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# Impact of Site Measurements on the Evaluation of Steel Railway Bridges

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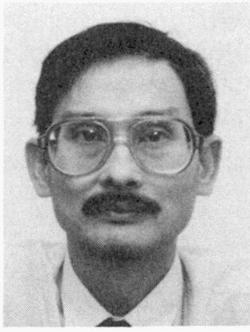

## Summary

This paper provides a summary of field measured static point stresses compared to theoretical stresses, as well as measured dynamic stresses compared to theoretical stresses including impact calculated as specified in the American Railway Engineering Association (A.R.E.A.) Manual. It also proposes values for the Alpha factor and percentages of the A.R.E.A. impact values that are more adequate for the evaluation of fatigue life.

## 1. Introduction

In anticipation of increased axle loads, the Engineering Department of the Canadian National Railway Company (CN) has undertaken a major bridge testing program since 1988 as an adjunct to its rating program. The purpose of this program is to ensure the safety of its aging bridge plant, to prolong its life and to prioritize replacement and strengthening programs. Most of the main lines are or will be supporting 130 tonne cars (286,000 lb.) in unit trains, and 6-axle 191 tonne (420,000 lb.) locomotives.

This paper summarizes and reviews the results of 69 full scale field tests [1] of fatigue sensitive members such as bottom flanges of plate girders & stringers and bottom chords of through trusses and deck trusses. The data presented is based on maximum recorded point stresses and maximum measured impact at the maximum recorded stresses at or near the fatigue limit state.

This paper also briefly discusses whether the use of the Alpha factor and impact factor as specified in the current American Railway Engineering Association (A.R.E.A.) Manual [2] is appropriate in estimating the fatigue life of railway bridges.

## 2. Testing

Static and dynamic effects were measured using a pre-weighed work train under controlled conditions. Generally, the work train consisted of one or two locomotives followed by six or more cars fully loaded and sometimes followed by three empty cars. The tests were conducted at various speeds ranging from crawl speed to a maximum of 110 km/hr. (70 mph) for freight trains and 180 km/hr. (110 mph) for passenger trains. The maximum allowable speed varied depending on the zone speed of the line.

## 3. Selection of spans for bridge testing

The basic concept of bridge rating and safe life evaluation used by CN's Bridge department is a multiple step procedure varying from a simple check against provisions similar to those contained in Chapter 15 of the A.R.E.A. Manual [2], to a full scale load testing and crack evaluation.

The first step involves checking critical details against the design provisions of the Manual. If they are adequate, no further action is warranted.

Next, a detailed analytical evaluation is made using the approved rating and fatigue procedures. If the span and details in question pass this test, no further action is warranted.

If the previous steps reveal structural inadequacies, and the cost of replacement or repair is high compared to the cost of a successful load test, the structure is then load tested. Line importance also plays a major role in selection of bridges for testing.

## 4. Description of bridges and spans tested

Between 1988 and 1995, CN's Bridge department has carried out over 69 field tests. The majority of the bridges tested were on the main line supporting traffic up to 40 Million Gross Tonnes per km. (70 MGTM). Most of the traffic in Eastern Canada is of mixed type while most of the traffic in Western Canada consists of unit trains.

The tests were conducted on various types of spans, a majority of which were built around the turn of the century. Included in these tests were 28 through truss spans, 13 deck truss spans, 6 through plate girder spans and 22 deck plate girder spans.

The truss spans investigated were of riveted construction. Generally, the construction was typical of turn of the century designs. The top chords and compression members were built-up sections, while the bottom chords and other tension members were either built-up members or eye bars with or without pin plates.

All the plate girder spans were of riveted construction except one welded span, and the beam span was built using rolled I-beams.

Decks were generally open deck timber. There were three ballasted type decks. The rails were generally 68 kg/m (136 lb./3ft) continuous welded rails on heavy tonnage lines with or without "Conley" expansion joints to 57 kg/m (115 lb./3ft) jointed rails on the low tonnage lines.

The substructures consisted of stone masonry or concrete piers & abutments, steel towers and pile bents. Conditions of the bearings ranged from satisfactory to poor. In order to simulate every day field conditions, approaches were not surfaced or tamped for the tests.

## 5. Alpha factor

The alpha factor is defined as the ratio of the field measured static live load stress to the theoretical static live load stress. Caution should be applied when using this factor for bridge rating and predicting the remaining life, since there is no built in safe guard against unintentional errors in testing and theoretical stresses are computed according to the rating guidelines, which do not necessarily reflect true boundary conditions.

## 6. Discussion of the results

The field measured stresses were compared with the theoretical stresses calculations based on simple analytical models (as used in normal bridge rating practice). All of the data was taken at temperatures above the freezing point (0°C). The measured data are in the raw format without any adjustments. The measured stresses do not include dead load and are typical of the live load stress ranges that cause fatigue damage in North American railway bridges.

Figures 1, 4 and 7 show the comparison of site measured static stresses (crawl speed) under work trains to theoretical stress. Figures 2, 5 and 8 show the comparison of maximum site measured dynamic stresses to theoretical stresses with full impact as defined by A.R.E.A. chapter 15. Figures 3, 6 and 9 show the comparison of site measured dynamic stresses to theoretical stresses with modified impact values (expressed as a percentage of the theoretical impact computed as specified in the A.R.E.A. Manual). Those reduced impact values were chosen in a conservative way, such that the measured stresses are still slightly lower than the total theoretical values with only a few exceptions.

The range of loaded lengths for the members tested are shown on each of the stress comparison figures. All plotted values are the maximum values recorded and do not represent the average cross-sectional stresses nor are the effects of bending, torsion or axial loading shorted out. Solid symbols indicate data from the ballasted deck structures.

### 6.1 Girder Spans

Measured static stresses in girder bottom flanges are plotted against the corresponding theoretical static stresses in Figure 1. The Alpha factor varied between 0.34 and 1.11. It is clear from the data that an Alpha factor of 0.85 as specified in the A.R.E.A. Manual is too low, and that an Alpha of 1.0 is a more appropriate assumption to make in evaluating most physically untested railway bridge girders. The two occurrences of an Alpha factor in excess of 1.0 came from two simple Deck Plate Girder spans. Figure 1 shows the advantage of bridge testing for most cases.

Figure 2 shows the same work trains with full impact (dynamic factor). The ratio of measured dynamic stresses to theoretical dynamic is less than 1.0 in all the recorded cases. Clearly, the impact formula specified in the A.R.E.A. Manual, originally derived for a rare event [3], is not appropriate for fatigue calculations.

Figure 3 is a modified version of Figure 2, with impact reduced to 10% of the A.R.E.A. value. Based on this data, one can conclude that for fatigue evaluation, it is quite safe to use an impact factor equal to 10% of the A.R.E.A. impact value, under certain operating conditions.

### 6.2 Stringers

As seen in Figure 4, the Alpha factor for stringer bottom flanges ranged from 0.32 to 1.35, and was generally less than 1.0, except for a few cases. In those few cases, the floor system had multiple stringers per rail. One stringer would record high stresses, while the adjacent ones would record low stresses. The uneven distribution of loads is due to small differences in elevation that prevent the ties from resting properly on some of the stringers. Again the Alpha factor needs to be 1.0, and not 0.85 or 0.8 (A.R.E.A. values for spans less than and greater than 23 meters respectively).

Figure 5 clearly illustrates that the ratio of measured dynamic stresses to theoretical stresses (including full theoretical impact) is less than 1.0.

Figure 6 shows that for fatigue life evaluation, it is safe to use a reduced impact factor equal to 25% of the theoretical impact specified in the A.R.E.A. Manual.

### 6.3 Truss Spans Bottom Chords

Figure 7 shows static stresses measured in the of bottom chords of the truss spans plotted against corresponding theoretical stresses. Except for one case, all the measured stresses were lower than the theoretical stresses. The Alpha factor varied from 0.42 to 1.0. The exception was a double track pin connected "fish belly" deck truss. The Alpha factor of 0.70 as specified in the A.R.E.A.

Manual is too liberal. A value of 0.95 might be more appropriate. However, it is our opinion that an Alpha factor of 1.0 should be used.

Figure 8 shows that, as in the earlier cases, the ratio of measured dynamic stresses to theoretical stresses (including full theoretical impact) is less than 1.0.

Figure 9 shows that even with no impact applied to the theoretical stresses, the measured dynamic stresses are still less than the theoretical stresses (with 1 exception). Our recommendation would be to use some nominal value for the theoretical impact (say equal to 10% of the A.R.E.A. value).

## 7. Concluding remarks

Measured static stresses and impacts outlined in this paper are generally lower than stresses calculated using conventional analytical techniques. In some cases remedial measures can be delayed for long periods of time. In overstressed member, testing will often point the way to less expensive retrofits, repairs or strengthening. In the majority of cases, bridge testing saves money.

Nevertheless, since the test data shows that there are many exceptions, it is not recommended to blindly assume that such is always the case. *A reasonable upper bound for static data on the three classes of members reviewed in this paper requires an Alpha factor of 1.0.* Without some field testing it is not appropriate to assume a lower value.

Impact factors originally designed to cover occasional occurrences during the life of a structure are much too conservative. The test data indicates that for fatigue evaluation purposes, impact factors lower than those specified in the A.R.E.A. Manual can be used. Our recommendation for appropriate impact values are: 10% of the A.R.E.A. value for girder bottom flanges, 25% of the A.R.E.A. value for stringer bottom flanges, and 10% of the A.R.E.A. value for truss bottom chords.

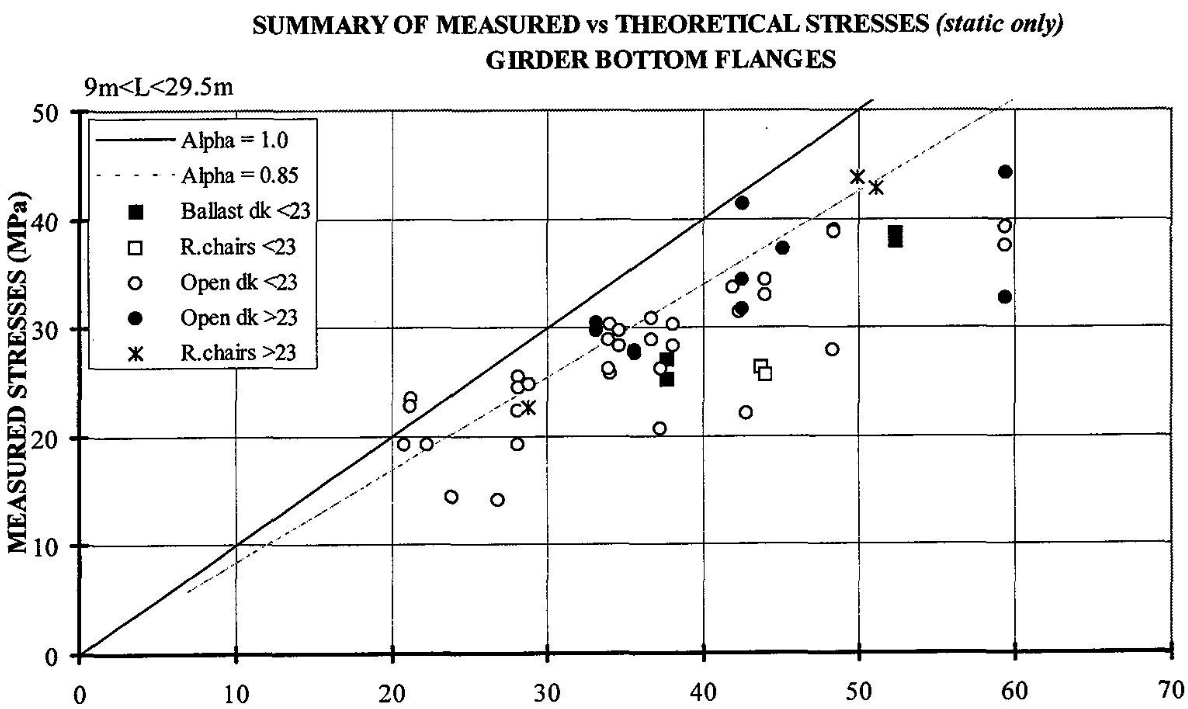


**Figure 1**

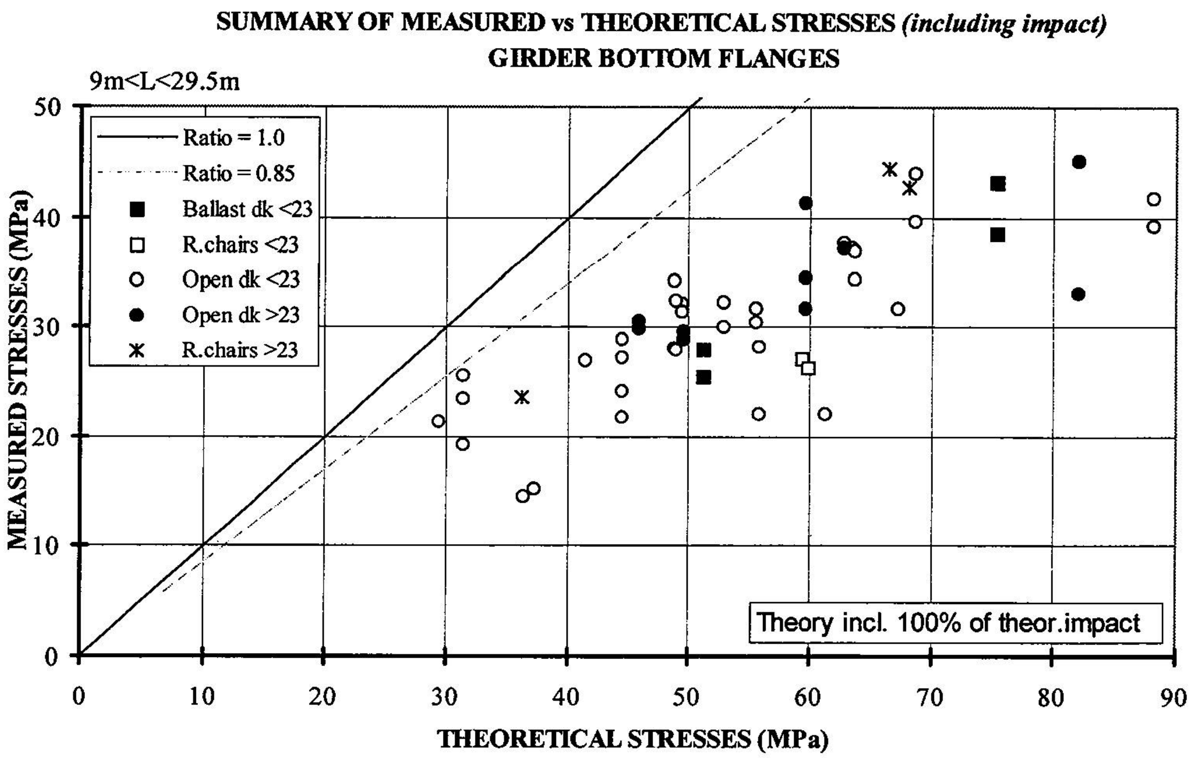


**Figure 2**

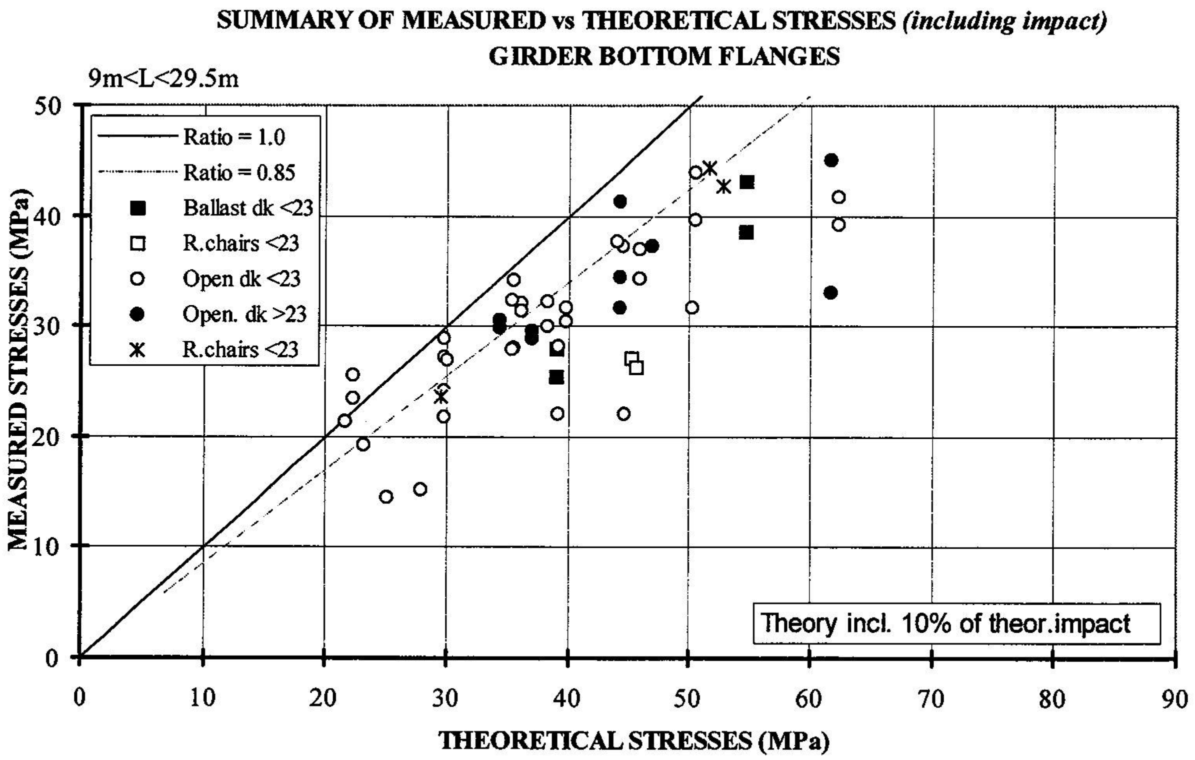


**Figure 3**

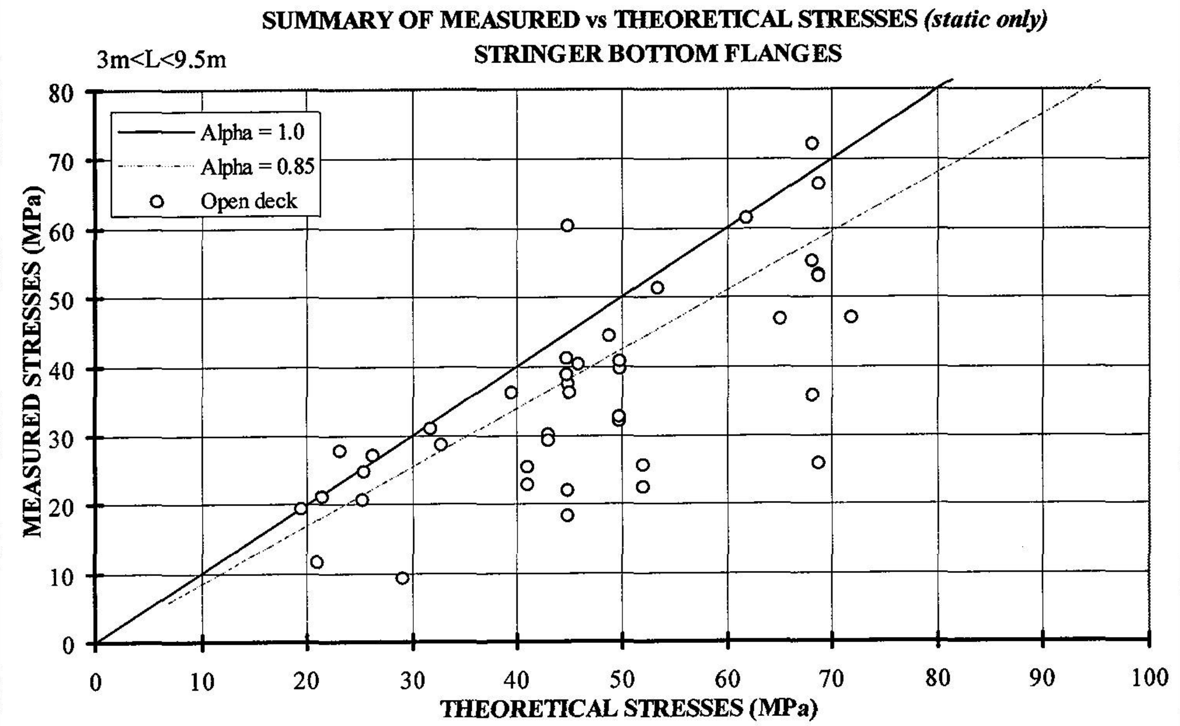


**Figure 4**

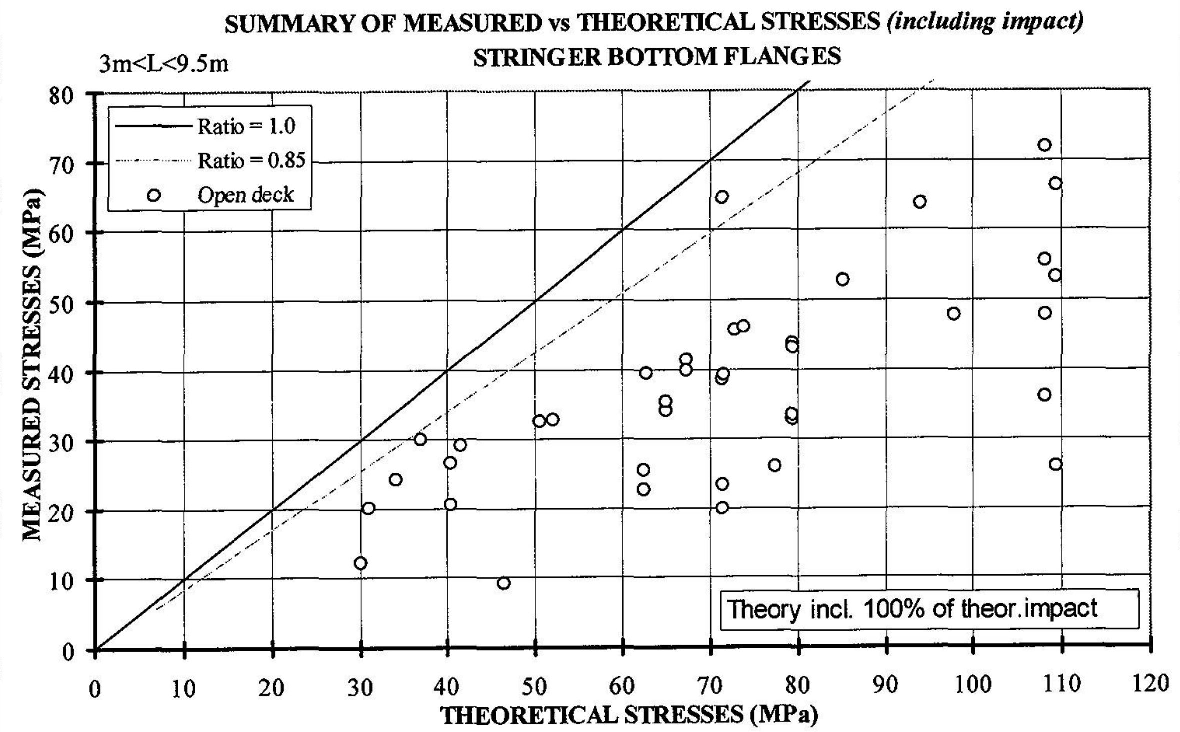


**Figure 5**

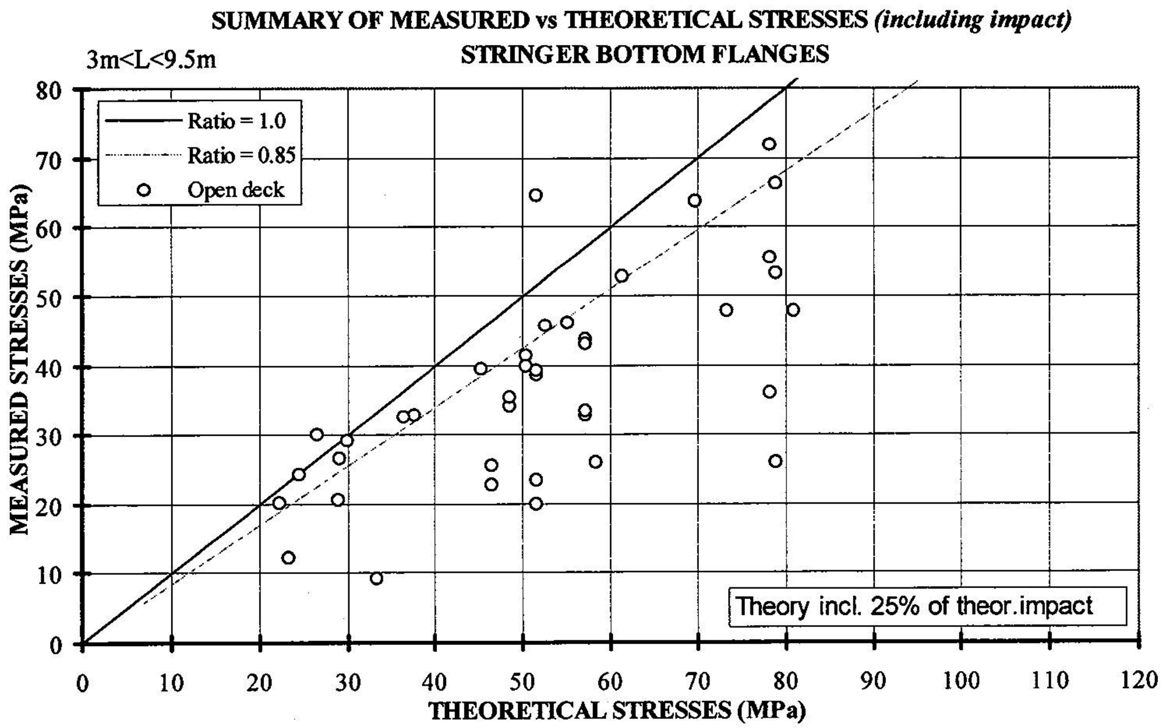


**Figure 6**

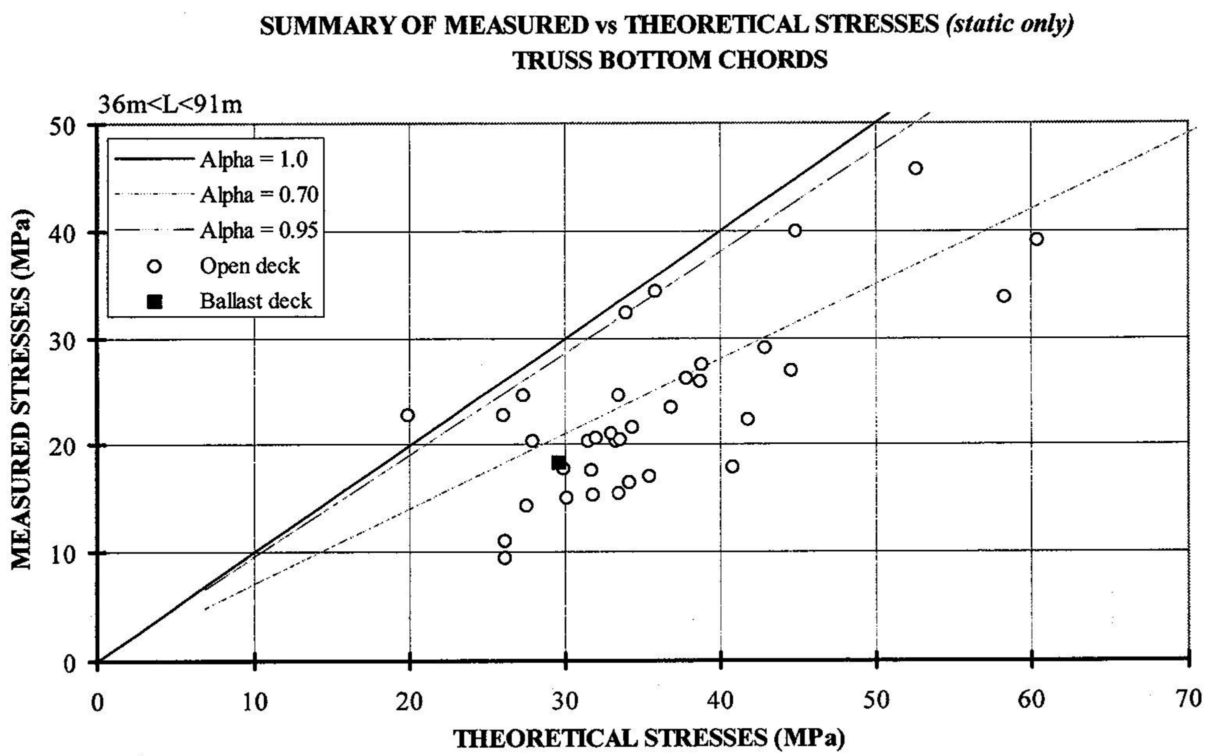


**Figure 7**

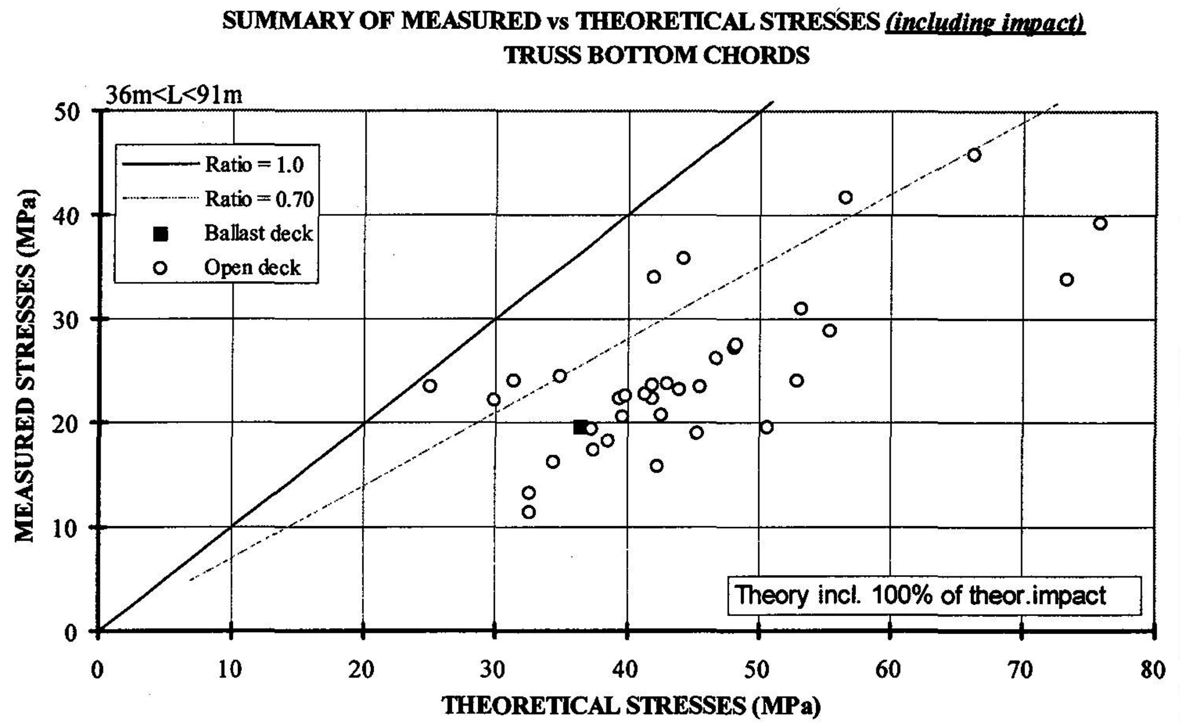


**Figure 8**

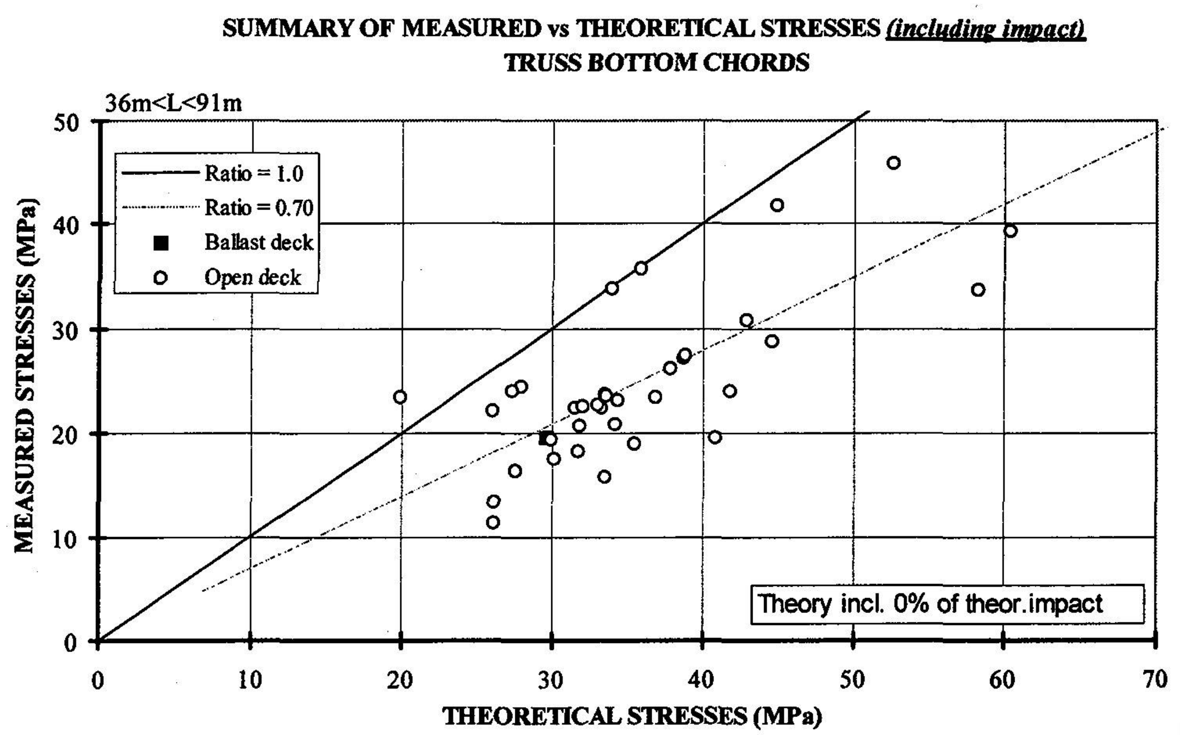


**Figure 9**

## Acknowledgments

The authors wishes to thank Romel Scorteanu, Project Engineer, who was responsible for the data acquisition, for his contribution, present and former members of the Rating and Testing groups at CN, and R.W. Richardson, Chief Engineer, for his continued support.

## Disclaimer

The data provided in this report is of a highly technical nature and greatly summarized. Readers need to be extremely cautious in its use.

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