Reliability-based Fatigue Assessment of Internally or Externally-restrained Concrete Deck Slabs

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ABSTRACT: Over last sixty years in North America, highway vehicle configurations have undergone revolutionary changes with longer and longer combinations of vehicles with larger number of axles in use. Furthermore, both axle weights and traffic volume on the highways have significantly increased over the years. Whereas concrete deck slabs in highway bridges have been known to have very high static wheel load carrying capacity owing to the internal arching action, the fatigue behaviour of deck slabs and related components under service loads have not yet been fully explored. Various researches have shown that with repeated application of wheel loads, the static load carrying capacity of concrete deck slabs tend to reduce over time due to the accumulation of the fatigue damage.

In externally restrained fibre-reinforced concrete deck slabs with steel straps, fatigue may be of concern in both the concrete and steel straps. In accordance with the Canadian Highway Bridge Design Code (CHBDC), steel straps, whether welded to girder flange or partially studded, belong to the Detail Category C for fatigue consideration with a constant amplitude threshold stress range of 69 MPa. For large number of cycles of applied stress range, the fatigue stress range limit could be reduced to as low as 34.5 MPa.

This paper provides a reliability-based methodology to assess fatigue performance of both new and existing deck slabs. Fatigue loading to be used to assess fatigue performance may be established from field monitoring of site-specific loading or stresses in critical bridge components. The reliability analysis has greater validity if the scientific data is obtained using structural health monitoring (SHM) techniques in the field. The procedure to carry out the fatigue assessment is demonstrated with the help of several examples.

It was analytically found that, barring severe environmental damage, the concrete deck slab designed by the CHBDC have adequate fatigue strength. However, the steel straps for some designs by the CHBDC provisions, particularly with thinner deck, may not perform satisfactorily for fatigue under the assumed traffic loads. The paper makes recommendations for revising the CHBDC requirement for minimum area and spacing of steel straps.
INTRODUCTION

Concrete deck slabs in highway bridges have been known to have significantly high static load capacities than those predicted by purely flexural theories. The higher load carrying capacities are primarily due to “in-plane compressive membrane action” (Hewitt and Batchelor 1975), or “internal arching action” (e.g., Mufti et al., 1993). While the static load carrying capacities of deck slabs are several times higher than the expected largest wheel loads including dynamic amplifications, the actual capacities are governed by the fatigue performance of the slabs under moving loads. However, various researchers (Matsui et al. 2001, Memon et al. 2003) have related the fatigue performance of concrete deck to its static punching shear capacity which has a high degree of variability due to variability in dimensions, material strengths, and analysis of punching shear capacity (Agarwal 2008-1) and can be determined using reliability principles. This paper discusses a reliability-based approach to assess the fatigue performance of concrete bridge decks and related components under wheel loads, both for new designs as well as for the evaluation of existing deck slabs. Fatigue loading to be used to assess fatigue performance may be established from field monitoring of site-specific loading or stress in critical bridge components. In this paper default values of fatigue loading for Canadian conditions have been used. Fatigue performance is examined for (a) internally restrained concrete deck slabs with FRP reinforcement, and (b) externally restrained deck slabs with steel straps.

FATIGUE BEHAVIOR OF DECK SLAB COMPONENTS

For internally restrained deck slabs, fatigue may be considered in the concrete and in the FRP reinforcement. For externally restrained deck slabs with steel straps, fatigue need to be assessed in both concrete and steel straps.

In the evaluation of existing deck slabs, long-term environmental effects on concrete strength are established through core testing. For externally restrained deck slabs with steel straps, the long-term environmental effect on straps is considered as loss of section due to corrosion of steel. It is proposed to consider a 0.01 mm/year loss of material all around the strap section. Environmental effect on the internal FRP reinforcement may be considered by lowering the specified strength by a suitable fraction depending upon the type of FRP material.

For internally restrained concrete deck slabs with FRP reinforcement, it is generally found that the stress in FRP reinforcement at the punching shear failure of deck is significantly lower than the specified tensile strength of FRP reinforcement, because of which the long-term environmental reduction in strength of FRP reinforcement has an insignificant effect on the static punching shear capacity of the concrete deck.

It has been observed that over time under the application of repeated loads, the residual deflections and crack widths in the deck slab grow (e.g. Memon et al., 2003). Because of potential stress concentrations in the FRP bars at the crack locations, there might a concern about the fatigue performance of the bars at the bottom of the slab. However, as shown by Agarwal (2009), stress concentration in the internal reinforcement requires bond between the reinforcement and the surrounding concrete and depends upon its effectiveness. This results in uncracked concrete around reinforcement to carry significant amount of tension which in turn increases the stiffness of deck and its load carrying capacity. Over time with repeated loading, this bond between reinforcement and concrete gradually diminishes due to progressive micro-cracking in the surrounding concrete, thus reducing tensile concrete contribution to deck stiffness and resulting in greater deflection and wider crack opening under the same load. This diminishing bond effectiveness also reduces stress concentration in the reinforcing bar. Stress concentration is greatest at the early loading stage when the bond is most effective, but total stress in the FRP reinforcing bar including stress concentration would still be too low compared...
to the tensile strength of the rebar to cause any premature fatigue failure of the rebar. Furthermore, the bond effectiveness and resulting increased load carrying capacity should not be relied upon at the ultimate limit state, and the punching failure load should be determined by conventional methods ignoring contribution of tension in concrete.

2.1 Fatigue in Concrete Deck

2.1.1 Fatigue life Curve

The general format used for fatigue life curve for metallic components is given in Eq. [1].

\[ N = \gamma F_{sr}^{-m} \]  

Where \( N \) is maximum number of cycles of uniform stress range \( F_{sr} \) that the component can sustain before fatigue failure, and \( \gamma \) and \( m \) are the constants. The equation can be rewritten as,

\[ F_{sr} = \left( \frac{\gamma}{N} \right)^{1/m} \]

For structural steel, the value of \( m = 3 \) has been used in the CHBDC (CSA 2006) for all fatigue detail categories. Value of \( \gamma \) depends upon the specific fatigue detail category.

Matsui et al (2001) and Memon et al (2003) observed a reduction in punching strength of a deck subjected to a number of cycles of repeated wheel loads and accounted for this behavior by relating the reduced ultimate strength under repeated loading to the initial static ultimate punching shear strength. Based on laboratory tests, Matsui et al (2001) developed the following equation for the fatigue life of concrete in deck,

\[ \log \left( \frac{P}{P_s} \right) = -0.07835 \log(N) + \log(1.52) \]

where, \( N \) is the number of cycles to failure under repeated cycles of a uniform load \( P \), and \( P_s \) is the initial static punching shear strength. Eq. [3] can be rewritten in a classical form of fatigue life curve as follows:

\[ \left( \frac{P}{P_s} \right) = 1.52N^{-0.07835} = 1.52N^{-1/12.76} \approx 1.5N^{-1/13} \]

Memon et al. (2003) developed a different form of relationship given by,

\[ \left( \frac{P}{P_s} \right) = 1.0 - \frac{\ln(N)}{30} \]

This has been further approximated into the classical form as follows:

\[ \left( \frac{P}{P_s} \right) = 1.19N^{-1/17} \]

In this equation, \( m = 17 \). Henceforth Eq. [6] has been used in this paper for fatigue performance assessment of the concrete deck.
2.1.2 Long-term Assessment of Fatigue in Concrete Deck

In the long-term, two aspects of the deck slab are worthy of consideration: (a) long-term reduction in static punching capacity due to the environmental effect on concrete quality, and hence on its strength, and loss of the cross-section of the steel straps, and (b) reduction of the static punching capacity of the deck slab due to accumulation of fatigue damage. Load-induced reduction in the static punching capacity over the time may be established by determining used-up fatigue life. Agarwal (2008-2) has proposed to use Eq. [7] to determine the reduced static punching shear capacity, \( P_{sR} \).

\[
\left( \frac{P_{sR}}{P_s} \right) = 1 - \left( 1 - \frac{P_a}{P_{sa}} \right)^{n_a/N_a}^{1.1}
\]

where, \( n_a \) is the total number of applied cycles of uniform wheel load \( P_a \) up to the time of evaluation, and \( N_a \) is the fatigue life to failure under the application of cycles of uniform wheel load \( P_a \). Value of \( P_s \) on the left-hand-side of the equation should be taken as the current value of \( P_s \) (including the effect of environmental deterioration and current condition of concrete and loss of strap section), and \( P_{sa} \) on the right hand side is an average of current value of \( P_s \) and the \( P_s \) corresponding the new deck. The index of 1.1 has been selected to be in the middle of the practical range, being 1.0 to 1.2 (Reifsnider et al. 1986). A value of 1.0 of the index gives most conservative estimate of the reduction in strength. As discussed below, \( P_a \) may be taken as the equivalent fatigue wheel load including the dynamic load allowance to represent the actual variable wheel load spectrum.

2.1.3 Determination of \( P_s \) for use in fatigue assessment of deck slab concrete in field

In the field, the initial static ultimate punching shear capacity would have potential statistical variation (Agarwal 2008-1) due to the statistical variation in material properties, dimensions, and analytical determination of the capacity. The statistical variation is assumed to be lognormal and characterized by a bias factor \( \delta_P \) and coefficient of variation \( V_P \). The following values of these parameters are recommended by Agarwal (2008-1).

For internally restrained deck slabs with FRP reinforcement- \( \delta_P = 1.29, V_P = 0.20 \)

For externally restrained deck slabs with steel straps- \( \delta_P = 1.19, V_P = 0.17 \)

It is suggested that the initial static punching shear strength \( P_s \) for fatigue considerations may be taken as that corresponding to an overall reliability index \( \beta = 2.0 \). Therefore,

\[
P_s = \delta_P P_n e^{-0.75 \beta V_P} = \delta_P P_n e^{-1.5 V_P}
\]

where, \( P_n \) is the calculated theoretical nominal punching shear strength, and 0.75 is the sensitivity factor for isolating safety margin on the resistance side.

2.2 Fatigue in External Steel Straps

For partially studded steel straps or those welded to the girder flanges, Detail Category C may be applied in accordance with CHBDC (Fig. 10.6, Examples 13 and 18), provided that the length of the welded detail is not longer than 50 mm in longitudinal direction of the strap. For Category C, the fatigue life constant \( \gamma = 1440 \times 10^9 \), \( m = 3 \), and the constant amplitude threshold stress range \( F_{ut} = 69 \) MPa. With large number of cycles, fatigue stress range limit \( F_{ut} \) could go down to as low as \( F_{ut/2} = 34.5 \) MPa.
3 EQUIVALENT FATIGUE WHEEL LOAD FOR DECK SLAB COMPONENTS, CANADIAN CONDITIONS

Wheel loads on the road are variable. However, an “equivalent fatigue wheel load” can be determined such that the cumulative fatigue effect of the variable wheel loads would be similar to that caused by the application of an equal number of cycles of the equivalent fatigue wheel load, $P_{fatigue}$, amplified by the dynamic load allowance, $I$.

The static value of $P_{fatigue}$ can be determined using the following equation based on Minor’s principle, where $n_i$ is number of occurrences of wheel load $P_i$, and $f_i$ is the normalized frequency of wheel load $P_i$.

$$[9] \quad P_{fatigue} = \left( \frac{\sum n_i P_i^m}{\sum n_i} \right)^{1/m} = \left( \sum f_i P_i^m \right)^{1/m}$$

For steel straps, this equation assumes that within the service load range, the relationship between the wheel load and corresponding stress in steel straps is nearly linear.

Using the wheel load distribution data given by Agarwal (2008-2), and $m=17$ for concrete deck slab and $m=3$ for steel strap, Eq. [9] gives the following values of static wheel load,

$P_{fatigue}$ for concrete deck slabs = 82.5 kN;
$P_{fatigue}$ for steel straps = 31.6 kN

According to the CHBDC provisions (CSA 2006), a dynamic load allowance (DLA) of 0.40 is applied where single axle load governs the design. Including the DLA, the total equivalent fatigue wheel load is $(1+I)P_{fatigue}$, and this load should not exceed the fatigue life, $P_f$, for the concrete deck slab. Also the stress in the strap caused by the wheel load $(1+I)P_{fatigue}$ should not exceed the fatigue stress limit for the steel strap.

$(1+I)P_{fatigue}$ for concrete deck slabs = $1.40 \times 82.5 = 115.5$ kN

$(1+I)P_{fatigue}$ for steel straps = $1.40 \times 31.6 = 44.3$ kN

It is noted that in the above calculations, $I$ is assumed to be constant for all wheel loads, whereas in reality it has been confirmed from field data that $I$ decreases with increase in wheel loads (Bakht et al, 2003).

The CHBDC provisions for steel components in the deck slab, calibrated for a dual-axle combination of 2 x 125 kN with a DLA of 0.3, are somewhat conservative. In accordance with CHBDC, for steel straps considered as deck components, the specified equivalent fatigue wheel load is given by $(1+I)P_{fatigue}$ for steel straps = $1.30 \times 0.62 \times (125/2) = 50.4$ kN. It is considered appropriate to use the conservative provision of CHBDC for steel straps.

4 FATIGUE EVALUATION OF EXAMPLE CASES

To demonstrate the application of above principles and approach, the procedure is applied to the example cases designed in accordance with the CHBDC.

4.1 Example Case A: Internally restrained deck Slab with GFRP reinforcement

The cast-in-place concrete deck slab is composite with steel girders and has internal GFRP reinforcement. The 175 mm thick deck slab is supported over 25” (635 mm) deep I-girders (W610x241) at a spacing of 2500 mm. The flange width and thickness are 329 mm and 31 mm, respectively. At top of the deck, 0.35% GFRP reinforcement is provided in each direction, with a clear cover of 35 mm to the top layer. At bottom, the 1.25% transverse GFRP reinforcement together with 0.35% longitudinal GFRP reinforcement is provided, with a clear cover of 35 mm
to the bottom layer. The specified compressive strength of concrete is 30 MPa. The specified tensile strength of the GFRP is 550 MPa and its modulus of elasticity is 40,000 MPa. In accordance with CHBDC, the wheel foot-print is taken as 250×600 mm.

4.1.1 Fatigue assessment of new deck slab:

For new designs, the bridge design life, according to the CHBDC, is 75 years. For Class A highways with two lanes, specified ADTT for fatigue evaluation specified by the CHBDC is 4000. The number of lifetime wheel load applications, \( N_{\text{life}} \), is given by Eq. [10],

\[
N_{\text{life}} = 365 \times y \times N_d \times p(ADTT)
\]

Where,

\( ADTT = \) average daily truck traffic on the highway \( = 4000 \) for Class A highway

\( N_d = \) average number of design cycles for truck passage \( = 5 \) (Agarwal 2008-2)

\( p = \) fraction of trucks in most critical lane \( = 0.85 \) for a two lane highway

\( y = \) bridge life \( = 75 \) years

\( N_{\text{life}} = 365 \times 75 \times 5 \times 0.85 \times 4000 = 465 \text{ Millions} \)

Using the analytical method of Agarwal (2008-1), theoretical nominal punching shear capacity \( P_n = 392 \text{ kN} \). From Eq. [8], \( P_s = 1.29 \times 392 \times e^{1.5 \times 0.20} = 374 \text{ kN} \).

From Eq. [6], lifetime fatigue capacity \( P_{\text{life}} \) is given by:

\[
P_{\text{life}} = 0.3679 \times 374 = 137.6 \text{ kN} > 115.5 \text{ kN} \text{ (OK)}
\]

4.1.2 Fatigue assessment after 50 years of use:

As discussed above, the environmental effects on the strength of the internal FRP reinforcement has little influence over the static punching shear capacity of the concrete deck slab. However, the deterioration of concrete condition, and hence the actual concrete strength would affect the static punching shear capacity. For this example of fatigue assessment, it is assumed that the specified concrete strength from statistical assessment of concrete core testing is 20 MPa. The remaining bridge design life is 25 years. For Class A highways with two lanes, specified ADTT for fatigue evaluation per CHBDC is 4000. Number of remaining lifetime wheel load applications, \( N_{\text{life}} \), is given by Eq. [10],

\( N_{\text{life}} = 365 \times 25 \times 5 \times 0.85 \times 4000 = 155 \text{ Millions} \)

Theoretical nominal punching shear capacity with reduced concrete strength of 20 MPa, \( P_n = 336 \text{ kN} \). From Eq. [8], \( P_s = 1.29 \times 336 \times e^{1.5 \times 0.20} = 321 \text{ kN} \).

For the first 50 years, applied number of cycles \( n_a = 155 \times 50/25 = 310 \text{ Millions} \), and \( P_a = 115.5 \text{ kN} \). Average static wheel load capacity over first 50 years, \( P_{sa} = (374+321)/2 \text{ kN} \).

From Eq. [6], fatigue life \( N_a \) under application of cycles of uniform load \( P_a \),

\[
N_a = \left( \frac{1.19P_{sa}}{P_a} \right)^{17} = \left( \frac{1.19 \times (374 + 321)/2}{115.5} \right)^{17} = 2610 \times 10^6
\]

From Eq. [7], reduced static punching shear capacity at 50 years is given by,
This gives, \( P_{sr} = 305 \) MPa

From Eq. [6], lifetime fatigue capacity \( P_{life} \) for the remaining life is given by:

\[
\frac{P_{life}}{305} = 1.19 \left( \frac{155000000}{374+321} \right)^{1/17} = 0.3924
\]

\( P_{life} = 0.3924 \times 305 = 119.7 \) kN  >  115.5 kN  (OK)

4.2  Example Case B:  Externally restrained FRC slab with steel straps welded to steel girder flanges

The 175 mm thick cast-in-place externally restrained concrete deck is composite with W610x241 steel girders at 2000 mm spacing and has external steel straps.  The straps with cross-sectional area of 715 mm\(^2\) are welded to the top girder flanges at 1250 mm spacing.  The deck slab has a 50 mm deep haunch on top of the girders.  Specified concrete compressive strength is 30 MPa, and the tensile strength of the strap steel is 350 MPa.  The wheel foot-print is taken as 250×600 mm.  Fatigue assessment is done for the new design.

4.2.1  Concrete Deck Slab:

Using the analytical method of Agarwal (2008-1), theoretical nominal punching shear capacity \( P_n = 553 \) kN.  From Eq. [8], \( P_s = 1.19 \times 553 \times e^{-1.5 \times 0.17} = 510 \) kN.

From Eq. [6], the lifetime fatigue capacity \( P_{life} \) is given by:

\[
\frac{P_{life}}{510} = 1.19 \left( \frac{465000000}{374+321} \right)^{1/17} = 0.3679,
\]

\( P_{life} = 0.3679 \times 510 = 187.6 \) kN  >  115.5 kN  (OK)

4.2.2  Steel Strap:

For \( (I+I)P_{fatigue} = 50.4 \) kN, the stress \( f_s \) in the critical strap from deck slab analysis under service loads is found to be 40.7 MPa.  Using \( N_{life} = 465 \) millions, for welded straps, Detail Category C:

\[
F_{sr} = \left( \frac{1440 \times 10^9}{465 \times 10^6} \right)^{1/3} = 14.6 \text{ MPa} < F_{sr}/2;
\]

Hence, the stress range limit \( F_{sr} = F_{sr}/2 = 34.5 \) MPa.  The above calculated stress range in strap of 40.7 MPa exceeds the stress range limit, so strap in this design example does not satisfy fatigue requirement of the CHBDC.  In order to satisfy the fatigue requirements, the area of each strap should be increased to at least 880 mm\(^2\).

4.3  Field Example- Salmon River Bridge

The cast-in-place externally restrained deck slab of the Salmon River Bridge (Mufti et al., 1999) is composite with steel girders and has external steel straps.  The 200 mm thick deck slab is supported over 1500 mm deep steel plate girders, with flange thickness of 28 mm and flange width of 350 mm, at 2700 mm spacing.  The straps with cross-sectional area of 1400 mm\(^2\) are welded to the top flanges of girders at a spacing of 1200 mm.  The deck slab has haunches over the girders with the average depth and width of 130 and 500 mm, respectively.  The specified concrete compressive strength is 35 MPa, and the tensile strength of strap steel is 350 MPa.  The wheel foot-print is taken as 250×600 mm.  Fatigue assessment is done for the new design.
4.3.1 Concrete Deck Slab of Salmon River Bridge:

From analysis, theoretical nominal punching shear capacity $P_n = 1194$ kN.

From Eq. [8], $P_s = 1.19 \times 1194 \times e^{-1.5 \times 0.17} = 1101$ kN.

From Eq. [6], lifetime fatigue capacity $P_{life}$ is given by:

$\left(\frac{P_{life}}{1101}\right) = 1.19 \times (465000000)^{-1/17} = 0.3679.$

$P_{life} = 0.3679 \times 1101 = 405$ kN $> 115.5$ kN (OK)

4.3.2 Steel Strap of Salmon River Bridge:

For $(I+I)P_{fatigue} = 50.4$ kN, the stress $f_s$ in the critical strap from deck slab analysis under service loads is found to be 20.2 MPa. The above calculated stress range in the strap of 20.2 MPa is well within the stress range limit of 35.5 MPa, hence the strap in this example satisfies fatigue requirement of the CHBDC.

5 CONCLUSIONS AND RECOMMENDATIONS

The paper provides a reliability-based methodology for the fatigue evaluation of internally and externally restrained deck slabs. An application of this methodology for Canadian conditions for bridge deck slabs designed by the empirical method of the CHBDC shows that: (a) the concrete deck slab will have adequate fatigue capacity, and (b) the steel strap design in accordance with the current CHBDC provisions may be inadequate in fatigue for some thinner deck slabs under the assumed traffic loads. It is recommended that consideration should be given to revising the code provisions for the cross-sectional area and spacing of the steel straps.

6 REFERENCES


Agarwal, AC. 2009. Local stress concentration in reinforcing bars in a deck due to concrete cracks. ISIS Canada, Winnipeg, Manitoba, Canada.


