TEST RESULTS FROM THE CONESTOGO RIVER BRIDGE

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The Conestogo River Bridge is a three span continuous steel plate girder structure with a concrete deck longitudinally prestressed for full composite action. The design live loading was an upper-bound representation of all observed truck loadings in Ontario, and a grid analysis was used instead of the usual live load distribution factors. A new load factor method was employed, and deflection and impact values provided by the AASHTO Specifications were abandoned in favour of a dynamic analysis, resulting in a very flexible structure with low natural frequencies. Other features include the use of high strength bolts as shear connectors, and slab test panels designed for membrane action with greatly reduced reinforcing steel percentages. On the basis of full scale load tests, the advisability of the regular use of these features on future bridges is determined. The testing program was conducted using two test vehicles, each of which could be loaded to 890 kN (200 kips). Static tests indicate good correlation between analytical and experimental strains, with load distribution significantly better than predicted by AASHTO Specifications. Dynamic tests indicate that this very flexible bridge, with a first mode frequency below the usual range of commercial vehicle frequencies, behaves as anticipated. The valuable information obtained from this project is incorporated in the new Ontario Highway Bridge Design Code.

The Ontario Ministry of Transportation and Communications has a research program comprised of in-house research and that performed by associated university and consulting engineering groups. The findings of these various research projects have culminated in the design and subsequent testing of a prototype bridge, under actual highway and environmental conditions. Proposals on such items as live loading, bridge dynamics, distribution factors, load factor design, shear connectors and slab membrane action could be assessed prior to the possible inclusion in the new Ontario Highway Bridge Design Code.

The crossing of the Conestogo River near Waterloo Ontario on Highway 85 was selected as the location of the prototype bridge. The structure was designed by the Structural Group of the Research and Development Division of the Ministry in 1973 with construction completed in the summer of 1975. Testing of the bridge began during construction, with much of the initial load testing conducted in 1975. Further static and dynamic tests have taken place since this first series and future tests are anticipated with the structure in use.

The resulting bridge is a three-span continuous, steel plate girder structure with an overall length of 113.7 m (373 ft) and a central span of 44.2 m (145 ft) as shown in Figures 1 and 2. The welded girders are haunched to a depth of 2.13 m (7 ft) over the piers, and 1.07 m (3 ft 6 in) at midspan. The deck between barrier walls is 10.36 m (34 ft) with two 3.66 m (12 ft) travelled lanes and two 1.52 m (5 ft) shoulders. Four girders are spaced at 2.64 m (8 ft 8 in) with cross frames at a maximum of 7.47 m (24 ft 6 in) centres, and bottom laterals connecting the outer pairs of girders (Figures 3 and 4). Composite action in the negative moment region is achieved by the longitudinal prestressing of the nominal 190 mm (7½ in) thick slab. The structure thus acts in a fully composite manner for the full length of the bridge under both superimposed dead load (consisting of sidewalk, curb and future waterproofing) and live load.

Design and Construction

The recommendations as outlined in the Proposed Ontario Bridge Design Load (POBDL) report (1) were used for most of the Conestogo River Bridge design. Items not covered in this report (web stiffening, fatigue, flange buckling, splices and connections) were designed using the applicable sections of the 1973 AASHTO Specifications (2) for Load Factor Design.

The POBDL live load – Case 3 (Figure 5) representing a design vehicle of 722.8 kN (162.5 kips), generally governed for the moment design. A maximum number of four vehicles (two per lane) was used since the structure is on a expressway class highway. Where critical moments arose by the loading of a shorter length influence line, a reduced UDL was used as in POBDL Case 2.

AASHTO specifies a distribution factor of S/1.68 (where S is the girder spacing in metres) for two or more traffic lanes on a concrete slab on steel girder type of bridge. It is generally accepted that this value tends to be overly conservative for certain types of bridges. A grid analysis program was used with the POBDL load to obtain the appropriate distribution factors for flexural effects at 0.4 end span, pier and 0.5 centre span. The computed distribution factors were found to vary with vehicle location. A value of S/2.29 was selected for use in the flexural design. Although somewhat unconservative for the exterior girder at the piers, it was felt that since the maximum negative moment did not occur with the vehicles directly over

Figure 1. The Conestogo River Bridge Under Static Test

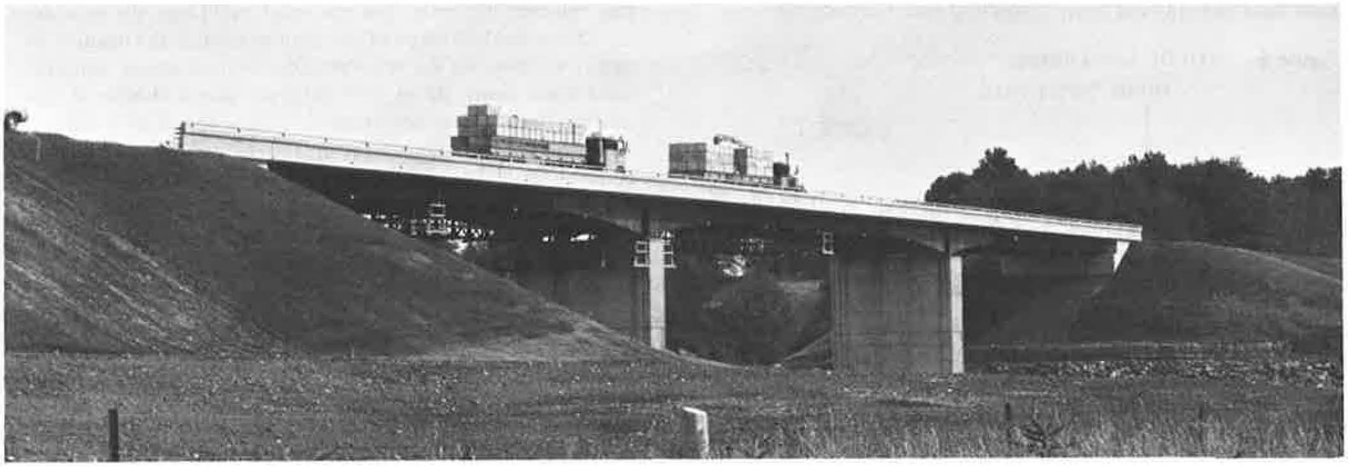


Figure 2. Bridge Elevation

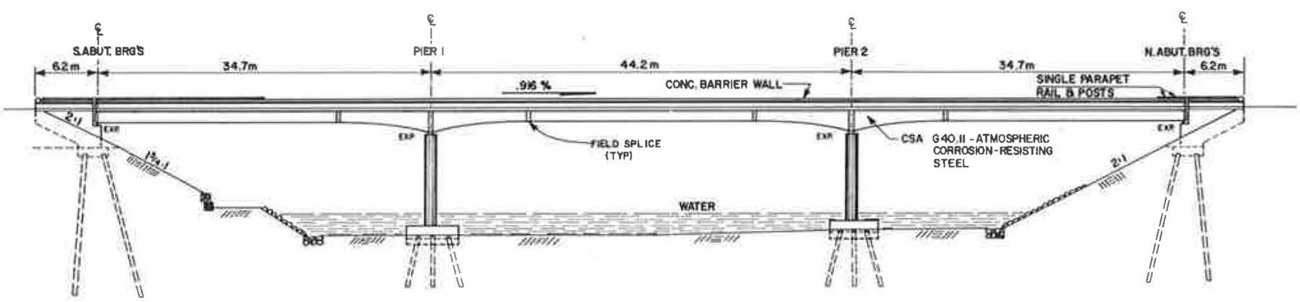


Figure 3. Half Bottom Chord Plan

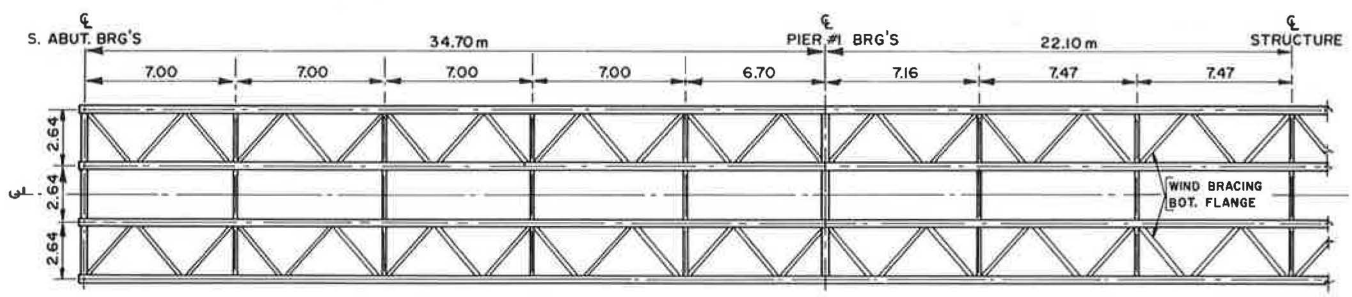
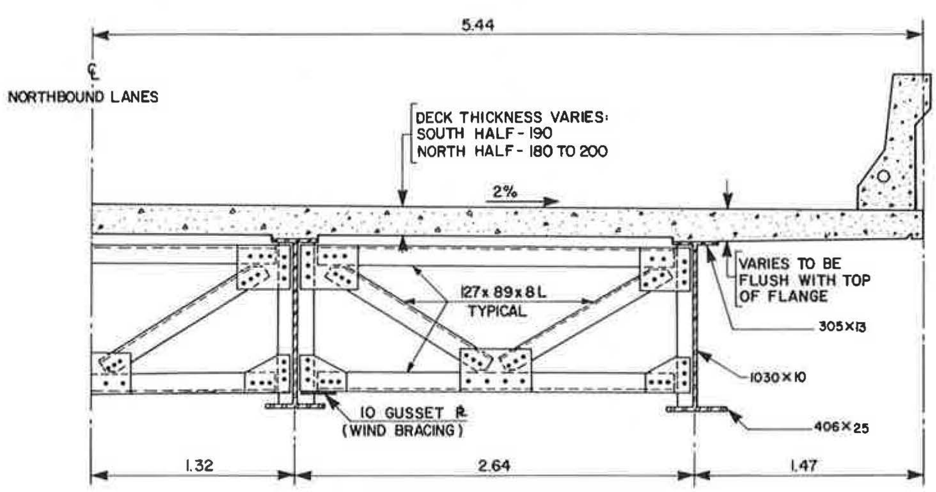
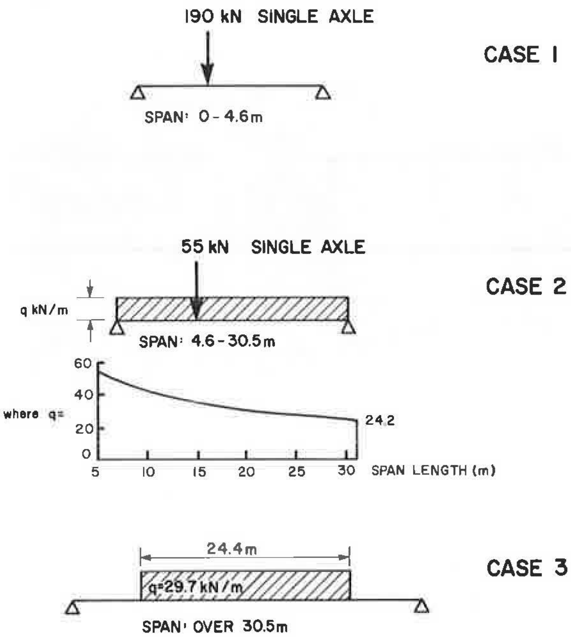


Figure 4. Half Cross Section



the pier, the values selected would be adequate for design purposes. This compares quite favourably with that observed in both field tests (3) and other theoretical investigations (4).

Figure 5. POBDL Live Loading



A live load variability factor of 1.10 (to account for the possibility of an illegal overload) was used in addition to the live load factor inherent in the 722.8 kN (162.5 kip) POBDL vehicle. This results in an overall live load factor of 1.276 on the maximum legally permitted load of 622.7 kN (140 kips). Dead load factors reflecting the variability of in-situ material weight vary with construction and fabrication methods (1.10 for structural steel and concrete deck slabs, 1.15 for concrete barrier wall, 1.33 for asphalt). In addition to these load factors, a capacity reduction factor of 0.85 was applied to the yield stress limit state for steel.

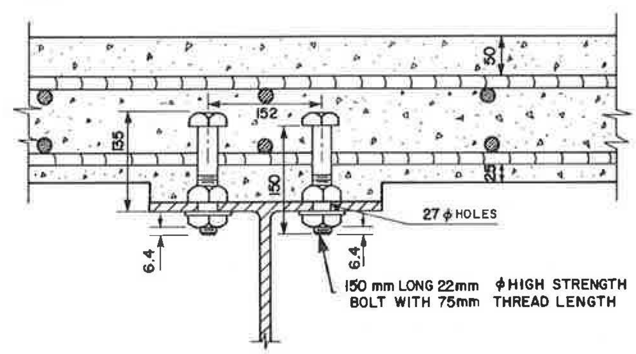
The impact factor is of the same configuration as the standard AASHTO provision, however its magnitude was chosen after a careful appraisal of the expected bridge-vehicle interaction. It was felt that if the first mode frequency could be sufficiently removed from the 2 to 5 Hz band of vehicle frequencies, the dynamic effects would be significantly lower than the almost 90 percent impact value measured in several field tests where such a frequency match was evident (5). A one-dimensional lumped mass analytical model was used to predict the natural frequencies. The resulting first mode frequency was 1.53 Hz with the second and third modes falling in the 2 to 5 Hz band. To control the likelihood of resonance and obtain a reliable design impact factor, a model of the bridge was constructed and tested with a vehicle of variable natural frequency and mass. The tests confirmed that resonance was unlikely in all but the second and third modes, and then only when the vehicle had been pre-excited by a bump prior to entry on to the bridge. A constant impact value of 45 percent of the maximum mid-span moment was used for design.

Girder bending stiffness was lowered considerably in order to remove the first mode frequency from the 2 to 5 Hz band. This increased deflections so that the live load deflection at mid-span was 100 mm (3.9 in) or 1/450 of the central span length. The current specification of 1/800 of the span length is based upon the theory that by limiting deflection, the causes of "objectionable" vibrations as perceived by pedestrians, would be alleviated. Since the Conestogo structure is on a limited access highway, pedestrians would not normally be on the structure and thus this deflection constraint could logically be waived.

Since the UDL type of live load as used in the bending design is not as severe for shear considerations as several concentrated loads, design shears were obtained using a number of axle configurations and vehicle weights. A load factor of 1.276 was used with a distribution factor of S/2.16 for computation of girder shear, while a simple span type of distribution was used for girder reactions at abutment and pier.

In order to prevent the concrete deck from cracking in the negative moment region and thus losing the fully composite action, it became necessary to longitudinally prestress the deck. The bolt configuration (Figure 6) consisting of a 22.2 mm (7/8 in) A325 Type 3 bolt with extra threaded length and two nuts in bearing on the top flange. A two nut system was required to pre-set the location of the bolt in the hole. The bolts were to be placed to one end of oversized holes in the top flange to allow shrinkage and elastic shortening from the slab stressing to take place freely without stressing the steelwork. These high strength bolt shear connectors were designed initially in accordance with the AASHTO Specifications for welded stud connectors and load factor design, using POBDL loading and substituting bolts for studs. They were thus designed for ultimate strength, then checked for fatigue.

Figure 6. Shear Connector Details



Although the use of high strength bolts as shear connectors has been studied (6), this particular assembly has not been tested before. Consequently a testing program was established to check if such a bolt would form as a shear connector at least as well as an equivalent diameter standard welded stud connector, if the load-slip relationship would be affected by the 4.8 mm (3/16 in) oversize hole and if the presence of threads in the shear plane would affect the fatigue life of the connector.

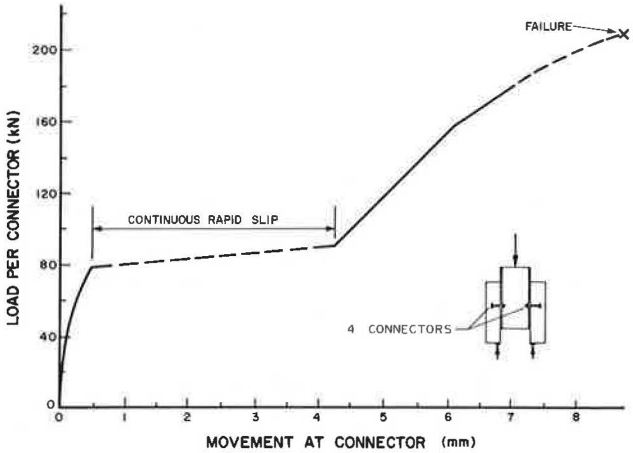
The results of the static slip tests are shown in Figure 7. Rapid slip takes place at 78 kN (17.5 kips) to 89 kN (20 kips) until the bolts come into bearing. This slip load is in excellent agreement with the proposed Ontario Highway Bridge Design Code value for slip resistance at the serviceability limit state for grit blasted studs of 79 kN (17.8 kips).

One specimen was taken to ultimate load, when the bolts sheared at an average load of 206 kN (46.2 kips) each. With the concrete strength of the slabs at 29.6 MPa (4300 psi), the calculated ultimate strength of an equivalent diameter stud shear connector is 167 kN (37.6 kips). Excluding the performance factor for the bolt of 0.67, the shear resistance of the bolt, according to the Ontario Highway Bridge Design Code is 185 kN (41.5 kips).

A second specimen was fatigue tested at a load range of

44.5 kN (10 kips) per bolt for 500,000 cycles with no slip. The load was incrementally increased, reaching 111 kN (25 kips) at 1,000,000 cycles with no slip. The test was terminated at 1,835,000 cycles because of fatigue failure of the steel testing beam. As also observed in Reference (7), fatigue is not a design factor for high strength bolt shear connectors.

Figure 7. High Strength Bolt Static Slip Test



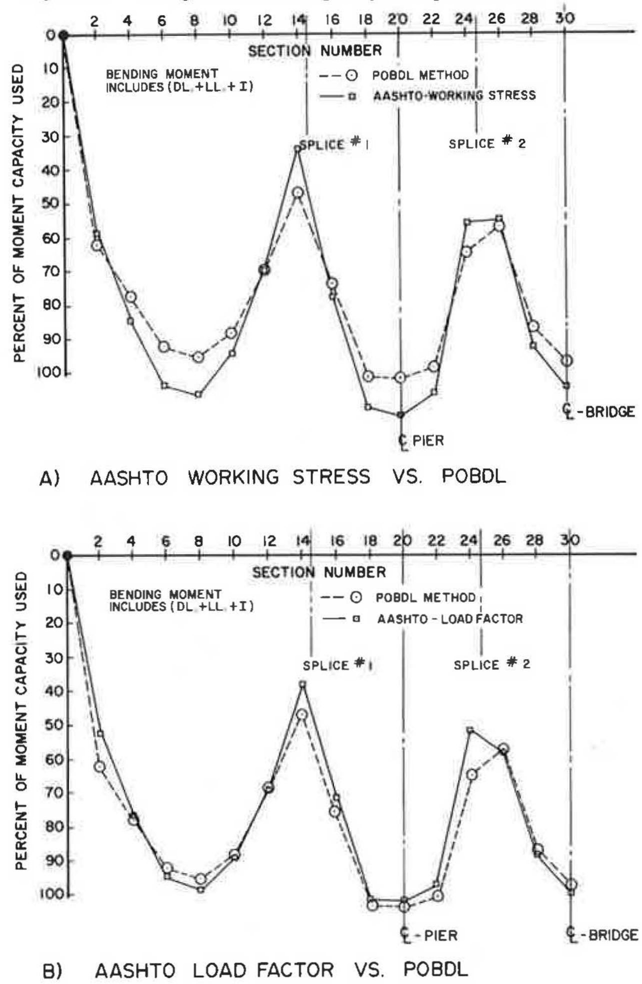
From this testing, it is confirmed that an A325 bolt, torqued to the specified tension, can be used as a shear connector. It can be designed as a bolt in shear, since its ultimate strength is not defined by the crushing of the concrete around the base of the shear connector as is the case of welded studs.

A portion of the deck was divided into test panels with varying slab thickness, transverse steel percentage and concrete cover. The slab design was based upon extensive theoretical and modelling research (8). This research indicated that deck slabs fail in punching shear, not in the assumed flexural mode, and have exhibited a very large factor of safety when compared with the AASHTO Specifications. The design and testing of these deck panels (in which the minimum transverse steel percentage is 0.2 percent) are covered in detail in reference (8). Except for these test panels, the deck slab was designed for transverse bending by the AASHTO working stress method and the distribution of wheel loads was in accordance with Article 1.3.2 (2). Consequently 19 mm (3/4 in) reinforcing bars were used at 190 mm (7½ in) centers (almost 1.0 percent), top and bottom with extra top steel for the cantilever slab portion. Due to longitudinal prestressing, the distribution steel was reduced to 13 mm (½ in) bars at 305 mm (12 in) centers top and bottom throughout.

A check on the capacity of the bridge was carried out using both the AASHTO working stress and load factor methods. Compariative results are plotted in Figure 8. AASHTO working stress shows an overstress condition of a maximum of 12 percent at the piers with the load factor method showing a similar capacity to the POBDL method. Since the differences between the capacities predicted by the three methods were sufficiently small, there was no concern over the adequacy of the Conestogo Bridge as designed with the new POBDL criteria.

Fabrication and erection of the structural steel followed conventional methods, with special emphasis placed on camber control. A vertical tolerance of ± 3.2 mm (± 1/8 in) was placed upon the setting of the formwork, reinforcing steel and prestressing ducts. This was necessary to allow a direct comparison between the slab design and the punching shear test results.

Figure 8. Design Moment Capacity Comparisons



The original construction method called for the high strength bolt shear connectors to be offset in oversize flange holes and remain in position during the placing of the deck slab. It was thought that a low torque could initially be placed on the bolts and consequently no movement would take place throughout the deck forming, re-bar placing and deck pour operations, yet allow relatively free movement of the slab during the post-tensioning procedure. However, a number of shear connectors were found to be out of place when a spot check was made after the deck was poured. More than 10 percent of the bolts would be incapable of the required movement, thus it was decided to abandon the concept of free slab movement and to fully torque the shear connectors prior to the stressing operation. From a design point of view, the additional prestress transfer load could be taken by the shear connectors without fear of slippage, and the compressive prestress in the steel girders would not be severe. It was felt that the some 15 percent loss of prestress at the pier would be offset in part by the increase in concrete strength that is normally found in the field, over that specified. The presence of the prestressing force is not considered to effect the ultimate load carrying capacity of the bridge.

Instrumentation and Testing

The test program and corresponding instrumentation were designed to answer the questions that arose with the formulation of the design and construction methods. It was hoped that information relative to the following subjects could be obtained from the prototype tests.

1. Girder erection stresses and deflected shape.
2. Nature and degree of prestress.
3. Theory of slab membrane action.
4. Effect of the barrier wall, sway frames and slab on distribution of load between girders.
5. Dynamic performance.

Reference (9) contains complete information concerning the variety of instrumentation used to monitor load, strain and deflection for the static and dynamic testing of the structure.

Prior to the construction of the barrier walls, a dynamic pull-down test was carried out. Top and bottom flange strains were recorded while jacking against a bulldozer on the shallow river bed at midspan. The results were compared to a similar test after the construction of the barrier walls.

Twelve static load locations were selected to produce maximum effects in the positive and negative moment regions. An incremental loading procedure was followed, using six load lifts with a maximum load of two 890 kN (200 kip) test vehicles. To explore the effect of the sway frames on the distribution of load to the girders, a set of static tests was repeated with a single test vehicle in a critical location for positive moments in the north end span. Strains were recorded with three instrumented sway frames in various combinations of unbolted and fully bolted conditions. In order to produce influence surfaces for bending moment, a single vehicle was successively positioned along the structure in 3.0 m (10.0 ft) increments in each of five test lanes. Top and bottom flange strains in the centrespan, pier and endspan were recorded.

The test vehicle, loaded to a weight of 368 kN (82.7 kips) made runs at 48, 64 and 80 km/h (30, 40 and 50 mph) in both directions in three lanes. The strain gauge output was recorded while the vehicle was on the bridge, and for 30 – 60 seconds of free vibration after the vehicle had left the bridge.

Analysis and Test Results

Construction and Dead Load

Good agreement was obtained between a continuous beam model and the measured cross sectional average values of bending moment and deflection. However, there was up to a 22% variation between girders in the mid-span deflections. This variation is due to slight differences in camber and a rotation of the pier sections allowed by the nominal clearance in the bolted girder splices. Significant erection stresses (172.3 MPa or 25 ksi) were observed in the bracing members since the sway frames were used to realign the girders. Apparent axial forces are present, even prior to stressing of the deck, and can be considered using the top and bottom flange strains as outlined in Reference (9). Average axial compressive strains of 110 micro-strain in the centre span were computed from the recorded top and bottom flange strains, which is of the correct order of magnitude for typical shrinkage strains. Very small axial strains were computed at the pier section.

Since the stressing of the deck was applied to the composite section and not the slab done as originally hoped, recording the resulting strains in the girders enabled slab prestress to be determined. Based upon top and bottom flange strains, the computed average sectional slab prestress compares very favourably with the design prestress, and in all cases is greater than the required service level prestress (Figure 9). The maximum steel prestress at the pier was of the order of 24.1 MPa (3.5 ksi) compression in the top flange (which is opposed by tensile live load stresses), with negligible stress in the bottom flange. Maximum top and bottom flange stresses were about 29.0 and 18.6 MPa (4.2 and 2.7 ksi) respectively in the centre span. It is felt that these stresses will pose no problems as in all cases where high compressive stress exists, the live load stress is either very small or tensile in nature. The axial forces are fairly uniform between girders indicating the prestressing effects were evenly distributed and there was an even shear transfer between the slab and girder.

Thermally induced stresses, were generally rather small over the period of observation. The largest stresses were found in the bottom chord member of the central sway frame (27.6 MPa or 4 ksi) for a cyclic ambient temperature variation of 9.4 C° (17 F°), which corresponds very closely to the stress one would expect if the member were fully restrained. Top chord and diagonal member stresses were very small as were the stresses in the bottom laterals. Since the stresses in the bottom chords of the exterior sway frames were also quite small, temperature stresses appear to be most significant in the interior panels where the largest degree of restraint against free movement is provided by girders.

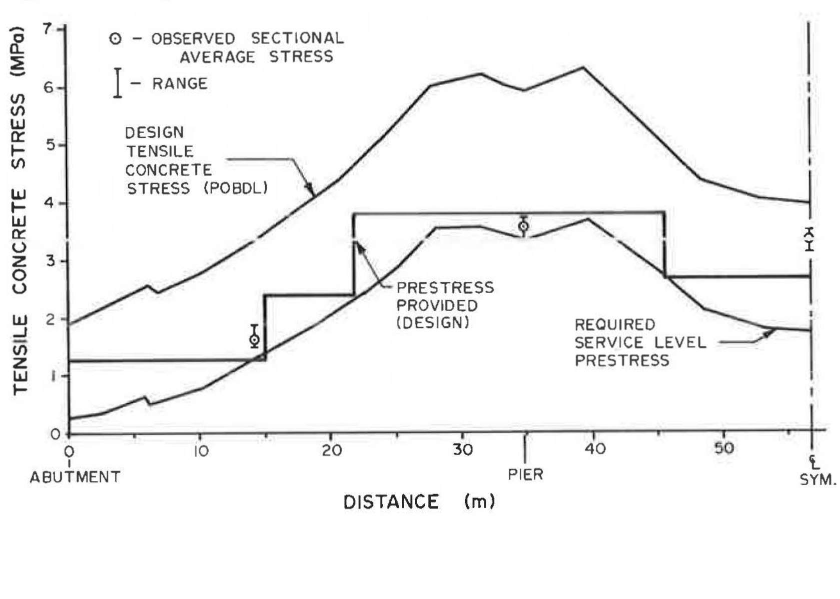
Static Tests

Grid analysis techniques have been used by several researchers (9) to model beam and slab bridges. Good results can be obtained with a careful choice of grid configuration and stiffness parameters. The choice of these parameters becomes more difficult with increasing girder depth and idealizations of the three-dimensional structure must be made.

A gridwork of elements was used conforming as closely as possible to the configuration of the structural steel. Longitudinal members were located at each main girder, with transverse members at each diaphragm or sway frame position. Bending and torsional moments of inertia, cross sectional area, bending and shear moduli of elasticity were required for each member. Since the varying haunch thicknesses at any given section resulted in less than a two percent difference in section properties, girder inertias were assumed to vary in the longitudinal direction only.

The barrier wall, cast in 6.1 m (20 ft) lengths with 3 expansion joints, could increase the stiffness of the exterior element

Figure 9. Longitudinal Deck Prestress



by almost 70 percent if fully active. However, the static pull down tests performed at the centerline of the bridge showed that there was no appreciable difference in either the mid-span deflection or girder stress for this symmetric loading. Good agreement was obtained between the test results assuming no contribution from the barrier wall to the section properties of the exterior element.

Several models of longitudinal element torsional stiffness were examined before suitable transverse stiffness was evident. It was found that by simply taking the torsional stiffness of the slab element by itself, the deflections and moments under non symmetric loading were overestimated by as much as 30 percent. The best model found assumed that the bottom lateral and sway frame systems provided lateral support to the exterior and interior girders. This in effect is similar to a closed section with the lateral system representing the bottom flange of the torsionally stiff box section. Each approach considered was examined statistically, comparing experimental and predicted response, with the above representation providing the necessary torsional stiffness to model the response satisfactorily.

Figure 10 shows two of the static test comparisons for the final load lift of 890 kN (200 kip) test vehicles. Linear behaviour was observed in all respects for the preceding load lifts.

A distribution factor was calculated at each of the three critical sections for every vehicle position in the static tests. A separate factor was obtained for interior and exterior girders shown in Figure 11 for positive and negative moment. In most cases the distribution factor computed from test data was considerably lower than that used for design purposes. The factors that exceeded the design value were accompanied by low sectional moments and were not critical. The only case that might be considered to be significant is when two vehicles are outside the design traffic lane, almost directly over the exterior girder, resulting in a large negative bending moment at the pier in the exterior girder. However, as in all of the tests with a truck on the shoulder, this respresents a more severe situation than normally contemplated in design. Similar trends in the variation of the distribution factor with load position were seen in the single vehicle crawl runs.

Axial forces in the instrumented sway frames were obtained for all static tests from the strain gauge data. In general, the largest forces were found in the bottom chord member of the sway frame closest to the loaded area. Maximum live load stresses were 86.6 MPa tensile (12. 4 ksi) and 61.5 MPa compressive (8.8 ksi). These stresses by themselves are not severe, however when coupled with observed erection stresses of up to 209.5 MPa (30 ksi), a severe stress condition could result in the connections. These observed live load stresses in the sway frame members agree quite favourably with those computed from

Figure 10. Static Bending Moments And Deflections

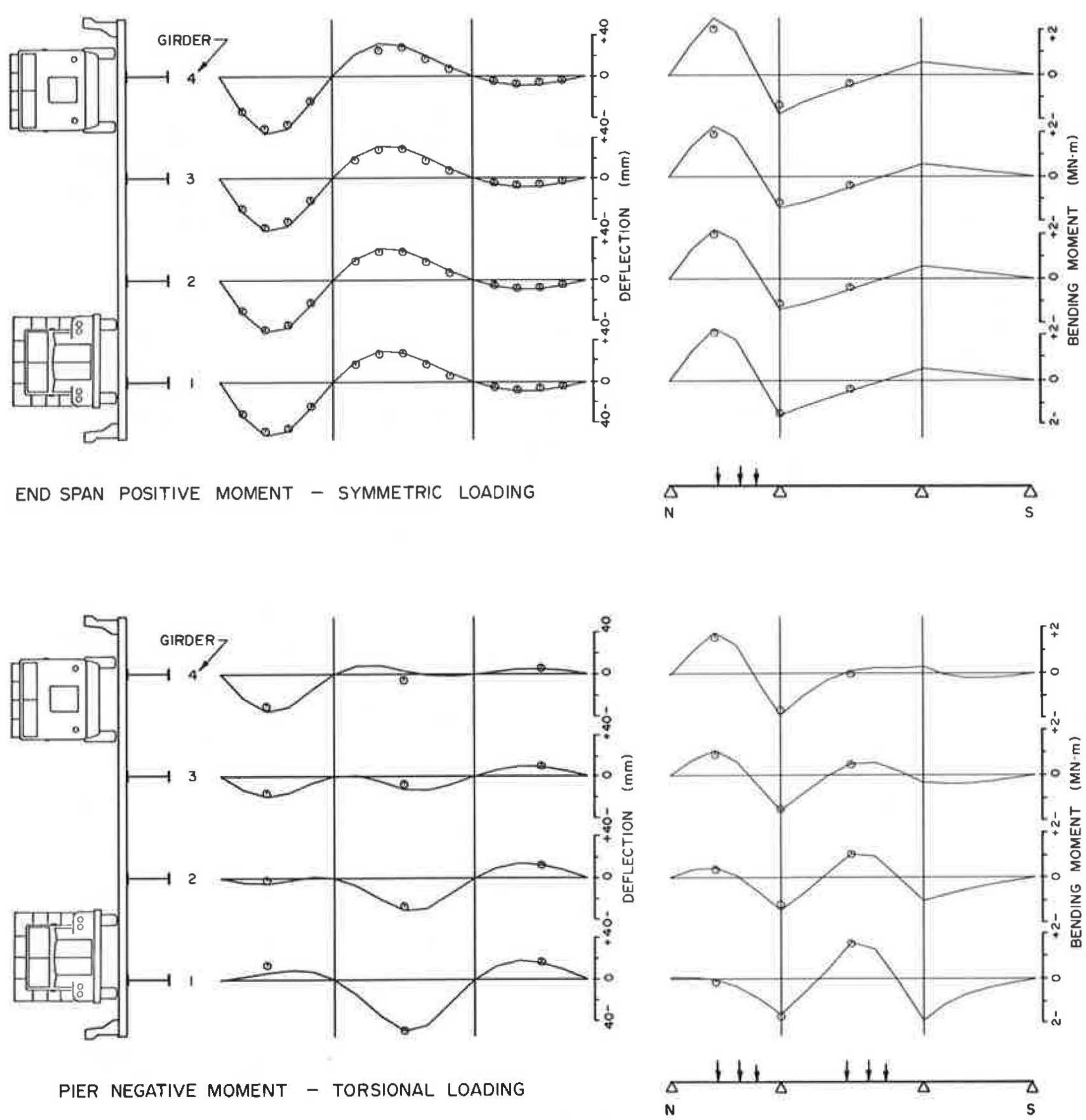
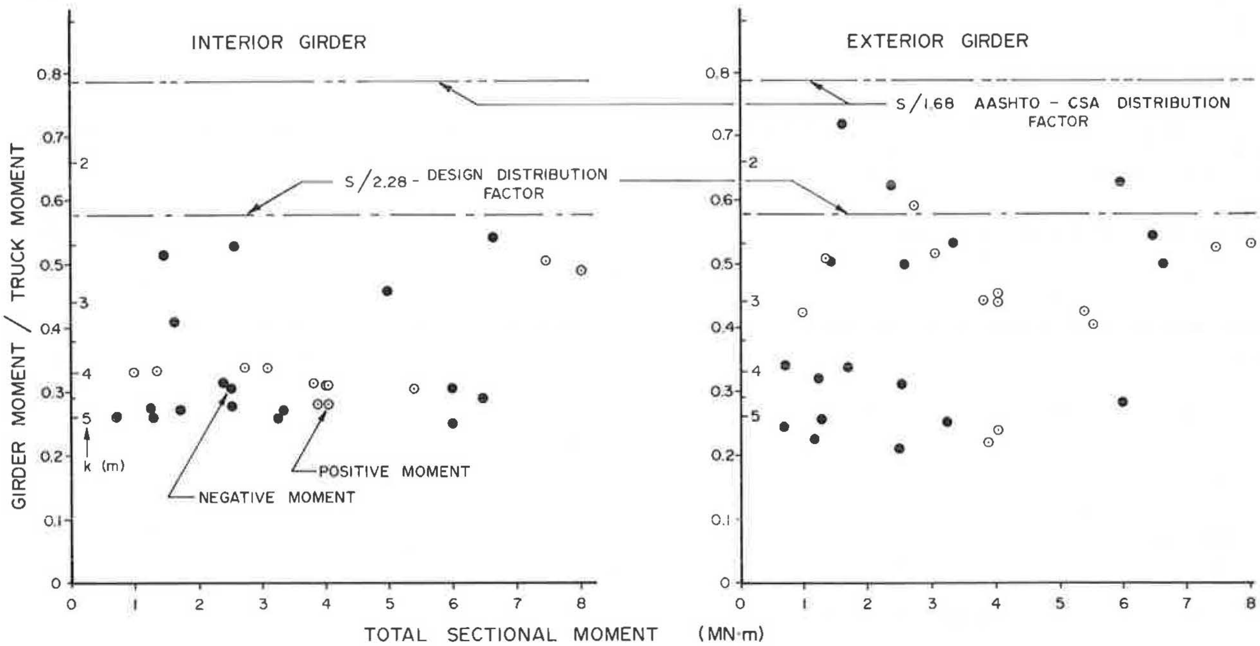


Figure 11. Distribution Factors From Static Tests



transverse moments and shears as determined from the grid analysis.

Similarly, forces were obtained for the instrumented bottom lateral members. In symmetrically loaded spans the stresses were tensile in the order of 14.0 MPa (2 ksi). Eccentric loading, producing longitudinal shear between an exterior pair of girders, engages the bottom laterals in a truss-like action. Alternate tensile and compressive forces were seen in adjacent members with a maximum observed stress of 64.3 MPa tension (9.2 ksi) and 55.9 MPa compression (8.0 ksi).

Transverse reinforcing bar stresses were obtained near the position of maximum positive longitudinal bending moment in the endspan. The maximum tensile stress found was 82.4 MPa (11.8 ksi) in the positive moment reinforcement of the central panel.

Stresses in the longitudinal reinforcing bars over the pier substantiated the design claim of full composite action throughout the length of the bridge superstructure. Applicable stresses are shown in Figure 12 showing definite composite action over the pier for several loading conditions. Maximum recorded live load tensile stresses in the longitudinal reinforcement at the pier was 10.5 MPa (1.5 ksi) corresponding to a concrete stress of 1.75 MPa (250 psi) which is well below the design tensile stress of 2.65 MPa (380 psi) shown in Figure 9.

Due to the high degree of preload in the sway frame members, they were not able to be completely removed as originally planned in the distribution tests. However, even in a semi-connected state there was enough restraint provided to the bottom flange of the girders to prevent a significant change in load distribution. The resulting changes in sway frame stresses were as much as ± 50% of their fixed state stresses. It is hoped that future tests may clarify the role that the bracing members play in the distribution of load.

From these results it is evident that the standard AASHTO distribution factor is extremely conservative while the figure of S/2.29 used in the design and substantiated by field tests is closer to reality. Researchers have recognized this problem and several methods have been proposed to account for the added lateral stiffness present in modern composite structures. Figure 13 shows the results of one such method (11) that involves the determination of actual stiffness parameters before a distribution factor can be obtained. The resulting distribution factors using the stiffness parameters from the Conestogo structure are not greatly different from that observed during the prototype tests or that used in the design.

Dynamic Tests

A natural frequency of 1.61 Hz was observed during the pull down tests with no discernable difference in frequency between the tests conducted before and after the barrier wall was constructed. The power spectrum shown in Figure 14 clearly identifies vibration modes at 1.68, 2.32, 2.66, 3.12, 3.44, 3.62, 6.08, 6.42 Hz. If the natural frequencies reported in Reference (10) for the bending modes are increased by a factor which gives a first mode frequency of 1.68 Hz, then frequencies of 1.68, 2.71, 3.63, 6.42 Hz are obtained which are in very close agreement with certain of the values from the field tests. The character of the damping is a mixture of Coulomb and viscous, with the Coulomb component dominant. The damping is between 0.3 and 0.5 percent of critical for both the second and third bending modes, if a viscous model is assumed and the values obtained using their logarithmic decrement. This value of damping is quite low when compared with that of non prestressed structures.

The dynamic amplification of bridge motion due to a vehicle passage was obtained by examination of the time histories of bridge response in the period while the vehicle was on the bridge and is considered to be representative of the impact factor used for bridge design. The distribution of dynamic amplification, although quite scattered, gave a mean dynamic amplification of 0.2 with a standard deviation of 0.09, giving a coefficient of variation of 0.45. If a normal distribution is assumed, these statistics give a probability of exceedance of the design impact factor of 0.45 of 2.75×10^{-3}.

Additional dynamic testing including the simultaneous presence of vehicles need still to be analysed to provide a complete view of the dynamic response of the bridge.

Figure 12. Stress Distributions At North Pier

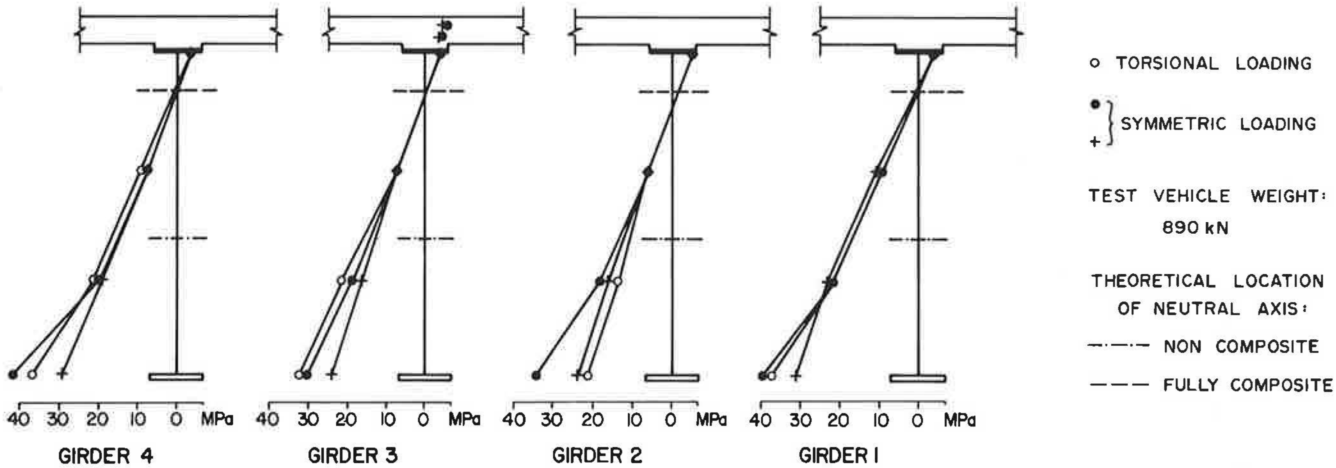


Figure 13. Variable Distribution Factor (After Sanders)

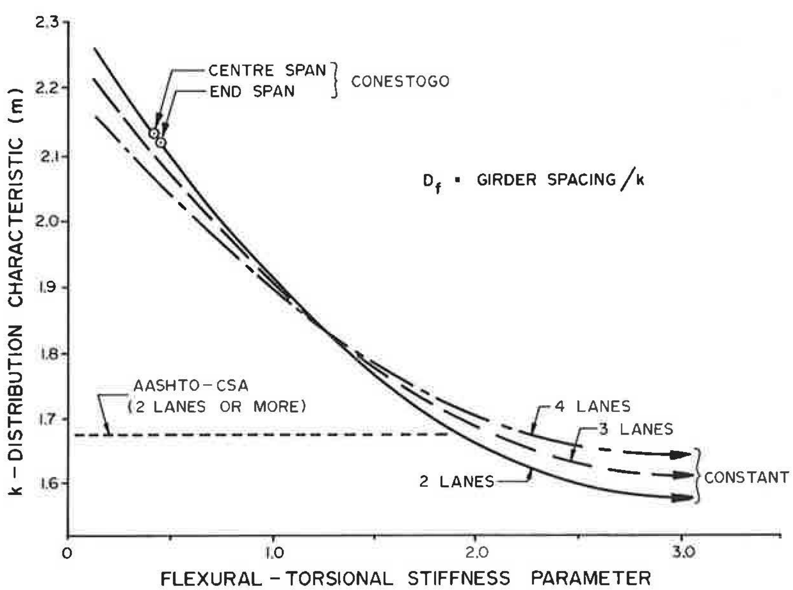
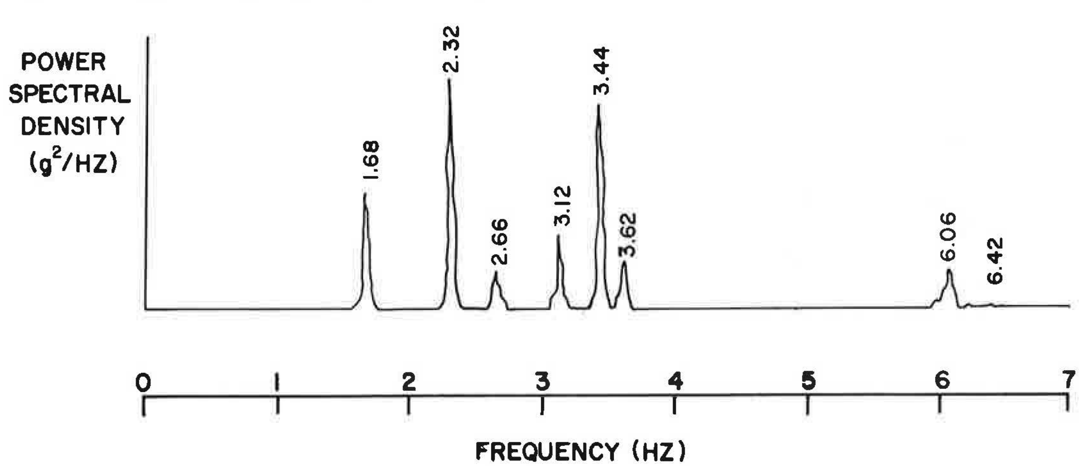


Figure 14. Typical Frequency Analysis Of Bridge Vibrations



Conclusions

1. Full composite action has been achieved for the entire length of the bridge using the method of longitudinal prestressing.
2. For this structure, the design criteria and methods used enable increased vehicle weights to be safely accommodated at no increase in materal compared to designs by AASHTO.
3. High strength bolts provide a satisfactory shear connection between the girder and slab.
4. The observed load distribution was significantly better than that suggested in present AASHTO specifications. The distribution factor used in the design is substantiated by the test results and current research.
5. The bracing system provides lateral restraint to the girder bottom flanges, increasing the torsional stiffness of the structure and considerably improving the distribution of load.
6. This very flexible structure with a first natural frequency below the usual range of vehicle frequencies appears to have satisfactory dynamic properties.
7. The Conestogo River Bridge responds in a manner which is consistent with the design assumptions.

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