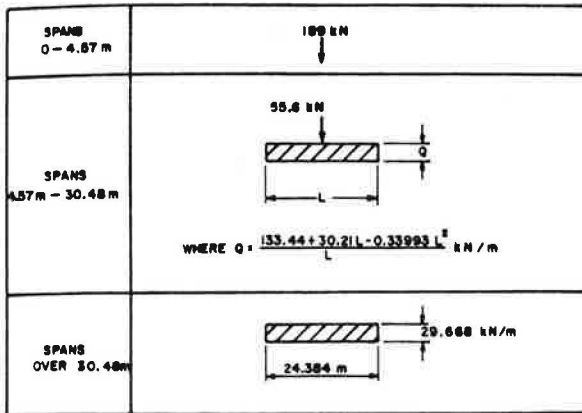
Note: Through-truss not evaluated. Design according to AASHTO HS-20 loading.

^aIndicate rating of HS7-HS9.

^bIndicate percentage of rated members.

of AASHTO H-20 loading). The discrepancy between the rating and the original design loading can likely be attributed to differences in design criteria and methods, or higher allowable working stresses in the original specifications (that is, the allowable tensile stress in 1931 specifications was 124.1 MPa (18.0 ksi) as compared to

Figure 7. Proposed Ontario bridge design load - RR186 (modified since this project to form the basis of the forthcoming Ontario Highway Bridge Code).



the allowable rating stress of 113.76 MPa (16.5 ksi)).

Load Factor Design. This is according to the most recent edition of the AASHTO Maintenance Manual (1974) [13] and partial load factors criteria [14].

1. To allow for the type of heavy traffic using the bridge, a hybrid concept was developed from AASHTO Codes [13, 16] and MTC Research Report No. RR186 [14]. Because it was economically impossible to restrict traffic for even a short period of time, a two-stage approach was taken. The first stage called for an immediate short-term upgrading/remedial works to carry loads of gross vehicle weight (GVW) less than 355.84 kN (80,000 pounds) and gross axle weight (GAW) less than 88.96 kN (20,000 pounds) [17]. The second stage called for long-term remedial work that would enable the bridge to carry the maximum proposed Ontario Bridge Design load. (GVW and GAW not to exceed 722.80 kN (162,500 pounds) and 189.04 kN (42,500 pounds) respectively) (Fig. 7). The short-term solution depended on implementation of all recommendations regarding traffic, weight and speed control on the structure [16]. Whenever guidance was not available from RR186, AASHTO Codes were used.

2. The structure capacity evaluated by the introduction of partial-load factors [14] satisfied the following general equation:

$$[(1-\mu) \phi \times \text{Capacity}] \geq [D \times FD + (1+i) m \times A \times FL]$$

where:

μ - deterioration factor

- = 0.10 (members immediately below deck and existing fasteners)
- = 0.05 (all other members)
- ϕ - capacity reduction factor
- = 0.80 (truss members)
- = 0.85 (remaining steel members and reinforced concrete in shear)
- = 0.90 (reinforced concrete in flexure)
- Capacity (moment, Shear or reaction)
- = $F_y \times$ flexure modulus etc., as per AASHTO
- D - dead load (moment, shear or reaction)
- FD - dead load variation factor
- = 1.10 (steel)
- = 1.15 (concrete)
- = 1.33 (asphalt)
- i - impact factor
- = as per AASHTO
- m - multiple presence indicator representing various possible combinations of simultaneous presence of more than one vehicle on the bridge ($m = 1$ for two loaded lanes)
- A - maximum allowable live-load (moment, shear or reaction)
- FL - Live-load variation factor
- = 1.10 for short-term
- 1.20 for long-term (Based on MTC's projected study of traffic growth for the period of 15 years.)

Safe load capacities of members with axial compression and bending were determined according to the load factor design method given by the AASHTO 1974 Maintenance Manual.

3. The types of vehicular loadings investigated were; proposed Ontario Bridge Design Load (Fig. 7) and axle combinations permissible by the Ontario Bridge formula [17].

Short-term Upgrading. The structure was strengthened to carry loads up to 355.84 kN (80,000 pounds) on a short-term basis and sign posted for load limits. The extent of major repairs was confined to the following:

1. Seating brackets under girders of trestle spans.
2. Prestressing to the bottom chord of bow-string truss and to the tension diagonal of 68.28 m (224 feet) decktruss.
3. Bracing members to increase the capacity of some diagonals in deck trusses.
4. Armouring of all joints in the deck.

Long-term Upgrading. While the short-term solution was arrived at hastily, on the basis of theoretical findings only, the target of achieving a load capacity of 722.80 kN (162,500 pounds) on a long-term basis (a phenomenal increase from the present day AASHTO standards), required a careful and efficient assessment of the full capabilities of the structure. Considering the question of total reliability of the various assumptions in a theoretical exercise, a combination of analytical and experimental procedure in the form of load tests on the actual structure was pursued. The main thrust of the testing program, details of which are described below was to verify or suitably influence the analytical method in its theoretical assessment. Apart from that, the test would provide, with a reasonable degree of confidence, ultimate proof of the load-carrying capacity of the structural units corresponding

to the application of selected loads. In view of the expected increase in the life of the structure and possible variations of traffic types during this period, some changes were made to the design and analysis criteria adopted earlier. The changes introduced also reflected the findings of the test program and were as follows:

1. Trusses were to be analysed as rigid-jointed plane frames.
2. Partial continuity of longitudinal girders in trestle spans were to be considered.
3. Flexural effects in the columns were to be suitably accounted for due to 2 above.
4. Thermal stress in the structure was to be relieved by repair or replacement of expansion bearings.
5. Floor beams were to carry loads without the help of composite action from the deck slab.
6. Impact factors were to be modified to 50 per cent (columns) and 30 per cent (trestle spans and trusses).
7. Refined methods of analysis considering

the interaction of the various elements of the bridge were to be used, where felt necessary, to draw upon the more realistic load distributing properties of the structure than assumed by simple methods. This approach appeared to be reasonable from the capability of the existing bridge to carry the test loads without signs of distress.

Numerical Example. An example shown in Figure 8 and Table 3 illustrates the differing results obtained depending on the type of analysis employed; Analysis I - simplified method according to AASHTO code Section 3 (13, 16); Analysis II - Finite Element Analysis Using "STRUDEL" Program.

The following procedure was adopted for Analysis I:

1. Proportion of axle load 'P' for the design of floor beams is $(S/6) \times P$ (Table 1.3.1 c).
2. No lateral distribution is considered (1.3.1 c) for calculating a) bending moments on floorbeams and b) proportion of load on girders.
3. Longitudinal distribution of load on girder is obtained as reactions on floor beams based on continuous slab analysis.
4. Using reactions determined from 2 & 3 above, bending moments in girders are calculated.

In Analysis II, the structure was modelled into an assemblage of beam and plate elements and the following assumptions were made:

1. Linear elastic analysis
2. Isotropic slab resting on floor beams without composite action; sidewalk slab ignored.
3. Concentrated wheel loads assigned to nodal points.

Figure 8. Trestle span (typical).

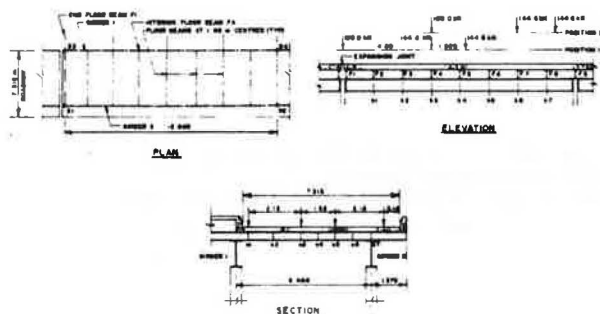


Table 3. Vehicular load distribution effects

Bending moment in floor beam (kN - m)

Floorbeam	Load Position	Location						
		a1	a2	a3	a4	a5	a6	a7
F1		41.9	100.1	158.4	137.0	105.9	21.9	-63.3
(adjacent to Exp. joint)	1	(17.2) ^a (17.0) ^c	(60.9) ^a (61.7) ^c	(112.3) ^a (114.9) ^c	(92.5) ^a (94.6) ^c	(80.1) ^a (80.5) ^c	(7.2) ^a (6.8) ^c	(-68.3) ^a (-69.3) ^c
F4		41.9	100.1	158.4	137.0	105.9	21.4	-63.3
(typical interior beam)	2	(7.5) ^a (8.0) ^c	(22.4) ^a (25.0) ^c	(56.7) ^a (69.2) ^c	(41.8) ^a (43.7) ^c	(43.5) ^a (45.0) ^c	(-1.0) ^b (4.1) ^c	(-43.4) ^a (-47.6) ^c

Bending moment in Girders (kN-m)

Girder	Load Position	Location						
		b1	b2	b3	b4	b5	b6	b7
1		212.6	425.6	637.9	658.3	514.1	342.5	171.5
	1	(207.1) ^a (205.2) ^c	(406.9) ^a (406.0) ^c	(597.0) ^a (612.5) ^c	(626.6) ^a (644.1) ^c	(494.9) ^a (494.0) ^c	(328.7) ^a (327.7) ^c	(163.5) ^a (163.1) ^c
2		299.7	599.4	899.0	928.0	724.6	482.9	241.8
	2	(283.8) ^a (281.0) ^c	(547.6) ^a (550.3) ^c	(793.8) ^a (818.8) ^c	(832.0) ^a (858.6) ^c	(654.8) ^a (657.0) ^c	(433.0) ^a (433.9) ^c	(216.6) ^a (217.0) ^c
1		225.5	451.1	676.6	651.0	625.4	599.9	382.3
	1	(216.0) ^a (221.0) ^c	(435.8) ^a (438.4) ^c	(631.6) ^a (648.0) ^c	(627.2) ^a (640.3) ^c	(601.3) ^a (599.9) ^c	(561.5) ^a (560.8) ^c	(361.8) ^a (361.5) ^c
2		318.0	635.8	953.8	917.7	881.7	845.6	538.7
	2	(285.8) ^a (289.2) ^c	(574.8) ^a (581.2) ^c	(830.3) ^a (856.3) ^c	(827.3) ^a (854.4) ^c	(796.1) ^a (797.1) ^c	(745.8) ^a (746.6) ^c	(487.6) ^a (488.2) ^c

^a Figures in () indicate Analysis II

^b -ve indicates hogging moments

^c Indicates results obtained by Analysis II, considering random slab elements with negligible thickness to represent scattered spalls in bridge deck.

4. External restraints in the direction of loads only at column supports.

5. Poisson's ratio in concrete = 0.15, in steel = 0.30.

Comparative study of the two methods indicated that simplified methods could lead to overestimation of capacity requirements as noted below:

1. Maximum bending moments in floor beams obtained from Analysis I are far in excess of those obtained from Analysis II (41% for Load Position 1 and 279% for Load Position 2).

2. Maximum bending moments in girders determined by Analysis I are slightly in excess of those obtained from Analysis II (5 to 15%); the aspect ratio (span/width = 2.33) of this grid reflects its predominant behaviour in the longitudinal direction.

3. Random spalls in bridge deck do not have any significant effect on the load distribution characteristics of the grid.

Field Tests

These tests involved experimental stress analysis by monitoring parts of the structure mainly through strain readings taken with the aid of electrical resistance strain gauges [8] and simultaneous complemental monitoring of these components by acoustic emission [11], as loads were applied in increments up to a predetermined proof load.

Briefly, the two systems differ in the sense that while electrical resistance strain gauges are quantitative, the acoustic emission is only qualitative. The latter provides an indication of the integrity of the structure at the selected points of interest by projecting warnings of incipient failures during test loading.

Objectives. The load testing program was confined to specific and representative areas of the structure evaluated as deficient by the theoretical work. The main objectives of the test were to determine the actual behaviour and the load distributing properties of the various components of the structure in its true three-dimensional state, a factor not accounted for

adequately in the analytical evaluation of the capacity of the structure. The main areas of investigation were:

1. For trestle spans, the possibility of longitudinal girders possessing more stiffness and capacity than could be predicted by the analytical assumptions of simply-supported spans between columns.

2. The effect of deck slab on the composite action of floor beams in flexure and on the webs of the floor beams directly over the longitudinal girders.

3. The load-moment transfer into the supporting columns from live load and the study of more realistic boundary conditions for the columns than the assumed end conditions (these effects were monitored for both static and dynamic loading).

4. The load distribution characteristics from decking system to main load-carrying members in trestle spans.

5. For trusses, the end conditions of the various members forming the truss were investigated, and the effects of thermal variations in the truss members caused by malfunctioning of existing expansion devices.

6. The study of impact factors based on dynamic tests.

7. The strength capacity of randomly deteriorated deck slab subjected to heavy axle loads.

Procedure. Two remote-controlled test vehicles (Fig. 9), capable of providing load lifts of about 765.06 kN (172 kips) each, were loaded incrementally using concrete blocks for the load. Dynamic effects were simulated by running the vehicles at various speeds over bumps created by laying plywood planks over the deck.

Conclusions. The following observations and conclusions were drawn from the above mentioned field tests.

1. For the floor beam and deck slab, the bare steel section of the floor beam has neutral axis (NA) at 267 mm (10.5 inches) from the soffit, while with full composite action with deck slab (without any shear lag), the theoretical NA should have been at 484 mm (19.07 inches). Table 4 shows

Figure 9. Two testing vehicles.

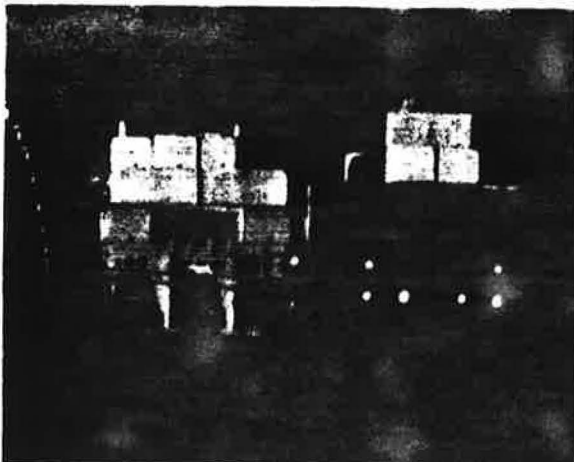
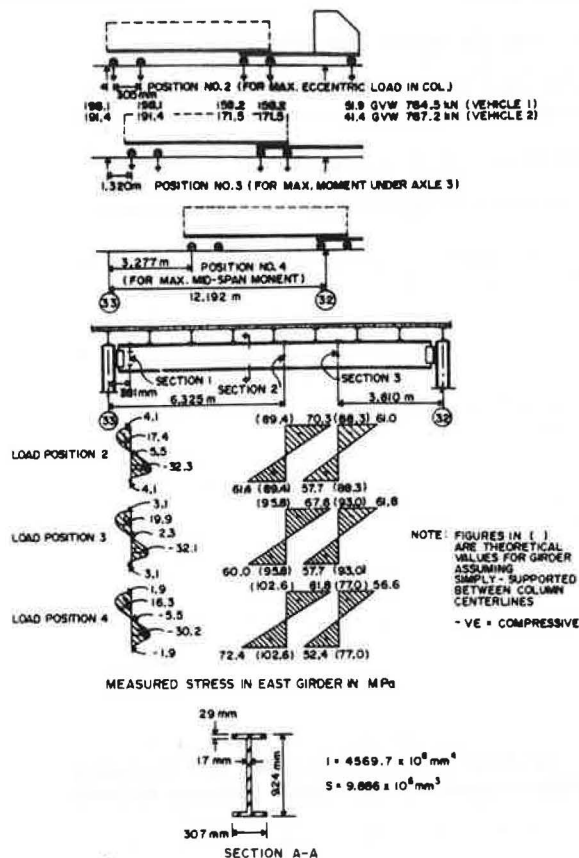


Table 4. Measured neutral axis (floor beam).

BEAM No. 1	LOAD CASE No. 1	DISTANCE OF NEUTRAL AXIS FROM BOTTOM OF BEAM (mm)
A	2	350
	3	314
	4	316
	7	349
B	2	308
	3	306
	4	289
	7	336
C	1	381
	2	347
	3	432
	4	348
	5	378
	6	333
	7	429

LOAD CASE NO.	TRANSVERSE POSITION OF VEHICLE	LONGITUDINAL POSITION
1	1	1
2	1	2
3	1	3
4	2	2
5	2	1
6	2	3
7	3	3

Figure 10. Test and analytical results (Trestle span).



scattered results indicating no fixed pattern. Thus there was a failure to predict the degree of composite action with any certainty.

2. Figure 10 indicates a comparison of test and analytical stresses in the longitudinal girder of the trestle span. The theoretical figures are consistently higher than the corresponding measured ones. This is explained by the existence of partial continuity of the girders through columns and non-functioning expansion joints restraining joint rotations over the supports. Measurement of thermal stresses in the girder suggested the desirability of reducing these by functioning expansion devices. Though the existing header angle connection between girder and column is theoretically a flexible one transferring pure shear only, field tests revealed the presence of some restraining moments at the junction.

3. Table 5 reveals that good correlation exists between the test and the analytical results of a rigid-jointed truss.

4. Impact factor derived as a ratio of maximum increase in static load stresses to maximum static load stress at any time is tabulated in Table 6 for various speeds and bump heights. A reasonable interpretation of the results suggests an estimate of impact factors higher than AASHTO criteria.

5. Acoustic emission monitoring detected no active cracks in the members tested. Minor reversible emissions noted in some members were attributed to normal relaxation of the structure in response to application and removal of the load. Joints monitored on the bowstring truss showed emissions corresponding to discreet movements but this activity did not increase systematically

Table 5. Comparison of field & analytical results (Bowstring truss).

Diagram of the Bowstring truss structure showing load positions and member forces. The diagram includes a horizontal baseline with points L₀, L₁, L₂, and L₃. Above the baseline, there are points U₁ and U₂. A truss structure is shown with members connecting these points. Two vehicles are shown on the baseline, with their wheel positions and weights indicated. The weights are 707.80 kN and 710.53 kN. The diagram also shows the total load and the load on each member.

50.84 41.37 148.52 157.86 148.52 157.86 182.01 178.72 182.01 178.72 - GVW 707.80 kN - VEHICLE 1
- GVW 710.53 kN - VEHICLE 2

Member	Test ^a	Analytical ^b	
P - kN	(Total load = 1414.46 kN)	(Total load = 1445.6 kN)	
M ₁ , M ₂ - kN-m			
L ₀ U ₁	P	1063.07	1214.30
	M ₁	-15.73	8.27
	M ₂	49.49	72.94
U ₁ U ₂	P	1169.82	1240.99
	M ₁	22.78	42.57
	M ₂	12.47	20.47
L ₀ L ₁	P	791.74	1112.0
	M ₁	-24.27	-8.13
	M ₂	62.64	96.67
L ₁ L ₂	P	809.54	1116.45
	M ₁	63.54	91.92
	M ₂	4.75	7.19

^a Moment values obtained outside of gusset plates
^b Moment values obtained at junction of members

Table 6. Impact factors computed from data of dynamic load test on span 2.

Diagram of the impact test setup showing a vehicle on a bridge span.

IMPACT FACTOR = MAXIMUM INCREASE IN STATIC LOAD STRESS AT ANY TIME / MAXIMUM STATIC LOAD STRESS AT ANY TIME x 100

VEHICLE SPEED # IN MPH	HEIGHT OF BUMP (IN)	IMPACT FACTOR				VIBRATING FREQUENCIES # x 100			
		COLUMN 3		COLUMN 5		UNDER MAX. STRESS		AFTER VEHICLE OFF THE SPAN	
		SECTION AA	SECTION BB	SECTION A'A'	SECTION B'B'	AA	BB	A'A'	B'B'
4.8	0	21*	12	5	8*	1.5	1.0		
16.0	0	32*	25	11	7	2.7	1.0	8.0	8.0
16.0	19	50*	33*	15	36*	3.5	1.5	8.0	7.5
48.3	19	39*	39*	25	40	8.0	8.0	8.0	8.0
48.3	38	52*	46*	32	38*	8.0	8.0	8.0	8.0

* vehicle brought to a sudden stop on the span

with the load, suggesting no active cracks but only possible slipping in the joint.
Laboratory Tests

Laboratory tests were grouped into the following categories:

Steel in Superstructure (chemical, mechanical, weldability, brittleness). Test coupons were taken from columns, floor beams and some truss members. Samples chosen for analysis were considered fairly representative of the bridge considering that almost all steel was fabricated by the open-hearth process according to C.E.S.A. specifications for steel highway bridges No. A6-1929.

Table 7. Properties of steel.

Mechanical

Sample Number	Yield point MPa	Tensile strength MPa	% Elongation (203 mm gauge length)
1M	235.8	384.0	30
2M	281.3	386.1	24
3M	266.8	409.6	32
4M	228.9	355.1	23
5M	279.2	386.8	27
6M	266.8	386.8	20
7M	279.2	376.5	29

Chemical

Sample Number	Carbon (C)%	Manganese (Mn)%	Phosphorus (P)%	Sulphur (S)%	Silicon (Si)%
1C	0.18	0.60	0.016	0.011	0.02
2C	0.14	0.49	0.018	0.014	0.04
3C	0.10	0.43	0.013	0.012	0.02
4C	0.13	0.44	0.012	0.015	0.06

Charpy V-notch

Sample Number	Test temperature (C°)	Charpy Energy (N-m)	Average Energy (N-m)
1V		14.9	
2V	-17.5°	13.6	15.1
3V		16.9	
4V		18.3	
5V		20.3	
6V	-6.6°	29.2	22.8
7V		24.4	
8V		21.7	

Note: 1 MPa = 0.145 ksi, 1 N-m = 0.738 ft-lb.

Carbon equivalent obtained from the chemical analysis based on the formulation $C_{eq} = C + Mn/3$ (a "worst case" formulation) rates the steel to be highly weldable. Weld tests performed on the samples indicated hardness of the heat affected zones not exceeding Vickers 223 with 300 being the practical limit, and suggested no problems of crack formations [19]. However, in view of the possibilities of high restraint and variations in the chemistry of material in the actual structure, a low strength, low hydrogen electrode E 7016 as specified in CSA code W59.1 - 1970 was adopted.

Charpy V-notch testing of specimens was conducted at two different temperatures. Considering, in bridges, the expected maximum rate of loading to be 1/10,000 times the strain rate in a Charpy Impact test [20], the fracture toughness obtained was considered satisfactory. Table 7 shows the results of analysis on some of the samples.

Concrete in the Deck. Concrete cores obtained from various locations in the deck were subjected to compression tests and chemical analysis to determine the level of soluble chloride ions. Table 8 shows consistently high compressive strengths and a remarkably high chloride ion content than present day researches on bridge deck deterioration would accept. Reasons for such high concentration remain unclear though possibilities of chloride admixtures in the concrete could not be ruled out.

Table 8. Properties of concrete.

Compressive strength

Core Number	Diameter mm	Height mm	Unit Weight kg/m ³	Compressive Strength MPa
1C	94	184	83.7	32.27
2C	94	140	86.6	45.16
3C	94	117	89.6	50.33
4C	94	165	81.9	42.95
5C	94	152	87.2	39.85
6C	94	156	86.6	51.30
7C	94	133	86.0	56.68
8C	94	156	87.8	50.54
9C	94	165	84.8	47.78
10C	94	171	85.4	36.61

Chloride Content

Core Number	Slide number ^a	p.H value	Chloride content kg (chloride)/m ³
1CH	1	11.6	4.56
	2	11.7	3.79
	3	11.7	1.94
2CH	1	12.0	5.47
	2	12.0	2.22
	3	12.1	0.33
3CH	1		10.06
	2		5.84
	3		4.91
4CH	1		7.95
	2		4.44
	3		1.64

^aSlide number 1 represented the wearing surface slide number 2 represented concrete to a depth of approximately 1-1/2 inches below surface and slide number 3 represented concrete approximately 2-1/4 inches below surface.

Fatigue Life

Complexity of factors attending the determination of the remaining fatigue life of an existing structure needs no special emphasis. Considering only the sketchy history of traffic available in the present case, attempted theoretical study did not seem satisfactory. However, it has been decided to replace the existing rivets by high strength bolts on all members subjected to tension and reversals as a measure to prolong the fatigue life of the structure [21]. Welded details for member strengthening have been kept to a minimum and totally avoided on members subjected to tension and stress reversals.

Details of some Strengthening Measures

At the time of writing this paper, the project is in various stages of design and construction. Some repair techniques are discussed briefly here. Strengthening in general, has been done by the addition of materials using both bolted and welded arrangements. Welded details have been avoided on tensile members and members subjected to reversals. This is to ensure lesser burden on future inspection and maintenance. Some compression members have been upgraded by bracing them against buckling (Fig. 11a). A combination of knee braces between columns & girders and cross braces between the webs of floor beams at the trestle spans

Figure 11a. Bracing details.

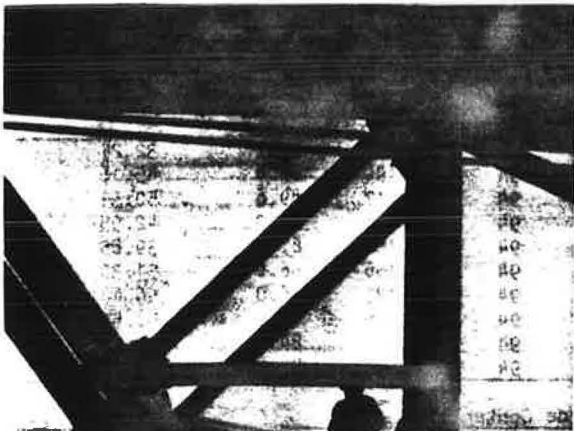


Figure 11b. Kneebraces.

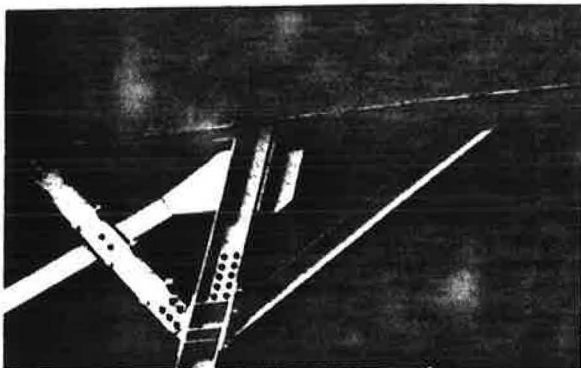
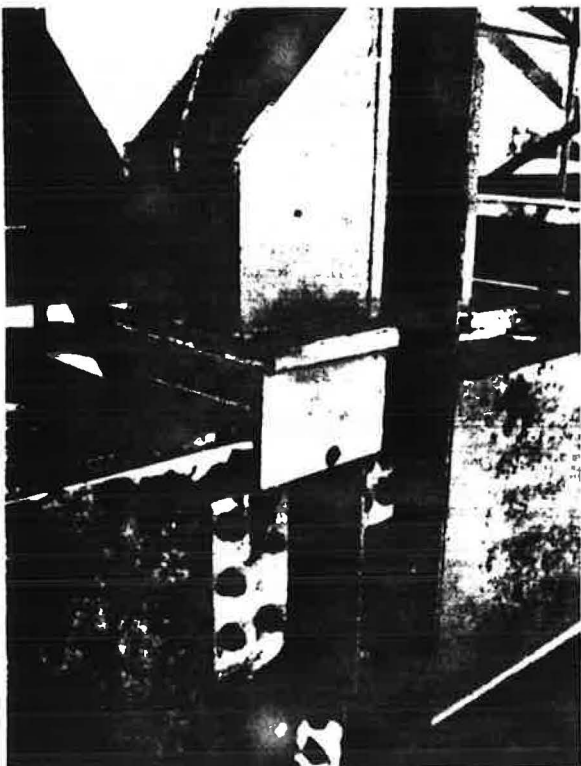


Figure 11c. Expansion bearing.



(Fig. 11b) reduces the possibility of failure of floor beam webs, improves the capacity of longitudinal girders by providing continuity over spans, strengthens the supporting columns against buckling and relieves partly the load on the header angle connections between girders and columns by sharing in the joint effects. Existing cast-steel bearing assemblies under expansion ends of trusses have been replaced by elastomeric type bearings. Trestle span expansion devices have been modified as shown in (Fig. 11c). Figs 12a, 12b, show details of some strengthening items.

Strengthening details have been proportioned according to load factor design such that;

1. Totally new members and compression members are stressed to yield limits.
2. New materials added to the original members (tension and reversals only) are stressed to a limit of $(F_y - F_o)$, where F_y is yield stress of original members and F_o is stress in original member from dead load only.

Cost of Repairs

Before deciding on a programme of long term upgrading, studies were made on the relative economics of (i) retaining the existing bridge for auto traffic only with minimal maintenance works and an adjacent new bridge for heavy truck traffic only and (ii) strengthening the existing bridge for one lane auto and one lane truck traffic and an adjacent new bridge for similar traffic. The total replacement of the bridge on the same location was not studied in detail considering the good salvage value of majority of the bridge components and the serious consequences of major shutdown of the crossing on the local industries.

The total estimated cost of repairs is about \$1.7 million. Repair works are being done in various phases to accommodate the availability of funds and the urgency of repair items. All major structural repairs are expected to be completed by September, 1979. Two contracts amounting to \$540,000 (about \$300,000 for short term up-grading) have already been completed and the next one amounting to \$744,000 is to begin in July 1978. Breakdown of repair costs is as follows:

Trestle Spans	-	\$ 348,000
Deck Truss (68.28 m)	-	\$ 54,000
Deck Trusses (51.82 m)	-	\$ 116,000
Bowstring Truss	-	\$ 51,000
Through Truss	-	\$ 62,000
Miscellaneous	-	\$1,000,000
(Deck Patching, Pier Spalls Expansion bearings, seals, approaches, replacing rivets by bolts, etc.)		

Conclusions

Superstructure

Generally, the safe load capacity of existing bridge components based on simplified methods of analysis and working stress design can be underestimated. Load factor design using refined methods of analysis to account for the complete interaction of the various elements of the bridge results in better assessment, as noted in the case of Perley Bridge. This approach appears to be satisfactory considering the ability of the existing bridge to carry the test loads without signs of

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