Evaluation of the Capacity of an Existing Steel Truss Bridge
Évaluation de la capacité d'un pont existant à treillis métallique
Ermittlung der Traglast einer bestehenden Stahlfachwerkbrücke

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SUMMARY
The paper presents the evaluation of the load capacity of a top deck steel truss bridge. Conventional analyses, load testing and extensive finite element analyses were used to evaluate the actual capacity. It is shown that conventional analyses are overly conservative compared to the actual bridge behaviour. A load testing programme, coupled with finite element analyses, led to important savings.

RÉSUMÉ
Cet article décrit l'évaluation d'un pont à trellis métallique avec dalle supérieure. Des analyses conventionelles, des essais de chargement et des analyses par éléments finis ont été utilisés afin d'évaluer la capacité réelle. On démontre que les analyses conventionelles sont trop conservatrices. Les essais de chargement couplés avec des analyses par éléments finis ont permis des économies appréciables.

ZUSAMMENFASSUNG
Der Beitrag berichtet von der Ermittlung der Traglast einer Stahlfachwerkbrücke mit oben liegender Fahrbahn. Dabei wurden herkömmliche Berechnungen, Belastungsversuche und umfangreiche Finite-Elemente-Berechnungen eingesetzt. Wie sich zeigt, sind die herkömmlichen Berechnungsverfahren zu konservativ im Vergleich zum tatsächlichen Verhalten der Brücken. Die Kombination von Probebelastungen und Finite-Elemente-Berechnungen ermöglichten bedeutende Einsparungen.

1. INTRODUCTION

Evaluation of existing bridges is a growing concern for bridge engineers. Several options are possible to evaluate the carrying capacity of an existing bridge: conventional or sophisticated analyses and bridge testing.

One can use conventional analyses coupled with design codes modified specifically for bridge evaluation. Frequently this lead to unrealistic evaluations of bridge carrying capacity. However conventional methods have several advantages. They are simple to use and many bridges can be analyzed rapidly. They are thus essential to classify bridges in categories to determine the priority of action and the type of intervention.

The degree of complexity of analyses can vary from simple static or empirical distribution factors to complex finite element. In an other perspective, load test can be performed. The tests can be set up to measure load effects at service load level in various instrumented members, they can be proof tests in which the load is added to the neighborhood of the ultimate capacity or at onset of nonlinear behavior, or they can be performed up to failure.

The combination of a properly carried load test and refined analyses approach an ideal situation. This type of action was undertaken for the Massawippi River bridge.

This paper presents the evaluation of the load carrying capacity of a deck slab steel truss bridge carried using a conventional approach, a load testing program and sophisticated analyses. The aim of this paper is to illustrate the steps involved in the strength evaluation procedure of an existing bridge. The emphasis is directed toward rational strength evaluation involving, when possible, refined analyses coupled with load testing.

2. BRIDGE DESCRIPTION

The Massawippi River bridge, located on Highway 108, south of Sherbrooke Québec, was built in 1937 and is owned by the Ministère des Transports du Québec (MTQ). This 183 m long bridge has eight simply supported spans: six with four reinforced concrete beams, and two with two 32.3 m steel trusses at a skew angle of about 53 degrees (Fig. 1). The top deck above the trusses is made of a 8.7 m wide and 220 mm thick reinforced concrete slab with a 125 mm asphalt cover. The deck also includes stringers and floor beams as shown in Fig. 2. Although the concrete slab was completely replaced some years ago, it was not mechanically connected to the steel beams so no composite action can theoretically be developed. Despite an important number of years in service, the bridge steel and concrete spans are still in good condition and only a limited number of elements require replacement or strengthening. Until recently, the bridge sustained an intense daily traffic of cars and trucks of any configuration and weight since the bridge had no posted weight limits. In an effort to identify all deficient bridges, the Bridge Department at the MTQ evaluates all substandard bridges, starting with those potentially critical located on highways. Considering its age and its location, the bridge evaluation became recently a priority. A conventional strength evaluation was then performed.

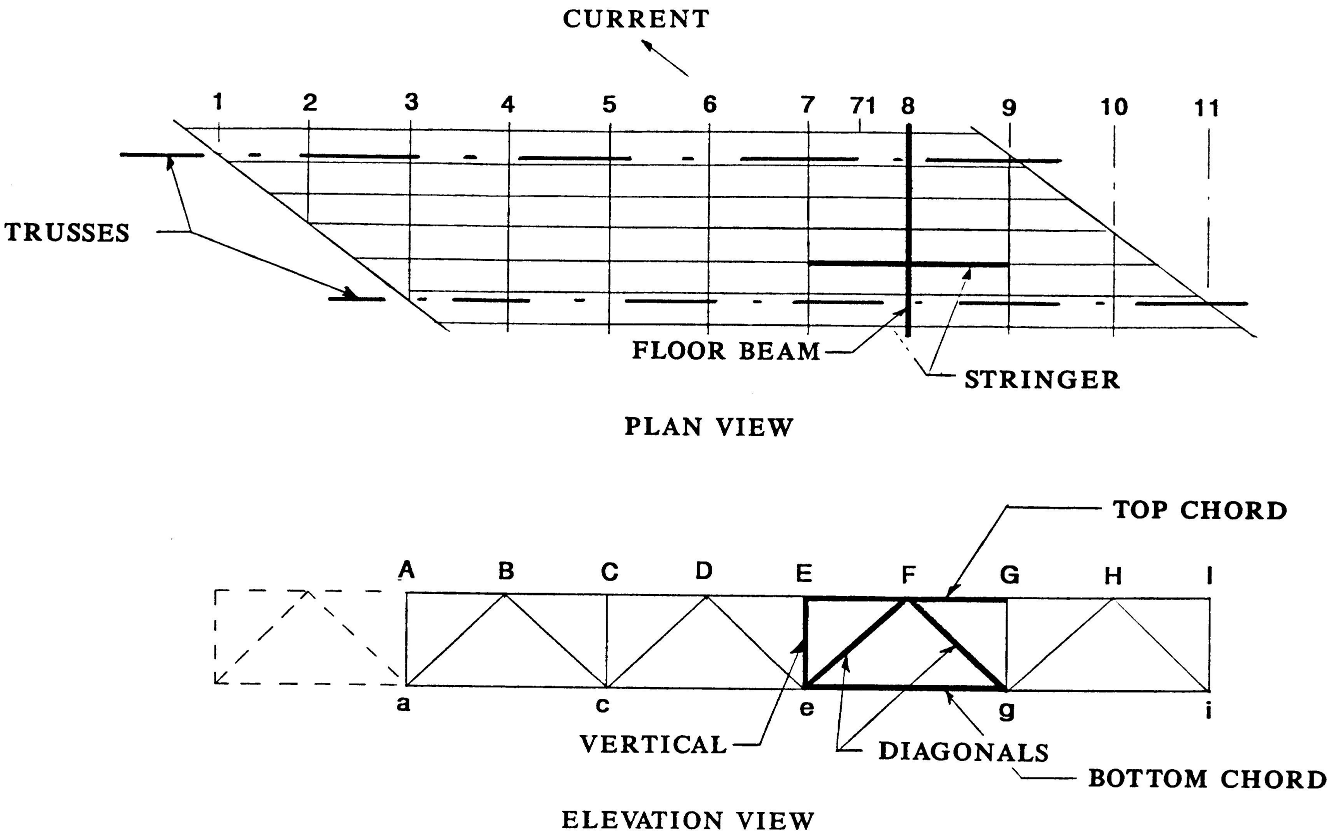

Fig. 1 Geometry of the bridge

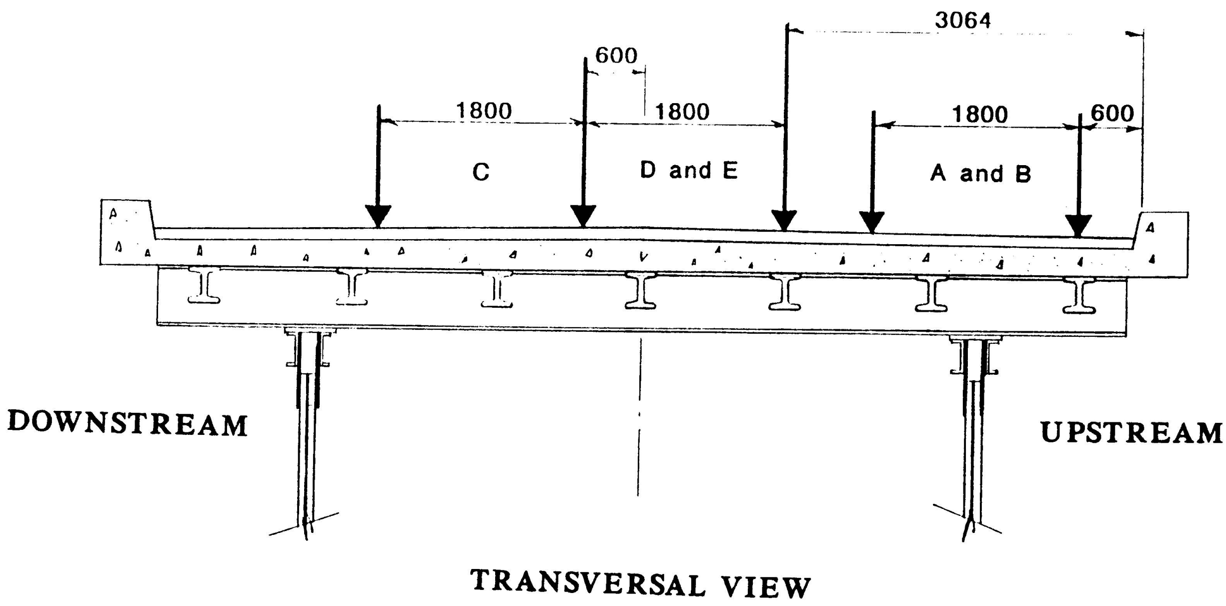

Fig. 2 Deck system

3. CONVENTIONAL STRENGTH EVALUATION

The MTQ Bridge Department evaluated the load carrying capacity of the Massawippi River bridge according to the latest provisions of the Canadian Bridge Code S6-M88 [1]. The assumptions used for the analysis were the same as in the original design. The dead and live load distribution between trusses were obtained following a conventional approach since the bridge structural system is satically determinate. The two trusses were assumed to behave independently, ignoring the transverse bracing in the analysis. Moreover, since no indication was available, no contribution of the deck, from either floor beams, stringers or the concrete slab could be included for the evaluation of the trusses. Also, the supports were assumed free to move horizontally at one end of the trusses.

The live load shearing between both trusses was considered following a conventional approach, with floor beams being simply supported on the top chords of the two trusses. The distribution factor, for the calculation of the most critical live load effects on a given truss according to clause 12 of S6-M88[1],is equal to 1.16 times the loading model. The bridge calculated first natural frequency of approximately 3.8 hertz produces a corresponding dynamic load allowance factor of 40%. The live load rating factors obtained from the evaluation are listed in Table 1 for the steel trusses and the concrete beams. The steel spans exhibited the lowest LLRF giving posting limits of 25, 31 and 36 tonnes for two-axle, semi-trailer and train-trailer trucks respectively.

Table 1: Live load rating factors (LLRF)

Members	2 axles-truck	Semi-trailler	Train-trailler
Tension chord	0.87	0.61	0.54
Compression chord	0.89	0.64	0.54
Tension diagonal:	1.03	0.72	0.64
Compression diag.	0.88	0.85	0.85
Posting (tonnes)	25	31	36

Considering the amount of vehicles traveling on the bridge daily, its location and its economical importance, it was decided to increase the bridge carrying capacity to legal load limits. However, due to the high cost of strengthening the steel trusses, it was decided to postpone the intervention to give engineers the time to explore other alternatives.

4. LOAD TESTING PROGRAM

In 1990, the MTQ acquired a mobile laboratory dedicated to bridge testing. This mobile unit has two data acquisition systems for static and dynamic load tests. The Massawippi River bridge test was the first duty for the mobile laboratory team and its new acquisition.

The instrumentation of the bridge lasted 15 days during which 49 strain gages were installed in a section of the upstream truss. Members instrumented within the truss were: 1 bottom chord, 2 top chords, 1 vertical member and 2 diagonals (Fig. 1). One member of a vertical bracing between the two trusses, 1 floor beam and 2 stringers were also instrumented. Finally, 1 strain gage was installed on the concrete slab soffit, parallel to a strain gage on a stringer, to give some indication on the composite action between the concrete slab and the steel beam.

The test itself took placed on September 26, 1990, and lasted 3 hours, during which the bridge was closed intermittently. The loading vehicle was a 43 tonne 5 axle semi-trailer truck, loaded with gravel, carefully weighted and measured. Five load paths along the bridge, named A to E (Fig. 2) and identified with marking lines on the road, were used to measure load effects on truss members.

For all members the axial force and bending moments were calculated from the strain measurements. For most of the truss members, the axial load was predominant. However even small bending moments measured justified the utilisation of 4 strain gages per members. Without then the measurements could have lead to unrealistic results with significant errors. The floor beams and stringers exhibited a certain degree of composite action with the concrete slab.

A comparison of load effects predicted by the conventional analysis and the corresponding measured values, clearly indicated a significant discrepancy between the behavior observed in the test and the one assumed in the analysis. The diminution of live load effects measured in the test, compared to the values obtained using the same assumptions as in the evaluation, were up to 67% for the tension and compression chords whereas it was 39% for the diagonals. This indicates that the bridge capacity obtained initially could be modified using different assumptions for the behavior.

Although field measurements represent the actual bridge behavior, test results were available only for 6 of the truss 25 main members. This amount of information is not sufficient to indicate clearly the reasons for such discrepancy and safely allow for any significant increase in the bridge load carrying capacity. It was therefore appropriate to proceed with sophisticated analyses using the finite element approach to study more deeply the bridge behavior.

5. FINITE ELEMENT ANALYSIS

Linear finite element analyses were carried out to model correctly the bridge. A 3-dimensional model was created in which steel trusses, floor beams, stringers, all bracing and the concrete slab were carefully discretized. To model adequately the bridge behavior, two factors were used for calibration: the longitudinal restraint of the truss supports and the participation of the deck system, including floor beams, stringers and the concrete slab, in the carrying mechanism.

The first parameter only accounts for the horizontal restraining action of free supports at one end of the trusses. Although they should theoretically allow free longitudinal movements of the bridge, the truss supports were rusted and suspected to be frozen. Analyses involved only two cases for the movement: free or fixed. To model the effects of the second parameter, two independent meshes were used to discretize the bridge. A first mesh describes the two trusses and the bracing system joining them (horizontal and vertical), and a second mesh for the deck system: the concrete slab, the floor beams and the stringers (Fig. 1). Vertical connection members between the two meshes were used to simulate various degrees of participation of the deck system in the load carrying mechanism.

Calibration of the two parameters was achieved with only one loading case. From the load effects in the bottom chord, it became rapidly obvious that the mobile supports were actually frozen horizontally. The second step in the calibration process involved only the stiffness of the connection member which was then modified in a trial and error process until satisfactory agreement between test results and analysis were obtained for this load case. The model was then considered adhequate to represent satisfactorily the bridge behavior and all load cases of paths A, B and C were analyzed. Some of the results are presented in Figs. 3 and 4 for the load path B, together with results of the conventional analysis used initially for the bridge evaluation and test measurements.

A close examination of the results indicates a significant reduction in load effects in top and bottom chords due to some composite action between the the trusses and the deck system. This interaction, although partial, increases the effective inertia of the two trusses which reduces the live load effects on horizontal chords. The skewness of the bridge deck induces an important load transfer between the two trusses through the vertical bracing. Frozen supports have also some influence on the load carrying mechanism making the bridge working like an arch.

These effects, important for the truss top and bottom chords, are however less significant for web members. For the diagonals and the vertical members, a less pronounced live load reduction is observed since vertical shear forces are mainly carried by the trusses. The modification in the load carrying mechanism can be observed by comparing reactions from dead load effects at truss supports. In Table 2 reactions calculated using the various approaches are presented. These results indicate that the dead load distribution is notably affected by the skew angle and the transversal bracing. However, for most of the truss members, the results indicate that the composite action has a more important effect than the frozen supports.

Table 2: Dead load reactions at supports

Support	Conventional 2D		Conventional 3D		Finite element	
	Free	Fixed	Free	Fixed	Free	Fixed
Other-most	661.5	661.5	507.4	579.9	436.1	523.1
Inner-most	697.0	697.0	853.3	827.0	922.4	863.8

6. EVALUATION OF THE NEW LOAD CARRYING CAPACITY

The confidence gained in the comparison of the finite element model with test results, allows one to proceed further and use the finite element model to calculate the live load effects in the truss members for posting. However, the evaluation of the actual carrying capacity using the finite element model must consider assumptions on the conservative side which can differ from those used for the comparison with test data. The contribution of the deck and the skewness of the bridge have a clear effect on the bridge behavior and thus on load effects supported by the bridge main members. These beneficial aspects of the bridge behavior are assumed to remain the same at service load level. However, the role played by frozen supports in the apparent gain in carrying capacity must be reconsidered.

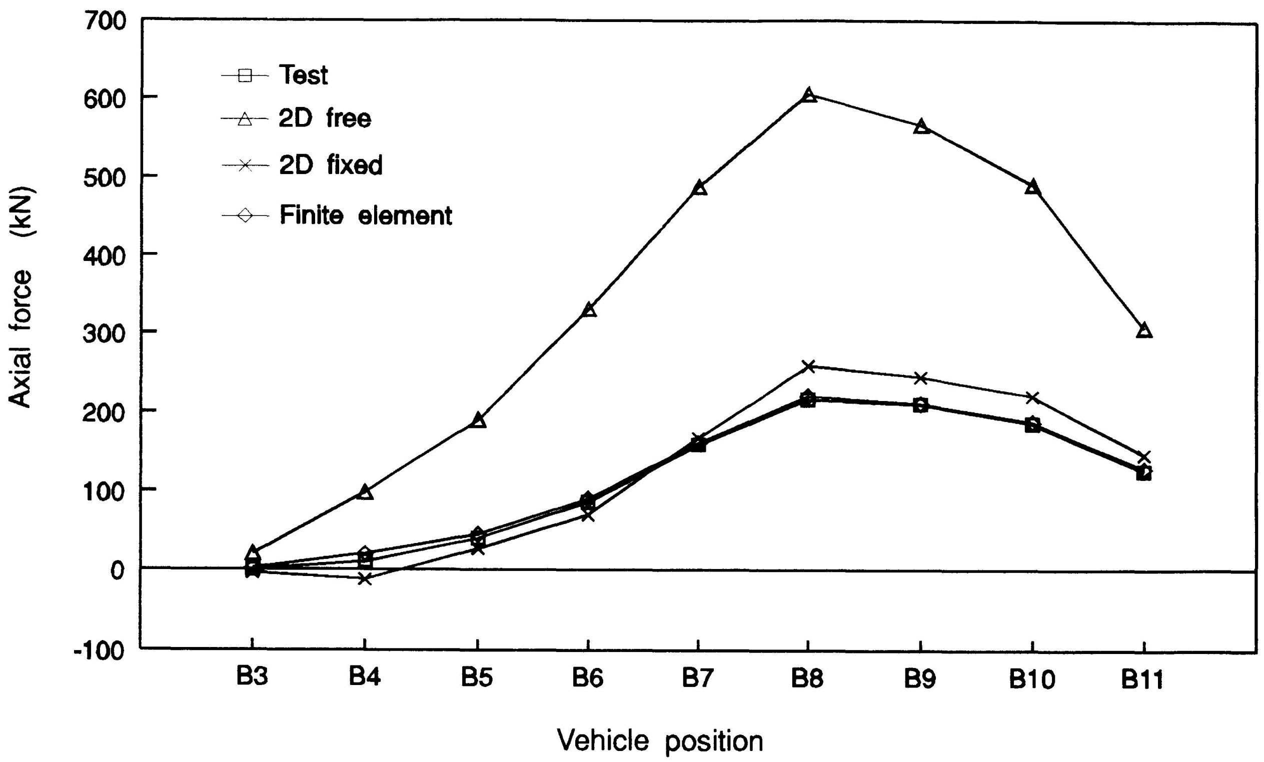

Fig. 3 Axial forces in the bottom chord

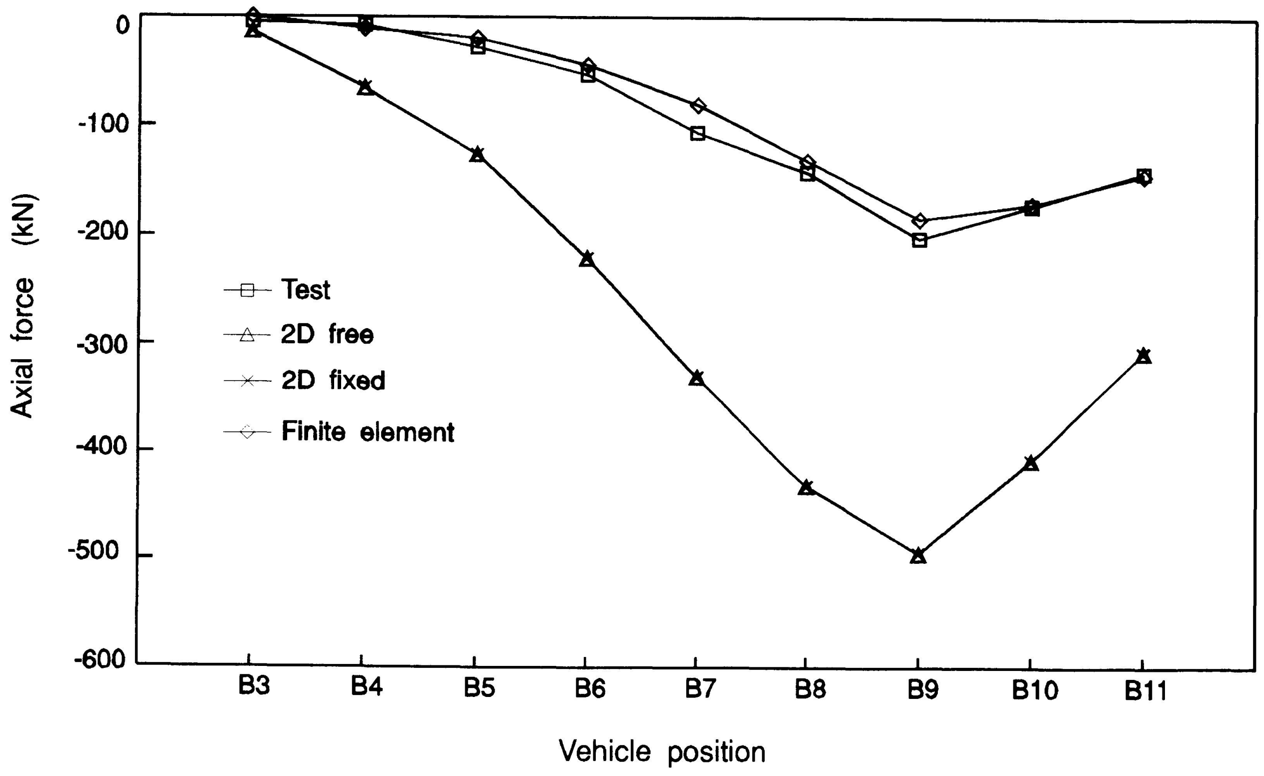

Fig. 4 Axial forces in the top chord

This assumption, although based on careful measurements and sensitive analyses, confront engineers with a dilemma: should the analysis to determine the actual carrying capacity be done with frozen supports or not? On the other hand, is it reasonable to repair the frozen supports to allow free longitudinal movements since the bridge apparently behaves correctly in its current situation? The answer are not simple and this matter requires special attention to be assess correctly.

For weight limit posting, both free and frozen support conditions at one end of the bridge were considered. Since it is difficult to determine accurately the bridge history, the dead load effects retained were those obtained from the conventional analysis or calculated using the finite element model with free or fixed supports, whichever produces the worst effects. For live load effects, the values obtained with the finite element model with free supports at one end were used.

An interesting fact observed in the analysis is that the most critical member governing posting is the vertical strut a-A (Fig. 2) located at the inner-most corner which carries higher compression forces due to load transfer between the two trusses through the vertical bracing. The weakness of this member was not identify in the conventional analysis. This mean that classical analysis although usually on the safe side, may sometimes be unconservative.

7. STRENGTHENING

Although posting limits did not apparently increase very much after all these efforts, the top and bottom chords, the weakest members previously, were not critical any more and only web members remain critical. Thus only the strengthening of a few members is now required, bringing down the costs by more than $200 000. This economy almost justifies by itself the acquisition of the mobile laboratory. The strengthening of the bridge was done in the summer of 1992.

8. CONCLUSION

The need for strengthening a bridge and the cost involved justified the utilization of a field testing campaign combined with refined analyses to increase the understanding of the load carrying mechanism of the bridge. This effort resulted in a slight increase in the actual safe carrying capacity of the bridge and in a significant reduction of members requiring strengthening, leading to important savings. The success of the project is an excellent example of the benefit obtained when advantages of two complementary approaches are efficiently combined.

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