Tentative Design Rules for Innovative Bridge Decks Comprising Sandwich Plate System Panels

D.J.L. Kennedy, A. Ferro, and R.A. Dorton,
Intelligent Engineering Canada Limited

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ABSTRACT

Bridge decks in Canada, a vulnerable part of our transportation infrastructure, have required rehabilitation that is costly and disrupts traffic. To increase the life of concrete decks concrete quality has been improved and various schemes used to reduce or eliminate cracking insofar as possible.

The Canadian Highway Bridge Design Code, CAN/CSA-S6-00, a Limit States Design document gives the basic requirements for the Ultimate, Fatigue and Serviceability Limit States, as applicable, for bridges comprising different materials such as steel, concrete, wood.

This paper presents the proposed Limit States Design rules for innovative bridge decks comprising Sandwich Plate System (SPS) Panels that replace concrete decks. The rules could form an integral part of Section 10 Steel Structures of the CHBDC.

SPS deck panels are shop fabricated by injecting a two-part thermosetting liquid elastomer into a cavity formed by two steel faceplates bounded by perimeter bars. The elastomer core bonds to the steel, acts as a web and provides continuous support to the faces precluding local plate buckling and eliminates closely-spaced stiffeners though stiffening plates may be added. The flexural stiffness and strength are tailored as required by using appropriate thicknesses for the sandwich elements. The Shenley Bridge constructed in 2003 in Québec is presented as an example. Expected advantages of SPS panels are reduced time for construction, weight and maintenance. The elastomer dampens vibrations naturally. Composite action with main structural elements increases their stiffness and flexural strength appreciably. Long life is foreseen.

INTRODUCTION

The title of this presentation contains the two concepts of (a) the development of design rules for (b) an innovative bridge deck system comprising Sandwich Plate System panels. Left unsaid is that these rules, to be consistent with the current Canadian Highway Bridge Design Code (CSA 2000), must be written in Limit States Design format. Therefore these three concepts are now presented - logically in reverse order - the order of their development.

LIMIT STATES DESIGN

Because limit states design is central to this paper its use in Canada is reviewed briefly. Although reinforced concrete structures had been designed using ultimate strength design for many years this strength design philosophy is considered to be simply a forerunner of limit states design. Not only were all limit states not described but the load and strength factors were not derived statistically. Although. the load factors were in about the correct proportions, both the load and "strength" factors were too high.

The introduction of Limit States Design into the non-communist world occurred in 1974, as far as we have been able to establish, when CSA Standard S16.1-1974 (CSA i974) was published for referencing in the National Building Code of Canada 1975 (ACNBC 1975). The S16 committee developed the load factors and load combination rules that were given in the NBCC as well as the resistance factors and resistances found in S16. This was trail blazing work and required five years to complete extending from January 1970 to December 1974 all to be able to write the inequality:

$$\phi R \geq \Sigma \alpha_i S_i \tag{1}$$

where ϕ = resistance factor
R = nominal resistance
Σ = summation over "i" load effects
α_I = i^{th} load factor, and
S_i = "i" load effect

Fig.1 shows the situation when this inequality is just satisfied for the postulated distributions of the sum of the load effects, $\Sigma \alpha_I S_i$, and some member resistance R as indicated by the vertical line when this condition is obtained. Note that the lower tail of the resistance gives some resistances less the upper tail of the effect of loads. Apart from selecting resistance and load factors to give the desired reliability index, β the advantages of Limit States Design are many. Working stress design by definition sets a

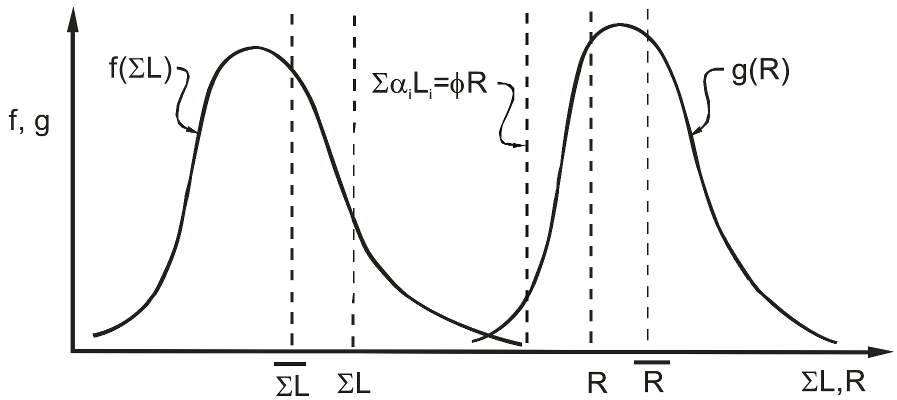


Fig. 1. Distribution curves when $\phi R = \Sigma \alpha_i L_i$

limit at a fraction (often 0.60 to give a margin of safety of 1.0/0.60 = 1.67) of a critical stress. For steel this was generally taken as the yield stress, σ_y, implying that the analysis is linearly elastic. But the only case when the yield stress is the true indicator of a failure condition is yielding of the gross cross section. Fracture occurs at the ultimate tensile strength. Buckling phenomena were accommodated by carrying over the same margin of safety to the "failure stresses". Although inelastic cross-sectional behaviour could be accommodated, geometric second order effects with distributed plasticity were not because first order analyses were followed. Using limit states design overcomes all these problems and assigns implicitly the best load and resistance factors.

A major impetus for the development of the first limit states design code for highway bridges by the Ontario Ministry of Transportation and Communications, (MTC 1979), about 25 years ago, was that bridges were being designed for much lighter trucks than those on the road. The discomforted bridge engineers knew that the apparent satisfactory performance was related to the excessive load factor on dead load that is implicit in working stress design. However, the ratio of dead load to live load, and therefore the ability of the bridges to carry heavier live loads, varied from bridge to bridge and even from member to member. Ministry engineers, based on extensive surveys of truck weights and lengths, devised the idealized five-axle truck weighing 700 kN or 157 kips shown in Fig. 2. Load effects, determined by taking subsets of the axles matched the maximum observed load from the truck surveys as shown in Fig.3, where the total weight of seven different contiguous axle groups versus the equivalent base length is plotted (Agarwal 1988). The axle groups, as numbered, comprise axle 4 by itself, axles 2 and 3, axles 1, 2 and 3, axles 2, 3 and 4, axles 1, 2, 3 and 4, axles 2, 3, 4 and 5, and lastly axles 1, 2, 3, 4 and 5. The OHBD truck not only reflected the actual vehicle loads on the highway, but also bore a direct relationship with the legal vehicle weights that were approximately and consistently 100 kN less than the observed maximum loads again as Fig. 3 indicates. The design truck in the new Canadian Highway Bridge Design Code for 2000 (CSA 2000) representing the minimum live load for bridges in the national highway system, has a total vehicle weight of 625 kN with axle positions as in the 1979 OHBD truck. With a load factor of 1.70, the total factored load is 1062 kN (239 kips) that is coincidentally just 2.5% more than the 1991 Ontario Highway Bridge Code.

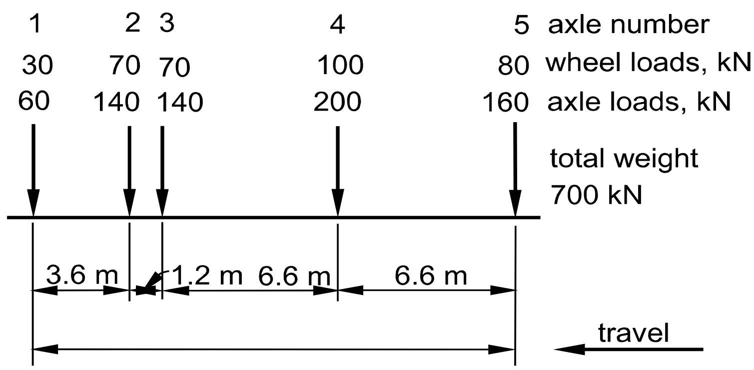


Fig.2. The 1979 Ontario Highway Bridge Design Truck

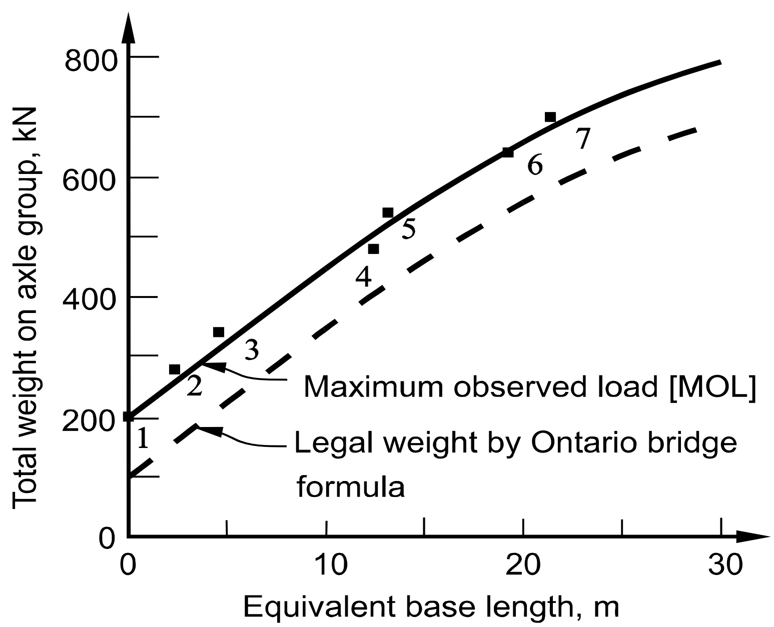


Fig. 3. MOL and 1979 OHBD truck, (MTC 1979)

The most recent development in limit states design usage in Canada has been the adoption of the companion action approach, commonly called Turkstra's Rule (Turkstra and Madsen 1980) in the 2005 edition of the NBCC. The full factored value of one transient load effect is added to a companion action value of another transient load [or loads] that could be expected to be present when the first transient load reaches its extreme value. Each of the transient loads is taken, in turn, at the full factored value with other loads at the reduced values. Their values are determined such that the probability of exceeding any sum of loads is about the same. The companion action approach provides more consistent reliability than the combination factor approach previously used. With an appropriate design philosophy, to what will it be applied?

SANDWICH PLATE SYSTEM PANELS

Fig. 4 shows one of only ten Sandwich Plate System panels being swung into position to create the entire deck of the Shenley Bridge in October 2003. Fast construction! The panels are next connected to each other by welding and slip critical bolted connections, then bolted in the same manner to the supporting steel girders to achieve full composite action and lastly the guard rails and a wearing surface are installed.

Fig. 4. Positioning one of ten Sandwich Plate System bridge deck panels

Fig. 5 shows the sandwich. The two steel plates are bonded to a compact polyurethane elastomer core. 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 6.0 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. The 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 a very broad 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.

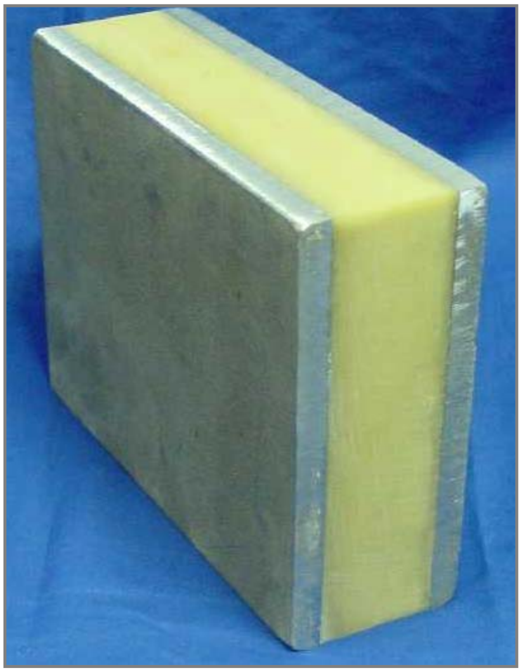
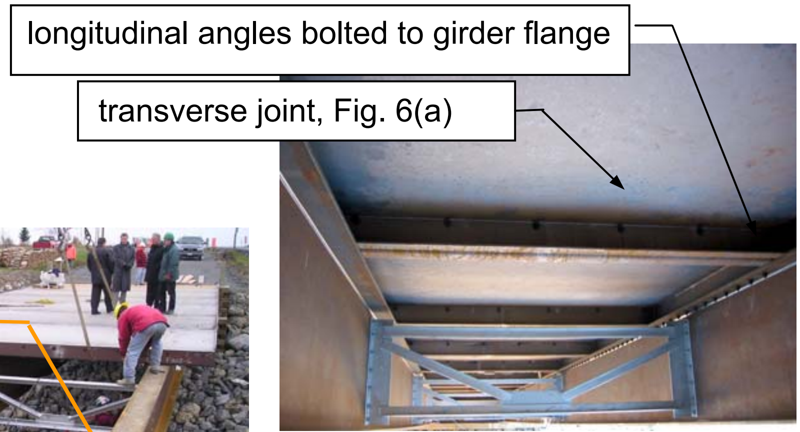


Fig. 5. The sandwich Plate System

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 generally shop fabricated (with obvious advantages) and assembled in the field. 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.

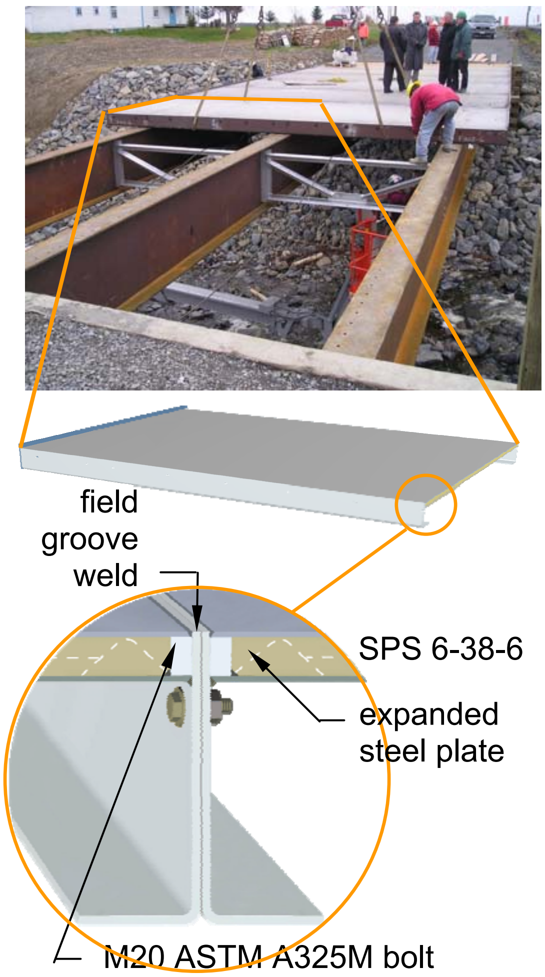
The SPS has been developed by Intelligent Engineering Limited (IE), in conjunction with industry partner, Elastogran GmbH, a member of the BASF Group. Research and development of the system has been conducted for the last twelve years. Intelligent Engineering has approvals from major ship classification societies such as the American Bureau of Shipping and Lloyds Register for the use of SPS in new builds and the rehabilitation of ships. Recent ship rehabilitations have been applied to the Canadian Coast Guard icebreaker, the “GRIFFON”, and to the National Oceanic Atmospheric Administration research vessel, the “WHITING”, of the United States government. Until recently, most applications have been in the maritime industry where the rehabilitation of the heavily loaded truck decks of ferries that are simply stiffened steel plate structures has been very successful. 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 damp 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 reinforcing bars may be a major problem), open steel grids, or other lightweight deck systems.

Returning to the Shenley Bridge, composite Fig. 6 provides erection views. It is not a big bridge - 7.11 m wide by 22 m long – nevertheless it is a bridge just as the short span Iron Bridge over the Severn River in Wales was when it was built and still is. The Shenley Bridge was designed in accordance with CSA Standard S6-00 (CSA-2000) for CL625 loading. The SPS panel designation is 6-38-6 indicating a total thickness of 50 mm out-to out of the SPS plate. The cross section in Fig, 6(a) shows the two 38 mm. deep perimeter bars welded to the top and bottom face plates. There are cold-formed edge angles 250x125x10 on each panel that are welded to the two face plates and to the perimeter bars at the top.



(b) underside of bridge

(c) top face coated with Stirling Eliminator for asphalt application



(a) positioning a prefabricated SPS deck panel

(d) completed bridge structure with asphalt coating and guardrails

Figure 6 - Shenley Bridge Erection

The field connection starts with slip critical bolting along the edge angles and is completed the V-groove weld between the edge angles at the top. The three longitudinal angles on each panel are bolted to the longitudinal girders to provide composite action for all subsequent loadings. Fig. 6(b) shows the transverse and longitudinal joints. Edge angles are also provided on both sides of the panels to which the guardrails are connected. The complete panels are very light and weigh only 35.5 pounds per square foot. Figs 6(c) and (d) show the deck ready to receive the asphalt and the AASHTO guard rail.

The bridge was tested statically and dynamically (IECL 2003b, 2004) as required by Transport Québec to assess its performance. The heaviest truck available caused a maximum bending moment of 20% of the fully factored CHBDC moment for the two lane bridge. Static deflections taken at 27 locations for five different load cases were in good agreement with predictions. They were consistent with full composite action that, for symmetric loading, is only about 1/3 of that without composite action. The comparison of measured and predicted strains through the depth of the girders shown on Fig. 7 also confirms this action. The cross sectional area of the SPS faceplates provides much compressive resistance and lifts the neutral axis to about 130 mm below the top flange of the girders.

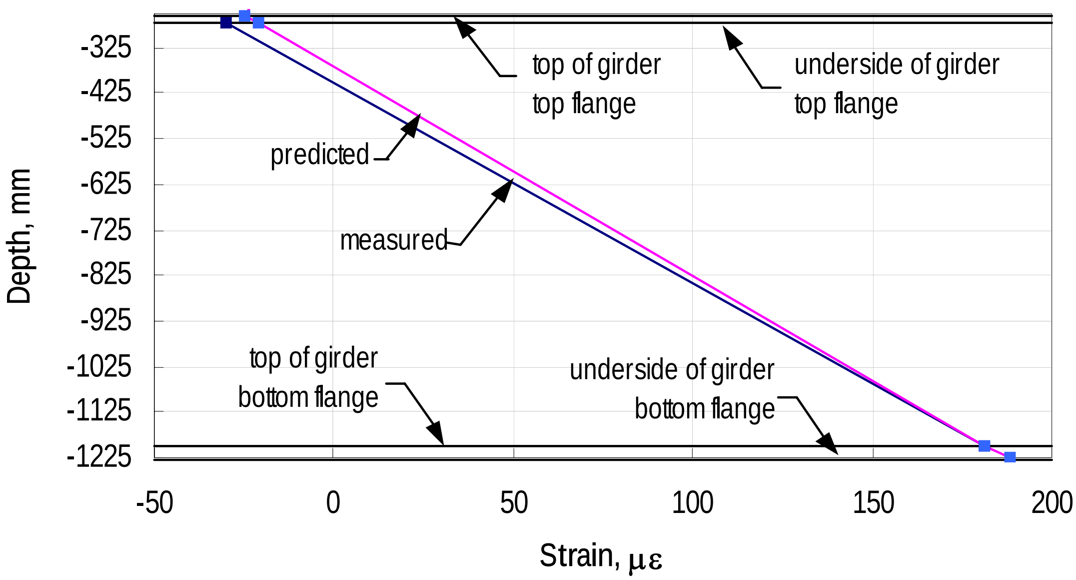


Fig. 7. Strain Distribution in Girder A for Load Case 5

Vibration tests were conducted prior to the application of the asphalt wearing surface by Murray and Setareh of Virginia Polytechnic Institute on 10 November 2003 to establish the natural frequencies and damping characteristics of the bridge. Frequencies of the first three modes of vibration were 5.8 Hz, 6.0 Hz and 15.3 Hz with respective damping ratios of 0.8%, 1.0% and 0.5%. The predicted natural frequency response for the first and second mode matches the measured values within 2%.

Numerous tests have shown that, at the typical bond stress ranges between the roughened steel and the elastomer, fatigue is not a problem on the steel-elastomer interface. However the fatigue resistance of the steel details must be examined. All bolted connections were, of course, designed as slip–critical cionnections.

Two types of welded connections are of chief concern in fatigue – the transverse square groove welds joining the top face plates to the edge angles and the transverse V groove weld joining the same edge angles together. CSA Standard S6-00 (CSA 2000) places both of these details in the low detail category E for a welded transverse deck plate splice with permant backing bar with a fatigue stress range for an unlimited number of cycles of one half the constant amplitude threshold stress range of 31 MPa. The latest AASHTO code is more liberal and assigns these to the higher Category D.

With this review of Limit States Design rules and of SPS deck panels we now propose design rules for the ULS – Ultimate Limit States - in particular for the SPS panels. The rules needed for the FLS – Fatigue Limit State – for steel details already exist in Section 10, Clause 10.17. The design rules required for the ULS follow in the next section of this paper. They require that material be added to the existing Section 10 Clauses as noted.

DESIGN RULES FOR SPS DECK PANELS

To integrate the new Clause on the design rules for SPS deck panels, herein designated as Clause 10.S, into Section 10, Steel Structures, of the CHBDC-00 additions and changes have to be made to a number of existing clauses in section 10. These proposals are first presented. Comparing these to existing clauses will show where changes are suggested.

10.2 Definitions

Faceplates - The steel plates that form the outer components of the sandwich plate system having inner surfaces suitably roughened to develop the needed bond strength on the steel-elastomer interfaces. Faceplates are generally flat but may be curved.

Perimeter Bars - steel bars of a depth equal to the desired thickness of elastomer and appropriately fastened by welding or other means to the two faceplates and flush with their perimeters to form the cavity into which the elastomer is injected, Perimeter bars may be supplemented or replaced by steel side plates or shapes to provide additional strength and stiffness to the SPS panel.

Sandwich Plate System - a structural system comprising steel faceplates with perimeter bars that form an air-tight cavity together with a structural elastomer core formed by injecting a two-part thermo-setting elastomer liquid into the cavity. Structural elements deeper than the SPS may be present on some or all sides or at intermediate locations to provide increased stiffness to the panel as a whole in one or orthogonal directions. The panel faces are suitably restrained during the setting process to limit expansive swelling.

Side plates - The steel plates that form the outer perimeter of the sandwich plate system having inner surfaces suitably roughened to develop the needed bond strength on the steel-elastomer interfaces. Side plates may be cold-formed angles projecting below the bottom faceplate as stiffeners or to accommodate fasteners and may project above the top faceplate to contain wearing surfaces or coatings.

Structural Elastomer Core - a two-part polymer comprising the correct proportions of a isocyanate and polyol that are mixed by high-pressure-impingement at the injection port(s) to set and bond automatically with the roughened faceplate surfaces to form a steel-elastomer-steel composite structural panel that acts as a unit.

Bubble Core – a structural elastomer core comprising sufficient two-part polymer to transfer the shear forces for the Limit State under consideration but containing spherical elastomer bubbles thereby reducing the amount and weight of elastomer in the core.(See Structural Elastomer Core.)

Composite Core – a structural elastomer core comprising sufficient two part polymer ribs in one direction or in orthogonal directions and of sufficient size to transfer the shear forces for the Limit State under consideration. (See Structural Elastomer Core.)

Solid Core - a structural elastomer core comprising solely a two-part polymer (See Structural Elastomer Core.)

10.3 Notation and Units

A_p	cross-sectional area of portion of perimeter bar of a given panel, mm^2
A_{gb}	tensile or compressive area of steel in one panel, mm^2
a	length of SPS panel, mm
b	width of SPS panel under consideration, mm
b_e	total effective width of two edge strips of an SPS panel, mm
C_e	Euler buckling strength = $\frac{\pi^2 EI}{L^2}$, N
F_{sr}	fatigue stress range, MPa, at number of cycles, N
I_b	flexural moment of inertia moment of inertia of panel of width b, mm^4
I_{po}	centroidal moment of inertia of perimeter bar of a given panel, mm^4
M_r	factored plastic moment resistance = $\phi_s ZF_y$, Nmm
M_p	fully plastic moment resistance = ZF_y, Nmm
M_y	yield moment resistance = SF_y, Nmm
m_p	factored plastic moment resistance per millimeter of length, Nmm/mm
P_y	yield load of an SPS panel of width b, for equal faceplate thicknesses, t_f, $P_y = 2t_f\, bF_y$, N

Q_b Shear parameter, mm^3

SPS 6-30-6 standard designation for a sandwich plate system indicating a 6 mm thick top faceplate, a 30 mm thick elastomer core and a 6 mm thick bottom faceplate.

t centre-to-centre distance of face plates

t_e thickness of elastomer core, mm

t_f thickness of one of a pair of faceplates of equal thicknesses,

U_1 factor to account for moment gradient and second order effects of axial force acting on deformed member

ϕ_{ib} resistance factor for interface bond = 1.0

ϕ_{st} resistance factor for steel in tension

τ_f factored bond stress

τ_r nominal interface bond resistance

ω_1 coefficient to determine equivalent bending effect in members subject to axial load and bending

10.4.2 Structural Steel

Structural steel shall conform to CAN/CSA-G40.21.The modulus of elasticity of steel, E_s, shall be taken as 200 000 MPa and the shear modulus of elasticity shall be taken as 76 000 MPa except in SPS faceplates and cold-formed pieces less than 10 mm in thickness respective values of 206 000 MPa and 79 000 MPa shall be used

10.4.13 Elastomer

10.4.13.1 Stress – strain data

The modulus of elasticity of structural elastomer may be taken as 750 MPa with a maximum strain of 25 000 □□ and the shear modulus as 275 MPa, unless otherwise approved, except that under extreme service temperatures below -20^{o}C or above 60^{o}C appropriate values based on published test data shall be used.

10.4.13.2 Mass density

The mass density of elastomer shall be taken as1100 kg/m^3.

10.4.13.3 Coefficient of Thermal Expansion

The coefficient of thermal expansion for elastomer that varies with temperature may be taken as *96* x 10^{-6}/^{o}C to 149 x 10^{-6}/^{o}C for temperatures from -30 ^{o}C to +30 ^{o}C unless otherwise specified.

10. 5.4 Fatigue Limit State

The requirements of Clause 10.17 shall be met for all steel components and of Clause 10.S.7 for elastomer-steel interfaces.

10.5.7 Resistance factors

(k) steel-elastomer interface, ϕ_b = 1.00 in conjunction with a nominal bond strength, τ_b = 6.0 MPa for ambient temperatures not exceeding 40 ^{0}C.

10.6.1 Deterioration Mechanisms

The deterioration mechanisms considered for components shall include corrosion. No deterioration need be considered for the elastomer enclosed within cavities and for the interior steel surfaces of such cavities.

10.7.2 Minimum thicknesses of Material

(e) When the minimum thickness of steel faceplates in SPS panels of 4 mm is used special attention shall be given to possible corrosion mechanisms. Otherwise the minimum thickness shall be taken as 6 mm.

10.7.4.1 Design

SPS panels shall not be cambered.

10.7.5 Welded attachments

All welds in SPS panels shall be made with continuous welds.

These changes and additions accompany the new Clause10.S following

10.S Sandwich Plate System Deck Panels

10.S.1 General

Clause 10.S applies to the design of Sandwich Plate System (SPS) decks comprising individual SPS panels joined together and to the supporting structure. Each panel comprises top and bottom steel faceplates bonded to an elastomer core that is bounded on all sides by integral longitudinal and transverse side plates that are attached to the faceplates and stiffen and support the SPS panels. Connections between the deck panels and other structural members shall be designed to ensure full composite interaction with them.

10.S.2 Construction Requirements for SPS deck panels

10.S.2.1 General

Clause 10.S.2 gives requirements for the construction of SPS deck panels for highway bridges and applies unless otherwise specified by the authority having jurisdiction. These requirements are provided to ensure compliance with the design philosophy of Section 10, Steel Structures.

10,S.2.2 Fabrication

Panels shall be fabricated in accordance with good shop fabrication practices to meet the requirements of Clause 10.24, this Clause and Clause 10.S.8 on splices and connections.

10.S.2.3 Preparation of steel-elastomer interfaces

When the elastomer is about to be injected, the interior surfaces of steel faceplates and edgeplates shall have been grit-blasted, and shall be dry, clean, and free of all contaminants. Flash rusting is permitted as specified by the authority having jurisdiction The abrasive used for grit blasting shall comprise angular profile steel grit of Rockwell C Hardness greater than 62 or alternatively angular corundum (aluminum oxide).

10.S.2.4 Dimensional Tolerances

Panels shall after fabrication, unless otherwise specified, meet the following tolerances:

- Thickness of faceplates, + 0. 50 mm, -0.00 mm
- Overall thickness of panel, +2 mm, - 1mm
- Straightness or flatness, 1/1000 of lesser overall panel dimension
- Width or length, joined to other panels, ± 1 mm
- Separation in joints between panels, 1 mm
- Width or length, not framed to other panels but framed to other steel, ± 2 mm

10.S.2.5 Restraint during Polymer Injection

The faceplate surfaces shall be restrained against expansion of the polymer during injection and initial curing to withstand the expected expansion pressures based on published data and to limit expansion of the upper faceplate to 1/600 of the least dimension of the panel.

10.S.3 Analysis

10.S.3.1 General

Unless otherwise approved, methods of analysis shall be as specified in Section 5, Section 10 as applicable, and one of the alternative analyses of this section.

10.S.3.2 Finite Element Analyses

Any finite element analysis shall be geometrically non-linear and based on the following general principles unless otherwise approved:

- model the boundary conditions of SPS panels and other bridge elements as they exist in the bridge,
- have a sufficiently fine mesh density in areas of high stress variation,
- model non-linear material properties of the steel or, as a conservative approximation, consider the steel to be linearly elastic to a maximum stress of F_y
- model the changes in the geometry of the structure as loads increase.

Note: *Kim and Hughes (2004 accepted) provide closed form solutions for the ultimate strength of SPS panels under in-plane edge compression and uniform lateral pressure.*

10.S.3.3 Standard Structural Analyses

In standard structural analyses, the factored resistances and stiffnesses of the SPS deck panels under different loading conditions and combinations shall be taken as given in Clauses 10.S.5.

10.S.4 Panel Design Dimensions

Panel dimensions shall be taken as the distance out-to-out of perimeter bars. The lesser panel dimension of rectangular panels shall be taken as the width, b, and the greater as the length, a.

10.S.5. Factored Resistances of SPS Panels

10.S.5.1 Cross-Sectional Properties of the SPS *per se*

The cross sectional properties of SPS panels shall be based on the cross sectional area of steel only. See the Commentary on Section 10. As a conservative approximation, the areas of the perimeter bars may be neglected.

10.S.5.2 Supports of SPS Panels

The supports for SPS panels on all sides shall have sufficient strength and stiffness to generate the reactions consistent with the critical failure mode or modes assumed in the analysis for the SPS panels themselves.

10.S.5.3 Panels Supported on Two Parallel Sides

The design of panels spanning in one direction, if not based on a finite element analysis as prescribed in Clause 10.S.3.2, shall be based on a standard structural analysis, see Clause 10.S.3.3, using the cross sectional properties given in Clause 10.S.5.1 and the factored resistances of Clauses 10.S.5.5.

10.S.5.4 Panels Supported on Four Sides

The design of panels supported on four sides, shall be based on a finite element analysis as prescribed in Clause 10.S.3.2, or on a standard structural analysis based on the effective width model of Clause 10.S.5.9 using the factored resistances of Clause 10.S. 5.9.

10.S.5.5 Panels in Bending

The factored moment resistance, M_r, of a panel bent uniaxially shall be taken as

$$M_r = \phi_s Z F_y = \phi_s M_p = \phi_s b t_f (t_f + t_e) F_y$$

where

Z = plastic section modulus of the panel as given in Clause 10.S.5.1

10.S.5.6 Panels in Tension

10.S.5.6.1 Cross-Sectional Area

Panels shall be proportioned based on the gross area of the steel faceplates and perimeter bars taken normal to the axis of the panel, unless connections reduce the gross cross-sectional area when the provisions of Clause 10.8.1 and 10.8.2 shall apply.

10.S.5.6.2 Factored Tensile Resistance

The factored tensile resistance, T_r, of an SPS panel with connections that do not reduce the gross sectional area shall be taken as:

$$T_r = \phi_s A_g F_y$$

where

A_g = $2bt_f + 2 A_p$
b = width of panel perpendicular to the panel axis
t_f = thickness of one of a pair of faceplates of equal thicknesses
A_p = cross-sectional area of perimeter bars of the given panel that are perpendicular to the panel axis

10.S.5.6.3 Axial Tension and Bending

Panels supported on two sides subjected to bending moments and axial tensile forces shall satisfy the relationship given in Clause 10.8.3 for Class 1 and 2 sections.

10.S.5.7 Panels in Compression

10.S.5.7.1 Cross-Sectional Area

Panels in compression shall be proportioned based on the gross area of the steel faceplates and perimeter bars taken normal to the axis of the panel.

10.S.5.7.2 Slenderness

The slenderness ratio of a panel shall not exceed 280.

10.S.5.7.3 Factored Compressive Resistance, Flexural Buckling

The factored compressive resistance, C_r, of an SPS panel, with no longitudinal edge or intermediate supports, conforming to the limits of Clause 10.S.5.7.2 shall be taken as given in Clause 10.9.3.1 with n = 2.24:

10.S.5.7.4 Axial Compression and Bending

Panels supported on two parallel sides only and subjected to coincident axial compression and uniaxial strong axis bending shall be proportioned in accordance with Clause 10.9.4.

where

C_r is as defined in Clause 10.S.5.7.3

M_r is as defined in Clause 10.S.5.5

10.S.5.8 Factored Shear on Elastomer-Steel Interface Due to Bending

The factored shear or bond stress, □$_{f,}$ on the elastomer steel interface shall be taken as

$$\tau_f = VQ/Ib$$

where

- V = the factored transverse shear resisted by the cross-section under consideration
- Q = the first moment of area of the steel faceplate about the neutral plane as given in Clause 10.S.5.1
- I = the flexural moment of inertia as given in Clause 10.S.5.1
- b = the net width of the elastomer carrying the shear V.

The factored shear stress, τ_f, shall not exceed the factored interface bond resistance

$$\tau_r = \phi_i b \tau_b = 1.00 \times 6.0 \text{ MPa as given in Clause 10.5.7}$$

10.S.5.9 Factored Resistances of Panels Supported on Four Sides

10.S.5.9.1 General

The supports for SPS panels on all sides shall meet the requirements of Clause 10.S.5.2.

10.S.5.9.2 Factored Bending Resistance

Unless a more complicated model is required because of specific loading conditions or is implemented for greater computational accuracy, the failure mode of a panel subject to a uniform out-of-plane distributed load, may be taken as that corresponding to negative fully plastic moment lines along all fixed supports at the inner boundary of the perimeter bars and positive fully plastic moment lines parallel to the long side of dimension "a" splitting into lines running at 45^0 to the four corners. The factored plastic moment resistance per millimetre of length shall be taken as:

$$m_p = \phi_s Z F_y = \phi_s t_f(t_f+t_e)F_y == \phi_s t_f t F_y$$

where

- t = the centre-to-centre distance of the face plates of equal thickness t_f and for the panel as a whole:
- $M_r = 3\phi_s t_f t F_y[1.14a + b]$ for panels with fixed edges and
- $M_r = \phi_s t_f t F_y[1.41a + b]$ for panels with simply supported edges

The load required to develop the failure condition shall be greater than or equal to the applied factored load.

Yield line patterns for other load distributions shall be consistent with the said distributions and the factored plastic moment resistance per millimetre of length given here.

10.S.5.9.3 Factored Compressive Resistance

The factored compressive resistance of an SPS panel with its edges parallel to the axis of loading supported laterally may be taken as the yield load on two edge strips of equivalent total width of:

$b_e = 3.00t\sqrt{E/\sigma_y}$ for plates with simply supported edges and
$b_e = 3.70t\sqrt{E/\sigma_y}$ for plates with fixed supported edges and

where

t = the centre to centre distance of the face plates of equal thickness t_f, thus giving:
$C_r = 2\phi_s b_e t_f F_y$

where

b_e is defined severally above for simply supported and fixed edge supports.

10.S.5.9.4 Axial Compression and Bending

Panels required to resist both bending moments and uniaxial compressive force shall be proportioned so that the overall panel strength satisfies

$$\frac{C_f}{C_r} + 0.80\frac{M_f}{M_r} \leq 1.0$$

where

C_f and M_f are the maximum load effects for the loading condition investigated and
C_r is as defined in Clause 10.S.5.9.3
M_r is as defined in Clause 10.S.5.9.2

10.S.5.9.5 Axial Tension and Bending

Panels required to resist both bending moments and axial tensile force shall be proportioned so that the overall panel strength satisfies

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1.0$$

where

T_f and M_f are the maximum load effects for the loading condition investigated
T_r is as defined in Clause 10.S.5.6.2
M_r is as defined in Clause 10.S.5.9.2

10.S.5.10 Longitudinal Stringers and Floor Beams

10.S.5.10.1 General

Longitudinal stringers and floor beams providing direct support for SPS panels shall have sufficient strength and stiffness to generate the vertical reactions consistent with the critical failure mode or modes assumed in the analysis for the SPS panels themselves.

10.S.5.10.2 Cross sectional properties

The cross sectional properties shall be based on the gross area of steel taken normal to the axis of the member including a width of the SPS panel, symmetric about the longitudinal axis not greater than that corresponding to a width-to-thickness ratio, b/t = $580/\sqrt{F_y}$, where "t" is the centre-to-centre distance of the face plates of equal thickness.

10.S.5.10.3 Bending Moments

The factored bending moments for longitudinal stringers and transverse floor beams shall be consistent with the respective support conditions and the governing load on the adjacent trapezoidal or triangular SPS panel areas extending from the member centre line to the positive yield lines on either side or as otherwise assumed in the analysis of the panels.

10.S.5.10.4 Longitudinal Stringers

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Longitudinal stringers required to resist both bending moments and an axial compressive force shall be proportioned so that

$$\frac{C_f}{C_r} + \frac{U_1 M_f}{M_r} \le 1.0$$

where

C_f and M_f = the maximum load effects including stability effects,
C_r is as defined in Clause 10.S.5.7.3 with A_g taken as defined in Clause 10.S.5.10.2 and an effective length, L = a,
M_r = the factored fully plastic moment or factored yield moment as appropriate for the width-thickness ratios of elements in compression but reduced in accordance with Clause 10.9.4.1 when lateral torsional buckling is a possibility due to a laterally unsupported compression flange.

U1 is as defined in Clause10.9.4.2 with ω = 1.0 in accordance with Clause 10.9.4.3.

10.S.5.10.5 Floor Beams

Floor beams with continuous lateral support provided to the compression flange, simply supported at the ends, and subject to bending about the major axis, shall proportioned so that

$M_r \geq M_f$

where

$M_r = \phi M_p$ for Class 1 and 2 sections as defined in Clause 10 9.2
$= \phi M_y$ for Class 3 sections as defined in Section 10.9.2 and
M_f = governing bending moment as defined in Clause 10.S.5.10.3

10.S,6. SPS Decks in Composite Construction

10.S,6.1 General

Connections shall be made between the supporting girders and the steel components of the SPS panels by bolting, welding or other approved means

10.S,6.2 Bolted Connections to Steel Girders

Bolted connections between SPS decks and steel girders shall be designed;

(a) to achieve full composite action for the ULS based on the factored shear resistance whether governed by the bearing resistance of the plate adjacent to the bolts or by the shear resistance of the bolts and
(b) as slip-critical connections at the FLS stress range for the anticipated number of cycles of the fatigue truck live,
(b) as slip-critical connections for the SLS

10.S,6.3 Welded Connections to Steel Girders

Welded connections between SPS decks and steel girders shall be designed;

(a) to achieve full composite action for the ULS based on the factored shear resistance of the weld
(b) for the FLS stress range for the anticipated number of cycles of the fatigue truck live load

10.S,6.4 Connections to Concrete Girders

Such connections may be made using bolts or other connectors acceptable to the authority having jurisdiction, grouted into the concrete girder using shear connection values based on appropriate published statistical data for the ULS and FLS as approved.

10.S,6.5 Control of permanent deflections

The normal stress in SPS decks and the flange of the steel girders due to serviceability dead and live loads shall not exceed 0.90Fy in both positive and negative moment regions

10.S.7. Structural Fatigue

10.S.7.1 Steel components

The steel components of SPS decks, including supporting longitudinal stringers and floor beams subject to fatigue loading shall meet the requirements of Clause 10.17 in respect to bridge decks.

10.S.7.2 Solid Elastomer Core

The shear stress on the interface between the steel and elastomer meeting the requirements of Clause 10.S.2.3 may be considered insensitive to fatigue.

10.S.7.3 Composite or Bubble Elastomer Core

The fatigue resistance for the interface between the steel and elastomer meeting the requirements of Clause 10.S.2.3 when composite or bubble cores are used shall be taken as $Fsr = (7580/N)^{1/12}$

10.S.8 Splices and Connections

10.S.8.1 General

Connections and splices shall be designed at the ULS to meet the requirements of Clause 10.18. See also Clause 10.S.7

10.S.8.2 Panel-to-Panel Splices

Welded panel-to-panel splices, generally made in the field, shall be made in such a manner that the elastomer adjacent to the splice connection is not charred in the welding process unless previous allowance has been made to accommodate this condition. Protection to the elastomer against charring is provided by (a) steel heat sinks to absorb the excess heat of the welding process or (b) distancing the weld from the elastomer such that the latter is not overheated or (c) a combination of the two.

10.S.8.3 Connections to SPS Panels

Where steel members are to be welded to SPS panels into which the elastomer has been already injected, the provisions of Clause 10.S.8.2 shall be followed.

10.S.9. Overlays

10.S.9.1 General

When an SPS overlay is used to rehabilitate an existing steel deck the general design principles enunciated herein shall be followed and shall be based on the remaining thickness of the existing steel deck as established by a statistical analysis of a sufficiently broad sample of field measurements, the new thicknesses of the top faceplate and the minimum thickness of the elastomer core unless a more detailed analysis is conducted accounting for the varying thickness of the core based on measured profiles of the steel deck at the time of rehabilitation. Existing cracks shall be re-welded unless it can be established to the satisfaction of the authority having jurisdiction that the probability of propagation in the rehabilitated deck is sufficiently remote.

10.S.9.2 Perimeter bars and top faceplate

Perimeter bars defining the minimum thickness of elastomer shall be welded in place on convenient longitudinal and transverse members to form cavities of such a volume that can be readily injected before setting of the elastomer takes place. The top faceplate is welded or otherwise connected to these to form an airtight cavity into which the elastomer is injected.

10.S.9.3 Ambient and work temperatures

The minimum ambient temperature and that of the steel cavity when the elastomer is injected in the field shall be 5^0C

METHODS OF ANALYSIS FOR COMBINED BENDING AND IN PLANE LOADING

This most important analytical problem common to virtually all deck panels is first addressed. Three methods of analysis are available for the ULS, the Ultimate Limit States.

A second order non-linear inelastic finite element analysis, such as in Ansys Release 9.0 of Ansys 2004, is now widely available and is the preferred approach because it minimizes the number of simplifications that need to be introduced. Correct modelling of the material properties, the loading and support conditions, and proper mesh density are required to generate valid analyses. Simple problems that are easily handled or verification against experimental results can be of value.

Kim and Hughes (2004 Accepted) have developed closed form second order solutions for SPS panels subject to uniform lateral pressure and in-plane compression. It is expected that this could be extended to other lateral loading conditions such as that for a bridge deck with tire loads distributed over relatively small areas.

The third solution is based on limit states design principles and simplified ultimate load analyses. The factored compressive resistance of an SPS panel, when both edges parallel to the axis of loading are supported, is taken as the yield load on two "equivalent total width" edge strips as given in Clause 10.S.5.9 for simply supported and fixed edges and on the perimeter bars. The factored bending resistance (without in-plane loading) is based on a simplified yield line pattern as shown in Fig. 8 with fully plastic bending moments per mm of length of $m_p = \phi_s Z F_y$ where Z is the plastic section modulus of the steel faceplates per mm. Yield line patterns for other loading conditions can be constructed in a similar manner.

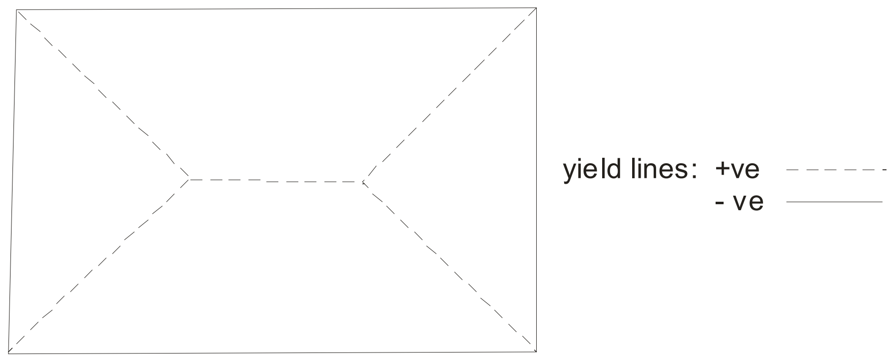

Fig. 8. Simplified yield line pattern, uniform load

Fig.9 shows the interaction diagram of axial load versus moment that is appropriate for SPS panels where C_r and M_r are obtained as described and C_f and M_f are the maximum load effects for the loading condition investigated.

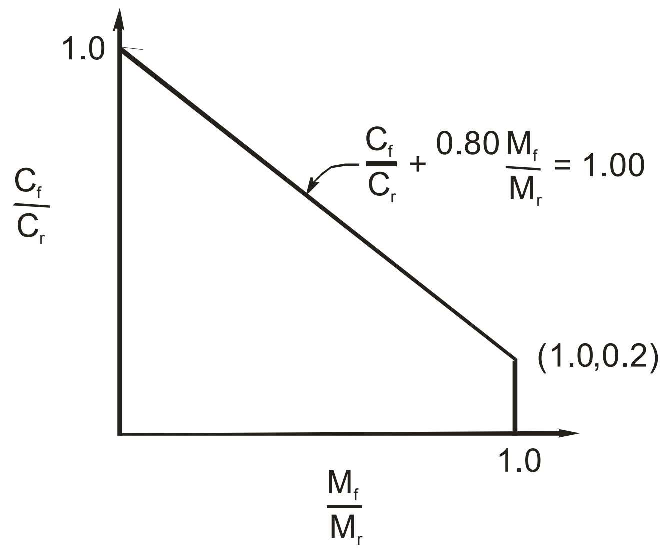

Fig. 9 Interaction diagram for axial load and moment

WHAT DOES THE DECK DO? HOW DOES IT DO IT?

When a deck is designed and constructed to meet the limit states design rules given herein, the deck is expected to perform its two primary functions of providing direct support for vehicular traffic and acting compositely with the longitudinal girders. (Other composite action may occur.). Deck panels are therefore subject to out of plane bending and to overall in-plane compressive or tensile forces depending on the sign of the overall bending moment at the section considered. Methods of analysis are presented in the previous section for the combined loading case and other considerations are discussed here. The panel supports have to generate the reactions consistent with the analysis of the panels.

Because the modular ratio of steel to elastomer is high and in the order of 280,only the steel components are considered to contribute to the cross-sectional properties and hence to carry all internal forces. Thus the contribution of the elastomer is minimal in this regard and the cross section is essentially one of steel.

The elastomer core, however, serves the two important and basic functions of transferring shear forces from one faceplate to the other and providing lateral support to the faceplate in compression. In its first role, the elastomer core spaces the faceplates apart and increases the stiffness and strength of the steel many times. Compared to a single 12 mm thick steel plate, an SPS 6-30-6 panel with a 30 mm elastomer core and the same area of steel has 6 times the plastic section modulus and 27.2 times the moment of inertia..

The second role of supporting the faceplate means that local buckling is precluded and the faceplate is as effective in compression as it is in tension on the gross section.

The factored resistances of panels supported on two parallel sides are thus simply derived for the steel cross section with resistance factors appropriate for steel members in tension, compression, shear and bending as the case may be and as given in Section 10 of the CHBDC. In buckling, the coefficient for axial buckling resistance, n, is taken as 2.24 because the residual stresses are low. In flexure because the panels are bent about their weak axis, lateral torsional buckling does not occur.

For fatigue loading, only steel details are considered unless the interface shear area between elastomer and steel is reduced substantially and sufficiently in a composite or bubble core to make the stress range on the interface high enough. Even then the S-N curve is very flat with a slope of about 1/12 as compared to 1/3 for steel details.

As noted previously, the soft elastomer core *per se* helps dampens vibrations.

Compared to concrete decks a major advantage of the SPS panels is the reduced weight of the latter. The Shenley Bridge panels, for example, weighed only 35.5 psf.

Durability of bridge decks is a major economic issue and it is suggested that SPS deck panels would be very durable and perform well. The bottom steel surfaces that are exposed to the atmosphere have simple geometry and as well are relatively well protected from the elements. The flat top surface is protected by the seal coat and asphalt.

Although the face plates are relatively thin, in extending across the full width of the bridge they add considerably to the area of longitudinal steel resulting in a neutral axis position somewhere near the top flange of the girders. Thus the capacity of the bridge in its primary role as a flexural member is considerably enhanced. This suggests that non symmetric girders with smaller top flanges could be considered.

Existing bridges with orthotropic steel decks that have suffered fatigue cracking can be rehabilitated by using the existing steel deck as the bottom faceplate, installing perimeter bars with a new top faceplate and injecting elastomer into the cavity. The new deck is many times stiffer at a cost in dead load that is relatively small. The potential propagation of existing cracks would be based on the new regime. This type of rehabilitation has been carried out in Germany by Thyssen-Krupp Stahlbau (IECL 2003a).

All in all, the SPS deck panels are seen to offer significant advantages for bridges in Canada whether for primary use or for rehabilitation.

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