

Ultimate Strength of an SPS Bridge – The Shenley Bridge, Québec, Canada

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Abstract

The cross section of the Shenley Bridge suggests that it is simply a slab-on-girder bridge, even if the slab acts compositely with the three longitudinal girders. However the deck panels, with moderately sized but widely spaced edge stiffeners, span transversely between the girders and longitudinally the main girders act as deck stiffeners even though they are rather large and very widely spaced. [Different proportions of deck thicknesses and stiffeners could give an orthotropic bridge approaching more usual proportions.] The fundamental difference of the Shenley Bridge from the usual orthotropic bridge is that the deck plate is *not* a single steel plate stiffened with trough stiffeners but a steel Sandwich Plate System (SPS) that is stiff enough by itself, self stiffened if you will, to span many times the span of a single steel plate thus reducing the number of stiffeners markedly. A Sandwich Plate System unit consists of two steel faceplates bonded to an elastomer core. The design of the Shenley Bridge, erected in November 2003, to carry gravity loads, particularly as related to the SPS deck panels is discussed and the results of full scale static load tests are presented and analysed.

Introduction



Figure 1. The Sandwich Plate System

In the (patented) Sandwich Plate System (SPS), two steel plates are bonded to a compact polyurethane elastomer core as shown in Figure 1. The elastomer, as a two part liquid, is injected into closed cavities formed by the steel faceplates and perimeter bars. The latter are not shown in the figure. To obtain a factored bond strength of 4 MPa or better on setting, the faceplates are grit blasted and have to be dry and free of grease, dirt and other contaminants when the elastomer is injected. An SPS designation, SPS 6-50-6, denotes the thicknesses of the three sandwich components – steel-elastomer-steel – in millimetres. In flexure, the plates act as flanges and the core as the web. The flexural stiffness and strength of a sandwich plate are many times those of a single steel plate and are tailored to meet particular structural requirements by selecting appropriate thicknesses for the sandwich elements. Shear is transferred from one steel plate to the other by the bonded elastomer without the need for fatigue prone steel-to-steel welds. Also the elastomer provides continuous support to the steel plates, precludes local buckling, and eliminates the need of closely spaced discrete stiffeners. The steel components are either shop fabricated (with obvious advantages) and assembled in the field or field fabricated and assembled. The steel cavities or units are fabricated using standard shop welding practices and assembled with welds and slip resistant bolted connections for dynamically loaded structures.

The SPS has been developed by Intelligent Engineering Limited (IE), in conjunction with industry partner, Elastogran GmbH, a member of the BASF Group. Research and development of the system has been conducted for the last ten years, with very favourable results on the key issues of structural performance, bond strength at the elastomer-steel plate interface and fatigue resistance. Intelligent Engineering has approvals from major ship classification societies such as the American Bureau of Shipping and Lloyds Register for the use of SPS in new builds and the rehabilitation of ships. The decks are not dissimilar to stiffened steel plates in civil engineering structures such as orthotropic decks of bridges. Other applications have been rehabilitation of orthotropic bridge deck panels in Germany and prefabricated SPS bridge deck panels in Austria. Both static and dynamic tests on a prototype SPS stadium riser, much lighter than its reinforced concrete counterpart were successful. The elastomer acts to dampen vibrations. The SPS has obvious applications wherever plate-like structures are needed and it is suggested can replace concrete slabs particularly where cracking and subsequent corrosion of rebars are a major problem.

The two-lane Shenley Bridge was constructed in November 2003, for Transports Québec in the municipality of Saint-Martin, Beauce-Sartigan county Figure 2(a) shows the bridge under construction with a transversely oriented SPS 6.4-38-6.4 prefabricated deck panel being positioned on the three steel plate girders. The deck comprises eight panels 7112 mm x 2400 mm and two panels at the ends 7112 mm x 1650 mm for an overall length of the bridge of 22.5 metres (73.8 ft). The SPS panels act compositely with singly symmetric plate girders 970 mm deep with a mass of 207 kg/m spaced at 2750 mm leaving overhangs of 806 mm on either side. The plate girder comprises a 300x20 top flange, a 925x12 web and a 350x25 bottom flange. The steel is ASTM A588 Grade 50 corrosion resistant steel with a specified minimum yield strength of 50 ksi (345 MPa).

As shown in Figure 2(a), the interconnection of adjacent SPS deck plates is achieved by pretensioned slip-critical bolts (at 500 mm on centres) that connect the webs of the transverse 250x125x9.5 mm cold-formed angles and a groove weld running along the top of the adjacent transverse angles. Figure 2(a) also shows an expanded steel plate within the elastomer. This is used to reinforce the elastomer because its coefficient of thermal expansion is considerably greater than that of the steel and at very low temperatures it tends to contract away from the steel. Cold-formed 9.5 mm thick angles extending 202 mm below the SPS plate are aligned with the longitudinal steel girders so that the SPS plates can be bolted to the girders (Figure 2(b)). ASTM A325 bolts in slip-critical connections are used. Similar to other steel bridge decks, a Stirling Eliminator coating is applied to the top faceplate prior to the asphalt application (Figure 2(c)). At the edges of the bridge the angles extend 50 mm above the top faceplate to provide a boundary for the asphalt surface. Breakaway AASHTO guardrail posts are bolted to the edge perimeter angles.

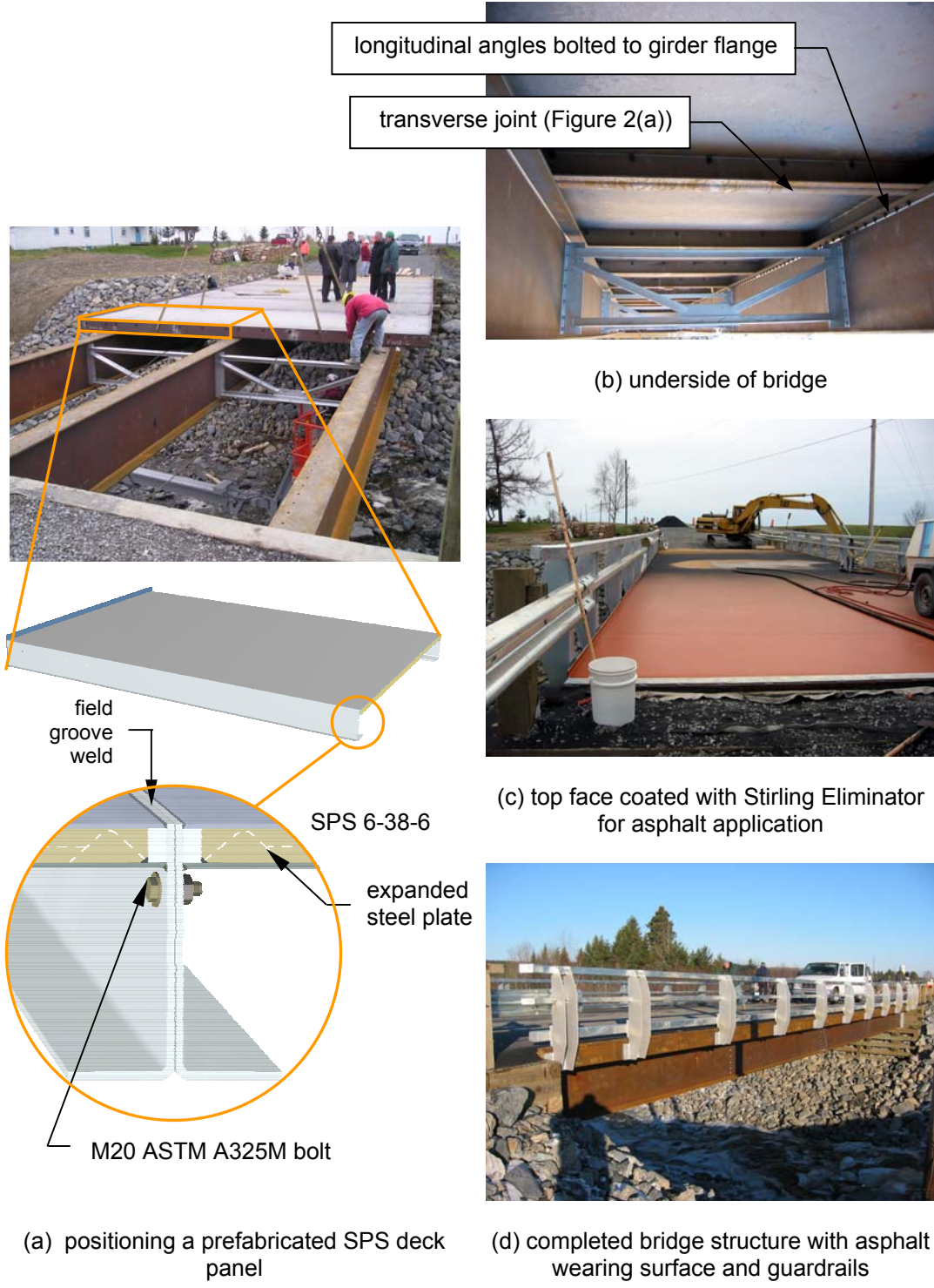


Figure 2. Shenley Bridge Assembly

Prefabrication of the SPS plates facilitates stricter quality control and rapid on-site assembly of the deck structure. Fewer fatigue prone details and improved vibration damping of the SPS deck plate system, which is exposed to heavy truck traffic, lead to an increased service life. The SPS bridge deck is lighter than conventional concrete deck structures resulting in lighter supporting substructures and improved seismic behaviour.

Design Loading and Criteria

The design live load used for the structural evaluation is based on the specified CL-625 truck given in the Canadian highway Bridge design Code, CSA Standard CSA-S6-00 (2). Figure 3 illustrates the wheel loads, axle loads, wheel and axle spacings and tire print sizes for the truck that has a total weight of 625 kN or 140.5 kips. Depending on the length of the influence line of the element being loaded seven subsets of the truck with from one to five axles are used to produce the maximum effect. For the ultimate limit states discussed here, a live load factor of 1.70 is used in conjunction with an incremental dynamic load allowance ranging between 0.25 and 0.50 depending on the number of axles considered and the element in question. Dead load factors range from 1.10 for factory produced components to 1.50 for wearing surfaces. A modification factor of 0.90 is applied to the live load for two-lane loading. For the serviceability limit states the live and dead load factors are taken as 0.90 and 1.00 respectively.

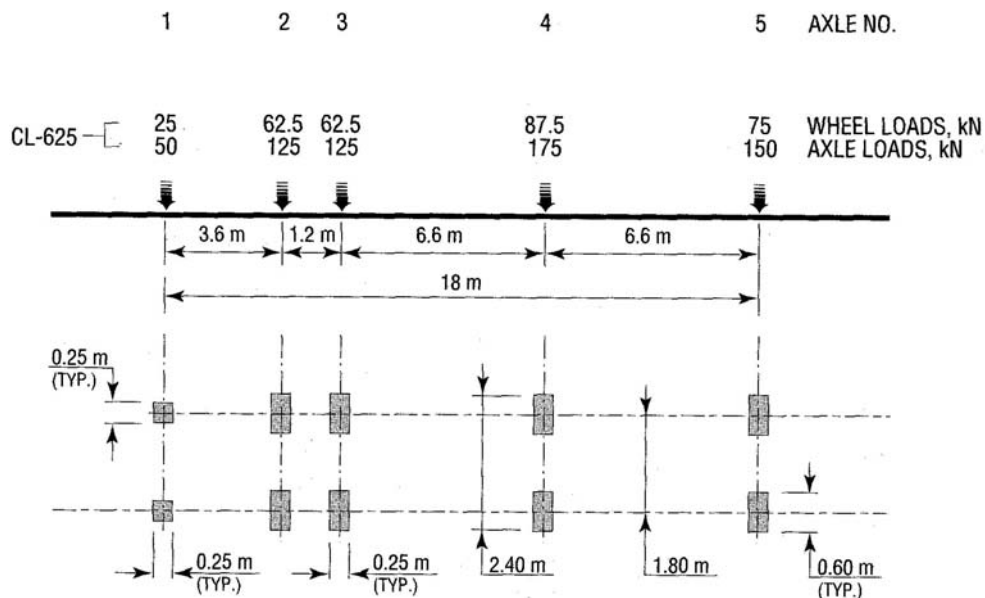


Figure 3. CL-625 truck from CSA 2000a

In the limit states design standard CSA-S6-00 (2), factored resistances are given as the product of three factors, the resistance factor, a relevant stress such as the ultimate tensile strength, the yield stress, or a critical buckling stress, and a corresponding geometric property such as the plastic section modulus for plastic design sections in flexure. From these, we deduce factored resistances for the SPS panels in tension and flexure based on 0.95 of the yield stress, $0.95\sigma_y$. For members in compression, the stress is limited to $0.90\sigma_y$ when buckling is not a consideration, The deflection due to live loads for the serviceability limit state is limited to 1/300 of the span.

Strength Evaluation. Finite element analyses using the program ANSYS (1) were used to determine the structural response of the SPS bridge deck plates and the bridge itself to two design trucks positioned side by side to produce several maximum load effects. The finite element model for the SPS 6.4-35-6.4 bridge deck plates, and the three supporting girders comprises SOLID45 solid elements for the steel with multi-linear isotropic properties with $E = 206\,000$ MPa, $\nu = 0.287$ and $\rho = 7850$ kg/m³ and Hyper 58 solid elements for the elastomer with $E = 750$ MPa, $\nu = 0.36$. The three load cases considered were: Case A, with truck axles 2 and 3 positioned directly between transverse angle web; Case B, with truck Axles 2 and 3 straddling the transverse angle web; and Case C, with the trucks positioned to produce the maximum global moment. The results of these analyses are presented in detail in Intelligent Engineering Limited (4) and summarised here.

Table 1 summarises the maximum stresses in the SPS panels. None of the steel stresses exceeds the factored resistance of 328 MPa. The locations where these maximum stresses occur are given in Intelligent Engineering Limited (4). For tire prints positioned adjacent to the webs of longitudinal and transverse angles, the normal and von Mises stresses tend to be highly localized in the plating over the webs and dissipate rapidly away from these locations. The maximum interface shear stress between the steel faceplates and elastomer in the longitudinal and transverse directions of 1.7 MPa is much less the factored resistance of 4 MPa. This is generally found to be the case.

A critical element of the SPS bridge deck plates is the V-groove welds as seen in Figure 2(a) joining adjacent panels when acted upon by negative bending moments that occur under the most severe loading event producing maximum tensile stresses in the weld. Dead and live loads applied after the welds are made are considered. For load case B with trucks positioned to straddle the transverse angle webs, a local model, including the pretensioned M20 ASTM A325M bolts gave a maximum factored tensile stress of 21 MPa that is more than acceptable. Fatigue at the root of the weld, not further discussed herein, was found not to be critical with an estimated fatigue life of 10^8 cycles under the fatigue truck loading.

Table 1. Summary of maximum stresses for SPS 6.4-35-6.4 bridge deck panels

Load Case	Stress in Steel, ≤ 328 MPa						Interface Shear Stress, ≤ 4.0 MPa	
	Longitudinal		Transverse		von Mises			
	Top Plate	Bottom Plate	Top Plate	Bottom Plate	Top Plate	Bottom Plate	Longit.	Transverse
A	41 -198	158 -130	244 -132	315 -324	241	320	1.1	1.7
B	58 -164	165 -145	123 -91	205 -198	144	218	1.1	1.1
C	42 -197	157 -135	243 -131	312 -321	239	317	1.1	1.7

The central supporting girder is stressed slightly more than the outside girders with an maximum average tensile stress across the bottom flange in the longitudinal direction of 150 MPa while the average compressive stress in the top flange of 18 MPa. This stress distribution is consistent with the deck acting compositely as discussed under the load test results. The stresses in all three supporting girders are much less than the factored resistance.

Serviceability Evaluation. Overall live load deflections of the bridge structure and local deflections of the deck were determined. The maximum overall deflection occurring in the deck near the central girder is 26 mm or about 1/850 of the span. The maximum local deflection in the deck under a tire print for load cases A and C is 3 mm or 1/600 of the span of the deck panel width.

Field Load Testing

Procedure. As part of the process required by the province of Québec for the evaluation of SPS technology for use in bridge structures, a static and dynamic load test programme was conducted, in accordance with the requirements of Section 14.16 of the Canadian Highway Bridge Design Code (2). The tests established the behaviour of the prototype SPS plate bridge and its ability to carry loads and provided field data to confirm the finite element analyses predictions. The static results are reported here.

Figure 4 gives the wheel loads, axle loads, wheel and axle spacings and tire print sizes for the test vehicle, the heaviest available, that was used for the static load test. Global and local deflections and strain measurements were taken for the five load cases of the test truck shown in Figure 5.

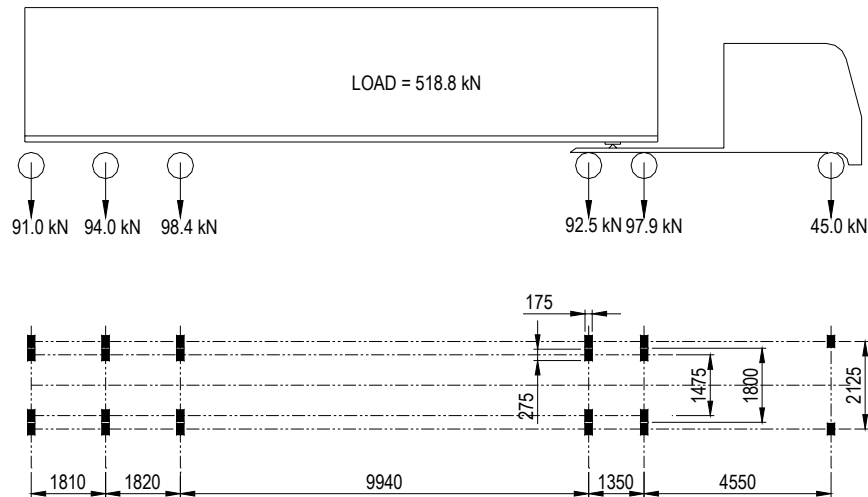
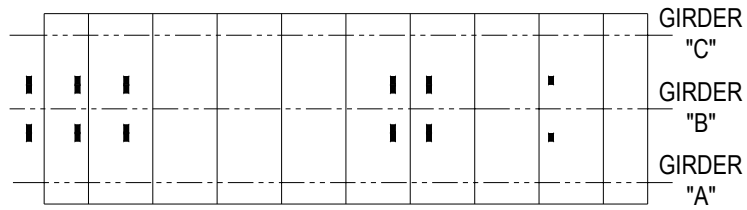


Figure 4. Load Test Vehicle

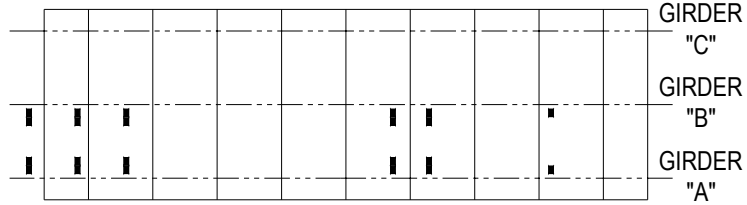
Load cases 1 to 4 examine the panel behaviour when either transverse or longitudinal, positive or negative, bending moments in an SPS deck panel are a maximum. Load case 4 also examines the torsional response of the bridge with the truck centred transversely between girders A and B. The fifth truck- position represents the load case for the maximum overall live bending moment of 1398 kN.m.

The factored live global bending moment for the ultimate limit states for two CL-625 trucks including the multi-lane use factor and a dynamic load allowance of 1.25 is 7086 kN.m. Hence, the applied truck live loads for the load test are 20% of the factored CL-625 truck bending moment or 38% of the specified bending moment of two CL-625 trucks. The bridge as built is somewhat heavier than anticipated because the fabrication sequence used allowed the bottom faceplate of the SPS to sag under self-weight resulting in a dead load moment of 2165 kN.m. equal to 0.94 of the factored dead load of 2299 kN.m. New fabrication procedures have been developed to eliminate the possibility of sagging.

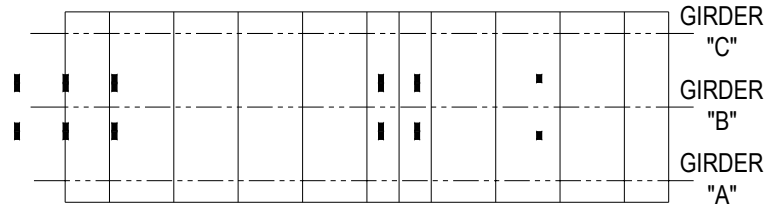
Field measurements. Global and local deflections and strains were measured for all five load cases. Global deflections were obtained using standard level survey techniques sighting with one of two theodolites on level rods graduated in millimetres placed on the asphalt surface at locations 1 through 22 as shown on Figure 6. Local longitudinal and transverse deflections within a panel were obtained to the nearest 0.1 mm by determining the deflection of the surface from an aluminum straight edge spanning between reference pads glued to the asphalt surface either over the transverse joints 2400 mm apart or over the longitudinal girders 2750 mm apart respectively. Figure 7 shows where the eight deflections with the bar oriented longitudinally (ΔL) and eight with a transverse orientation (ΔT) were measured.



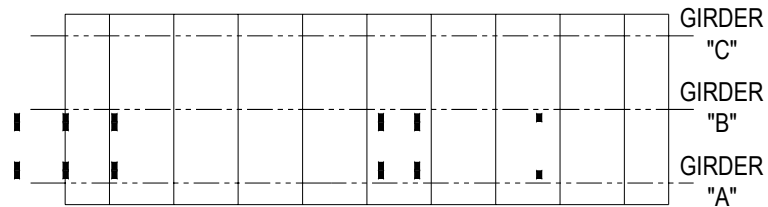
(a) Load Case 1 – truck centred, tandem axles straddling a transverse joint



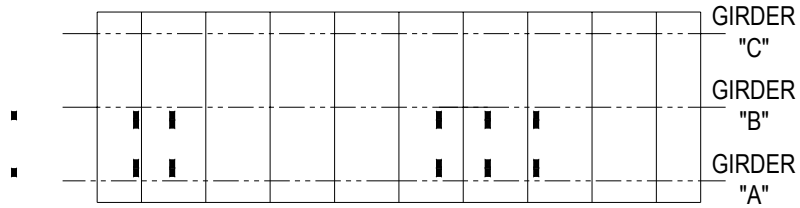
(b) Load Case 2 –As Case 1, but truck centred between girders A and B,



(c) Load Case 3 – truck centred, tandem axles centred within a panel



(d) Load Case 4 – As Case 3 but truck centred between girders A and B



(e) Load Case 5 – Maximum overall bending moment

Figure 5. Test Load Cases

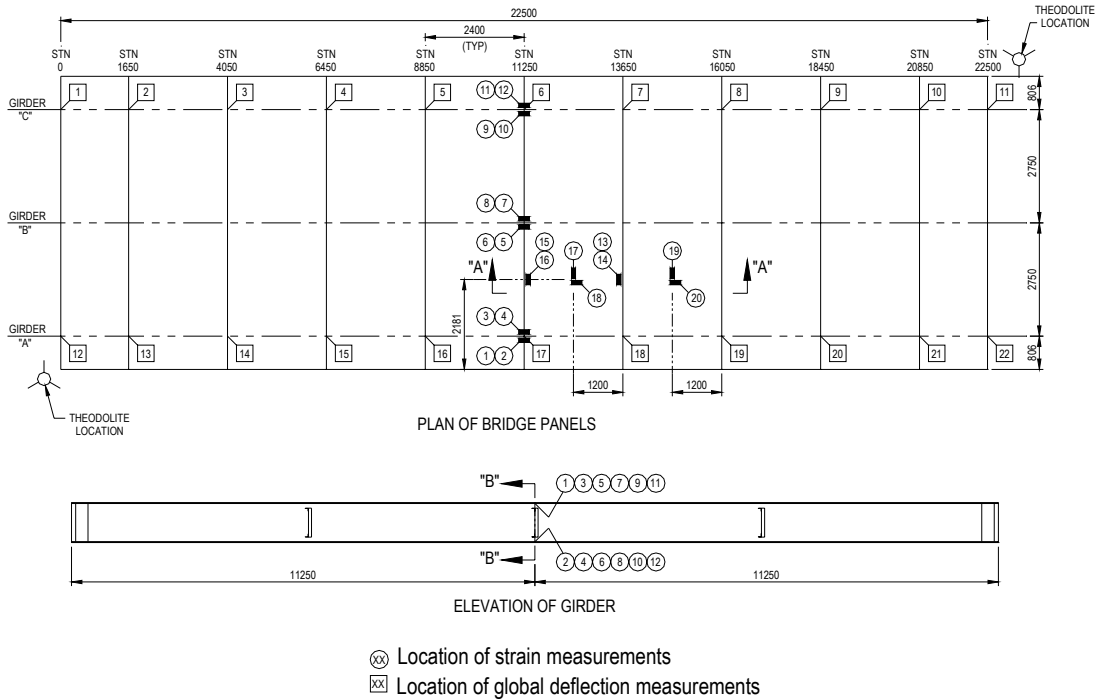


Figure 6. Location of global deflection and strain measurements

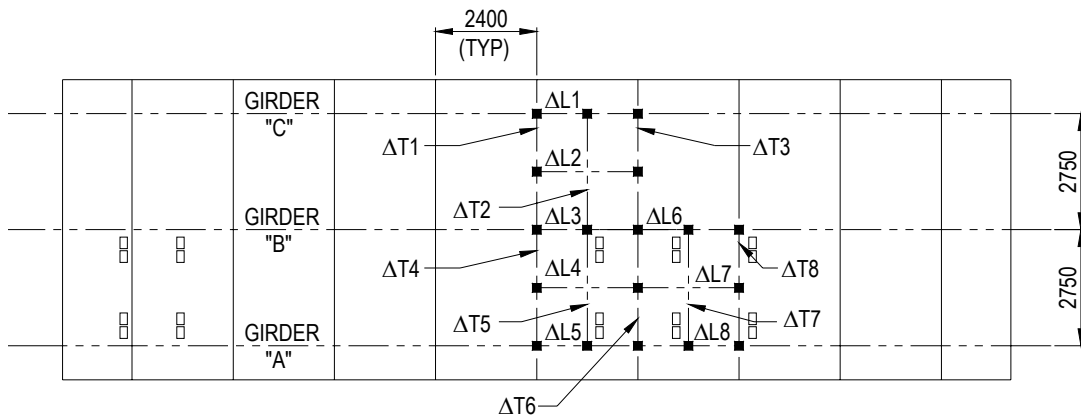


Figure 7. Location of local deflection measurements

Strains were measured at 20 locations as shown on Figure 6. Twelve strain gauges (numbers 1 to 12) were located on the flanges of the girders to measure longitudinal strains. Two pairs of gauges (numbers 13 & 16 and 14 & 15) were located on the underside of the deck and transverse angle flange, to measure the transverse strains in the angle. Two pairs of gauges (numbers 17 & 20, and 18 & 19) were located centrally between longitudinal angles on the underside of the SPS plate to measure both transverse and longitudinal strains in the bottom faceplate.

Analytical Model. The finite element model is the same as that for the strength evaluation except that it now included SOLID45 brick elements with a modulus of elasticity of 4300 MPa for the 50 mm asphalt wearing surface. The wheel loads are applied as surface pressures to the deck surface via contact elements TARGE170 and CONTA173 that allow the tires to follow the deformation of the SPS faceplate and the mass of the guardrail system was taken into account. The bolted interface between the SPS panels and the girders is considered to be bonded, i.e., with no slip in the bolted slip-critical connections. The dead weight of the bridge was applied to the model in the first load step and then vehicle loads. The analytical results reported for deflections and strains are those due to the vehicle loads.

Field Load Test Results. Table 2 gives the global test and predicted deflections for outside girders A and C at midspan under the test truck live load. The deflections are small with a maximum predicted deflection of 10.8 mm for Girder C of load case 5. In spite of the fact that errors in measurement of a few millimetres result in test values at variance with predicted values, six of the ten measurements are within one millimetre and three others within 3 mm of the predicted value. The test value of 6 mm for load case 5 is considered to be in error because adjacent observation points on either side of midspan had an average value of 14 mm within about 3 mm of the predicted value. Without composite action between the SPS deck and the girders as assumed in the analyses, the measured deflections would have been about 2.9 times as large. Therefore these measurements confirm that composite action was achieved.

The local deflections are small, with a maximum measured deflection of 2.7 mm and generally agree with those predicted. Errors show up as sporadic differences. The crux of the local deflections is that they attest to the stiffness of the SPS deck. When the maximum measured deflection is increased to the CHBDC serviceability load level, the deck meets the serviceability deflection criterion with a deflection of only 1/331 of the span.

Table 2. Global test and predicted live-load deflections at midspan

Load Case	Girder	Deflection, mm	
		Test	predicted
1	A	6	5.8
1	C	5	5.8
2	A	3	3.1
2	C	9	9.0
3	A	6	5.5
3	C	4	5.5
4	A	7	2.8
4	C	8	8.6
5	A	1	3.7
5	C	6	10.8

The predicted and measured strains match closely with an average difference of the measured strain from that predicted of 11 $\mu\epsilon$ that is 0.19 times the average predicted

strain of $59 \mu\epsilon$ for the 17 gauges that functioned.. Gauge numbers 14, 18 and 20 malfunctioned. For the ASTM A588 Grade 50 corrosion resistant steel the specified minimum yield strain is approximately $1750 \mu\epsilon$. The largest recorded strain was $189 \mu\epsilon$ for load case 5 and increasing this strain in proportion to the factored design moment to the test moment for load case 5, gives a maximum strain for the factored CL-625 truck of $1175 \mu\epsilon$. This resulting strain does not even yield the steel to say nothing of the inelastic reserve capacity of the structure that could be considered for the ultimate limit state. The maximum live load local strain recorded in the deck plate was $51 \mu\epsilon$ for transverse gauge 17 in load case 3 as compared to a predicted strain of $47 \mu\epsilon$. This strain is consistent with the small local deck deflections. Had gauges 18 and 20 functioned they would have recorded the combination of the local plate tensile bending stresses and the overall compressive bending stresses due to composite action.

Figure 8 shows the measured strains and predicted strains through the depth of longitudinal girder A for load case 5. The close correspondence of the measured strains to the predicted strains that are based on the assumption that fully composite action is obtained between the SPS deck and the steel girders and with the neutral axis about 130 mm below the top of the steel girder confirms this assumption. The very significant compressive force of the moment resisting couple, not carried in the steel girders or longitudinal framing members of the SPS deck, must therefore be carried in the top and bottom faceplates. No buckling problems exist because the elastomer supports the faceplates and precludes local buckling.

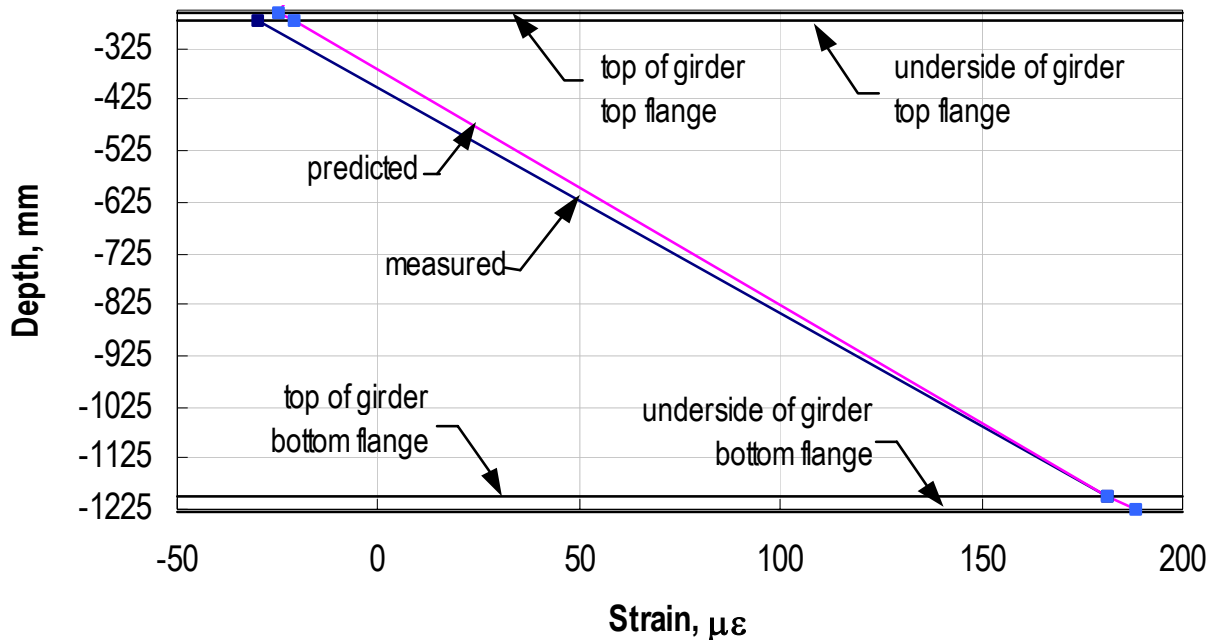


Figure 8. Strain Distribution in Girder A for Load Case 5

Summary and Conclusions

The analyses show that when the bridge is fully loaded with a CL-625 truck in each lane, the maximum stress under the factored dead and live loading is still less than the yield stress. Beyond this, the factored resistance of the bridge would take into account the inelastic straining of the steel components. The total reserve to develop the fully plastic moment is estimated to be about 2.9 times the fully factored moment due to dead and live loads when a multiple of 1.00 would be sufficient. The performance was corroborated, in so far as possible, by loading the bridge with the heaviest truck available to about 20% of the factored live load bending moment of a CL-625 truck in each lane. At this loading, the behaviour was linearly elastic and matched the analytical results closely.

The deck has a number of interesting characteristics. It is lightweight with a weight of about 43% of that of a 200 mm thick concrete deck. As compared to a lightweight orthotropic steel deck, the deck is stiff with reduced deck curvatures and small panel deflections. The predicted strains through the depth of the steel girder are in good agreement with those measured. The neutral axis location about 130 mm below the top of the girders indicates that full composite action of the SPS deck and girders was obtained as assumed in the analytical model. This is confirmed by calculating the deflections of the bridge under a symmetric loading case and comparing that to that computed with and without taking into account the contribution of the deck.

By using an SPS deck, the common problems associated with corrosion of reinforcing bars in concrete decks are obviated. The Stirling Lloyd Eliminator bonds the asphalt to the top faceplate of the SPS deck and should provide a long life without need for asphalt replacement particularly because the stiffness of the SPS deck and corresponding reduced deck curvatures reduces straining of the asphalt.

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